



Flood Risk & Drainage Assessment

Proposed Warehouses | Land off Lawton Street

New Smithfield Market | Manchester | M11 2WJ

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1.0 Introduction

It is proposed to develop land which is located off Lawton Street at Smithfield New Market in Manchester for the provision of two new warehouse buildings with associated goods vehicle access and turning hardstanding together with car and trailer parking as shown below in figure 1.



FIGURE 1 PROPOSED SITE PLAN¹ (REDUCED, NTS)

The site currently has a mixture of uses with the central area used for car parking, the western and eastern areas used for commercial units, the central southern part used as a controlled vehicle access and egress point for the markets, whilst the south eastern area comprises a bankside and a lorry parking area.

Non-residential developments comprising: the provision of a building or buildings where the total floorspace to be created is 1000 square metres or more, or; where the floor area is not yet known, a site area of 1 hectare or more are classified as 'major development' and in line with paragraph 045 of the Planning Practice Guidance for Flood Risk and Coastal Change (PPG-FRCC) the Local Planning Authority (LPA) are required to consult with the Lead Local Flood Authority (LLFA) on surface water drainage. Paragraph 169 of the 2021 NPPF states that 'major developments should incorporate sustainable drainage systems unless there is clear evidence that this would be inappropriate'.

¹ Cube Architecture & Design drawing number AS763-03G

The notional development extent is shown on appended key plan [1] within a redline area of approximately 2.828ha. The proposed building areas including canopies total approximately 1.254ha. The Environment Agency Flood Map for Planning does not show the site to be affected by fluvial flooding. The EA Long Term Flood Risk Maps show the site to be susceptible to flood risk from both surface water and reservoir sources. The Manchester City, Salford City and Trafford Councils Level 2 Hybrid SFRA (MST SFRA) does not highlight any groundwater or known / historical sewer flooding problems at or in the vicinity of the site. Drawing SS_4.5 of the MST SFRA shows the site area located within the 'Conurbation Core' critical drainage area.

Whilst the proposed site is in flood zone 1, the development area exceeds 1ha and is at risk of surface water flooding. Therefore, in accordance with the footnote to paragraph 167 of the National Planning Policy Framework (NPPF 2021), a site-specific flood risk assessment is required. The development will only be considered appropriate if the FRA demonstrates that: 'within the site, the most vulnerable development is located in areas of lowest flood risk; development is appropriately flood resilient and resistant; it incorporates sustainable drainage systems, unless there is clear evidence that this would be inappropriate; any residual risk can be safely managed; and safe access and escape routes are included where appropriate, as part of an agreed emergency plan'.

The following plan shows the location of the site in the context of the local area and identifies features referred to in this FRDA.

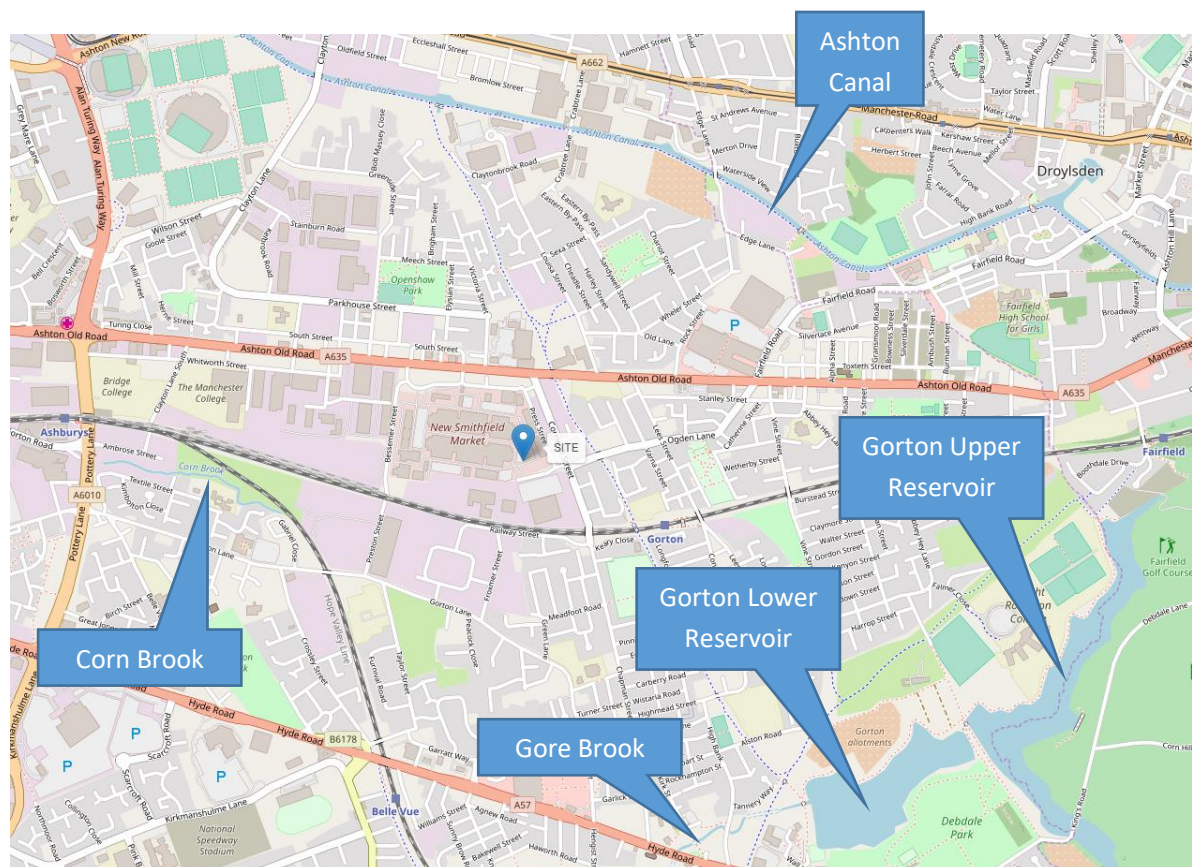


FIGURE 2 SITE LOCATION²

² <https://www.openstreetmap.org/copyright>

2.0 Flood Risk Avoidance & Compatibility

2.1 Development Vulnerability

According to table 2 of PPG-FRCC, buildings used for 'offices, general industry, storage and distribution' are classified as 'less vulnerable' with respect to flood risk. The matrix in Table 3 demonstrates this type of land use is appropriate in flood zone 1. This is subject to passing the Sequential Test, which requires land in flood zone 1 to be considered in preference to flood zones 2 or 3. It is noted in the guidance that table 3 does not reflect the need to avoid flood risk from sources other than rivers and the sea.

In summary, whilst the location in flood zone 1 means it is not necessary to apply the sequential or exception tests in respect of fluvial sources, the interaction of the proposed development for other potential flood mechanisms needs to be considered as recommended by Paragraph 33 of PPG-FRCC: 'Any development proposal should take into account the likelihood of flooding from other sources, as well as from rivers and the sea. The sequential approach to locating development in areas at lower flood risk should be applied to all sources of flooding, including development in an area which has critical drainage problems, as notified to the local planning authority by the Environment Agency, and where the proposed location of the development would increase flood risk elsewhere'.

2.2 Flood Risk Management

The following section discusses site specific sources of flooding. Where it is not possible to avoid sources of flooding, the consequences of flooding to or as a result of a development can be mitigated by site and building design. For example, flood resilience measures can be integrated into the building design if a flood avoidance strategy is not viable. This process needs to be considered in relation to the expected depth and speed of onset of flooding, with the site and building made safe for their lifetime by taking into account the effects of climate change.

Paragraph 50 of PPG-FRCC directs developers to 'seek opportunities to reduce the overall level of flood risk in the area and beyond'. It advises that 'this can be achieved, for instance, through the layout and form of development, including green infrastructure and the appropriate application of Sustainable Drainage Systems' (SuDS). A drainage strategy is considered later in this document.

3.0 Potential Flood Mechanisms

3.1 Fluvial Sources

The nearest significant open channel ordinary watercourse is Corn Brook situated approximately 0.6km to the south west and at circa 12m below site levels. There is a drain shown on ordnance survey plans alongside the railway circa 200m south of the site where ground levels are approximately 0.7m below site levels. It is expected that the drain has an outfall to Corn Brook that is culverted under Thorpeness

Square (refer to figures 3 and 4 below) and therefore when the watercourse surcharges, this may cause flooding along Preston Street and to the land west of Preston Street, constrained by the railway line embankment. Gore Brook is the nearest main watercourse located approximately 1.3km to the south in Sunny Brow Park, again at around 12m below site levels.

The following extract of the online Flood Map for Planning demonstrates that the site is in flood zone 1 and is therefore at low risk (less than 1 in 1,000 annual probability) of river or sea flooding. The flood map does not provide information on flooding from other sources such as groundwater, direct runoff from fields or overflowing sewers. It does not show the effect of flood defences or climate change on the extent of flooding.

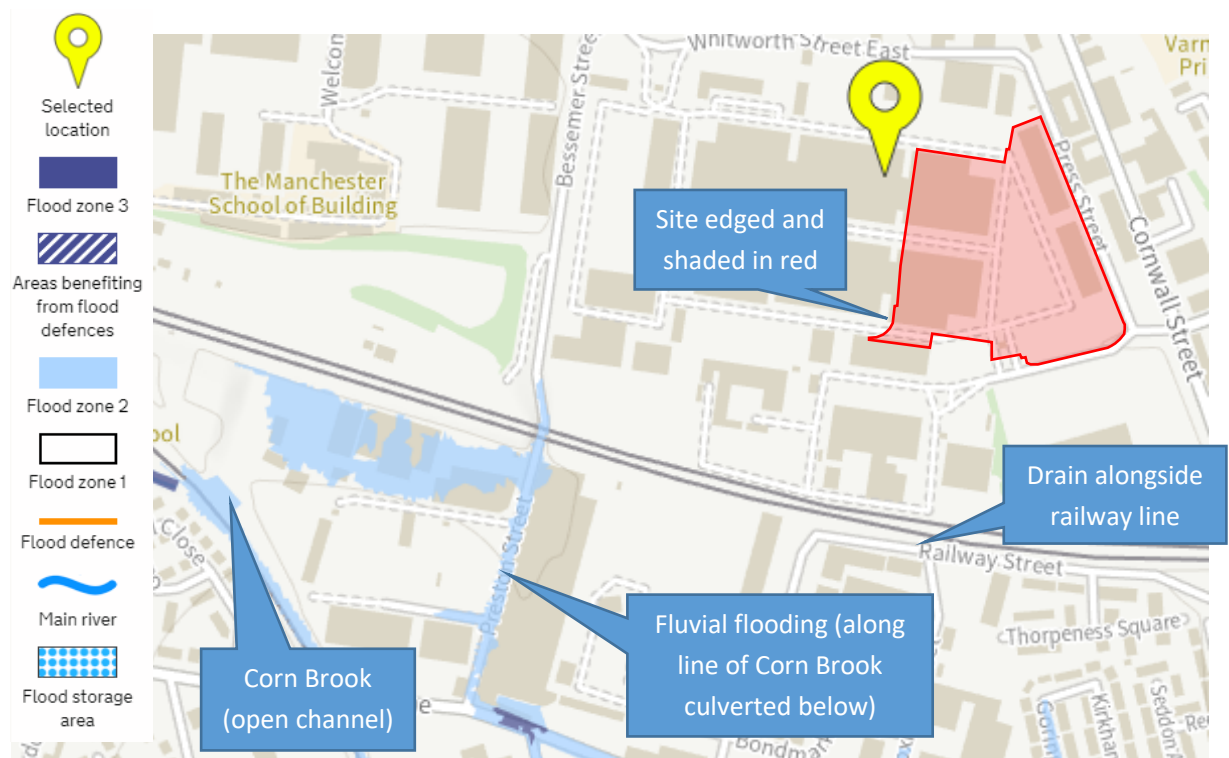


FIGURE 3 EXTRACT EA FLOOD MAP FOR PLANNING

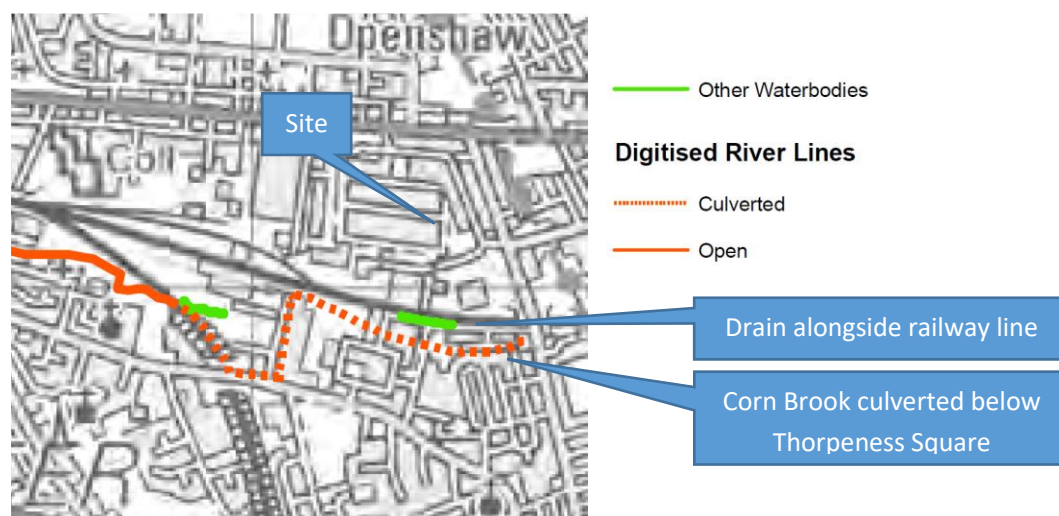


FIGURE 4 EXTRACT FROM SFRA MAPS

Flood extents for the smaller ordinary watercourses are generally not shown on the EA Flood Map for Planning, however, the SFRA confirms that the EA have a hydraulic model of Corn Brook³ which extends from Railway Street, Gorton to its confluence with the Manchester Ship Canal at Pomona Docks.

3.2 Groundwater Sources

The SFRA notes that ‘according to Greater Manchester sub-regional SFRA there are no known reports of groundwater flooding in the Manchester, Trafford and Salford authorities’ and that ‘reduced abstraction has also led to groundwater rebound in central Manchester, although levels have now stabilised’.

Mine water rebound involves rising groundwater levels resulting from the cessation of mine dewatering and the SFRA comments that the Coal Authority who are responsible for monitoring this change consider that ‘minewater levels in Manchester, Salford and Trafford have recovered or almost recovered. Some areas are likely to experience a slight rebound, but the effects of this will be minimal’.

The site is not shown on EA maps to be in a groundwater source protection zone but is located over a major aquifer that has a high vulnerability to groundwater pollution typically characterised by high leaching soils and the absence of low permeability superficial deposits. The bedrock, shown on BGS geology maps as ‘Chester formation – sandstone’ is designated as a principle aquifer, whilst superficial deposits are recorded by the BGS to be ‘Till’ and are designated a Secondary A aquifer. Soilscape maps describe near ground soils as ‘slowly permeable seasonally wet acid loamy and clayey soils’ with ‘impeded drainage to stream network’.

