PRICE & MYERS

Roehampton Gate Cafe
Richmond Park

Flood Risk Assessment and Drainage Strategy Report

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Job Number: **29081**

Date	Revision	Notes/Amendments/Issue Purpose
12 April 2024	0	Draft
3 July 2024	1	Planning

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Contents

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Acronyms	
AOD	Above Ordnance Datum
BGL	Below Ground Level
CIRIA	Construction Industry Research and Information Association
DCG	Design and Construction Guidance
DEFRA	Department for Environment, Food and Rural Affairs
EA	Environment Agency
FFL	Finished Floor Level
FRA	Flood Risk Assessment
LLFA	Lead Local Flood Authority
NPPF	National Planning Policy Framework
NSTS	Non-Statutory Technical Standards for SuDS
PPG	Planning Practice Guidance
SFRA	Strategic Flood Risk Assessment
SuDS	Sustainable Drainage Systems
SWMP	Surface Water Management Plan

1 Introduction

Price & Myers have been commissioned to undertake a Flood Risk Assessment (FRA) for the proposed development at Roehampton Gate Café, in Richmond Park, London.

The National Planning Policy Framework (NPPF) states that an appropriate FRA will be required for all development proposals of 1ha or greater in Flood Zone 1 and for any development within Flood Zones 2 or 3. The EA's indicative floodplain map shows that the site is in Flood Zone 1 and the site area is less than 1 ha, however this report has been prepared for BREEAM purposes and will therefore assess the flood risk from all sources.

This FRA has been carried out in accordance with the NPPF and the accompanying Planning Practice Guidance (PPG) "Flood Risk and Coastal Change". This FRA also incorporates advice and guidance from the Environment Agency (EA), the London Borough of Richmond Strategic Flood Risk Assessment (SFRA) (March 2021) and Surface Water Management Plan (December 2021) and CIRIA documents.

This report will also outline the proposed drainage strategy for the site including a detailed SuDS assessment. The surface water drainage strategy is in accordance with:

- the Department for Environment, Food and Rural Affairs (DEFRA) "Non-Statutory Technical Standards for Sustainable Drainage Systems" (NSTS);
- the Lead Local Flood Authority (LLFA) guidance document on SuDS/Drainage;
- BREEAM guidance documents.

2 Site Description and Location

The site is located in the north-east of Richmond Park, 200m south-east of the Roehampton Gate. It is bound by Priory Lane (shown on some maps as Horse Ride) to the west, The Alton Primary School to the north-east, and Richmond Park Golf Course to the south. The post code for the site is SW15 5JP and the national grid reference (NGR) is TQ213741. The total site area is 0.721 ha.

The site levels fall from east to west, with a high point of 10.5m AOD along the eastern boundary and a low point of 9.1m AOD on Priory Lane to the west. The levels continue to fall to the west of the site to the Beverley Brook, which is located approximately 200m to the west. The Beverley Brook runs south to north and outfalls to the River Thames approximately 3 km north-east of the site.

The existing site comprises of car parking, an existing café building, a bike hire kiosk, a toilet block, and soft landscaping.

Figure 2.1: Existing site, showing site boundary

2.1 Existing Drainage

The site and surrounding car park are served by separate foul water and surface water drainage networks. A CCTV survey was undertaken in September 2021. It shows that the surface water network serves approximately 0.8ha of area including the car park outside of the red line boundary. The surface water network discharges into a Thames Water surface water sewer which outfalls to the Beverley Brook. The drainage CCTV survey can be found in Appendix A

The Thames Water sewer records show that a 675mm diameter surface water sewer is located to the north of the site. The sewer flows from east to west and discharges to the Beverley Brook.

The sewer records also show a 900x600 brick egg-shaped combined water sewer, which flows from south to north. There is a foul water drainage network on the site, which connects into the sewer via a private foul water manhole to the north-east of the proposed café building. The Sewer Record can be found in Appendix B.

Figure 2.2: Thames Water sewer records, showing Beverley Brook to the south-west and the site boundary in green

3 Development Proposal

It is proposed to build a new café, with toilets and bicycle hire space. Alterations are also proposed to the hard and soft landscaping. An extract from the development proposals is available in Figure 3.1 below and the plans are available in full in Appendix C.

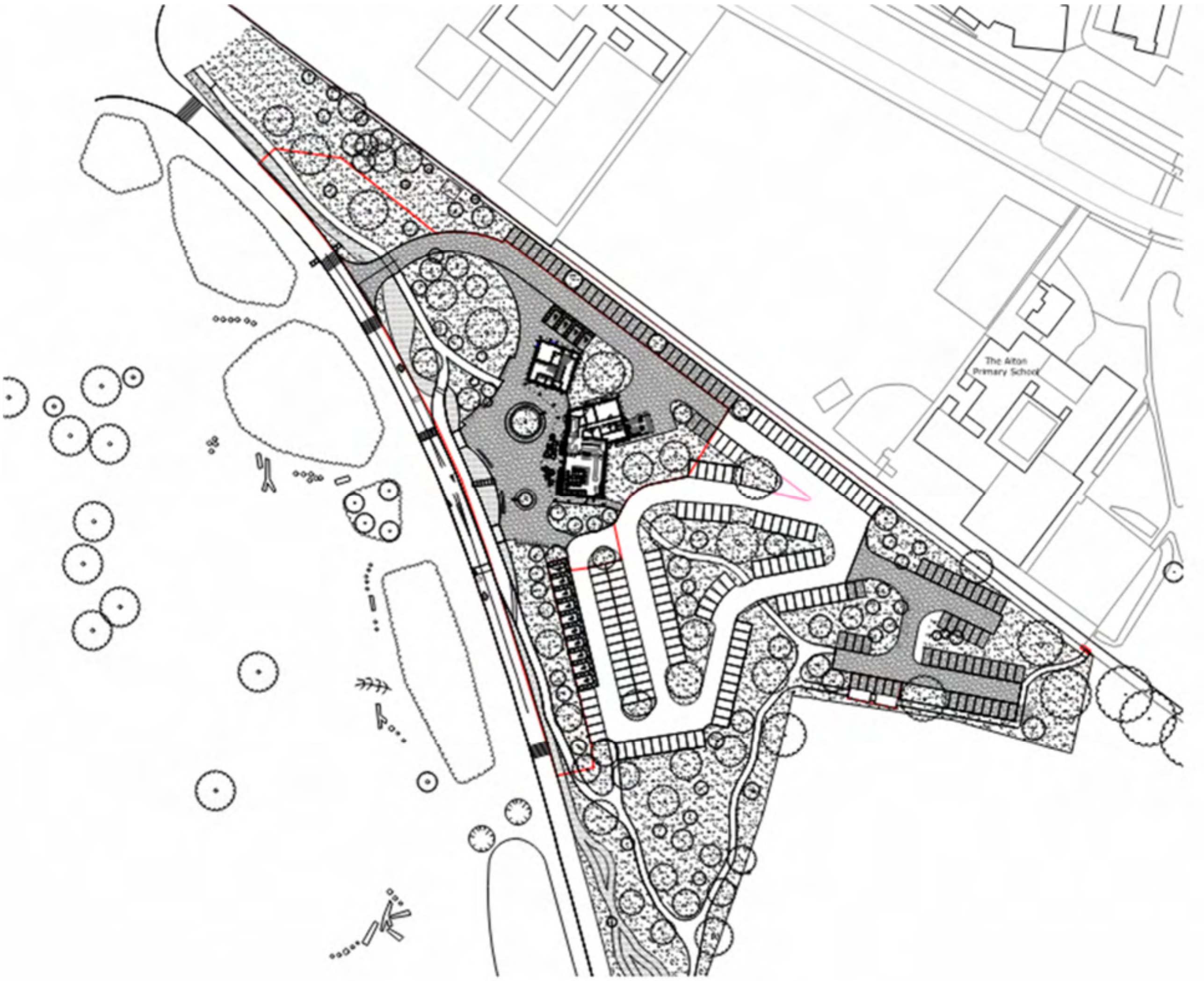


Figure 3.1: Proposed site layout (David Morley Architects)

4 Flood Risk Assessment

4.1 Flood Risk from Watercourses and Tidal Flooding

The EA's flood map for planning shows that the site is located in Flood Zone 1 which is defined as land assessed as have a probability of fluvial flooding of less than 0.1%. Developments in this flood zone do not have any restrictions, provided they do not increase the risk of flooding elsewhere.

Figure 4.1: EA Flood Map for Planning

4.2 Flood Risk from Groundwater

Groundwater flooding occurs when water originating from sub-surface permeable strata emerges from the ground, typically after prolonged rainfall.

The SFRA online mapping service provides information on three metrics for groundwater flooding, as follows:

- The EA 'susceptibility to groundwater flooding' map allocates a susceptibility rating to each 1km grid square. The site is shown to be in a grid square with a susceptibility to groundwater flooding between 25% and 49.9%.
- The Greater London Authority (GLA) 'increased potential for elevated groundwater' map is based on the BGS 1:50,000 geology map. The site is shown to be within the area of increased potential for elevated groundwater, with 'permeable superficial' ground conditions.
- The BGS 'susceptibility to groundwater flooding version 6' map also uses the BGS 1:50,000 geology map and allocates a rating for the potential for groundwater flooding to occur. The site is located in an area where there is potential for groundwater flooding to occur at the surface.

A ground investigation was undertaken in March 2024 by GEA Ltd which located groundwater at depths of around 2.00 m BGL in all borehole locations. The groundwater rose to depths of around 0.50 m on completion of each borehole. GEA's preliminary findings reported that this was likely the result of the collapse of the borehole sides from around 3.00 m depth due to the water pressure.

It should be noted that February 2024 was a very wet month across Southern England, with 239% of average rainfall recorded, making it the wettest February on record in this region. March 2024 was also wetter than average, with 50% more rainfall than usual falling across the month in England (Met Office Climate Summaries, www.metoffice.gov.uk). This is likely to have raised groundwater levels significantly at the time of the site investigation.

As groundwater was encountered at 2m below ground level during a prolonged period of wet weather, the risk of groundwater flooding is considered to be low.

4.3 Flood Risk from Surface Water and Overland Flows

Surface water flooding occurs when intense rainfall is unable to soak into the ground or enter a drainage system due to blockages or the capacity of the system being exceeded. Overland flows can also be generated by burst water mains, failed dams and any failure in a system storing or transferring water.

The EA's indicative Surface Water Flooding Map, Figure 4.2, shows that there is an area of "High" (more than 3.3%) risk of annual flooding from surface water on the site. This area correlates to an area of low topography on the site, where surface water may pond before draining away. The maps also indicate that there are areas of "Medium" (between 1% and 3.3%), "Low" (between 0.1% and 1%) and "Very Low" (less than 0.1%) risk of annual flooding from surface water.

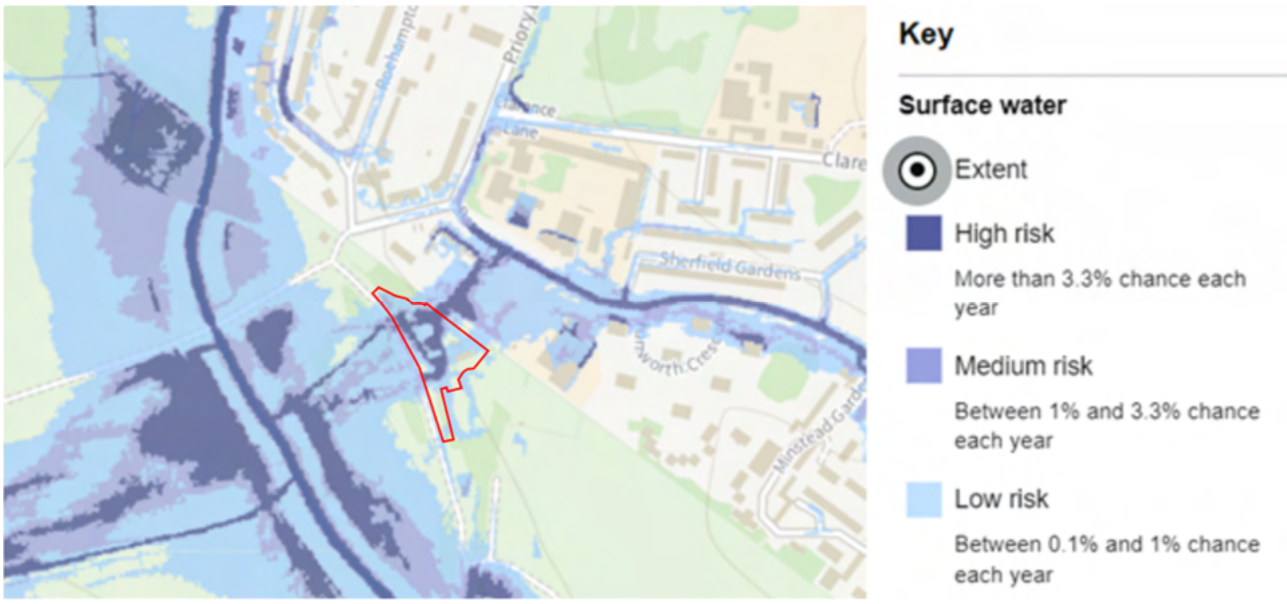


Figure 4.2: Environment Agency Surface Water Flood Risk Map

The surface water flood depth maps are shown in Figure 4.3, and the surface water velocity map is shown in Figure 4.4. These show that there is an area of surface water ponding in the low lying areas of the site in a 1 in 30 year event; the ponding is generally below 30cm deep but in one small area is between 30 and 90 cm deep. In a 1 in 100 year event, a clear flow path can be seen on the velocity map, crossing the site from north-east to south-west.

The Surface Water Flooding Map is based off LiDAR scans of the topography. The map does not take into account the presence of culverts and buried pipes. As described in Section 2.1, there is an existing surface water sewer that originates from the east and connects to the Beverley Brook to the west. It is likely that much of the surface water shown in the flow path in Figure 4.4 will be conveyed by this surface water sewer.

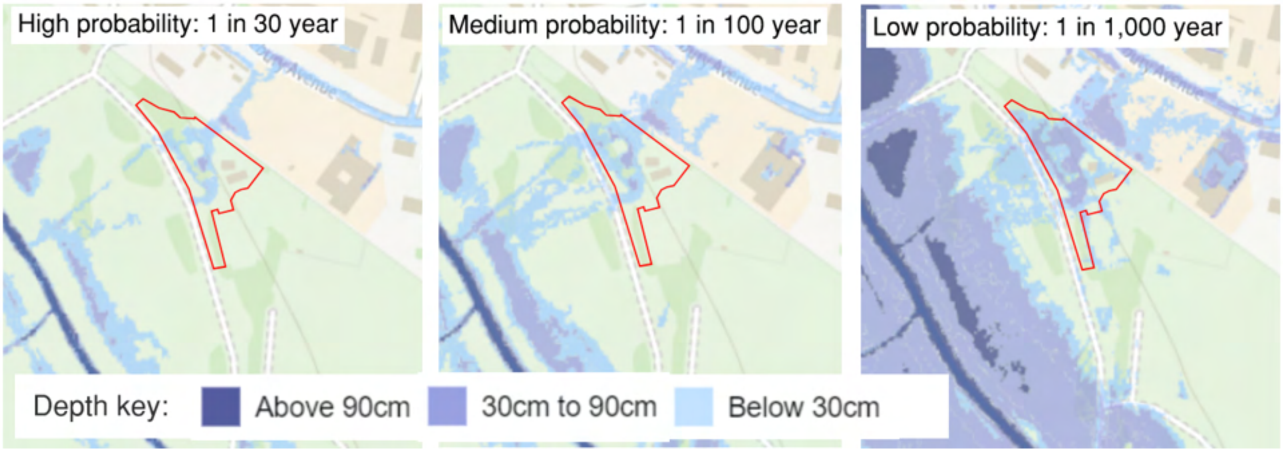


Figure 4.3: Environment Agency Surface Water Flood Depth Map

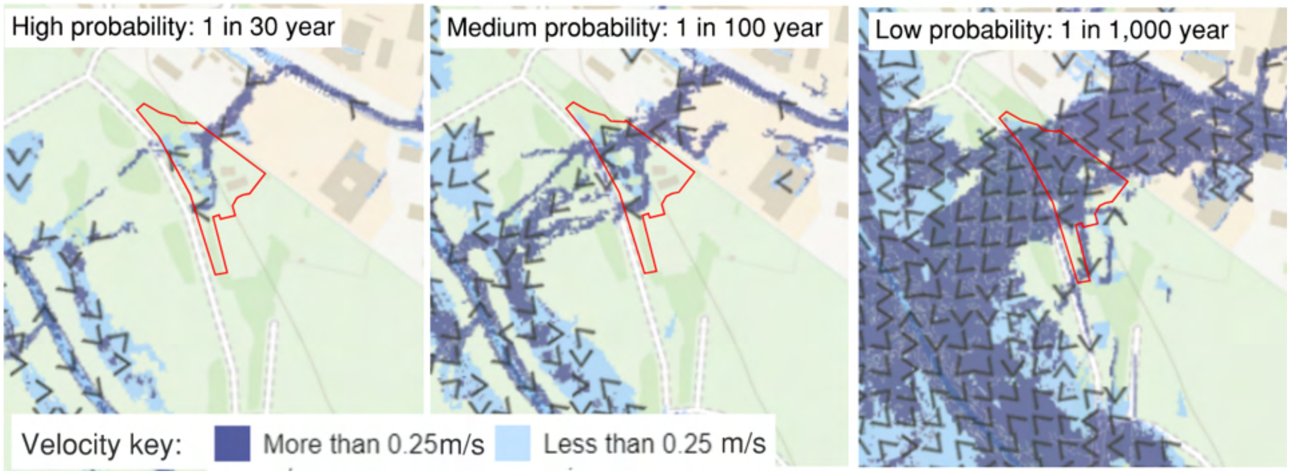


Figure 4.4: Environment Agency Surface Water Flood Velocity Map

4.3.1 Mitigation Measures for Surface Water and Overland Flows

The site levels will be altered as part of the development proposals. The finished floor level of the new café and bike hire building is proposed to be set at 9.85m AOD, which is above the surrounding ground levels. The surface water flow path crossing the site from the north-east will be directed to the north of the building, and will follow the existing flow path off-site towards the Beverley Brook to the west.

Surface water originating on the site will be managed in an appropriately designed surface water drainage system, as described in Section 5.

4.4 Flood Risk from Reservoirs

The EA provides information on flood risk from reservoirs. The map showing the maximum extent of flooding from reservoirs was updated in 2021 to show the combined effects of flooding from reservoirs and rivers. The Figure 4.5 shows that the site is not at risk of reservoir flooding when river levels are normal, or when there is flooding from rivers.