The nearest BGS borehole⁴ drilled in 1970 is circa 150m to north of the site and logs record initial fill material to 1.37m (4ft6in) below ground level over stiff silty and stiff sandy clay soils to the depth of the excavation at approx. 9.1m (30ft) bgl, with a groundwater strike at 6.4m (21ft) bgl and slow seepage with a standing level of approx. 8.1m (26ft6in) bgl.

A ‘SUDS Map’ from the 2008 Greater Manchester Sub-Regional SFRA shows the site within an area coloured orange signifying ‘Attenuation - above highly vulnerable aquifer’. This, together with the EA groundwater vulnerability map classification, suggest that there is a pathway for contaminants at ground level to reach groundwater in the bedrock aquifer via the intermediary superficial soils. However, based on the BGS borehole record and soilscape description of near ground soils, it is not considered that a linkage is available and similarly that the reverse flow of ground water from aquifer to issue at ground level is not a viable mechanism.

The report⁵ for intrusive ground investigation undertaken at the proposed development location by E3P has been reviewed. It advises that ‘Groundwater strikes were encountered as seepages from 0.40m with the shallow boreholes to 5.00m bgl. Within the deeper cable hole a groundwater strike was encountered at 7.30m bgl within a sand band between two clay bands’. The report also comments that ‘During the

³ Atkins (2009) River Medlock and Corn Brook Flood Mapping Study (2D Infoworks CS)

⁴ BGS ID: 937543 : BGS Reference: SJ89NE1126, British National Grid: 388580,397260

⁵ Phase II Geoenvironmental Site Assessment, New Smithfield Market, Barton and Redman, E3P, July 2020, Reference: 12-757-R2-1,

initial monitoring visits, groundwater levels were observed to increase minimally. The greatest groundwater change occurred within WS210 where levels increased by 0.04 m over the monitoring period. Window sample locations are mostly set within a low-permeability clay stratum and groundwater may be a result of surface water collecting in the well rather than a measure of true groundwater’.

There are no basements or similar subterranean elements proposed as part of the new building that would require special investigation or measures.

3.3 Pluvial Sources

Rain falling over the ground surface that is not infiltrated into the ground due to incapacity of soils, saturation of soils or the presence of development via paved areas will either pond locally or flow overland depending on the prevailing terrain. This is commonly referred to as surface water runoff and the EA provide online maps (RoFSW) that show the susceptibility of land to the risk of surface water flooding.

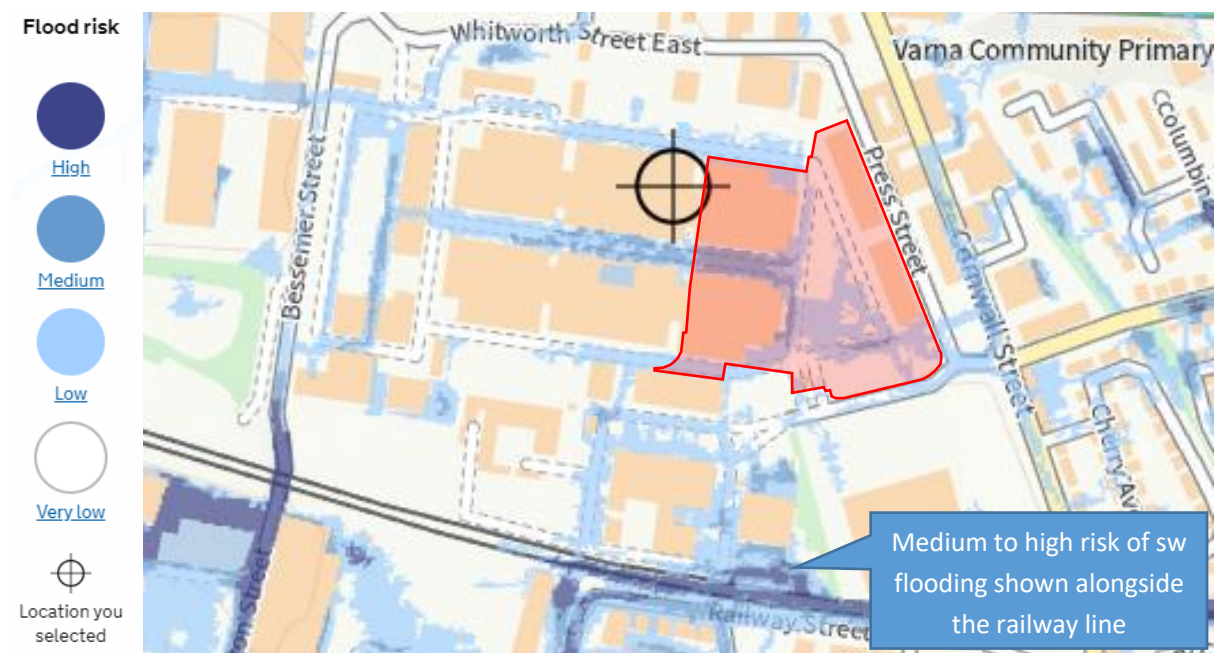


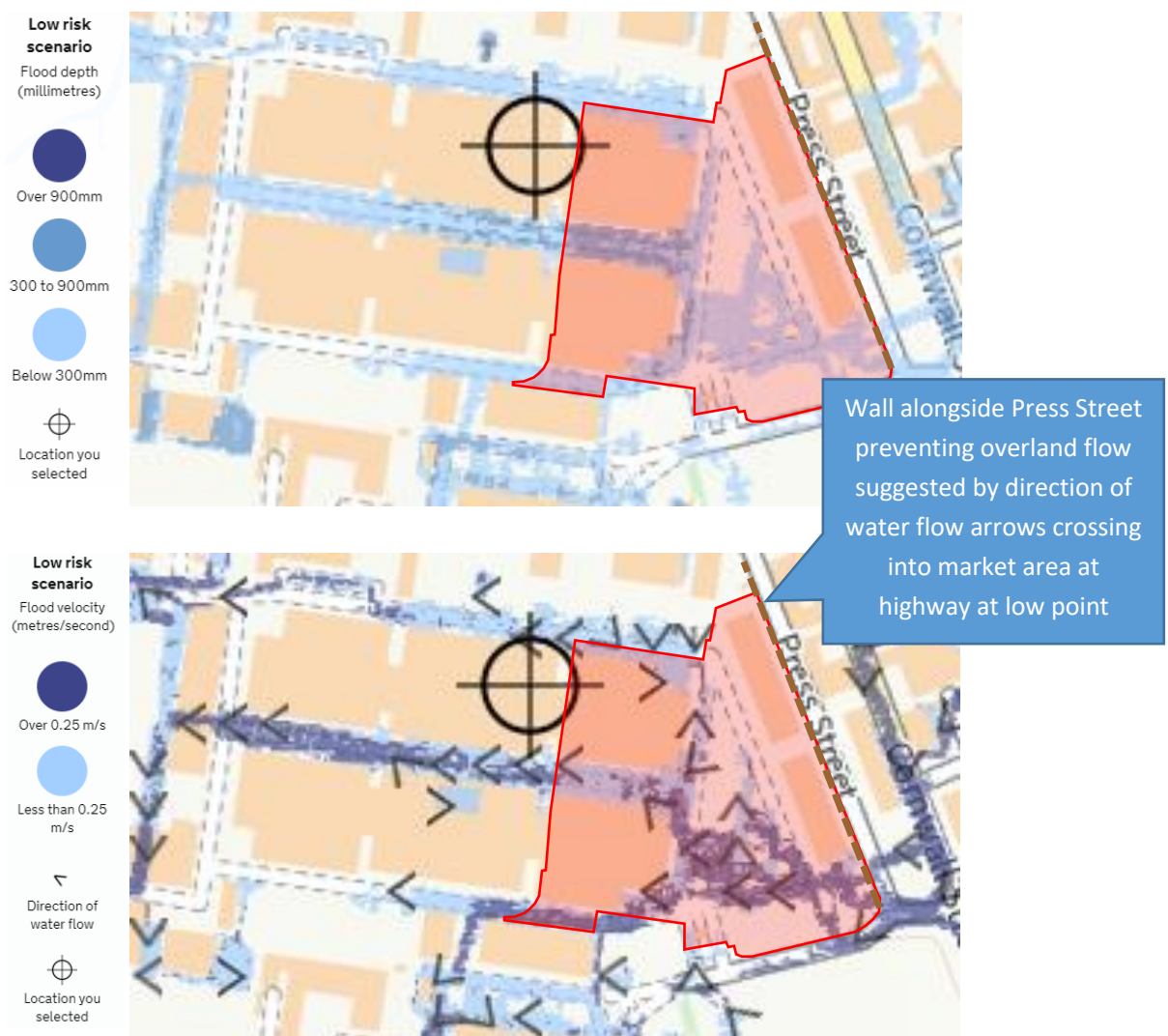
FIGURE 5 EXTRACT EA FLOOD RISK FROM SURFACE WATER MAP

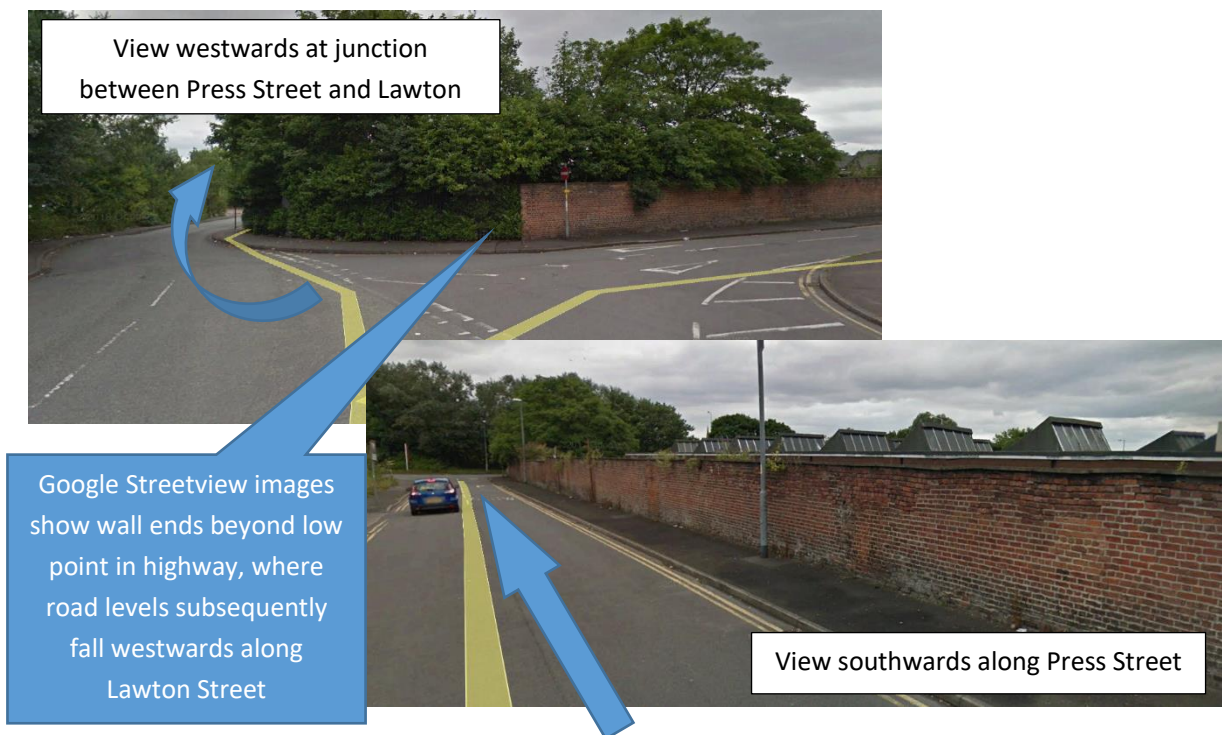
The RoFSW maps are the result of modelling which allows for four rainfall scenarios together with the form and type of terrain in terms of level data, ground roughness and adjustments to suit rural and urban areas where losses will occur due to natural infiltration and the presence of drainage systems. It should be noted⁶ that ‘due to the modelling techniques used, the mapping picks out depressions in the ground surface and simulates some flow along natural drainage channels, rivers, low areas in floodplains, and flow paths between buildings. Although the maps appear to show flooding from ordinary watercourses,

⁶
https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/297432/LIT_8988_Obf634.pdf

they should not be taken as definitive mapping of flood risk from these as the conveyance effect of ordinary watercourses or drainage channels is not explicitly modelled. Also, structures (such as bridges, culverts and weirs) and flood risk management infrastructure (such as defences) are not represented’.

The central area of the site is shown to be at ‘low’ risk of flooding from surface water runoff. This means that each year this area has a chance of flooding of between 1% and 0.1% (between 1in100year and 1in1000year return). The RoFSW map guidance notes that ‘flooding from surface water is difficult to predict as rainfall location and volume are difficult to forecast. In addition, local features can greatly affect the chance and severity of flooding.’ When local features are considered, the overland flood flow route suggested as shown on the following surface water flood velocity map extract is unlikely to occur because of the presence of a wall alongside Press Street that is unlikely to be included in the digital terrain model used to derive the RoFSW maps.





Using the current surface water flood map dataset, the hazard associated with the low-risk flood extents is shown below (figure 6). With reference to the Flood Hazard Rating Table extracted from DEFRA guidance FD2321 (see below) the Hazard to people classification for areas shaded in green are deemed to be 'very low hazard - caution'.

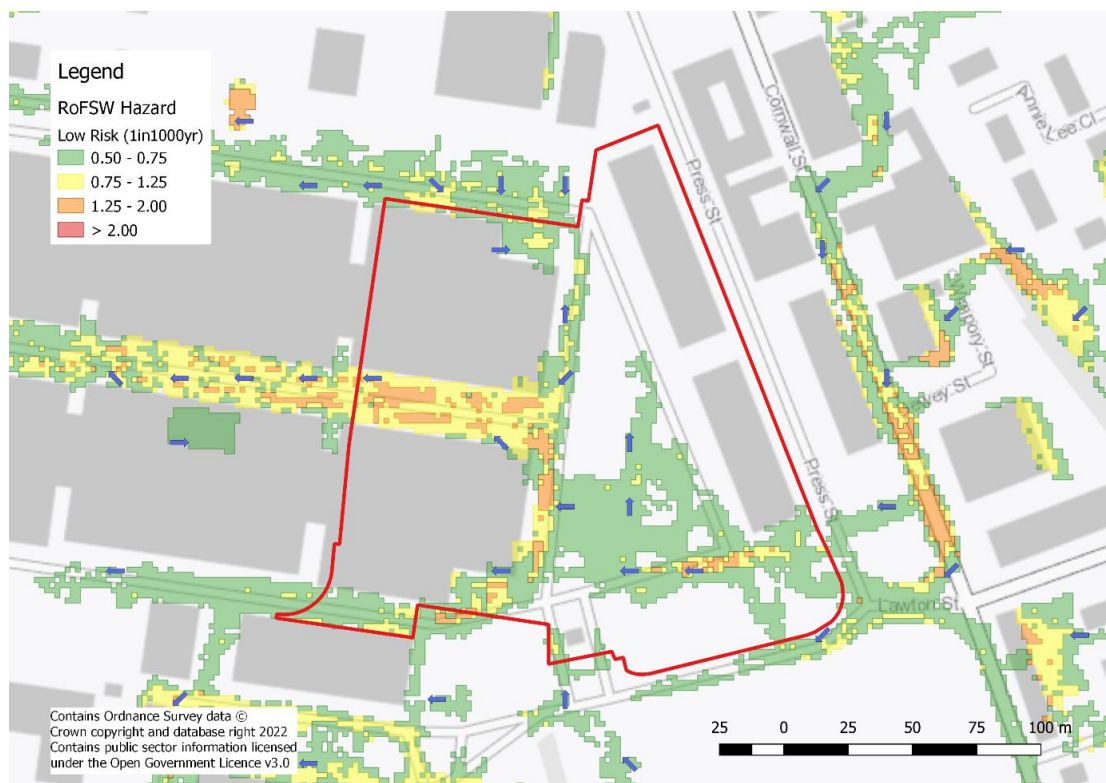


FIGURE 6 ROFSW HAZARD - LOW RISK SCENARIO (1in1000yr)

This table is recommended for development planning and control use.