Figure 4.5: Environment Agency Risk of Reservoir Flooding Map

4.5 Flood Risk from Sewers

Sewer flooding occurs when the flow entering the sewerage network is greater than the capacity of the sewers. Thames Water sewer records show that there are two sewers crossing the site as described in Section 2.1. The combined sewer crossing the site from south to north is approximately 2.1m deep to soffit. There are no known issues of this sewer surcharging on the site.

There is a surface water sewer crossing the site from east to west. In the event of surcharge, the flows would follow the topography towards the Beverley Brook to the west of the site. The flood risk to the development from sewers is therefore considered to be low.

4.6 Summary of Flood Mitigation Measures

The flood risk from all sources except surface water is considered to be low or very low. The flood risk from surface water will be managed by raising the levels of the buildings and modifying the external levels. The surface water flows originating on the site will be managed in an appropriately designed surface water system, as described in Section 5.

5 Surface Water Runoff Assessment

5.1 Existing Runoff

The total site area is approximately 7,210m^2 or 0.721ha, of this approximately 1,490m^2 or 0.149ha is impermeable, and a further 1,760m^2 or 0.176ha is semi-permeable compacted hardcore. The hardcore is assumed to be 50% permeable, with the other 50% assumed to be managed in the below ground drainage system. Therefore, the total impermeable area is assumed to be 2,370m^2 or 0.237ha As described in Section 2.1, the site currently drains to the Beverley Brook via a private drainage system.

The existing runoff rate into ExS01 from the site for the design storm events was calculated using the modified rational method as shown below:

$Qx_{ex} = 2.78 \times A \times i$

Where 'x' is the return period in years, 'A' is the catchment area in ha and 'i' is the rainfall intensity in mm/hr provided by the FEH method.

$Q1_{ex} = 2.78 \times 0.237 \times 23.78 = 15.7$ l/s
$Q30_{ex} = 2.78 \times 0.237 \times 86.08 = 56.7$ l/s
$Q100_{ex} = 2.78 \times 0.237 \times 127.95 = 84.3$ l/s

5.2 Design Criteria

The DEFRA Non-Statutory Technical Standards for sustainable drainage systems (NSTS) requires that the drainage system must be designed so that, unless an area is designated to hold and/or convey water as part of the design, flooding does not occur:

- on any part of the site in a 1 in 30 year rainfall event;
- in any part of a building (including basement) or in any utility plant susceptible to water, in a 1 in 100 year rainfall event.

The current EA guidance on ensuring that new development does not increase flood risk elsewhere gives values for climate change allowance on rainfall intensity based on the expected lifetime of the building and the location of the site. For development with a lifetime between the years 2061 and 2100, the "central" allowance must be used for the 2070s epoch, which covers the period 2061 to 2125.

The central allowance for the 2070s epoch in the London Management Catchment is 20% in the 1 in 30 year rainfall event and 25% in the 1 in 100 year rainfall event. This allowance has been applied to the rainfall to carry out the drainage design.

5.2.1 Greenfield Runoff Rates

The London Plan and NSTS state that developments should aim to achieve greenfield runoff rates. The greenfield runoff rates for storm events of several different return periods were calculated using the FEH method based on the site area of 0.721ha. The results are summarised below and the supporting documentation is available in Appendix D.

$Q1_{Gr}$ = 2.3 l/s
Q_{Bar} = 2.8 l/s
$Q30_{Gr}$ = 8.8 l/s
$Q100_{Gr}$ = 10.3 l/s

5.3 Surface Water SuDS Strategy

The London Plan states that developments should ensure that surface water runoff is managed as close to its source as possible utilising sustainable methods (SuDS). There should be a preference for green over grey infrastructure in line with the following drainage hierarchy outlined in Policy SI 13 of the London Plan.

5.3.1 Rainwater Used as a Resource (e.g., Rainwater Harvesting)

Rainwater harvesting promotes the storage and re-use of rainwater collected from roofs and hard surfaced areas. This type of system contributes to the reduction of runoff rates and volumes within a development.

Given that water demand at the site is expected to be highest in the summer when days of rainfall are fewest[1], it is not considered an effective use of resources to install rainwater harvesting systems. Furthermore, the capacity of rainwater harvesting systems to attenuate rainwater depends on the water use within the building, and rainwater harvesters provide no attenuation if the harvester is already full. This means that during the winter months when rainfall is highest and building use is expected to be lowest, the rainwater harvesting system would provide minimal benefit.

The site is located in managed parkland, which relies on a healthy groundwater table for the health of the trees and plants. It is therefore proposed to utilise permeable surfaces where practicable to encourage groundwater recharge.

5.3.2 Rainwater Infiltration to Ground at or Close to Source

Due to high groundwater the ground investigation report could not complete infiltration testing and conclude that it is not viable to dispose of surface water to the ground via soakaways. It is proposed to use permeable surfaces across all hard-standing areas of the site with the exception of the access road. The ground investigation can be found in appendix E.

5.3.3 Rainwater Attenuation in Green Infrastructure Features for Gradual Release (for example green roofs, rain gardens)

It is proposed to include a green roof on part of the main roof of the building, which will offer benefits for biodiversity, as well as water treatment and will reduce runoff during normal rainfall events. A swale of 120m^3 volume is proposed to the west of the development that will provide attenuation storage, improve water quality and provide biodiversity and amenity benefits.

[1] See Met Office UK climate averages for Kew Gardens weather station: https://www.metoffice.gov.uk/research/climate/maps-and-data/uk-climate-averages/gcpuckhb6

5.3.4 Rainwater Discharge Direct to a Watercourse

It is proposed to connect to the existing surface water drainage within the site which ultimately connects to the Beverley Brook watercourse to the west.

5.3.5 Controlled Rainwater Discharge to a Surface Water Sewer or Drain

It is proposed to restrict the discharge rate to the surface water drain to as low as reasonably practicable.

5.3.6 Controlled Rainwater Discharge to a Combined Sewer

Not required.

5.4 Water Quality

SuDS have the potential to provide water quality benefits to runoff entering the environment. The SuDS Manual states that SuDS should be designed to treat runoff to reduce the risk of environmental pollution. Chapter 26 of The SuDS Manual sets out the ‘simple index approach’ to water quality risk management. This approach has been used to quantify the benefits of the proposed SuDS scheme on water quality.

Step 1 of the simple index approach is to identify the pollution hazard indices for the proposed land use. Table 5.1 below shows the associated pollution hazard indices for the café roof, the access road and the car park:

Land Use	Pollution Hazard Level	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Other roofs	Low	0.3	0.2	0.05
Individual property driveways, residential car parks, low traffic roads and non-residential car parking with infrequent change	Low	0.5	0.4	0.4

Table 5.1: Pollution Hazard Indices, taken from Table 26.2 of The SuDS Manual (CIRIA, 2015)

Step 2 of the simple index approach is to select SuDS with a total pollution mitigation index that equals or exceeds the pollution hazard index. The swale discharges to a watercourse. The permeable paving discharges directly to the ground.

The indicative surface water pollution mitigation indices for a swale are shown in Table 5.2.

Type of SuDS Component	TSS	Metals	Hydrocarbons
Swale	0.5	0.6	0.6

Table 5.2: Surface Water Pollution Mitigation Indices, taken from Table 26.3 of The SuDS Manual

As the mitigation indices for the swale are higher than the pollution indices for the roof and low traffic road, the swale provides adequate treatment to the surface water before entering the watercourse.

Characteristics of the material overlying the proposed infiltration surface, through which the runoff percolates	TSS	Metals	Hydrocarbons
Permeable pavement*	0.7	0.6	0.7
* All features to be underlain by a soil with good contaminant attenuation potential of at least 300mm in depth.			

Table 5.3: Groundwater Pollution Mitigation Indices, taken from Table 26.4 of the SuDS Manual.

The indicative groundwater pollution mitigation indices for a permeable pavement are shown in Table 5.3.

As the mitigation indices for the permeable pavement are higher than the pollution indices for the non-residential car parking with infrequent change, incorporation of a permeable pavement will provide suitable prevention from pollution to the groundwater.

5.5 Biodiversity and Amenity

The SuDS Manual describes that in addition to water quantity and water quality, key aspects of SuDS are amenity and biodiversity. The green roof and swale will provide biodiversity benefits providing additional spaces for local plants, inspects and animals.

5.6 Proposed Discharge Rates and Attenuation Volume

The discharge rate for the proposed development is proposed to be restricted by a vortex flow control device. Building Regulations Part H states that the minimum pipe size should be 75mm. Therefore, the flow control device is proposed to have a 75mm opening in order to reduce the risk of blockages. Therefore, the proposed discharge rate will be 3.6 l/s when the head of water is 1.8m. This runoff rate will be the limit for all storm events including the design storm with the climate change allowance (see Section 5.2).

The attenuation storage volume is proposed to be provided on site in a swale, in the sub-base of the permeable paving and a below ground attenuation tank. The storage volume than can be contained within the swale is approximately 120m^3, based on a base width of 1m, a depth of 0.6m and side slopes of 40%. The storage volume that can be contained within the porous sub-base is approximately 85.7m^3 based on an area of 952m^2, a sub-base depth of 300mm and a porosity of 0.3. For attenuation calculations the infiltration through the permeable paving subbase has been considered negligible.

The below ground attenuation tank has been sized to provide the additional storage needed to prevent flooding of the site in line with the NSTS. Preliminary hydraulic modelling calculations show that the size of the tank will be approximately 19 m^3. An indicative plan of the drainage strategy including the locations of the attenuation features is provided in Appendix F.

The hydraulic modelling calculations are provided in Appendix G.

5.7 Exceedance Routes and Overland Flows

The NSTS state that 'the design of the site must ensure that, so far as is reasonably practicable, flows resulting from rainfall in excess of a 1 in 100 year rainfall event are managed in exceedance routes that minimise the risks to people and property'. Flows in rainfall events exceeding the design storm will be managed in exceedance routes.

Site levels and finished floor levels of the new buildings have been designed so that exceedance flows will be away from the buildings and towards areas of car parking and soft landscaping. The exceedance flow routes are shown on the plan available in Appendix H.

5.8 Summary of SuDS Measures

The proposed SuDS include a green roof, swale and permeable paving. These have been incorporated to maximise biodiversity and amenity benefits, whilst providing adequate water treatment benefits. The peak flow rate has been controlled with a vortex flow control device and a connection proposed to the existing drainage system. An indicative drainage strategy plan is provided in Appendix F.

6 Surface Water Maintenance Strategy

The successful implementation and operation of a SuDS system depends on a robust and clear maintenance strategy being implemented. The following measures should form part of the site's proposed management plan. The SuDS will be maintained by The Royal parks and will form part of the overall maintenance regime for the site. Maintenance schedules from the manufacturers of specific products, e.g. green roofs, should supersede this outline strategy.

SuDS Element	Maintenance		
	Activity	**Required Action**	**Typical Frequency**
Green Roofs	**Monitoring / Inspections**	Inspect all components including soil substrate, vegetation, drains, irrigation systems, membranes and roof structure for proper operation, integrity of waterproofing and structural stability	Annually and after severe storms
		Inspect soil substrate for evidence of erosion channels and identify any sediment sources	
		Inspect drain inlets to ensure unrestricted runoff from the drainage layer to the conveyance or roof drain system	
		Inspect underside of roof for evidence of leakage	
	Regular Maintenance	Remove debris and litter to prevent clogging of inlet drains and interference with plant growth	Half yearly and annually or as required
		During establishment i.e. year one, replace dead plants as required	Monthly -but usually responsibility of manufacturer
		Post establishment, replace dead plants where > 5% of coverage	Annually in autumn
		Remove fallen leaves and debris from deciduous plant foliage	Half yearly or as required
		Remove nuisance and invasive vegetation, including weeds	
		Mow grasses, prune shrubs and manage other planting (if appropriate) as required – clippings should be removed and not allowed to accumulate	
	Remedial Actions	If erosion channels are evident, these should be stabilised with extra soil substrate similar to the original material, and sources of erosion damage should be identified and controlled	As required
		If drain inlet has settled, cracked or moved, investigate and repair as appropriate	

SuDS Element	Maintenance		
	Activity	**Required Action**	**Typical Frequency**
		Clear perforated pipework of blockages	As required
Swale	**Regular Maintenance**	Remove litter and debris	Monthly, or as required
		Cut the grass, manage vegetation and remove nuisance plants	Monthly at start, then as required
		Inspect inlets, outlets and overflows for blockages, and clear if required	Monthly
		Inspect for ponding, compaction and silt accumulation	Monthly or when required
		Inspect vegetation coverage	Monthly for 6 months, quarterly for 2 years, then half yearly
		Inspect inlets and facility surface for silt accumulation, establish appropriate silt removal frequencies	Half yearly
	Occasional Maintenance	Reseed areas of poor vegetation growth, alter plant types to better suit conditions, if required	As required or if bare soil Is exposed over 10% of more of the swale treatment area.
	Remedial Actions	Repair erosion or other damage by re-turfing or reseeding	As required
		Relevel uneven surfaces and reinstate design levels	
Permeable Paving	**Monitoring / Inspections**	Initial inspection	Monthly for three months after installation
		Inspect for evidence of poor operation and/or weed growth – if required, take remedial action	Three-monthly, 48 hours after large storms in first six months
		Inspect silt accumulation rates and establish appropriate brushing frequencies	Annually
		Monitor inspection chambers	Annually
	Regular Maintenance	Brushing and vacuuming -standard cosmetic sweep over whole surface	Once a year after autumn leaf fall
		Rubbish and litter removal	As required
	Remedial Actions	Remediate any landscaping which through vegetation maintenance or soil slip, has been raised to within 50mm of the level of the paving.	As required
		Remedial work to any depressions, rutting and cracked or broken blocks considered detrimental to the structural performance or a hazard to users, and replace lost jointing material	
		Rehabilitation of surface and upper substructure by remedial sweeping	Every 10 to 15 years or as required

SuDS Element	Maintenance		
	Activity	**Required Action**	**Typical Frequency**
Attenuation tank	**Monitoring / Inspections**	Inspect all inlets, outlets, vents, overflows and control structures to ensure they are working as they should	Annually or after severe storms
		Inspect and identify any elements that are not operating correctly.	Monthly for three months, then half yearly or as required.
	Regular Maintenance	Remove sediments / debris from catch pits / gullies and control structures	Annually, after severe storms or as required
	Remedial Actions	Repair inlets, outlets, vents, overflows and control structures.	As required

Table 6.1: SuDS Maintenance Strategy as taken from The SuDS Manual

Effective SuDS design must assess all foreseeable risks during construction and maintenance. These must be mitigated during the detailed design stages where effective design will aim to avoid, reduce and mitigate risks.

This process will also require input from the principal contractor who will ensure the construction of SuDS components are carried out in a safe and sustainable manner.

7 Foul Water Assessment

As outlined in Section 2.1, there is an existing foul water drainage network that serves the existing development which outfalls to the public sewer that crosses the site.

It is proposed to connect the new foul water drainage network to the existing drainage that connects to the public sewer.

A 'Pre-development Enquiry' will be submitted to Thames Water to confirm that capacity exists within the receiving public sewers. A Section 106 application will be required for a new indirect connection to the sewer.

8 Conclusions

- The site is located in the north-east of Richmond Park, 200m south-east of the Roehampton Gate. The post code for the site is SW15 5JP and the national grid reference (NGR) is TQ213741. The total site area is 0.721 ha.

- It is proposed to build a new café, with toilets and bicycle hire space. Alterations are also proposed to the hard and soft landscaping.

- The proposed development is considered to be at very low or low risk of flooding from rivers and tidal sources, reservoirs, groundwater and sewers.

- There is a surface water flow path crossing the site which is likely to affect the external areas of the site. The risk to the building has been mitigated by raising the finished floor levels and landscaping the site so that the falls are away from buildings in all directions. The surface water flow path will continue to cross the site, but it will be directed away from the buildings.

- Therefore, the proposed redevelopment has an acceptable flood risk within the terms and requirements of NPPF.

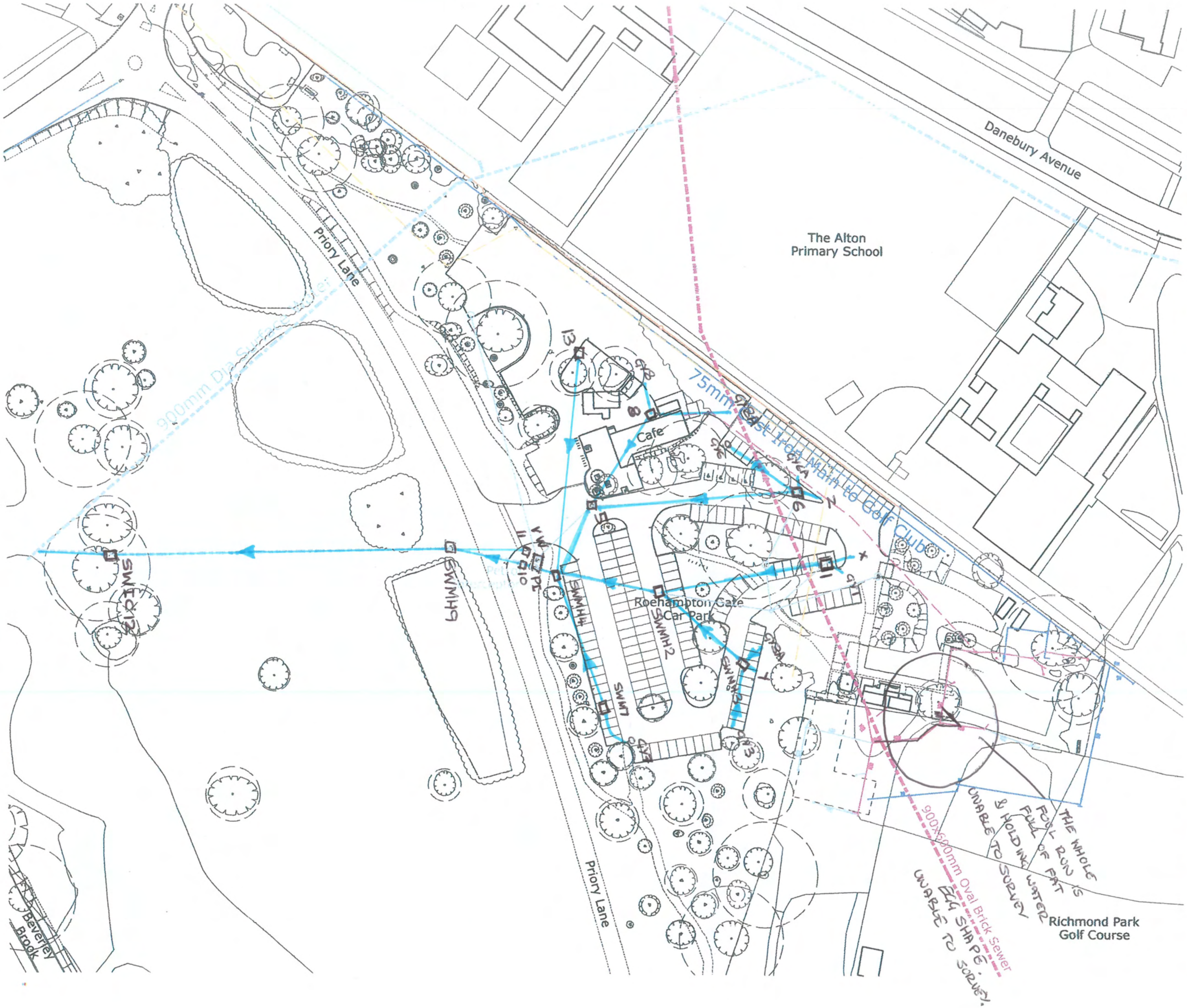
- The surface water drainage strategy has been developed in compliance with the NSTS. The SuDS measures proposed include a green roof, swale and permeable paving.