Table 4 – Hazard to People Classification using Hazard Rating ($HR = d \times (v + 0.5) + DF$) for (Source Table 13.1 of FD2320/TR2 – Extended version)

HR	Depth of flooding - d (m)												
	DF = 0.5				DF = 1								
Velocity v (m/s)	0.05	0.10	0.20	0.25	0.30	0.40	0.50	0.60	0.80	1.00	1.50	2.00	2.50
0.0	0.03+0.5= 0.53	0.05+0.5= 0.55	0.10+0.5= 0.60	0.13+0.5= 0.63	0.15+1.0= 1.15	0.20+1.0= 1.20	0.25+1.0= 1.25	0.30+1.0= 1.30	0.40+1.0= 1.40	0.50+1.0= 1.50	0.75+1.0= 1.75	1.00+1.0= 2.00	1.25+1.0= 2.25
	0.03+0.5= 0.53	0.06+0.5= 0.56	0.12+0.5= 0.62	0.15+0.5= 0.65	0.18+1.0= 1.18	0.24+1.0= 1.24	0.30+1.0= 1.30	0.36+1.0= 1.36	0.48+1.0= 1.48	0.60+1.0= 1.60	0.90+1.0= 1.90	1.20+1.0= 2.20	1.50+1.0= 2.55
0.3	0.04+0.5= 0.54	0.08+0.5= 0.58	0.15+0.5= 0.65	0.19+0.5= 0.69	0.23+1.0= 1.23	0.30+1.0= 1.30	0.38+1.0= 1.38	0.45+1.0= 1.45	0.60+1.0= 1.60	0.75+1.0= 1.75	1.13+1.0= 2.13	1.30+1.0= 2.30	1.58+1.0= 2.58
0.5	0.05+0.5= 0.55	0.10+0.5= 0.60	0.20+0.5= 0.70	0.25+0.5= 0.75	0.30+1.0= 1.30	0.40+1.0= 1.40	0.50+1.0= 1.50	0.60+1.0= 1.60	0.80+1.0= 1.80	1.00+1.0= 2.00	1.50+1.0= 2.50	2.00+1.0= 3.00	2.50+1.0= 3.50
	0.08+0.5= 0.58	0.15+0.5= 0.65	0.30+0.5= 0.80	0.38+0.5= 0.88	0.45+1.0= 1.45	0.60+1.0= 1.60	0.75+1.0= 1.75	0.90+1.0= 1.90	1.20+1.0= 2.20	1.50+1.0= 2.50	2.25+1.0= 3.25	3.00+1.0= 4.00	3.75+1.0= 4.75
1.0	0.08+0.5= 0.58	0.15+0.5= 0.65	0.30+0.5= 0.80	0.38+0.5= 0.88	0.45+1.0= 1.45	0.60+1.0= 1.60	0.75+1.0= 1.75	0.90+1.0= 1.90	1.20+1.0= 2.20	1.50+1.0= 2.50	2.25+1.0= 3.25	3.00+1.0= 4.00	3.75+1.0= 4.75

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

The depth of surface water flooding shown on the RoFSW where the hazard rating is between 0.75 and 2.00 (yellow and orange shading on figure 6) exceeds 0.3m as shown below (figure 7), but is generally less than 0.25m/s (refer to figure 8).

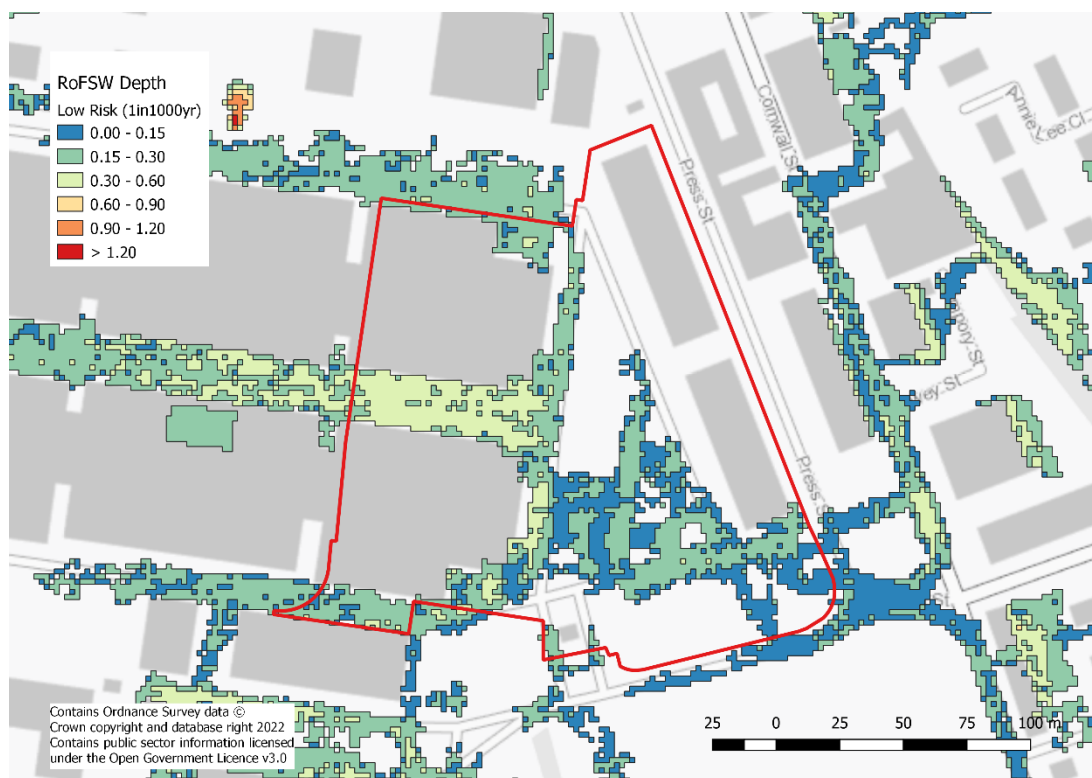


FIGURE 7 ROFSW DEPTH - LOW RISK SCENARIO (1in1000yr)



FIGURE 8 RoFSW VELOCITY - LOW RISK SCENARIO (1in1000YR)

Taking the effect of the wall into account, it is unlikely that the extent of surface water flooding shown on the RoFSW map will actually occur within the development area and instead overland runoff would be routed down Lawton Street to the south of the site.

However, as there is a significant area of hardstanding at the site, it is expected that there will be the potential for ponding of surface water generated from roof and paved areas within the site in extreme conditions and in the event of blockage of sewers, etc. The velocity of such food water is likely to be less than 0.25m/s and less than 300mm and so would have hazard rating of less than 0.75.

Existing ground levels across the proposed warehouse building footprints vary between approx. 67.65m at the east of unit 2 up to approx. 66.90m at the western end of unit 1. Therefore, it is expected that unit 2 will have a finished floor level (FFL) of 67.70m, with localised grading undertaken to the eastern side to ensure a standard 150mm clearance above external finished ground levels. Unit 1 can then be set with a FFL of 67.20m that provides at least 300mm above the lowest existing external ground level allowing freeboard against potential overland flood flows as suggested by the RoFSW maps. The appended drainage layouts therefore show these FFLs.

3.4 Artificial Drainage System Sources

An extract of the sewer network at and in the vicinity of the site has been obtained from United Utilities (UU) and is shown (reduced, nts) below (figure 9) with the site outline in orange. This shows that there is a combined water sewer crossing centrally through the site in proximity to the proposed buildings. Whilst UU's records annotate this as 100mm concrete and 300mm brick, it is understood that the sewer is

actually 900mm brick and is a relic part of the Collyhurst 'V' interceptor sewer. The sewer has been camera surveyed by Lanes Group Plc and is shown on a record drainage layout plan for the New Smithfield marked area. The Lanes survey notes a weir wall at the upstream end adjacent to the south east site corner (at UU chamber 6107) and so it appears that flows in the 1000mm brick sewer under Lawton Street turn northwards to flow in the 1400mm concrete sewer under Press Street, with only an overflow able to pass into the 900mm brick sewer crossing through the site.

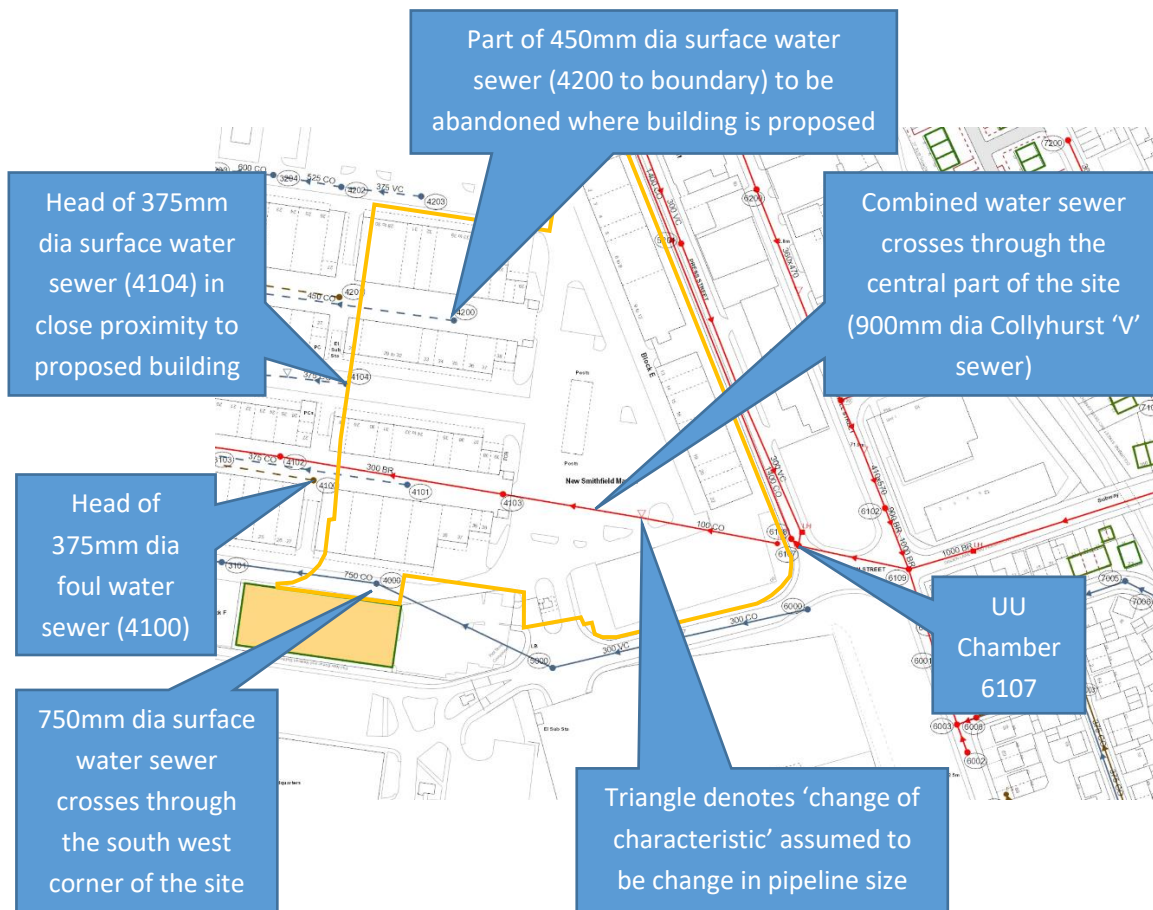


FIGURE 9 UNITED UTILITIES SEWER RECORDS

The sewer records also show a surface water sewer alongside the south of the site under Lawson Street. This is 300mm diameter and then increases to 750mm diameter as it changes direction and crosses under the south west corner of the site. Three more surface water sewers are shown on the sewer records that also flow westwards and are aligned alongside and adjacent to the existing buildings at the market with the northern sewer conflicting with the proposed footprint of unit 1. It is therefore expected that this length of sewer will be abandoned, subject to formal agreement with United Utilities.

The SFRA advises that 'where the SFRA has identified that there is a risk from surface water flooding, any water that surcharges the sewer system would be expected to follow similar flow paths and pond in similar low spots, although the volume of water that emerges from the system will be entirely dependent on the reason for the network surcharging (which could be due to rainfall beyond the design level of the sewer system, sewer capacity issues or blockage or failure)'.

Given the site area and surrounding land has almost entirely impermeable cover, the sewers and pipelines draining the area will already cater for surface water runoff, but may only have a limited capacity offering a particular standard of flood protection. Exceedance of the pipeline capacities will result in localised flooding above ground level. The preceding section has examined the potential for surface water flooding and overland flows at the site and in line with the SFRA guidance, it is expected that flood water not managed by drainage systems will affect similar areas.

It should also be noted that the new surface water drainage system will also reduce peak discharge rates to the sewer and will provide capacity to attenuation rainfall intensities up to and including 1in100year return period, with an increase applied to allow for the predicted effects of climate change over the lifetime of the development.

3.5 Infrastructure Failure Mechanisms

The EA provide maps to show the potential extent of flood water that could occur in the event of a reservoir failure event. There are 2 flooding scenarios shown on the reservoir flood maps. They are a 'dry-day' and a 'wet-day'. The 'dry-day' scenario predicts the flooding that would occur if the dam or reservoir failed when rivers are at normal levels. The 'wet day' scenario predicts how much worse the flooding might be if a river is already experiencing an extreme natural flood.

It should be noted that the EA reservoir flood maps show the largest area that might be flooded if a reservoir were to fail and release the water it holds. The map displays a worst-case scenario and is only intended as a guide. It is not a prediction of what will happen. The site and surrounding area are shown to be affected as shown on the extract in figure 11 below.

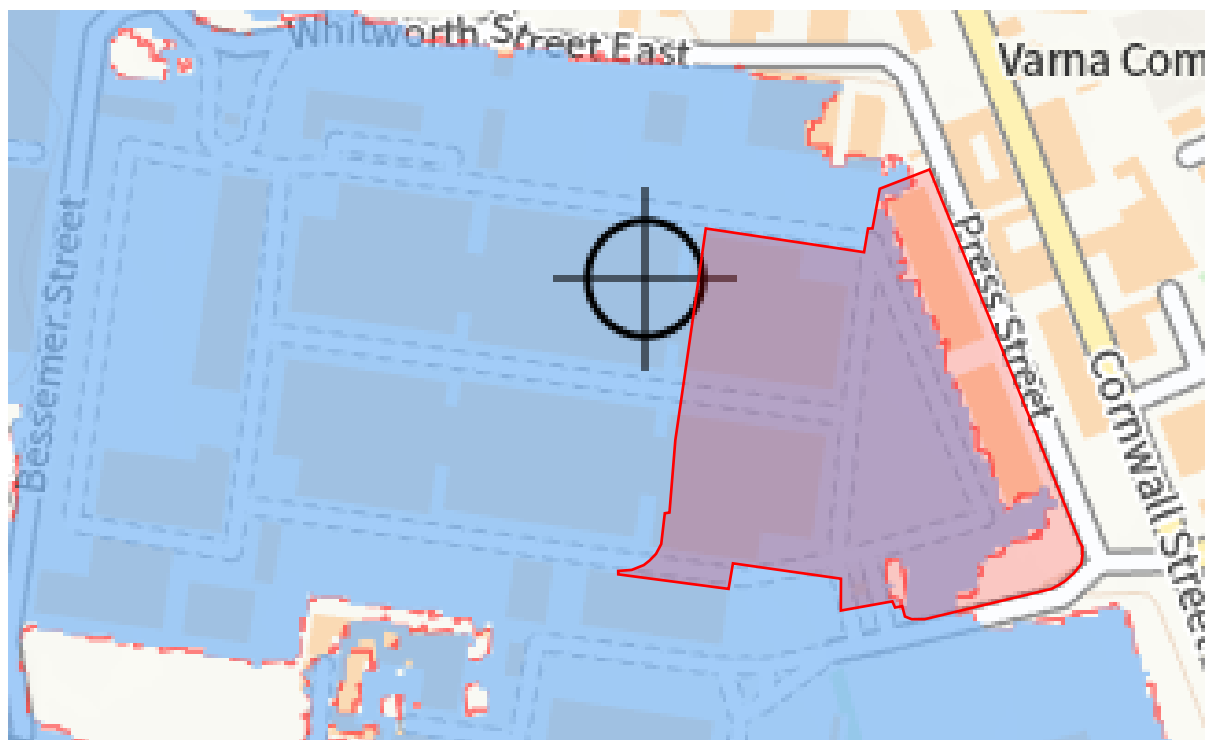


FIGURE 10 POTENTIAL EXTENT OF FLOODING DUE TO A RESERVOIR FAILURE EVENT

The reservoirs that could affect the development site area are Audenshaw No.1 (grid reference SJ9170096700) and Audenshaw No.3 (grid reference SJ9110096700). At the site location, the extent of flooding from either reservoir is the same

Gorton Reservoir situated circa 1.3km to the south east was originally built as a drinking water supply and is owned by United Utilities. It is understood that the reservoir is no longer used but is currently maintained at its normal maximum water level as a leisure facility. Due to its size, maintenance of the reservoir falls under the provisions of the Reservoirs Act and the Reservoir Inspectorate. The Audenshaw Reservoirs situated circa 2.5km eastwards from the site are also maintained by United Utilities.

The SFRA describes Ashton Canal as a broad canal which are characteristically shallow with 'raised embankments in places and mainly used today for tourism, carrying narrow boats and other small boats'. Ashton Canal is approximately 1.0km to the north of the site and is managed by the Canal and River Trust (formerly British Waterways). The SFRA comments that 'these canals do not pose a direct flood risk as they are controlled water bodies with regulated flows. However, there are residual risks associated with lower probability events such as breaching of embankments. The residual risk associated with canals is unknown as it depends on a number of factors including: the source of water into the canal, materials used within the canal embankments and the condition of those embankments. The risk is limited due to continued maintenance by British Waterways ... The main flood hazards on the Ashton Canal are perceived to arise from overtopping and/or breaching. Specific issues include:

- a potentially high inflow to the canal with flow towards Manchester during flood conditions which exceeds the canal capacity at Manchester,
- possible blockage of the bywash at Ancoats in flood conditions and a consequent overtopping of the canal banks.

A historic flood event from 2007 relating to the Ashton Canal is recorded on the National Historic Flood Map that is maintained by the EA. This is referred to in the SFRA and is described as 'flooding reported at Ancoats'.

4.0 The Impact of Climate Change

The effect of changes in the climate have to be considered in respect of flood risk. Principally the effect of changes to rainfall intensity, duration and frequency is critical for surface water flood risk, rate of rise in river and minor watercourse water levels together with the extent of flooding when watercourse banksides and drainage system capacities are exceeded.

The Environment Agency provide advice⁷ for climate change allowances that should be considered in flood risk assessments for both river flows and rainfall intensities. The development location is situated in Irwell Management Catchment area of the North West River Basin District. The site is in flood zone 1 and so for 'less vulnerable' development, the central allowance categories for river flow increase are

⁷ <https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances>

appropriate and the guidance suggests that the total potential change anticipated for the '2080s' (2070 to 2115) is 35%.

The EA provided updated guidance in May 2022 for anticipated changes in extreme rainfall intensity in small and urban catchments. In the Irwell Management Catchment, the following allowances apply for the 2070s epoch:

Annual Exceedance Rainfall Event	Central Allowance	Upper End Allowance
3.3% (1in30year return period)	30%	40%
1% (1in100year return period)	30%	45%

It is evident from review of the preceding section that an increase in main river flow will not have a direct impact on the site as the extent of flood zones 2 and 3 will not be significantly altered and not to the extent that the site would be affected.

The climate change increases for rainfall are considered as part of the drainage design for the development.

5.0 Flooding – Development Interaction

5.1 The Impact of Flooding on The Development

As discussed in the preceding sections, the site is considered to be at either low or very low risk of flooding from all sources except for potential overland flood flow and ponding as informed by the RoFSW maps. The proposed building finished floor levels can be selected to mitigate the risk of inundation, whilst it is acceptable for some surface flooding to the external paved areas.

5.2 The Impact of Development on Flood Risk

The provision of the warehouse buildings together with associated external hardstanding for vehicle access, manoeuvring and parking will mostly replace existing impermeable areas of hardstanding currently used for vehicle access, parking and a security gatehouse. However, there will be the loss of some open greenfield land in the form of soft landscaped banking. There appears to be a loss of 0.282ha permeable area, but a re-provision of 0.139ha permeable area, resulting in a decrease of permeability between pre and post development conditions.

This decrease is unlikely to result in a significant increase in the surface water flood risk that is identified on the EA's RoFSW map. However, a drainage system following best practice guidance and in line with the principles discussed in this FRDA will better manage surface water runoff within the site and so will have a beneficial influence as rain falling at the site can be intercepted and attenuated with controlled release into the surface water drainage system.

5.3 Mitigation Measures

To mitigate the potential increase in the rate and volume of runoff that will be drained from the site, the use of Sustainable Drainage Systems (SuDS) should be implemented in line with local and national planning policy as far as reasonably practicable with a view to replicating greenfield characteristics. However, there is by inspection a limit to what can be undertaken at the site in terms of soft / green vegetated solutions. Strategic and site level evidence record fine grained soils across the site that have a low infiltration capacity.

The design of the warehouse buildings is expected to utilise steel framing with lightweight roof and wall cladding, which are not typically compatible with even extensive green roof coverings due to the increase in loading that would need to be sustained and adverse effect this has on the efficiency and therefore cost of the steel frame.

The use of rainwater harvesting is feasible in respect of a large catchment being available via the roof area. However, the minimal occupancy of the building that will generally be used for storage would make the use of harvested rainwater inefficient as its primary purpose would be for toilet flushing in office / personnel areas. The potential to reduce the overall volume of surface water runoff from site via diversion into a harvesting system is reliant on the above ground building services (water supply and distribution) design.

The incorporation of rainwater harvesting would not alter the volume of attenuation storage needed to manage surface water runoff at the site which is sized on the basis that harvesting is not effective (for example due to limited use or mechanical or electrical failure resulting in the harvesting tank being full under rainfall conditions), but the overall volume of water leaving the site for disposal to the local watercourse directly or via surface water sewers would be reduced over the course of each year. It is understood that rainwater harvesting is not currently included in the development proposals, but as noted above, this will not adversely affect the management of surface water to control flood risk at the development.

It is not considered that there is an unusual risk of groundwater or flood water from a sewer fracture or leak adversely affecting the building that would require special flood resilience construction for the ground floor and sub-structure parts of the building generally. Sewer records show that the existing combined water sewer crossing the site is in the order of 5m depth. There are no basement areas identified and it is expected that conventional building design will be employed with sub-structure materials that are inherently robust in respect of prevention of damp entry, etc.

Runoff from roof and paved areas within the site will be managed by the proposed drainage system, so the area shown on the RoFSW to be susceptible to overland flooding is unlikely to be so extensive. However, for robustness to allow safe routing of overland flows that may arise due to residual surface water runoff from areas adjacent to the site or exceedance flows arising from within the site, it is recommended that the building finished floor level is set at least 300mm above the lowest existing ground level to the west of the site = $66.90 + 0.3 = 67.20\text{m AOD}$.

6.0 Drainage Assessment

6.1 Foul Water Drainage

As presented via figure 9 in section 3.4, the local sewerage is the responsibility of United Utilities. The head of the nearest foul water sewers (UU chambers 4100 and 4201) would present logical connection points for the proposed warehouse buildings, subject to a Section 106 agreement with United Utilities under the Water Industry Act.

6.2 Surface Water Drainage

6.2.1 Outfall destination hierarchy

As discussed in section 3.2, the underlying ground conditions are not deemed appropriate for the disposal of surface water runoff via infiltration. The nature of fine-grained clay material found in a borehole near the site from another investigation corresponds to low permeability characteristics that are not suitable for infiltration drainage techniques such as soakaways, etc. The Ground Investigation Report by E3P states that 'The Made Ground and soils are cohesive in nature and the falling head tests undertaken have proven the ground to be impermeable with limited soakage potential'.

A positive outfall will therefore be required and it is proposed that surface water runoff from the site should be discharged to one or more of several surface water sewers that cross or are within proximity to the development site, subject to agreement with United Utilities via a S106 application. Various sources of sewer and site drainage records, together with some camera survey information have been reviewed and used to determine the location and level of sewers and drains at the site. However, it is recommended that existing drainage in and around the site is surveyed as soon as possible to confirm the precise line, level, connectivity and condition of these assets, whether they are to be re-used or made redundant and abandoned.

6.2.2 Discharge rate

The subject site is predominantly brownfield land and whilst it clearly drains to a conventional drainage system as evident from a number of gullies shown on the topographical survey, it is possible that the Lead Local Flood Authority will require as close to a greenfield rate of discharge as practicable in line with the non-statutory technical standards for SuDS published by Defra. This has been calculated as shown on MicroDrainage sheet 1 included in Appendix A to be $q_{bar} = 5.6\text{l/s/ha}$. The development runoff area has been determined to be 2.599ha (as shown on appended key plan [1]) and so a limiting rate of $5.6 \times 2.599 = 14.5\text{l/s}$ would be appropriate on this basis.

6.2.3 Water Quality

The 2015 SuDS Manual provides best practice for surface water drainage including water quality aspects. Table 4.3 notes that the pollution hazard level for roofs (excluding residential) and non-residential car

parking with infrequent change is 'low', whilst the level for commercial yard and delivery areas and non-residential parking for frequent change is 'medium'. The corresponding water quality management requirements for discharge to surface waters may be addressed by following the simple index approach.

It is considered that the use of trapped gullies, silt traps and catchpits within the proposed drainage system will be adequate mitigation to address the low pollution hazard level. In terms of the paved areas through the site, these are deemed to be medium risk (except the pedestrian pathways that will either shed to vehicular paved areas or drain to rainwater down pipe gully pots).

The paved areas across the site are to be constructed using a conventional surfacing of either reinforced concrete or heavy-duty asphalt to suit robustness to sustain not only gross vehicle weight and frequency of axle loading criteria, but also the action of wheels from manoeuvring. Therefore, separators have been included in the drainage system to manage silts and oils.

6.2.4 Rainfall Intensity

It is necessary to provide capacity in the drainage system for runoff due to 1in30year return period rainfall with runoff exceeding this up to and including 1in100year return period intensities increased for climate change managed safely on site. Due to the site constraints (ie proposed finished levels, lack of open space, proximity of properties to boundaries, etc) it has been decided to initially check the attenuation tank sizes to suit capacity for 1in100year+40% rainfall intensities. The full system of tank, chambers and pipes offers a greater total storage capacity and so will be checked for rainfall intensities up to and including 1in100year+45% in line with the latest guidance.

6.2.5 Climate Change

As noted in section 4.0, 1in30year rainfall intensities should have a 40% increase applied in line with the latest guidance, whilst 1in100year rainfall intensities should have a 45% increase applied. The EA also advise that developments should be designed so that for the upper end allowance in the 1% annual exceedance probability event:

- there is no increase in flood risk elsewhere,
- the development will be safe from surface water flooding.

6.2.6 Urban Creep

Commercial developments typically have a high percentage of impermeability which limits the potential for future building or paved area extensions under permitted development rules. This is reflected in LASOO guidance to the non-statutory technical standards which note that urban creep allowances only apply to residential development. Specifically, at this site the roof and paved areas account for 91.9% of the drainage assessment site (refer to appended key plan [1]) and so an allowance for urban creep is not considered as appropriate or necessary.

6.2.7 Attenuation Sizing

The site drainage has been split into two networks to suit discharging to two outfall locations. The northern network (network A) is to drain to the 450mm dia surface water sewer between UU chambers 4200 and 3200 via a new chamber (with existing sewer between 4200 and new chamber abandoned due to new warehouse footprint). The southern network (network B) is to drain to the 750mm dia surface water sewer at UU chamber 4000.

The catchment for the northern network is to comprise the roof and paved areas shown on appended key plan [2] which total 1.489ha. The remaining roof and paved areas total 1.110ha. Therefore, on a pro-rata basis, the overall discharge rate of 14.5l/s will be split between northern and southern networks as 8.3l/s and 6.2l/s respectively.

Micro Drainage sheets 2-5 demonstrate that a tank (tank A) comprising three layers of cellular units (0.4m high layers = 1.2m overall height, based on Wavin Aquacell or Polypipe Polystorm units) with a plan area of 17m wide x 50m long (850sqm) is adequate as it will provide a capacity of 969m³ when a void ratio of 0.95 is allowed. The volume needed to attenuate the critical 12hour duration winter rainfall event of 1in100year return period with intensity increased by 40% for climate change is 953.3m³ with a peak discharge rate of 8.2l/s.

Micro Drainage sheets 6-9 demonstrate that a tank (tank B) comprising five layers of cellular units (0.4m high layers = 2.0m overall height, based on Wavin Aquacell or Polypipe Polystorm units) with a plan area of 9.5m wide x 40m long (380sqm) is adequate as it will provide a capacity of 722m³ when a void ratio of 0.95 is allowed. The volume needed to attenuate the critical 12hour duration winter rainfall event of 1in100year return period with intensity increased by 40% for climate change is 717.5m³ with a peak discharge rate of 6.2l/s.