- Surface water will discharge into the Beverley Brook via an existing drainage network. The peak rate of discharge will be controlled using a vortex flow control device with a 75mm opening in order to reduce the risk of blockages. Attenuation will be provided in the swale and below ground attenuation tank.

- Foul water will discharge into the existing combined sewer via the existing drainage connection.

Appendix A
Drainage CCTV Survey

VINCI - ROEHAMPTON GATE CAFE.



0 50m N

Tree survey information provided by Canopy Consultancy and dated 22/09/20. Refer to tree survey for full information.

- Foul Sewer/ Drain
- Surface Water Drain
- Mains Water Supply
- Mains Gas Supply
- Telecoms

P3	06/07/21	Drain Lines Update
P2	13/05/21	General Update
P1	12/05/21	First Issue

DavidMorley

Roehampton Gate, Richmond Park

The Royal Parks

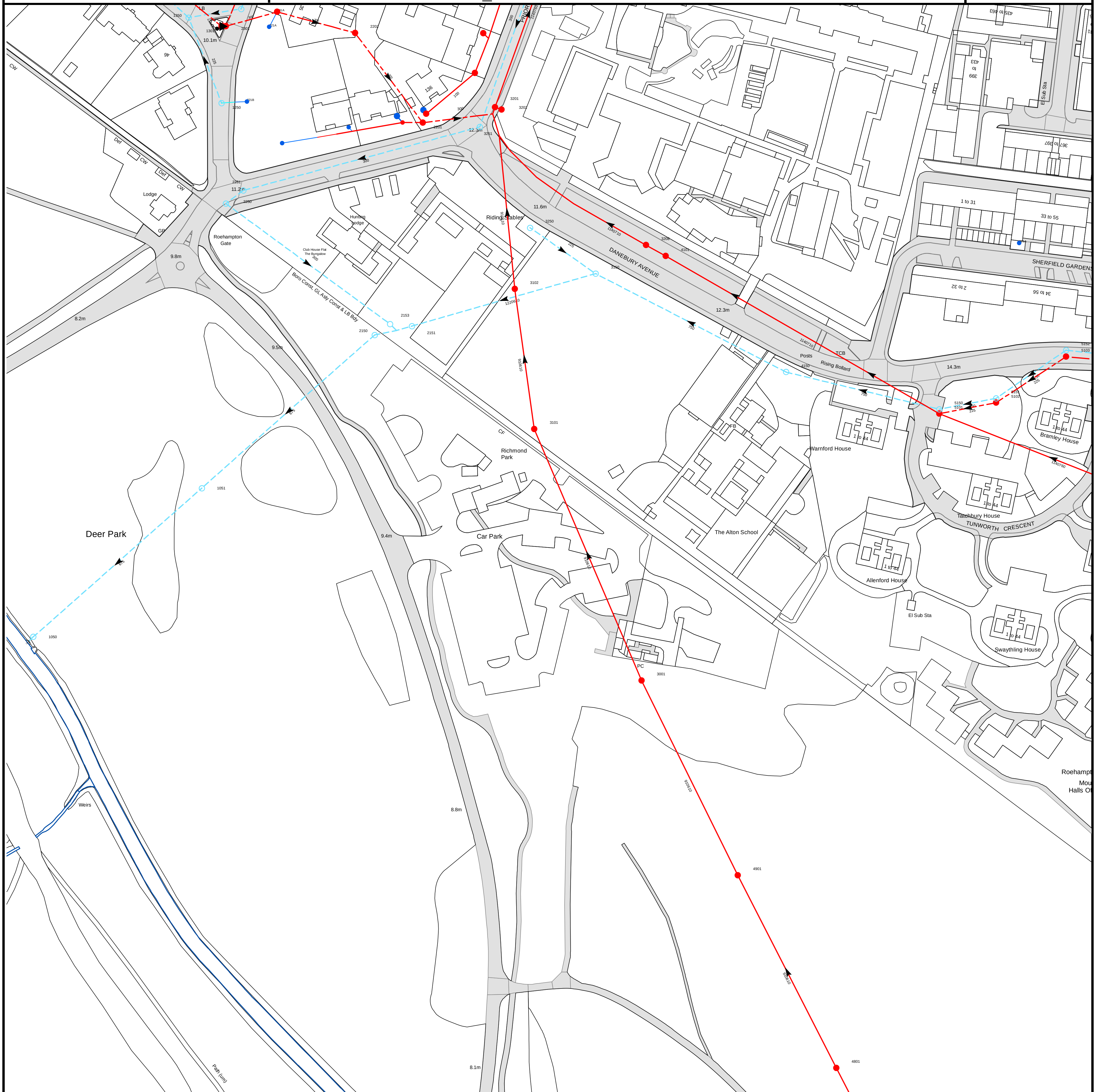
Site Services Plan

732-00-008 P3

06/07/2021 1:1000 @ A3

Appendix B
Sewer Records

Asset Location Search Sewer Map - ALS/ALS Standard/2021_4556442

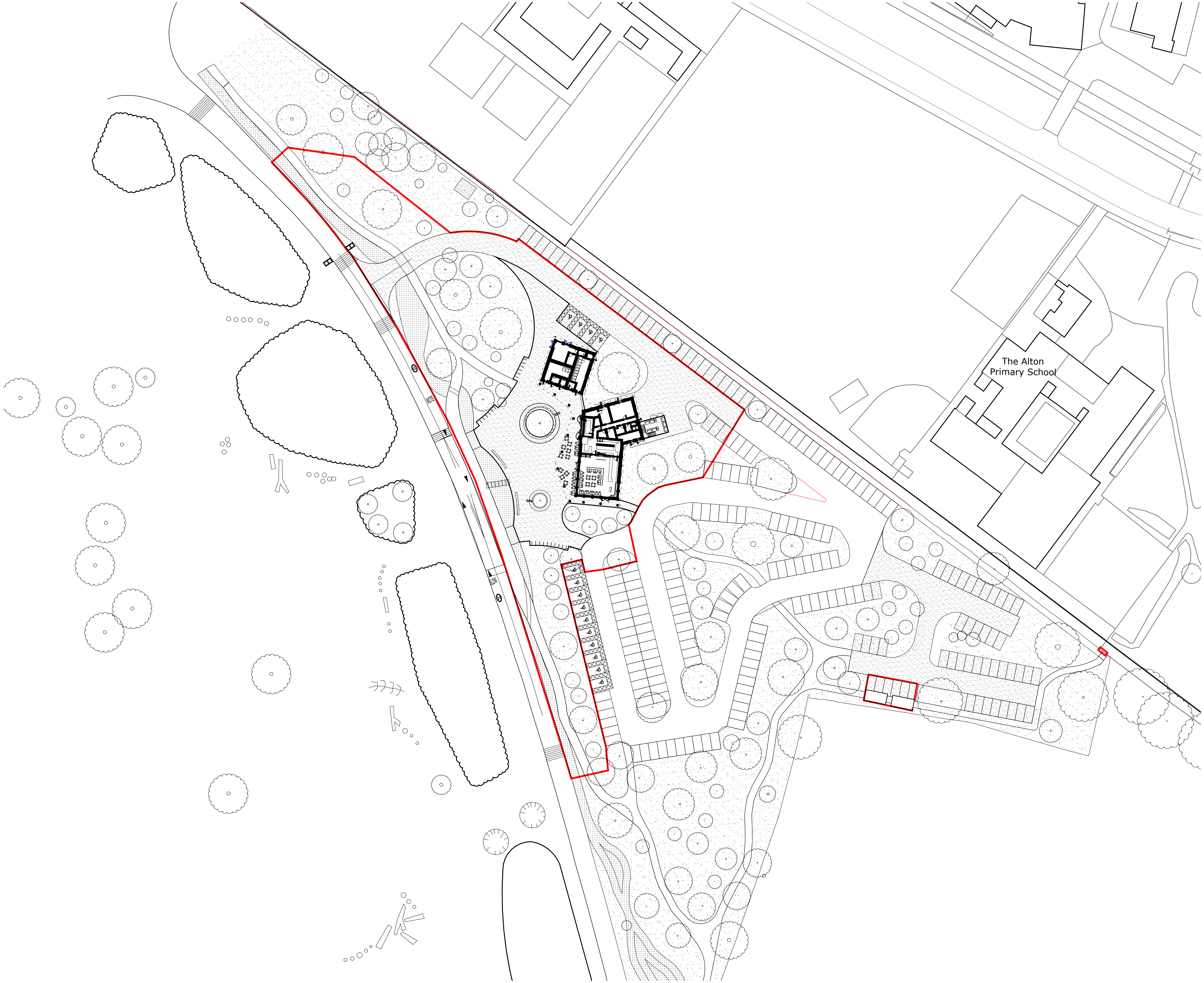


The width of the displayed area is 500 m and the centre of the map is located at OS coordinates 521350,174062

The position of the apparatus shown on this plan is given without obligation and warranty, and the accuracy cannot be guaranteed. Service pipes are not shown but their presence should be anticipated. No liability of any kind whatsoever is accepted by Thames Water for any error or omission. The actual position of mains and services must be verified and established on site before any works are undertaken.