6.2.8 Network Simulation

The tanks have been incorporated within two drainage networks as shown on appended key plan [4] so that they can be simulated to allow for time of flow and backwater effects, etc. The full drainage networks have then been modelled with the pipe areas shown on appended key plan [3]. The full network details are provided on MicroDrainage pages 10-15 For network A and 16-21 for network B. Tank A is as sized in section 6.2.7 above, but tank B has been slightly reduced to 9.5m x 39m x 2m.

It can be seen from the summary of simulation results for each network (MicroDrainage pages 22-23 and pages 24-25 for northern and southern networks respectively), that there is no flooding for 1in30 year rainfall intensities that have been increased by 40% to allow for the predicted effects of climate change.

When simulated for 1in100yr+45% rainfall events, there is limited flooding for a short duration (15minute) winter storm as highlighted on appended key plan [5]. The floodwater can be accommodated at ground level without causing a hazard and so can be deemed as acceptable.

The peak discharge rate for the northern network is 8.2l/s and for the southern network, the peak rate does not exceed 6.1l/s. These total 14.3l/s, compared to the design total limiting rate of 14.5l/s. The maximum water level in tank A is 65.641m and in tank B is 65.666m compared to tank top levels of 65.65m and 65.70m, demonstrating that the tank sizes are adequate.

6.2.9 Exceedance

In the event of system failure due either to blockage, collapse or a storm event exceeding the design rainfall intensity, runoff will be routed above ground. A flood flow plan is therefore required to assess the exceedance scenario and understand whether the flow route and areas of ponding will present a hazard or adverse consequence.

The flood flow analysis facility of MicroDrainage has been used to assess the effect of a deluge of 50mm depth rainfall across the proposed site terrain as shown on the appended key plan [6]. It can be seen that generally runoff would accumulate at low points in paved areas to car parking and service yard areas. As the prevailing fall of land is west to east, there is the potential for runoff to accumulate alongside the eastern elevations of the warehouse buildings too.

6.2.10 Construction, Operation & Maintenance

Paragraph 085 of the PPG-FRCC outlines several other key factors relating to the design of sustainable drainage systems: 'When planning a sustainable drainage system, developers need to ensure their design takes account of the construction, operation and maintenance requirements of both surface and subsurface components, allowing for any personnel, vehicle or machinery access required to undertake this work. Any sustainable drainage system should be designed so that the capacity takes account of the likely impacts of climate change and likely changes in impermeable area within the development over its lifetime and continues to provide effective drainage for properties. Whether maintenance and operation requirements are economically proportionate should be considered by reference to the costs that would be incurred by consumers for the use of an effective drainage system connecting directly to a public sewer'.

The appended drainage layout plans have been prepared in cognisance of construction, operation and maintenance aspects of both temporary and permanent conditions and this has influenced the location and depth of chambers, pipes, tank, etc relative to end use of the site (ie frequency and type of traffic and location of buildings relative to drainage assets. It is expected that this will be more rigorously audited as part of detailed drainage design and ongoing iterative changes to suit alterations to the development assuming the proposals are granted planning approval.

Aside from the final built form, it is important that the temporary conditions and/or timing of the pipe and storage facility installations are considered both by drainage designer and contractor to ensure that adequate cover or other protection is available to prevent damage by construction vehicle or material loadings. This should be addressed in line with standard CDM practice at construction stage and/or via risk assessment documentation.

Once installed the system must be accessible for inspection and maintenance in line with current good practice. A good benchmark is that for adoptable drainage systems which should comply with the new design and construction guidance DCG published by WaterUK (that has replaced Sewers for Adoption). DEFRA Standards S10 to S14 inclusive provide more guidance in relation to structural integrity, designing for maintenance considerations and construction of sustainable drainage systems.

To safeguard the long-term efficacy of the drainage system which would remain in private ownership and due to material selection and provision of offline capacity, etc would not qualify for adoption by United Utilities, the most likely solution would be to ensure inspection and maintenance via a private management company, which is likely to either be or administered by the site owner or operator.

7.0 Recommendations and Conclusions

7.1 Further Investigation & Consultation

It is recommended that existing drainage within the site and extending to at least the nearest upstream and downstream chambers are cctv surveyed to record the line, level, size, connectivity and condition. The need to abandon, accommodate or divert existing drainage to suit the development proposals can then be assessed.

Once the existing drainage has been surveyed to allow the final selection of suitable outfall points, United Utilities should be contacted via a developer enquiry to ascertain whether flows to existing drainage that will directly or indirectly drain to their sewers is acceptable at each connection point. Alternatively, a connection agreement under S106 of the Water Industry Act can be progressed but may result in either abortive work or delay if the assumed connection points and flow rates are not accepted.

7.2 Conclusions

This Flood Risk and Drainage Assessment has reviewed all readily available information to determine the sources of flooding that could affect the site. The proposed development area is at low risk of flooding from most sources except for overland flows resulting from exceedance of surface water or other sewerage systems.

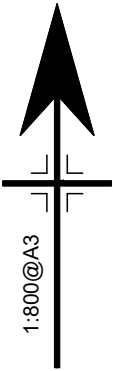
No special mitigation measures are deemed as necessary to be incorporated into the development proposals other than ensuring the building finished floor level is at least 300mm above the adjacent existing ground levels to the west of the development area to mitigate the risk of adverse effect from overland flooding that is predicted (RoFSW maps).

A drainage scheme is presented in the appendix showing how attenuation of runoff can be implemented to suit the constraints of the site and in line with the principles discussed in the drainage assessment allowing provisional finished floor levels of 67.20m to unit 1 and 67.70m to unit 2.

Appendix A – Key Plans & MicroDrainage Calculations

[1] POST DEV RUNOFF

PROPOSED WAREHOUSES
LAND OFF LAWTON STREET
NEW SMITHFIELD MARKET
MANCHESTER
M11 2WJ

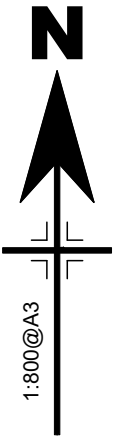


	ROOF	1.254HA
	PAVING	1.345HA
	TOTAL CONNECTED AREA	<u>2.599HA</u>
	NOTIONAL DEVELOPMENT EXTENTS	



[2] NETWORK CATCHMENTS

PROPOSED WAREHOUSES
LAND OFF LAWTON STREET
NEW SMITHFIELD MARKET
MANCHESTER
M11 2WJ

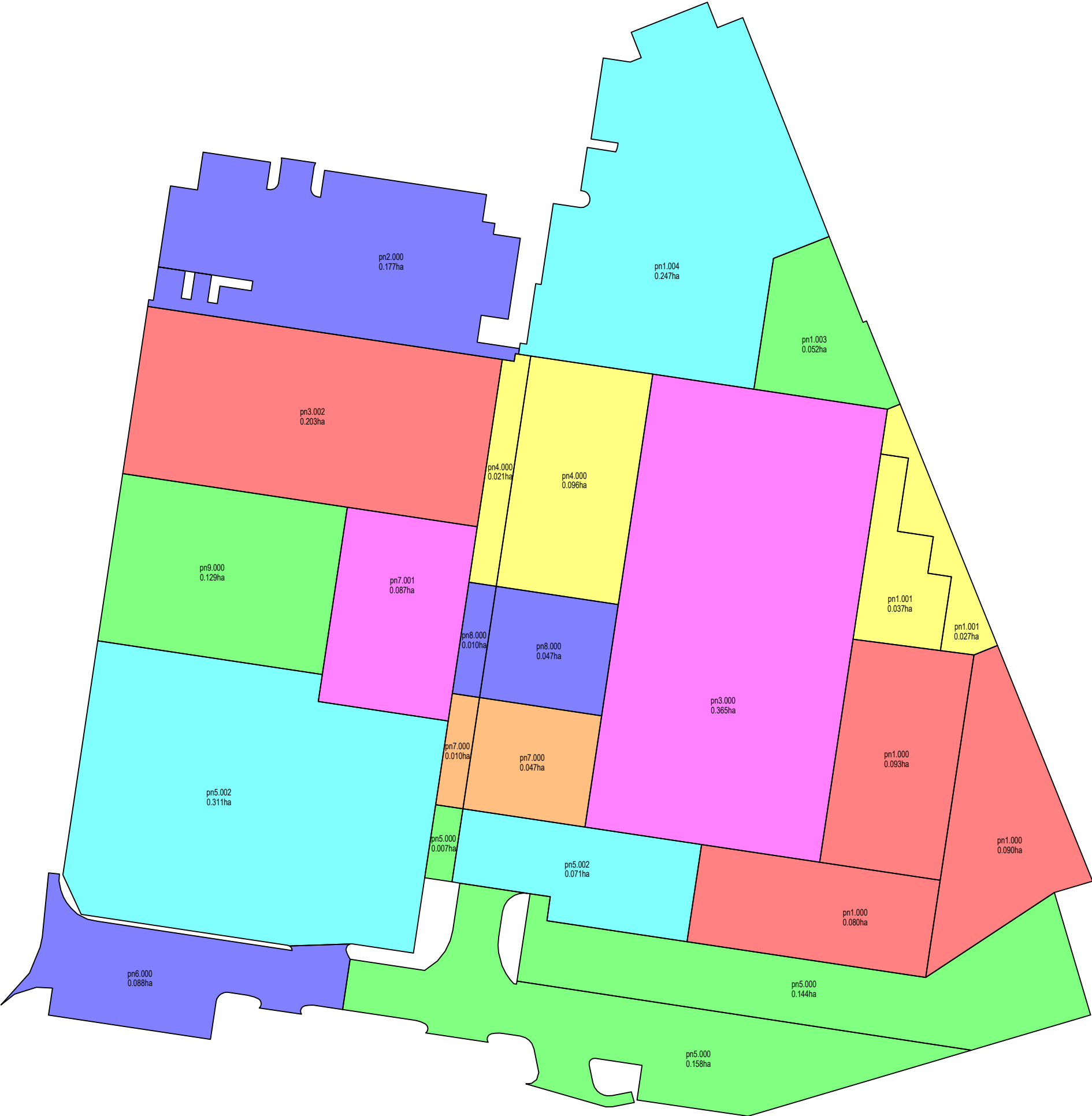
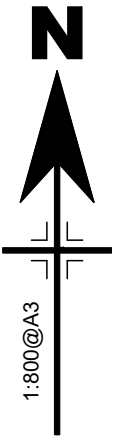


	NETWORK A	1.489HA
	NETWORK B	1.110HA
	TOTAL CONNECTED AREA	<u>2.599HA</u>
	NOTIONAL DEVELOPMENT EXTENTS	



[3] PIPE AREAS

PROPOSED WAREHOUSES
LAND OFF LAWTON STREET
NEW SMITHFIELD MARKET
MANCHESTER
M11 2WJ



[4] NETWORKS

PROPOSED WAREHOUSES
LAND OFF LAWTON STREET
NEW SMITHFIELD MARKET
MANCHESTER
M11 2WJ

ATTENUATION TANK A: 17M WIDE X 50M LONG (850SQM
PLAN AREA) X 1.2M HIGH, 95% VOID, CAPACITY = 969M3.

FLOW CONTROL: DESIGN FLOW 8.3L/S, DESIGN HEAD 1.25M
HYDROBRAKE OPTIMUM REFERENCE SHE-0130-8300-1250-8300

NETWORK A = PIPELINES 1-4

NETWORK B = PIPELINES 5-10

ATTENUATION TANK B: 9.5M WIDE X 39M LONG (370.5SQM
PLAN AREA) X 2.0M HIGH, 95% VOID, CAPACITY = 703.95M3.

FLOW CONTROL: DESIGN FLOW 6.2L/S, DESIGN HEAD 2.05M
HYDROBRAKE OPTIMUM REFERENCE SHE-0102-6200-2050-6200



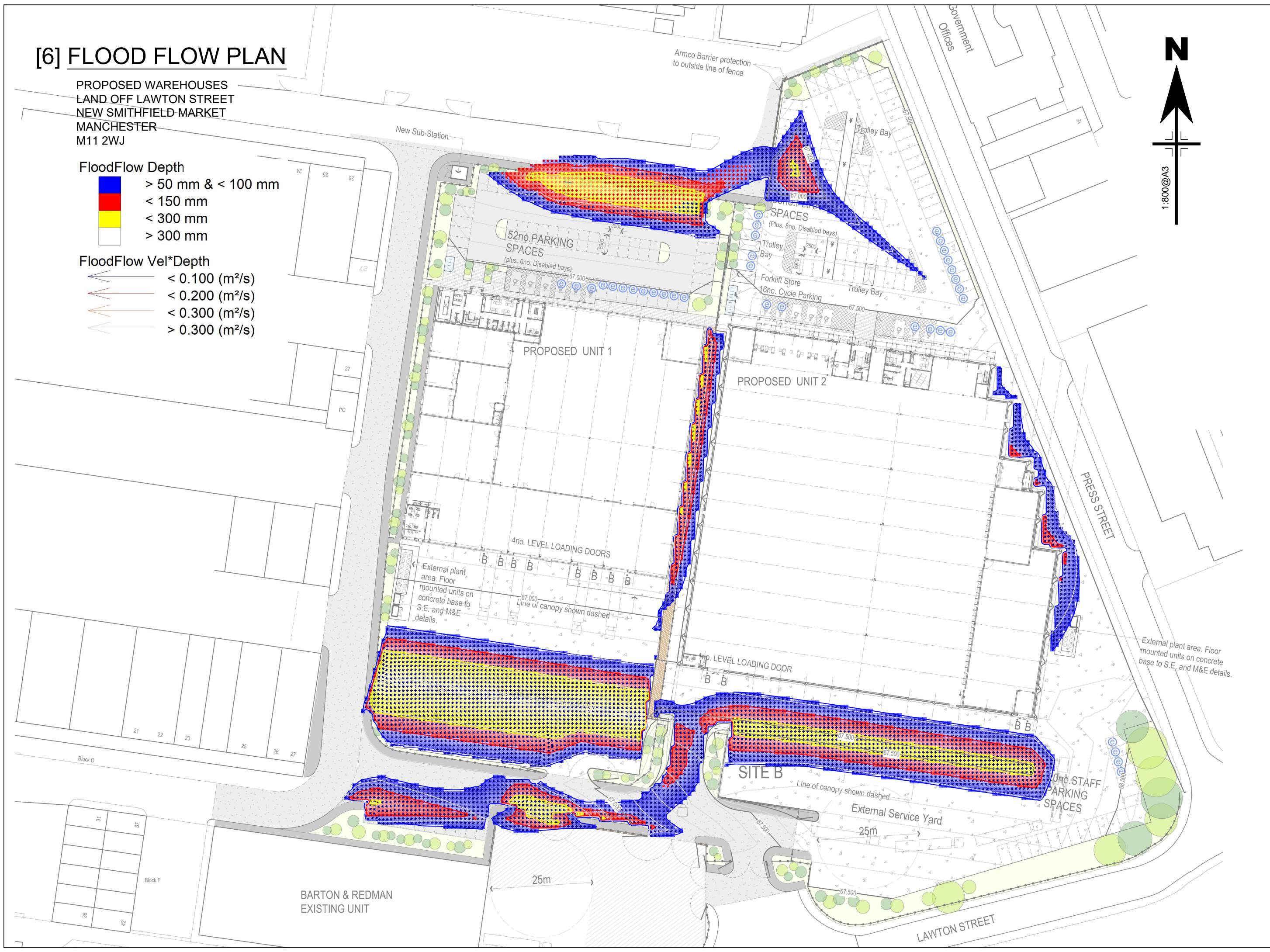
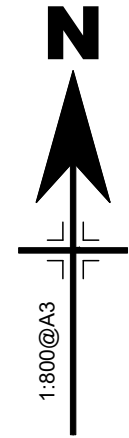
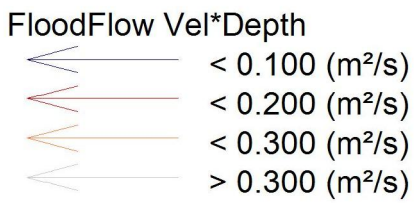
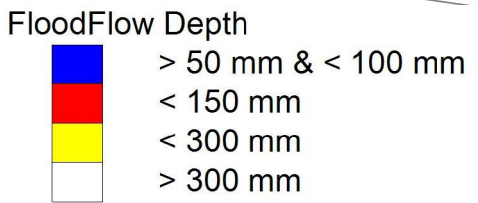
[5] 1IN100YR+45% FLOODING