Based on the Ordnance Survey Map with the Sanction of the controller of H.M. Stationery Office, License no. 100019345 Crown Copyright Reserved.

Appendix C
Development Proposal



© David Morley Architects
Do not scale off drawing.
Check all dimensions on site and advise any discrepancies before commencing work.
All dimensions in millimetres unless otherwise noted.

0 50m

N

Key

Rev.	Date	Description
P2	01/07/24	Issue for information
P1	12/04/24	Draft Issue for Planning

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Project
Roehampton Gate, Richmond Park

Client
The Royal Parks

Title
Site Plan

Project and Drawing Number
732-15-001

Revision
P2

Date
12/04/2024

Scale
1 : 1000 @ A3

Appendix D
Greenfield Runoff Rates

hr wallingford

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Harvey Doran
Site name:	29081
Site location:	Roehampton Cafe

Site Details

Latitude:	51.45269° N
Longitude:	0.25522° W
Reference:	2357498535
Date:	Jul 01 2024 17:09

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

Runoff estimation approach

FEH Statistical

Site characteristics

Total site area (ha):	0.721

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Calculate from dominant HOST
HOST class:	19
BFI / BFIHOST:	0.234
Q_{MED} (l/s):	2.42
Q_{BAR} / Q_{MED} factor:	1.14

Hydrological characteristics

	Default	Edited
SAAR (mm):	596	596
Hydrological region:	6	6
Growth curve factor 1 year:	0.85	0.85
Growth curve factor 30 years:	2.3	2.3
Growth curve factor 100 years:	3.19	3.19
Growth curve factor 200 years:	3.74	3.74

Notes

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

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

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

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

(3) Is SPR/SPRHOST ≤ 0.3?

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

Greenfield runoff rates

	Default	Edited

Q_{BAR} (l/s):	2.75	2.75
1 in 1 year (l/s):	2.34	2.34
1 in 30 years (l/s):	6.32	6.32
1 in 100 year (l/s):	8.77	8.77
1 in 200 years (l/s):	10.28	10.28

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

Appendix E
Ground Investigation Report

GROUND INVESTIGATION PRELIMINARY FINDINGS

Project Name:	*Roehampton Gate Cafe, Priory Lane, Richmond Park, London, SW15 5JR*	Date of instruction:	23-January-24
Client:	*The Royal Parks*	Date of fieldwork	14 & 15-March-24
GEA ref:	*J24024*	Date of preliminary report:	22-March-24
GEA Project Engineer and contact details:	*Jack Bonnewell* *Office: 01727 824666* *Email: jack@gea-ltd.co.uk*	Proposed date of final report:	16-April-24
		Preliminary logs attached	Y / ~~N~~
		Site plan attached	Y / ~~N~~

Summary of desk study findings:

Reference to historical maps indicates that at the time of the earliest map studied, dated 1868, the site was unoccupied and formed part of Richmond Park, with the Roehampton Gate entrance located around 90 m to the north. Beverley Brook was in existence from approximately 160 m to the southwest. Little development occurred on site until 1993, when the existing car park to the south and hardstanding areas on site were established, and the 2006 map shows the existing café. No other significant changes are recorded to the latest map studied, dated 2023. BGS records indicate the site to be underlain by superficial Head Deposits over Kempton Park Gravel, which are in turn underlain by the London Clay Formation.

Brief description of the site:

The site is located in London Borough of Richmond Upon Thames, approximately 1.84 km south west of Barnes London Overground Station. The site area is situated on the northeastern extent of Richmond Park. The site is bounded to the north and northeast by the grounds of The Alton Primary School whilst the remainder of the site is bounded by the grounds of Richmond Park itself. The site is occupied by a collection of single storey timber structures in use as a café and bicycle hire office surrounding by areas of car parking. The site is generally flat and level and includes a mixture of hard and soft landscaped areas, with soft landscaping comprising small grass verges and with a mixture of semi-mature to mature predominantly deciduous trees. At the time of the investigation, saturated ground conditions were observed in the park, located at approximately 120 m west of the site area adjacent to Beverley Brook, with large areas of perched water present at ground level.

Summary of ground conditions:

The investigation encountered the expected ground conditions in that, beneath a generally consistent thickness of made ground, Head Deposits were encountered over Kempton Park Gravel, below which was the London Clay Formation. The made ground comprised a dark grey silty slightly gravelly clay with gravel of fine sub rounded flint and occasional brick to depths between 0.20 m and 0.30 m. In some locations, below which, made ground soils comprised a dark brown sandy gravelly clay with gravel comprising fine to coarse sub rounded to sub angular flint, brick, and occasional clinker, coal, and ash to depths between 0.50 m and 0.70 m. The Head deposits and Kempton Park Gravel soils extended to depths of around 2.20 m where proved and at least 3.00 m elsewhere. These soils generally comprised firm sandy gravelly clay to depths of around 1.10 m to 1.60 m over medium dense to dense silty sand, with a decrease in density with depth attributed to groundwater. The underlying London Clay Formation was encountered in Borehole No.5 only from a depth of 2.20 m and comprised firm brown mottled blue grey clay, which was proved to the base of the borehole at 3.00 m. 'Blowing sand' conditions were encountered whereby sand under the pressure of groundwater is forced up into the drilling equipment, causing the rods to 'bind', and these conditions prevented advancing boreholes deeper than 3 m. Four of the boreholes were continued with continuous SPTs and increasing blow counts were recorded with depth, but the depth to the top of the London Clay is unclear. Geotechnical laboratory testing is underway and will determine the plasticity index of the head deposits and the London Clay Formation. Apart from the extraneous fragments of anthropogenic material contained in the made ground, no visual or olfactory evidence of contamination was encountered within any exploratory location. Borehole No 1 was aborted at a depth of 0.60 m due to being unable to penetrate due to the density of soils encountered with a SPT 'N' value of greater than 50.

Summary of groundwater conditions:

Groundwater was encountered at depths of around 2.00 m in all locations and had risen to depths of around 0.50 m on completion of each borehole, which is likely the result of the collapse of the borehole sides from around 3.00 m depth due to the water pressure. A falling head test could not be undertaken due to the shallow groundwater level encountered within the boreholes, and shallow soakaways are therefore unlikely to be feasible.

Details of the proposed development, upon which preliminary design recommendations are based:

It is understood that the proposals include replacing the existing café, bicycle hire kiosk and toilet block with larger buildings to provide greater space for each, along with reconfiguring the existing car park.

Preliminary foundation recommendations:

It should be possible to adopt light to moderately loaded spread foundations bearing on the firm clay or just into the top of the dense sand. Excavations should be controlled in order to avoid over-digging as groundwater will be encountered. Moderate width pad or strip foundations may be designed to apply a net allowable bearing pressure of around 100 kN/m^2 if bearing on firm clay, and in this respect it would be prudent to have a suitably qualified engineer inspect the base of foundation excavations. Additionally, foundations may need to be deepened in accordance with NHBC guidelines and the minimum founding depth will be determined by the results of the geotechnical laboratory testing, although high shrinkability clay should be assumed at this stage. Where deepening results in an uneconomic foundation design, or where the ingress of water into excavations makes pouring traditional concrete foundations difficult, consideration may need to be given to raft foundations, or to driven or bored piled foundations or steel helical screw piles.

Details of any other ground related issues (inc contamination) that may affect the site:

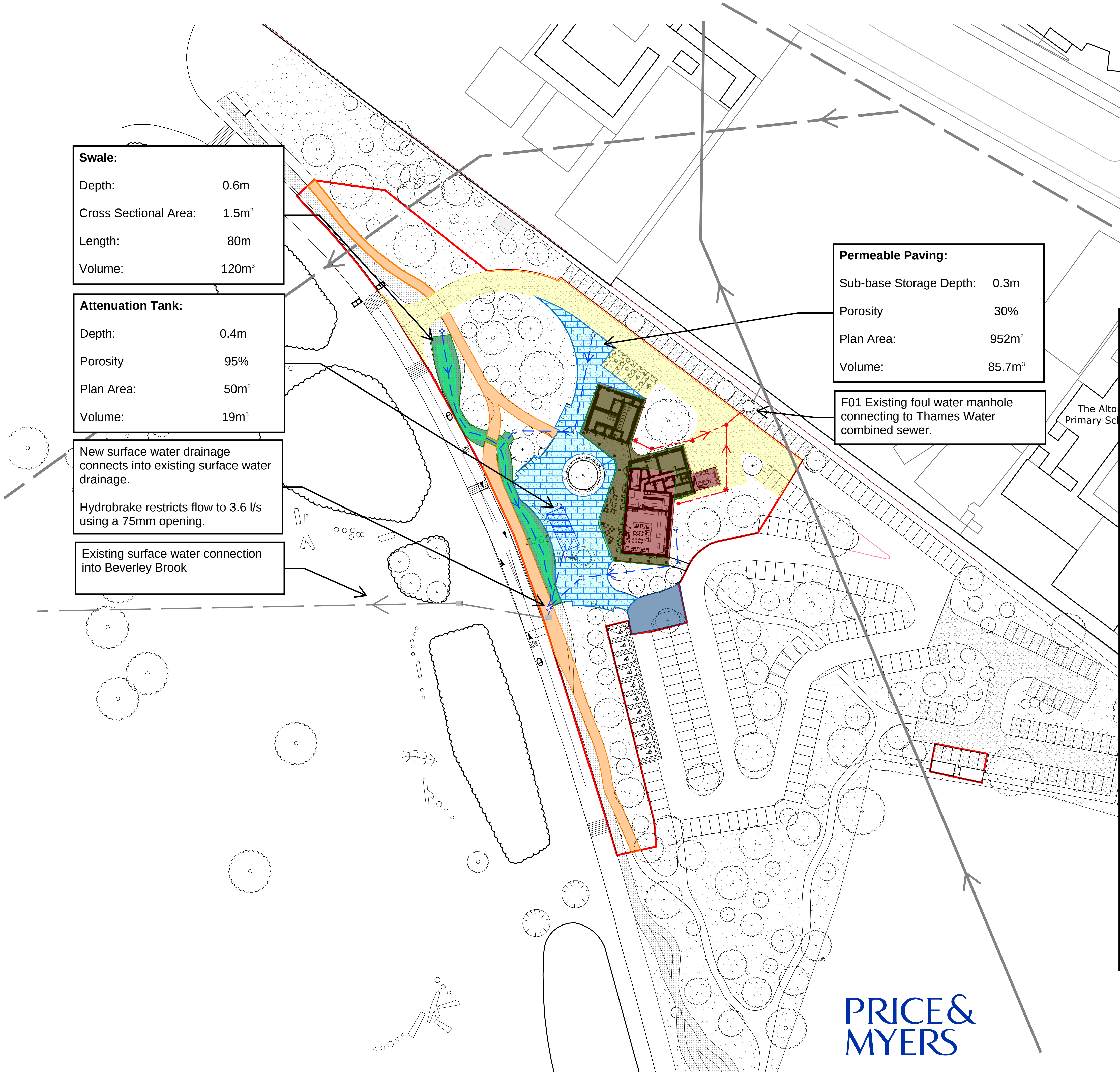
Apart from the extraneous fragments of anthropogenic material contained in the made ground, no visual or olfactory evidence of contamination was encountered. Chemical laboratory testing of soils is currently underway, and results will be included in the final report.

In order to provide most comprehensive report possible for the above site GEA require the following information:

Please provide information on the proposed development including anticipated loads and designs, if available at this stage. If consideration is being given to raft foundations, please provide proposed pressure distributions or initial assessment of proposed pressure at this stage.

The information provided by this summary and any attached sheets is preliminary and is subject to change in the light of any laboratory testing which will be completed prior to the issue of the final report, and following a full review of all of the information from the investigation as part of our Quality Management procedures. Any design decisions made on the basis of this information are therefore made at the risk of the client and GEA accepts no liability in this respect.

Appendix F
Drainage Layout



Area Key

Site Outline	7,210m²
Proposed Permeable Paved Areas	952m²
Proposed Vehicular Areas -Formally Drained	1,078m²
Retained Vehicular Areas- Drainage Retained	135m²
Proposed Green Roof Area	465m²
Proposed Roof Area	329m²
Proposed Pedestrian Pathway- Drains to Ground	591m²

Drainage Key

- Proposed Swale
- Proposed Rock Wool Attenuation Tank
- Proposed Surface Water Manhole
- Proposed Vortex Flow Control
- Proposed Surface Water Pipework
- Existing Surface Water Pipework
- Existing Surface Water Manhole
- Proposed Foul Water Manhole
- Proposed Foul Water Pipework
- Existing Foul Water Manhole
- Existing Foul Water Drain
- Existing Combined Water Sewer
- Existing Surface Water Sewer

PRICE & MYERS

Job No.	29081	Page	SK 604	Rev	6
Date	2024 07 03	Eng	HD	Chd	TS
Job	Roehampton Gate Cafe				

Appendix G
Hydraulic Model

Design Settings

Rainfall Methodology	FEH-22	Minimum Velocity (m/s)	1.00
Return Period (years)	100	Connection Type	Level Soffits
Additional Flow (%)	0	Minimum Backdrop Height (m)	0.200
CV	1.000	Preferred Cover Depth (m)	1.200
Time of Entry (mins)	5.00	Include Intermediate Ground	✓
Maximum Time of Concentration (mins)	500.00	Enforce best practice design rules	x
Maximum Rainfall (mm/hr)	500.0		

Simulation Settings

Rainfall Methodology	FEH-22	Skip Steady State	x	Check Discharge Rate(s)	x
Winter CV	1.000	Drain Down Time (mins)	350	Check Discharge Volume	x
Analysis Speed	Normal	Additional Storage (m^3/ha)	0.0		

Storm Durations

60	180	360	600	960	2160	4320	7200	10080
120	240	480	720	1440	2880	5760	8640	

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
2	0	0	0
30	20	0	0
100	25	0	0

Node S8 (FC) Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	x	Sump Available	x
Invert Level (m)	8.320	Product Number	CTL-CHE-0075-3600-1800-3600
Design Depth (m)	1.800	Min Outlet Diameter (m)	0.100
Design Flow (l/s)	3.6	Min Node Diameter (mm)	1200

Node S3(Swale) Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	9.150
Side Inf Coefficient (m/hr)	0.00000	Porosity	1.00	Time to half empty (mins)	

Depth (m)	Area (m^2)	Inf Area (m^2)	Depth (m)	Area (m^2)	Inf Area (m^2)
0.000	80.0	0.0	0.600	320.0	0.0

Node S8 (FC) Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	7.859
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	

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	50.0	0.0	0.400	50.0	0.0	0.401	0.0	0.0

Node S4 (PP) Carpark Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Invert Level (m)	9.350	Slope (1:X)	999.0
Side Inf Coefficient (m/hr)	0.00000	Time to half empty (mins)		Depth (m)	0.300
Safety Factor	2.0	Width (m)	30.700	Inf Depth (m)	
Porosity	0.30	Length (m)	31.000		

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)	Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
2 year 60 minute winter	28.360	11.281	30 year +20% CC 1440 minute winter	8.798	3.508
2 year 120 minute winter	21.658	8.615	30 year +20% CC 2160 minute winter	6.123	2.456
2 year 180 minute winter	17.545	6.946	30 year +20% CC 2880 minute winter	4.807	1.917
2 year 240 minute winter	14.652	5.828	30 year +20% CC 4320 minute winter	3.456	1.372
2 year 360 minute winter	11.149	4.414	30 year +20% CC 5760 minute winter	2.771	1.096
2 year 480 minute winter	8.965	3.566	30 year +20% CC 7200 minute winter	2.352	0.930
2 year 600 minute winter	7.492	2.999	30 year +20% CC 8640 minute winter	2.071	0.818
2 year 720 minute winter	6.502	2.593	30 year +20% CC 10080 minute winter	1.868	0.739
2 year 960 minute winter	5.144	2.045	100 year +25% CC 60 minute winter	136.802	54.416
2 year 1440 minute winter	3.662	1.460	100 year +25% CC 120 minute winter	87.846	34.943
2 year 2160 minute winter	2.615	1.049	100 year +25% CC 180 minute winter	66.706	26.408
2 year 2880 minute winter	2.095	0.836	100 year +25% CC 240 minute winter	53.841	21.416
2 year 4320 minute winter	1.562	0.620	100 year +25% CC 360 minute winter	39.582	15.670
2 year 5760 minute winter	1.290	0.510	100 year +25% CC 480 minute winter	31.191	12.407
2 year 7200 minute winter	1.124	0.444	100 year +25% CC 600 minute winter	25.704	10.290
2 year 8640 minute winter	1.013	0.400	100 year +25% CC 720 minute winter	22.070	8.801
2 year 10080 minute winter	0.933	0.369	100 year +25% CC 960 minute winter	17.203	6.838
30 year +20% CC 60 minute winter	100.526	39.987	100 year +25% CC 1440 minute winter	11.904	4.747
30 year +20% CC 120 minute winter	65.350	25.994	100 year +25% CC 2160 minute winter	8.190	3.285
30 year +20% CC 180 minute winter	49.628	19.647	100 year +25% CC 2880 minute winter	6.358	2.535
30 year +20% CC 240 minute winter	39.976	15.901	100 year +25% CC 4320 minute winter	4.476	1.777
30 year +20% CC 360 minute winter	29.255	11.582	100 year +25% CC 5760 minute winter	3.523	1.393
30 year +20% CC 480 minute winter	22.965	9.135	100 year +25% CC 7200 minute winter	2.946	1.164
30 year +20% CC 600 minute winter	18.888	7.561	100 year +25% CC 8640 minute winter	2.560	1.012
30 year +20% CC 720 minute winter	16.205	6.462	100 year +25% CC 10080 minute winter	2.285	0.903
30 year +20% CC 960 minute winter	12.639	5.024			