PROPOSED WAREHOUSES
LAND OFF LAWTON STREET
NEW SMITHFIELD MARKET
MANCHESTER
M11 2WJ



[6] FLOOD FLOW PLAN

PROPOSED WAREHOUSES
LAND OFF LAWTON STREET
NEW SMITHFIELD MARKET
MANCHESTER
M11 2WJ



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Hillside Metal Bridge Durham, DH6 5NX	Proposed Warehouses New Smithfield Market Manchester	
Date 28/01/2022 File 146B31.SRCX	Designed by cje Checked by	
Innovyze	Source Control 2020.1.3	

ICP SUDS Mean Annual Flood

Input

Return Period (years) 1 SAAR (mm) 860 Urban 0.000
Area (ha) 1.000 Soil 0.450 Region Number Region 10

Results 1/s

QBAR Rural 5.6
QBAR Urban 5.6

Q1 year 4.9

Q1 year 4.9
Q30 years 9.5
Q100 years 11.6

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Date 09/08/2022 File TANK_A.SRCX	Designed by cje Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+40%)

Half Drain Time : 997 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	64.846	0.396	0.0	8.2	8.2	319.9	O K
30 min Summer	64.980	0.530	0.0	8.2	8.2	427.7	O K
60 min Summer	65.122	0.672	0.0	8.2	8.2	542.7	O K
120 min Summer	65.267	0.817	0.0	8.2	8.2	659.8	O K
180 min Summer	65.344	0.894	0.0	8.2	8.2	722.1	O K
240 min Summer	65.391	0.941	0.0	8.2	8.2	759.8	O K
360 min Summer	65.439	0.989	0.0	8.2	8.2	799.0	O K
480 min Summer	65.464	1.014	0.0	8.2	8.2	818.9	O K
600 min Summer	65.473	1.023	0.0	8.2	8.2	826.3	O K
720 min Summer	65.473	1.023	0.0	8.2	8.2	825.9	O K
960 min Summer	65.458	1.008	0.0	8.2	8.2	813.6	O K
1440 min Summer	65.421	0.971	0.0	8.2	8.2	784.0	O K
2160 min Summer	65.361	0.911	0.0	8.2	8.2	736.0	O K
2880 min Summer	65.301	0.851	0.0	8.2	8.2	687.0	O K
4320 min Summer	65.167	0.717	0.0	8.2	8.2	579.3	O K
5760 min Summer	65.036	0.586	0.0	8.2	8.2	473.5	O K
7200 min Summer	64.927	0.477	0.0	8.2	8.2	385.5	O K
8640 min Summer	64.837	0.387	0.0	8.2	8.2	312.2	O K
10080 min Summer	64.764	0.314	0.0	8.2	8.2	253.5	O K
15 min Winter	64.895	0.445	0.0	8.2	8.2	359.2	O K
30 min Winter	65.045	0.595	0.0	8.2	8.2	480.6	O K
60 min Winter	65.207	0.757	0.0	8.2	8.2	611.4	O K
120 min Winter	65.371	0.921	0.0	8.2	8.2	744.1	O K
180 min Winter	65.461	1.011	0.0	8.2	8.2	816.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	117.448	0.0	327.6	22
30 min Summer	79.010	0.0	440.9	37
60 min Summer	50.812	0.0	567.2	66
120 min Summer	31.621	0.0	706.0	126
180 min Summer	23.637	0.0	791.7	184
240 min Summer	19.105	0.0	853.2	244
360 min Summer	14.037	0.0	940.4	362
480 min Summer	11.286	0.0	1008.2	482
600 min Summer	9.522	0.0	1063.1	602
720 min Summer	8.282	0.0	1109.8	720
960 min Summer	6.640	0.0	1176.9	862
1440 min Summer	4.854	0.0	1209.1	1112
2160 min Summer	3.541	0.0	1423.7	1516
2880 min Summer	2.828	0.0	1515.7	1932
4320 min Summer	2.055	0.0	1652.6	2732
5760 min Summer	1.637	0.0	1754.7	3464
7200 min Summer	1.371	0.0	1837.4	4184
8640 min Summer	1.186	0.0	1906.7	4928
10080 min Summer	1.049	0.0	1968.3	5552
15 min Winter	117.448	0.0	367.0	22
30 min Winter	79.010	0.0	493.8	37
60 min Winter	50.812	0.0	635.3	66
120 min Winter	31.621	0.0	790.8	124
180 min Winter	23.637	0.0	886.7	182

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Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max E Outflow (l/s)	Max Volume (m³)	Status
240 min Winter	65.516	1.066	0.0	8.2	8.2	860.5	O K
360 min Winter	65.576	1.126	0.0	8.2	8.2	909.2	O K
480 min Winter	65.609	1.159	0.0	8.2	8.2	936.0	O K
600 min Winter	65.625	1.175	0.0	8.2	8.2	949.1	O K
720 min Winter	65.631	1.181	0.0	8.2	8.2	953.3	O K
960 min Winter	65.622	1.172	0.0	8.2	8.2	946.0	O K
1440 min Winter	65.572	1.122	0.0	8.2	8.2	905.8	O K
2160 min Winter	65.494	1.044	0.0	8.2	8.2	842.7	O K
2880 min Winter	65.407	0.957	0.0	8.2	8.2	772.6	O K
4320 min Winter	65.219	0.769	0.0	8.2	8.2	621.2	O K
5760 min Winter	65.006	0.556	0.0	8.2	8.2	449.3	O K
7200 min Winter	64.848	0.398	0.0	8.2	8.2	321.3	O K
8640 min Winter	64.730	0.280	0.0	8.2	8.2	226.4	O K
10080 min Winter	64.650	0.200	0.0	8.0	8.0	161.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	19.105	0.0	955.6	240
360 min Winter	14.037	0.0	1053.7	356
480 min Winter	11.286	0.0	1129.6	472
600 min Winter	9.522	0.0	1187.7	584
720 min Winter	8.282	0.0	1228.5	696
960 min Winter	6.640	0.0	1270.4	914
1440 min Winter	4.854	0.0	1236.3	1158
2160 min Winter	3.541	0.0	1594.9	1624
2880 min Winter	2.828	0.0	1697.5	2080
4320 min Winter	2.055	0.0	1851.1	3024
5760 min Winter	1.637	0.0	1965.6	3696
7200 min Winter	1.371	0.0	2058.5	4400
8640 min Winter	1.186	0.0	2135.8	5016
10080 min Winter	1.049	0.0	2204.9	5648

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	18.000	Shortest Storm (mins)	15
Ratio R	0.350	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.489

Time (mins) Area			Time (mins) Area		
From:	To:	(ha)	From:	To:	(ha)
0	4	0.920	4	8	0.570

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Model Details

Storage is Online Cover Level (m) 66.900

Cellular Storage Structure

Invert Level (m) 64.450 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	850.0	0.0	1.200	850.0	0.0	1.201	0.0	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference MD-SHE-0130-8300-1250-8300
Design Head (m) 1.250
Design Flow (l/s) 8.3
Flush-Flo™ Calculated
Objective Minimise upstream storage
Application Surface
Sump Available Yes
Diameter (mm) 130
Invert Level (m) 64.410
Minimum Outlet Pipe Diameter (mm) 150
Suggested Manhole Diameter (mm) 1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.250	8.3	Kick-Flo®	0.789	6.7
Flush-Flo™	0.368	8.2	Mean Flow over Head Range	-	7.2

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.7	0.800	6.7	2.000	10.3	4.000	14.4	7.000	18.8
0.200	7.8	1.000	7.5	2.200	10.8	4.500	15.2	7.500	19.4
0.300	8.2	1.200	8.1	2.400	11.3	5.000	16.0	8.000	20.0
0.400	8.2	1.400	8.7	2.600	11.7	5.500	16.7	8.500	20.6
0.500	8.1	1.600	9.3	3.000	12.5	6.000	17.5	9.000	21.2
0.600	7.9	1.800	9.8	3.500	13.5	6.500	18.1	9.500	21.8

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Summary of Results for 100 year Return Period (+40%)

Half Drain Time : 1142 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	64.362	0.662	0.0	5.3	5.3	238.9	O K
30 min Summer	64.587	0.887	0.0	5.3	5.3	320.1	O K
60 min Summer	64.826	1.126	0.0	5.3	5.3	406.5	O K
120 min Summer	65.066	1.366	0.0	5.3	5.3	493.2	O K
180 min Summer	65.196	1.496	0.0	5.4	5.4	539.9	O K
240 min Summer	65.274	1.574	0.0	5.5	5.5	568.4	O K
360 min Summer	65.358	1.658	0.0	5.7	5.7	598.4	O K
480 min Summer	65.400	1.700	0.0	5.7	5.7	613.9	O K
600 min Summer	65.417	1.717	0.0	5.8	5.8	620.0	O K
720 min Summer	65.418	1.718	0.0	5.8	5.8	620.3	O K
960 min Summer	65.396	1.696	0.0	5.7	5.7	612.3	O K
1440 min Summer	65.341	1.641	0.0	5.6	5.6	592.2	O K
2160 min Summer	65.252	1.552	0.0	5.5	5.5	560.3	O K
2880 min Summer	65.166	1.466	0.0	5.3	5.3	529.3	O K
4320 min Summer	65.003	1.303	0.0	5.3	5.3	470.3	O K
5760 min Summer	64.847	1.147	0.0	5.3	5.3	414.1	O K
7200 min Summer	64.694	0.994	0.0	5.3	5.3	358.9	O K
8640 min Summer	64.506	0.806	0.0	5.3	5.3	291.0	O K
10080 min Summer	64.346	0.646	0.0	5.3	5.3	233.1	O K
15 min Winter	64.443	0.743	0.0	5.3	5.3	268.3	O K
30 min Winter	64.696	0.996	0.0	5.3	5.3	359.5	O K
60 min Winter	64.966	1.266	0.0	5.3	5.3	456.9	O K
120 min Winter	65.240	1.540	0.0	5.5	5.5	556.0	O K
180 min Winter	65.390	1.690	0.0	5.7	5.7	610.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	117.448	0.0	244.2	22
30 min Summer	79.010	0.0	328.5	37
60 min Summer	50.812	0.0	422.8	66
120 min Summer	31.621	0.0	526.3	126
180 min Summer	23.637	0.0	590.1	184
240 min Summer	19.105	0.0	635.9	244
360 min Summer	14.037	0.0	700.8	362
480 min Summer	11.286	0.0	751.4	482
600 min Summer	9.522	0.0	792.1	602
720 min Summer	8.282	0.0	821.6	720
960 min Summer	6.640	0.0	848.9	860
1440 min Summer	4.854	0.0	827.6	1112
2160 min Summer	3.541	0.0	1060.9	1512
2880 min Summer	2.828	0.0	1129.4	1932
4320 min Summer	2.055	0.0	1231.4	2768
5760 min Summer	1.637	0.0	1307.6	3576
7200 min Summer	1.371	0.0	1368.9	4400
8640 min Summer	1.186	0.0	1420.4	5184
10080 min Summer	1.049	0.0	1466.6	5752
15 min Winter	117.448	0.0	273.5	22
30 min Winter	79.010	0.0	367.1	37
60 min Winter	50.812	0.0	473.6	66
120 min Winter	31.621	0.0	589.4	124
180 min Winter	23.637	0.0	660.9	182

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Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max E Outflow (l/s)	Max Volume (m³)	Status
240 min Winter	65.484	1.784	0.0	5.9	5.9	644.0	O K
360 min Winter	65.588	1.888	0.0	6.0	6.0	681.4	O K
480 min Winter	65.646	1.946	0.0	6.1	6.1	702.5	O K
600 min Winter	65.676	1.976	0.0	6.1	6.1	713.3	O K
720 min Winter	65.687	1.987	0.0	6.2	6.2	717.5	O K
960 min Winter	65.677	1.977	0.0	6.1	6.1	713.8	O K
1440 min Winter	65.604	1.904	0.0	6.0	6.0	687.4	O K
2160 min Winter	65.489	1.789	0.0	5.9	5.9	645.9	O K
2880 min Winter	65.365	1.665	0.0	5.7	5.7	601.0	O K
4320 min Winter	65.120	1.420	0.0	5.3	5.3	512.5	O K
5760 min Winter	64.885	1.185	0.0	5.3	5.3	427.8	O K
7200 min Winter	64.641	0.941	0.0	5.3	5.3	339.8	O K
8640 min Winter	64.337	0.637	0.0	5.3	5.3	230.1	O K
10080 min Winter	64.149	0.449	0.0	5.3	5.3	162.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	19.105	0.0	712.2	240
360 min Winter	14.037	0.0	785.0	356
480 min Winter	11.286	0.0	836.9	472
600 min Winter	9.522	0.0	868.4	584
720 min Winter	8.282	0.0	877.9	696
960 min Winter	6.640	0.0	875.5	912
1440 min Winter	4.854	0.0	854.7	1154
2160 min Winter	3.541	0.0	1188.2	1620
2880 min Winter	2.828	0.0	1264.9	2080
4320 min Winter	2.055	0.0	1379.2	2984
5760 min Winter	1.637	0.0	1464.6	3864
7200 min Winter	1.371	0.0	1533.3	4760
8640 min Winter	1.186	0.0	1591.1	5352
10080 min Winter	1.049	0.0	1642.8	5944

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Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	18.000	Shortest Storm (mins)	15
Ratio R	0.350	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.110

Time (mins)	Area	Time (mins)	Area
From: To: (ha)		From: To: (ha)	
0 4 0.775		4 8 0.334	

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Model Details

Storage is Online Cover Level (m) 66.900

Cellular Storage Structure

Invert Level (m) 63.700 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	380.0	0.0	2.000	380.0	0.0	2.001	0.0	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference MD-SHE-0102-6200-2050-6200
Design Head (m) 2.050
Design Flow (l/s) 6.2
Flush-Flo™ Calculated
Objective Minimise upstream storage
Application Surface
Sump Available Yes
Diameter (mm) 102
Invert Level (m) 63.670
Minimum Outlet Pipe Diameter (mm) 150
Suggested Manhole Diameter (mm) 1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.050	6.2	Kick-Flo®	0.906	4.2
Flush-Flo™	0.441	5.3	Mean Flow over Head Range	-	5.0

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.4	0.800	4.8	2.000	6.1	4.000	8.5	7.000	11.1
0.200	4.8	1.000	4.4	2.200	6.4	4.500	9.0	7.500	11.4
0.300	5.2	1.200	4.8	2.400	6.7	5.000	9.4	8.000	11.8
0.400	5.3	1.400	5.2	2.600	6.9	5.500	9.9	8.500	12.1
0.500	5.3	1.600	5.5	3.000	7.4	6.000	10.3	9.000	12.5
0.600	5.2	1.800	5.8	3.500	8.0	6.500	10.7	9.500	12.8

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STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Surface Network 1

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales

Return Period (years)	1	PIMP (%)	100
M5-60 (mm)	18.000	Add Flow / Climate Change (%)	0
Ratio R	0.350	Minimum Backdrop Height (m)	0.200
Maximum Rainfall (mm/hr)	50	Maximum Backdrop Height (m)	1.500
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	1.200
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	500

Designed with Level Soffits

Time Area Diagram for Surface Network 1

Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	0.920	4-8	0.570

Total Area Contributing (ha) = 1.489

Total Pipe Volume (m³) = 34.827

Network Design Table for Surface Network 1

- Indicates pipe length does not match coordinates

« - Indicates pipe capacity < flow

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
1.000	56.999	0.315	181.0	0.263	4.00	0.0	0.600	o	300	Pipe/Conduit	
1.001	52.987	0.260	203.8	0.065	0.00	0.0	0.600	o	375	Pipe/Conduit	
1.002	26.231	0.130	201.8	0.000	0.00	0.0	0.600	o	375	Pipe/Conduit	
1.003	31.542	0.155	203.5	0.052	0.00	0.0	0.600	o	375	Pipe/Conduit	
1.004	8.649	0.043	201.1	0.247	0.00	0.0	0.600	o	450	Pipe/Conduit	
2.000	45.000	0.203	221.7	0.177	4.00	0.0	0.600	o	300	Pipe/Conduit	
1.005	2.660#	0.007	380.0	0.000	0.00	0.0	0.600	o	450	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
1.000	46.86	4.82	66.000	0.263	0.0	0.0	0.0	1.17	82.4	33.3
1.001	44.27	5.51	65.610	0.327	0.0	0.0	0.0	1.27	139.8	39.2
1.002	43.11	5.86	65.350	0.327	0.0	0.0	0.0	1.27	140.5	39.2
1.003	41.81	6.27	65.220	0.379	0.0	0.0	0.0	1.27	139.9	43.0
1.004	41.50	6.37	64.990	0.627	0.0	0.0	0.0	1.43	227.4	70.5
2.000	47.27	4.71	65.300	0.177	0.0	0.0	0.0	1.05	74.4	22.6
1.005	41.37	6.42	64.947	0.804	0.0	0.0	0.0	1.04	164.9	90.1

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Network Design Table for Surface Network 1

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
1.006	5.243	0.040	131.1	0.000	0.00	0.0	0.600	o	450	Pipe/Conduit	
1.007	9.110#	0.050	182.2	0.000	0.00	0.0	0.600	ooo	300	Triple Pipe	
3.000	45.764	0.380	120.4	0.365	4.00	0.0	0.600	o	375	Pipe/Conduit	
4.000	38.652	0.390	99.1	0.118	4.00	0.0	0.600	o	225	Pipe/Conduit	
3.001	6.211	0.320	19.4	0.000	0.00	0.0	0.600	o	375	Pipe/Conduit	
3.002	9.110#	0.050	182.2	0.203	0.00	0.0	0.600	o	375	Pipe/Conduit	
1.008	8.608	0.040	215.2	0.000	0.00	0.0	0.600	o	300	Pipe/Conduit	
1.009	27.275	0.110	248.0	0.000	0.00	0.0	0.600	o	300	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
1.006	41.23	6.46	64.840	0.804	0.0	0.0	0.0	1.77	282.2	90.1
1.007	40.85	6.60	64.800	0.804	0.0	0.0	0.0	1.16	246.3	90.1
3.000	48.32	4.46	65.500	0.365	0.0	0.0	0.0	1.65	182.2	47.7
4.000	48.20	4.49	65.660	0.118	0.0	0.0	0.0	1.31	52.2	15.4
3.001	48.09	4.52	65.120	0.482	0.0	0.0	0.0	4.13	456.1	62.8
3.002	47.62	4.63	64.800	0.686	0.0	0.0	0.0	1.34	147.9	88.4
1.008	40.47	6.73	64.450	1.489	0.0	0.0	0.0	1.07	75.5«	163.2
1.009	39.22	7.19	64.410	1.489	0.0	0.0	0.0	0.99	70.3«	163.2

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PIPELINE SCHEDULES for Surface Network 1

Upstream Manhole

- Indicates pipe length does not match coordinates

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
1.000	o	300	S21	67.500	66.000	1.200	Open Manhole	1200
1.001	o	375	S20	67.600	65.610	1.615	Open Manhole	1350
1.002	o	375	S19	67.600	65.350	1.875	Open Manhole	1350
1.003	o	375	S18	67.290	65.220	1.695	Open Manhole	1350
1.004	o	450	S2 CP	67.110	64.990	1.670	Open Manhole	1350
2.000	o	300	S3A	66.625	65.300	1.025	Open Manhole	1350
1.005	o	450	S3 CP	67.200	64.947	1.803	Open Manhole	1350
1.006	o	450	PI_A	67.437	64.840	2.147	Open Manhole	1350
1.007	ooo	300	S4 CP	67.030	64.800	1.930	Open Manhole	2100
3.000	o	375	S1A	67.560	65.500	1.685	Open Manhole	1350
4.000	o	225	RE1	67.090	65.660	1.205	Open Manhole	375
3.001	o	375	S1	67.200	65.120	1.705	Open Manhole	1350
3.002	o	375	S5 CP	67.100	64.800	1.925	Open Manhole	2100
1.008	o	300	TANK_A	66.881	64.450	2.131	Open Manhole	100
1.009	o	300	S6 FC	67.160	64.410	2.450	Open Manhole	1800

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
1.000	56.999	181.0	S20	67.600	65.685	1.615	Open Manhole	1350
1.001	52.987	203.8	S19	67.600	65.350	1.875	Open Manhole	1350
1.002	26.231	201.8	S18	67.290	65.220	1.695	Open Manhole	1350
1.003	31.542	203.5	S2 CP	67.110	65.065	1.670	Open Manhole	1350
1.004	8.649	201.1	S3 CP	67.200	64.947	1.803	Open Manhole	1350
2.000	45.000	221.7	S3 CP	67.200	65.097	1.803	Open Manhole	1350
1.005	2.660#	380.0	PI_A	67.437	64.940	2.047	Open Manhole	1350
1.006	5.243	131.1	S4 CP	67.030	64.800	1.780	Open Manhole	2100
1.007	9.110#	182.2	TANK_A	66.881	64.750	1.831	Open Manhole	100
3.000	45.764	120.4	S1	67.200	65.120	1.705	Open Manhole	1350
4.000	38.652	99.1	S1	67.200	65.270	1.705	Open Manhole	1350
3.001	6.211	19.4	S5 CP	67.100	64.800	1.925	Open Manhole	2100
3.002	9.110#	182.2	TANK_A	66.881	64.750	1.756	Open Manhole	100
1.008	8.608	215.2	S6 FC	67.160	64.410	2.450	Open Manhole	1800
1.009	27.275	248.0	S7	66.760	64.300	2.160	Open Manhole	1350

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Area Summary for Surface Network 1

Pipe Number	PIMP Type	PIMP Name	PIMP (%)	Gross Area (ha)	Imp. Area (ha)	Pipe Total (ha)
1.000	User	-	100	0.080	0.080	0.080
	User	-	100	0.090	0.090	0.170
	User	-	100	0.093	0.093	0.263
1.001	User	-	100	0.027	0.027	0.027
	User	-	100	0.037	0.037	0.065
1.002	-	-	100	0.000	0.000	0.000
1.003	User	-	100	0.052	0.052	0.052
1.004	User	-	100	0.247	0.247	0.247
2.000	User	-	100	0.177	0.177	0.177
1.005	-	-	100	0.000	0.000	0.000
1.006	-	-	100	0.000	0.000	0.000
1.007	-	-	100	0.000	0.000	0.000
3.000	User	-	100	0.365	0.365	0.365
4.000	User	-	100	0.096	0.096	0.096
	User	-	100	0.021	0.021	0.118
3.001	-	-	100	0.000	0.000	0.000
3.002	User	-	100	0.203	0.203	0.203
1.008	-	-	100	0.000	0.000	0.000
1.009	-	-	100	0.000	0.000	0.000
	Total			Total	Total	Total
				1.489	1.489	1.489

Free Flowing Outfall Details for Surface Network 1

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
1.009	S7	66.760	64.300	65.140	1350	0

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Online Controls for Surface Network 1

Hydro-Brake® Optimum Manhole: S6 FC, DS/PN: 1.009, Volume (m³): 7.5

Unit Reference	MD-SHE-0130-8300-1250-8300
Design Head (m)	1.250
Design Flow (l/s)	8.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	130
Invert Level (m)	64.410
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.250	8.3	Kick-Flo®	0.789	6.7
Flush-Flo™	0.368	8.2	Mean Flow over Head Range	-	7.2

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	4.7	0.800	6.7	2.000	10.3	4.000	14.4	7.000	18.8
0.200	7.8	1.000	7.5	2.200	10.8	4.500	15.2	7.500	19.4
0.300	8.2	1.200	8.1	2.400	11.3	5.000	16.0	8.000	20.0
0.400	8.2	1.400	8.7	2.600	11.7	5.500	16.7	8.500	20.6
0.500	8.1	1.600	9.3	3.000	12.5	6.000	17.5	9.000	21.2
0.600	7.9	1.800	9.8	3.500	13.5	6.500	18.1	9.500	21.8

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Storage Structures for Surface Network 1

Cellular Storage Manhole: TANK_A, DS/PN: 1.008

Invert Level (m) 64.450 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	850.0	0.0	1.200	850.0	0.0	1.201	0.0	0.0

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STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Surface Network 2

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales			
Return Period (years)	1	PIMP (%)	100
M5-60 (mm)	18.000	Add Flow / Climate Change (%)	0
Ratio R	0.350	Minimum Backdrop Height (m)	0.200
Maximum Rainfall (mm/hr)	50	Maximum Backdrop Height (m)	1.500
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	1.200
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	500

Designed with Level Soffits

Time Area Diagram for Surface Network 2

Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	0.775	4-8	0.334

Total Area Contributing (ha) = 1.110

Total Pipe Volume (m³) = 39.180

Network Design Table for Surface Network 2

- Indicates pipe length does not match coordinates

« - Indicates pipe capacity < flow

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
5.000	53.198	0.475	112.0	0.309	4.00	0.0	0.600	o	450	Pipe/Conduit	
6.000	52.655	0.325	162.0	0.088	4.00	0.0	0.600	o	300	Pipe/Conduit	
5.001	20.075	0.140	143.4	0.000	0.00	0.0	0.600	o	525	Pipe/Conduit	
5.002	45.670	0.150	304.5	0.383	0.00	0.0	0.600	o	600	Pipe/Conduit	
5.003	3.407#	0.020	170.4	0.000	0.00	0.0	0.600	o	450	Pipe/Conduit	
5.004	3.407	0.020	170.4	0.000	0.00	0.0	0.600	o	450	Pipe/Conduit	
5.005	2.838#	0.020	141.9	0.000	0.00	0.0	0.600	ooo	300	Triple Pipe	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
5.000	48.32	4.46	65.300	0.309	0.0	0.0	0.0	1.92	305.4	40.4
6.000	47.28	4.71	65.300	0.088	0.0	0.0	0.0	1.23	87.1	11.3
5.001	46.56	4.89	64.750	0.397	0.0	0.0	0.0	1.87	404.5	50.1
5.002	44.53	5.44	64.610	0.780	0.0	0.0	0.0	1.39	393.1	94.1
5.003	44.40	5.48	64.460	0.780	0.0	0.0	0.0	1.55	247.3	94.1
5.004	44.28	5.51	64.340	0.780	0.0	0.0	0.0	1.55	247.3	94.1
5.005	44.15	5.55	64.320	0.780	0.0	0.0	0.0	1.32	279.5	94.1

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Network Design Table for Surface Network 2

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
7.000	16.942	0.150	113.3	0.057	4.00	0.0	0.600	o	225	Pipe/Conduit	
8.000	15.674	0.140	112.3	0.057	4.00	0.0	0.600	o	225	Pipe/Conduit	
7.001	26.767	0.200	133.5	0.087	0.00	0.0	0.600	o	225	Pipe/Conduit	
9.000	40.770	0.275	148.3	0.129	4.00	0.0	0.600	o	225	Pipe/Conduit	
7.002	35.140#	1.360	25.8	0.000	0.00	0.0	0.600	o	300	Pipe/Conduit	
10.000	2.838#	0.020	141.9	0.000	4.00	0.0	0.600	o	300	Pipe/Conduit	
5.006	1.541	0.030	51.4	0.000	0.00	0.0	0.600	o	300	Pipe/Conduit	
5.007	18.076	0.100	180.8	0.000	0.00	0.0	0.600	o	300	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
7.000	49.33	4.23	65.810	0.057	0.0	0.0	0.0	1.23	48.8	7.6
8.000	49.41	4.21	65.800	0.057	0.0	0.0	0.0	1.23	49.0	7.6
7.001	47.63	4.62	65.660	0.201	0.0	0.0	0.0	1.13	44.9	25.9
9.000	47.60	4.63	65.735	0.129	0.0	0.0	0.0	1.07	42.6	16.6
7.002	46.83	4.82	65.385	0.329	0.0	0.0	0.0	3.11	219.5	41.8
10.000	50.00	4.04	65.440	0.000	0.0	0.0	0.0	1.32	93.2	0.0
5.006	44.11	5.56	63.700	1.110	0.0	0.0	0.0	2.20	155.4	132.5
5.007	43.24	5.82	63.670	1.110	0.0	0.0	0.0	1.17	82.4«	132.5

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PIPELINE SCHEDULES for Surface Network 2

Upstream Manhole

- Indicates pipe length does not match coordinates

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
5.000	o	450	S8	67.660	65.300	1.910	Open Manhole	1350
6.000	o	300	S9 CP	66.750	65.300	1.150	Open Manhole	1350
5.001	o	525	S10	66.825	64.750	1.550	Open Manhole	1500
5.002	o	600	S11	66.630	64.610	1.420	Open Manhole	1500
5.003	o	450	S12	66.630	64.460	1.720	Open Manhole	1500
5.004	o	450	PI_B	66.749	64.340	1.959	Open Manhole	1350
5.005	ooo	300	S14 CP	66.760	64.320	2.140	Open Manhole	2100
7.000	o	225	RE3	67.080	65.810	1.045	Open Manhole	225
8.000	o	225	RE2	67.090	65.800	1.065	Open Manhole	225
7.001	o	225	S17	67.085	65.660	1.200	Open Manhole	1350
9.000	o	225	RE4	67.150	65.735	1.190	Open Manhole	225
7.002	o	300	S16	67.110	65.385	1.425	Open Manhole	1200
10.000	o	300	S13 CP	66.680	65.440	0.940	Open Manhole	2100
5.006	o	300	TANK_B	66.777	63.700	2.777	Open Manhole	100
5.007	o	300	S15 FC	66.800	63.670	2.830	Open Manhole	1800

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
5.000	53.198	112.0	S10	66.825	64.825	1.550	Open Manhole	1500
6.000	52.655	162.0	S10	66.825	64.975	1.550	Open Manhole	1500
5.001	20.075	143.4	S11	66.630	64.610	1.495	Open Manhole	1500
5.002	45.670	304.5	S12	66.630	64.460	1.570	Open Manhole	1500
5.003	3.407#	170.4	PI_B	66.749	64.440	1.859	Open Manhole	1350
5.004	3.407	170.4	S14 CP	66.760	64.320	1.990	Open Manhole	2100
5.005	2.838#	141.9	TANK_B	66.777	64.300	2.177	Open Manhole	100
7.000	16.942	113.3	S17	67.085	65.660	1.200	Open Manhole	1350
8.000	15.674	112.3	S17	67.085	65.660	1.200	Open Manhole	1350
7.001	26.767	133.5	S16	67.110	65.460	1.425	Open Manhole	1200
9.000	40.770	148.3	S16	67.110	65.460	1.425	Open Manhole	1200
7.002	35.140#	25.8	TANK_B	66.777	64.025	2.452	Open Manhole	100
10.000	2.838#	141.9	TANK_B	66.777	65.420	1.057	Open Manhole	100
5.006	1.541	51.4	S15 FC	66.800	63.670	2.830	Open Manhole	1800
5.007	18.076	180.8	UU 4000	66.740	63.570	2.870	Open Manhole	1650

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Area Summary for Surface Network 2

Pipe Number	PIMP Type	PIMP Name	PIMP (%)	Gross Area (ha)	Imp. Area (ha)	Pipe Total (ha)
5.000	User	-	100	0.158	0.158	0.158
	User	-	100	0.144	0.144	0.302
	User	-	100	0.007	0.007	0.309
6.000	User	-	100	0.088	0.088	0.088
5.001	-	-	100	0.000	0.000	0.000
5.002	User	-	100	0.311	0.311	0.311
	User	-	100	0.071	0.071	0.383
5.003	-	-	100	0.000	0.000	0.000
5.004	-	-	100	0.000	0.000	0.000
5.005	-	-	100	0.000	0.000	0.000
7.000	User	-	100	0.010	0.010	0.010
	User	-	100	0.047	0.047	0.057
8.000	User	-	100	0.010	0.010	0.010
	User	-	100	0.047	0.047	0.057
7.001	User	-	100	0.087	0.087	0.087
9.000	User	-	100	0.129	0.129	0.129
7.002	-	-	100	0.000	0.000	0.000
10.000	-	-	100	0.000	0.000	0.000
5.006	-	-	100	0.000	0.000	0.000
5.007	-	-	100	0.000	0.000	0.000
				Total	Total	Total
				1.110	1.110	1.110

Free Flowing Outfall Details for Surface Network 2

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
5.007	UU 4000	66.740	63.570	0.000	1650	0

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Online Controls for Surface Network 2

Hydro-Brake® Optimum Manhole: S15 FC, DS/PN: 5.007, Volume (m³): 8.0

Unit Reference	MD-SHE-0102-6200-2050-6200
Design Head (m)	2.050
Design Flow (l/s)	6.2
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	102
Invert Level (m)	63.670
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points			Head (m)	Flow (l/s)	Control Points			Head (m)	Flow (l/s)
Design Point (Calculated)			2.050	6.2	Kick-Flo®			0.906	4.2
Flush-Flo™			0.441	5.3	Mean Flow over Head Range			-	5.0

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.4	0.800	4.8	2.000	6.1	4.000	8.5	7.000	11.1
0.200	4.8	1.000	4.4	2.200	6.4	4.500	9.0	7.500	11.4
0.300	5.2	1.200	4.8	2.400	6.7	5.000	9.4	8.000	11.8
0.400	5.3	1.400	5.2	2.600	6.9	5.500	9.9	8.500	12.1
0.500	5.3	1.600	5.5	3.000	7.4	6.000	10.3	9.000	12.5
0.600	5.2	1.800	5.8	3.500	8.0	6.500	10.7	9.500	12.8

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Storage Structures for Surface Network 2

Cellular Storage Manhole: TANK_B, DS/PN: 5.006

Invert Level (m) 63.700 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	370.5	0.0	2.000	370.5	0.0	2.001	0.0	0.0

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

Simulation Criteria

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

Number of Input Hydrographs 0 Number of Offline Controls 0 Number of Time/Area Diagrams 0
Number of Online Controls 1 Number of Storage Structures 1 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR M5-60 (mm) 18.000 Cv (Summer) 0.750
Region England and Wales Ratio R 0.350 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0
Analysis Timestep 2.5 Second Increment (Extended)
DTS Status ON
DVD Status OFF
Inertia Status OFF

Profile(s) Summer and Winter
Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
Return Period(s) (years) 30, 100
Climate Change (%) 40, 45

PN	US/MH Name	Duration (mins)	First (X) Surcharge	Water Surcharged			Flow / Cap.	Half Drain Pipe		Status
				Level (m)	Depth (m)	Volume (m³)		Time (mins)	Flow (l/s)	
1.000	S21	15	30/15 Summer	66.686	0.386	0.000	1.37		107.0	SURCHARGED
1.001	S20	15	30/15 Summer	66.210	0.225	0.000	0.86		111.5	SURCHARGED
1.002	S19	15	30/15 Summer	66.018	0.293	0.000	0.96		117.5	SURCHARGED
1.003	S18	15	30/15 Summer	65.923	0.328	0.000	1.03		127.7	SURCHARGED
1.004	S2 CP	15	30/15 Summer	65.786	0.346	0.000	1.23		187.7	SURCHARGED
2.000	S3A	15	30/15 Summer	65.870	0.270	0.000	1.02		71.3	SURCHARGED
1.005	S3 CP	15	30/15 Summer	65.660	0.263	0.000	2.05		255.6	SURCHARGED
1.006	PI_A	15	30/15 Summer	65.441	0.151	0.000	1.58		255.8	SURCHARGED
1.007	S4 CP	960	30/15 Summer	65.299	0.199	0.000	0.13		24.5	SURCHARGED
3.000	S1A	15	30/15 Summer	66.165	0.290	0.000	0.91		151.6	SURCHARGED
4.000	RE1	15	30/15 Summer	66.247	0.362	0.000	1.00		49.6	SURCHARGED
3.001	S1	15	30/15 Summer	65.855	0.360	0.000	0.88		195.3	SURCHARGED
3.002	S5 CP	15	30/15 Summer	65.598	0.423	0.000	2.56		272.4	SURCHARGED
1.008	TANK_A	960	30/15 Summer	65.298	0.548	0.000	0.40	838	22.7	SURCHARGED
1.009	S6 FC	960	30/15 Summer	65.302	0.592	0.000	0.13		8.2	SURCHARGED

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

Simulation Criteria

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

Number of Input Hydrographs 0 Number of Offline Controls 0 Number of Time/Area Diagrams 0
 Number of Online Controls 1 Number of Storage Structures 1 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR M5-60 (mm) 18.000 Cv (Summer) 0.750
 Region England and Wales Ratio R 0.350 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0
 Analysis Timestep 2.5 Second Increment (Extended)
 DTS Status ON
 DVD Status OFF
 Inertia Status OFF

Profile(s) Summer and Winter
 Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
 Return Period(s) (years) 30, 100
 Climate Change (%) 40, 45

PN	US/MH Name	Duration (mins)	First (X) Surcharge	Water Surcharged			Flow / Cap.	Half Drain Pipe		Status
				Level (m)	Depth (m)	Volume (m³)		Time (mins)	Flow (l/s)	
1.000	S21	15	30/15 Summer	67.503	1.203	3.138	1.65		129.0	FLOOD
1.001	S20	15	30/15 Summer	67.022	1.037	0.000	1.11		144.0	SURCHARGED
1.002	S19	15	30/15 Summer	66.729	1.004	0.000	1.16		142.1	SURCHARGED
1.003	S18	15	30/15 Summer	66.558	0.963	0.000	1.31		162.7	SURCHARGED
1.004	S2 CP	15	30/15 Summer	66.296	0.856	0.000	1.70		258.5	SURCHARGED
2.000	S3A	15	30/15 Summer	66.361	0.761	0.000	1.34		93.0	FLOOD RISK
1.005	S3 CP	15	30/15 Summer	66.056	0.659	0.000	2.78		346.7	SURCHARGED
1.006	PI_A	15	30/15 Summer	65.650	0.360	0.000	2.16		349.8	SURCHARGED
1.007	S4 CP	960	30/15 Summer	65.642	0.542	0.000	0.18		32.5	SURCHARGED
3.000	S1A	15	30/15 Summer	67.078	1.203	0.000	1.18		197.2	SURCHARGED
4.000	RE1	15	30/15 Summer	67.090	1.205	0.200	1.29		64.0	FLOOD
3.001	S1	15	30/15 Summer	66.493	0.998	0.000	1.19		263.4	SURCHARGED
3.002	S5 CP	15	30/15 Summer	66.036	0.861	0.000	3.48		370.9	SURCHARGED
1.008	TANK_A	960	30/15 Summer	65.641	0.891	0.000	0.18	1192	10.5	SURCHARGED
1.009	S6 FC	960	30/15 Summer	65.644	0.934	0.000	0.13		8.2	SURCHARGED

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30 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Surface Network 2

Simulation Criteria

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

Number of Input Hydrographs 0 Number of Offline Controls 0 Number of Time/Area Diagrams 0
 Number of Online Controls 1 Number of Storage Structures 1 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR M5-60 (mm) 18.000 Cv (Summer) 0.750
 Region England and Wales Ratio R 0.350 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0
 Analysis Timestep 2.5 Second Increment (Extended)
 DTS Status ON
 DVD Status OFF
 Inertia Status OFF

Profile(s) Summer and Winter
 Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
 Return Period(s) (years) 30, 100
 Climate Change (%) 40, 45

PN	US/MH Name	Duration (mins)	First (X) Surcharge	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Half Drain Time (mins)	Pipe Flow (l/s)	Status
5.000	S8	15	100/15 Summer	65.521	-0.229	0.000	0.48		134.2	OK
6.000	S9 CP	15	100/15 Summer	65.445	-0.155	0.000	0.46		37.6	OK
5.001	S10	15	30/15 Summer	65.367	0.092	0.000	0.54		154.1	SURCHARGED
5.002	S11	15	30/15 Summer	65.330	0.120	0.000	0.80		272.7	SURCHARGED
5.003	S12	15	30/15 Summer	65.228	0.318	0.000	2.25		273.0	SURCHARGED
5.004	PI_B	480	30/15 Summer	65.103	0.313	0.000	0.33		40.2	SURCHARGED
5.005	S14 CP	480	30/15 Summer	65.102	0.482	0.000	0.26		39.6	SURCHARGED
7.000	RE3	15	30/15 Summer	66.588	0.553	0.000	0.57		24.6	SURCHARGED
8.000	RE2	15	30/15 Summer	66.585	0.560	0.000	0.57		24.6	SURCHARGED
7.001	S17	15	30/15 Summer	66.539	0.654	0.000	2.01		83.9	SURCHARGED
9.000	RE4	15	30/15 Summer	66.208	0.248	0.000	1.36		55.0	SURCHARGED
7.002	S16	15		65.570	-0.115	0.000	0.69		138.8	OK
10.000	S13 CP	15		65.440	-0.300	0.000	0.00		0.0	OK
5.006	TANK_B	480	30/15 Summer	65.102	1.102	0.000	0.18	936	10.9	SURCHARGED
5.007	S15 FC	480	30/15 Summer	65.130	1.160	0.000	0.07		5.3	SURCHARGED

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Hillside Metal Bridge Durham, DH6 5NX	Proposed Warehouses New Smithfield Market Manchester	
Date 09/08/2022 File NETWORK_B.MDX	Designed by cje Checked by	
Innovyze	Network 2020.1.3	

100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Surface Network 2

Simulation Criteria

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

Number of Input Hydrographs 0 Number of Offline Controls 0 Number of Time/Area Diagrams 0
 Number of Online Controls 1 Number of Storage Structures 1 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR M5-60 (mm) 18.000 Cv (Summer) 0.750
 Region England and Wales Ratio R 0.350 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0
 Analysis Timestep 2.5 Second Increment (Extended)
 DTS Status ON
 DVD Status OFF
 Inertia Status OFF

Profile(s) Summer and Winter
 Duration(s) (mins) 15, 30, 60, 120, 240, 360, 480, 960, 1440
 Return Period(s) (years) 30, 100
 Climate Change (%) 40, 45

PN	US/MH Name	Duration (mins)	First (X) Surge	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)	Flow / Cap.	Half Drain Time (mins)	Pipe Flow (l/s)	Status
5.000	S8	15	100/15 Summer	66.119	0.369	0.000	0.61		170.3	SURCHARGED
6.000	S9 CP	15	100/15 Summer	66.063	0.463	0.000	0.54		44.6	SURCHARGED
5.001	S10	15	30/15 Summer	65.977	0.702	0.000	0.68		196.4	SURCHARGED
5.002	S11	15	30/15 Summer	65.925	0.715	0.000	1.10		376.2	SURCHARGED
5.003	S12	15	30/15 Summer	65.736	0.826	0.000	3.03		368.3	SURCHARGED
5.004	PI_B	960	30/15 Summer	65.667	0.877	0.000	0.24		29.7	SURCHARGED
5.005	S14 CP	960	30/15 Summer	65.666	1.046	0.000	0.19		29.3	SURCHARGED
7.000	RE3	15	30/15 Summer	67.080	1.045	0.429	0.71		30.7	FLOOD
8.000	RE2	15	30/15 Summer	67.090	1.065	0.221	0.75		32.5	FLOOD
7.001	S17	15	30/15 Summer	67.027	1.141	0.000	2.57		107.1	FLOOD RISK
9.000	RE4	15	30/15 Summer	66.628	0.668	0.000	1.81		73.3	SURCHARGED
7.002	S16	960		65.668	-0.017	0.000	0.07		13.4	OK
10.000	S13 CP	960		65.666	-0.074	0.000	0.00		0.1	OK
5.006	TANK_B	960	30/15 Summer	65.666	1.666	0.000	0.19	1229	11.4	SURCHARGED
5.007	S15 FC	960	30/15 Summer	65.693	1.723	0.000	0.09		6.1	SURCHARGED

Appendix B – Drainage Plans