Results for 2 year Critical Storm Duration. Lowest mass balance: 91.98%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
360 minute winter	S3(Swale)	264	9.335	0.185	5.5	21.7240	0.0000	OK
360 minute winter	S4 (PP)	264	9.335	0.185	6.5	0.0524	0.0000	OK
360 minute winter	S8 (FC)	264	9.334	1.475	6.7	20.8342	0.0000	SURCHARGED
360 minute winter	S7	264	9.335	0.993	3.3	0.2809	0.0000	SURCHARGED
360 minute winter	S6	264	9.335	0.689	1.6	0.1949	0.0000	SURCHARGED
360 minute winter	S5	264	9.335	0.536	0.8	0.1516	0.0000	SURCHARGED
60 minute winter	S1	33	9.399	0.049	4.4	0.0138	0.0000	OK
360 minute winter	S2	264	9.335	0.083	2.6	0.0235	0.0000	OK
360 minute winter	S9 (Outfall)	264	7.661	0.026	2.7	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
360 minute winter	S3(Swale)	S3-S4	S4 (PP)	3.6	0.493	0.091	0.3351	
360 minute winter	S4 (PP)	S4-S8	S8 (FC)	6.5	0.873	0.065	1.2254	
360 minute winter	S8 (FC)	Outfall	S9 (Outfall)	2.7	1.306	0.064	0.0112	53.3
360 minute winter	S7	S7-S8	S8 (FC)	-1.8	0.720	-0.019	0.5493	
360 minute winter	S6	S6-S7	S7	1.5	0.739	0.062	0.2164	
360 minute winter	S5	S5-S6	S6	0.8	0.435	0.038	0.2152	
60 minute winter	S1	S1-S2	S2	4.4	0.566	0.104	0.1154	
360 minute winter	S2	S2-S3	S3(Swale)	2.6	0.499	0.069	0.4666	

Results for 30 year +20% CC Critical Storm Duration. Lowest mass balance: 91.98%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
360 minute winter	S3(Swale)	344	9.564	0.414	13.9	67.4500	0.0000	FLOOD RISK
360 minute winter	S4 (PP)	344	9.564	0.414	11.8	56.6440	0.0000	FLOOD RISK
360 minute winter	S8 (FC)	344	9.563	1.704	7.3	21.1143	0.0000	FLOOD RISK
360 minute winter	S7	344	9.563	1.221	4.2	0.3455	0.0000	FLOOD RISK
360 minute winter	S6	344	9.563	0.917	4.3	0.2595	0.0000	FLOOD RISK
360 minute winter	S5	344	9.563	0.764	2.1	0.2162	0.0000	FLOOD RISK
360 minute winter	S1	344	9.564	0.214	4.6	0.0605	0.0000	OK
360 minute winter	S2	344	9.564	0.312	6.7	0.0882	0.0000	FLOOD RISK
360 minute winter	S9 (Outfall)	344	7.662	0.027	3.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
360 minute winter	S3(Swale)	S3-S4	S4 (PP)	-5.8	0.478	-0.145	0.3812	
360 minute winter	S4 (PP)	S4-S8	S8 (FC)	6.9	0.999	0.068	1.3045	
360 minute winter	S8 (FC)	Outfall	S9 (Outfall)	3.0	1.342	0.071	0.0120	104.1
360 minute winter	S7	S7-S8	S8 (FC)	4.2	0.769	0.043	0.5493	
360 minute winter	S6	S6-S7	S7	4.2	0.767	0.173	0.2164	
360 minute winter	S5	S5-S6	S6	2.1	0.452	0.104	0.2152	
360 minute winter	S1	S1-S2	S2	4.5	0.501	0.106	0.5802	
360 minute winter	S2	S2-S3	S3(Swale)	6.5	0.623	0.174	0.7685	

Results for 100 year +25% CC Critical Storm Duration. Lowest mass balance: 91.98%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
480 minute winter	S3(Swale)	352	9.660	0.510	11.5	92.8303	0.0000	FLOOD RISK
360 minute winter	S4 (PP)	280	9.696	0.546	15.8	81.4837	0.0000	FLOOD RISK
480 minute winter	S8 (FC)	352	9.687	1.828	5.3	21.2666	0.0000	FLOOD RISK
480 minute winter	S7	352	9.682	1.340	4.5	0.3793	0.0000	FLOOD RISK
360 minute winter	S6	272	9.673	1.027	5.8	0.2906	0.0000	FLOOD RISK
60 minute winter	S5	35	9.709	0.910	9.6	0.2574	0.0000	FLOOD RISK
480 minute winter	S1	344	9.660	0.310	4.9	0.0877	0.0000	FLOOD RISK
480 minute winter	S2	344	9.660	0.408	7.1	0.1154	0.0000	FLOOD RISK
480 minute winter	S9 (Outfall)	352	7.663	0.028	3.1	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
480 minute winter	S3(Swale)	S3-S4	S4 (PP)	-5.5	0.442	-0.139	0.3812	
360 minute winter	S4 (PP)	S4-S8	S8 (FC)	6.7	1.057	0.067	1.3045	
480 minute winter	S8 (FC)	Outfall	S9 (Outfall)	3.1	1.358	0.074	0.0124	127.5
480 minute winter	S7	S7-S8	S8 (FC)	4.5	0.813	0.046	0.5493	
360 minute winter	S6	S6-S7	S7	5.8	0.754	0.238	0.2164	
60 minute winter	S5	S5-S6	S6	9.4	0.760	0.471	0.2152	
480 minute winter	S1	S1-S2	S2	4.8	0.474	0.114	0.5858	
480 minute winter	S2	S2-S3	S3(Swale)	7.0	0.594	0.187	0.7685	

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Design Settings

Rainfall Methodology	FSR	Maximum Time of Concentration (mins)	500.00
Return Period (years)	100	Maximum Rainfall (mm/hr)	500.0
Additional Flow (%)	0	Minimum Velocity (m/s)	1.00
FSR Region	England and Wales	Connection Type	Level Soffits
M5-60 (mm)	20.000	Minimum Backdrop Height (m)	0.200
Ratio-R	0.400	Preferred Cover Depth (m)	1.200
CV	1.000	Include Intermediate Ground	✓
Time of Entry (mins)	5.00	Enforce best practice design rules	x

Simulation Settings

Rainfall Methodology	FSR	Skip Steady State	x
FSR Region	England and Wales	Drain Down Time (mins)	350
M5-60 (mm)	20.000	Additional Storage (m^3/ha)	0.0
Ratio-R	0.400	Check Discharge Rate(s)	x
Winter CV	1.000	Check Discharge Volume	x
Analysis Speed	Normal		

Storm Durations

15	30

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
2	0	0	0
30	20	0	0
100	25	0	0

Rainfall

Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)	Event	Peak Intensity (mm/hr)	Average Intensity (mm/hr)
2 year 15 minute winter	99.345	40.058	30 year +20% CC 30 minute winter	147.308	59.399
2 year 30 minute winter	64.388	25.963	100 year +25% CC 15 minute winter	305.911	123.351
30 year +20% CC 15 minute winter	226.279	91.242	100 year +25% CC 30 minute winter	200.847	80.987

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Results for 2 year Critical Storm Duration. Lowest mass balance: 96.86%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	S3(Swale)	23	9.262	0.112	22.6	11.4692	0.0000	OK
30 minute winter	S4 (PP)	34	9.234	0.084	20.7	0.0238	0.0000	OK
30 minute winter	S8 (FC)	35	9.228	1.369	29.1	20.7037	0.0000	SURCHARGED
30 minute winter	S7	35	9.229	0.887	9.4	0.2510	0.0000	SURCHARGED
30 minute winter	S6	35	9.230	0.659	8.8	0.1866	0.0000	SURCHARGED
30 minute winter	S5	35	9.234	0.435	4.3	0.1232	0.0000	SURCHARGED
15 minute winter	S1	10	9.436	0.086	12.4	0.0243	0.0000	OK
15 minute winter	S2	10	9.366	0.114	18.1	0.0322	0.0000	OK
30 minute winter	S9 (Outfall)	35	7.660	0.025	2.6	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
30 minute winter	S3(Swale)	S3-S4	S4 (PP)	12.4	0.872	0.311	0.1426	
30 minute winter	S4 (PP)	S4-S8	S8 (FC)	20.7	1.816	0.207	0.8738	
30 minute winter	S8 (FC)	Outfall	S9 (Outfall)	2.6	1.286	0.061	0.0107	15.0
30 minute winter	S7	S7-S8	S8 (FC)	8.8	1.215	0.090	0.5493	
30 minute winter	S6	S6-S7	S7	8.8	1.298	0.124	0.4889	
30 minute winter	S5	S5-S6	S6	4.3	0.880	0.216	0.2152	
15 minute winter	S1	S1-S2	S2	12.2	0.722	0.289	0.2502	
15 minute winter	S2	S2-S3	S3(Swale)	17.9	1.274	0.476	0.3241	

Results for 30 year +20% CC Critical Storm Duration. Lowest mass balance: 96.86%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	S3(Swale)	33	9.420	0.270	67.4	36.1937	0.0000	SURCHARGED
30 minute winter	S4 (PP)	33	9.420	0.270	50.4	15.5644	0.0000	SURCHARGED
15 minute winter	S8 (FC)	13	9.486	1.627	83.7	21.0203	0.0000	FLOOD RISK
30 minute winter	S7	19	9.523	1.181	20.4	0.3342	0.0000	FLOOD RISK
30 minute winter	S6	19	9.556	0.985	20.0	0.2788	0.0000	FLOOD RISK
30 minute winter	S5	19	9.566	0.767	14.8	0.2171	0.0000	FLOOD RISK
15 minute winter	S1	10	9.502	0.152	28.2	0.0431	0.0000	OK
15 minute winter	S2	11	9.452	0.200	41.1	0.0565	0.0000	OK
30 minute winter	S9 (Outfall)	19	7.662	0.027	2.9	0.0000	0.0000	OK

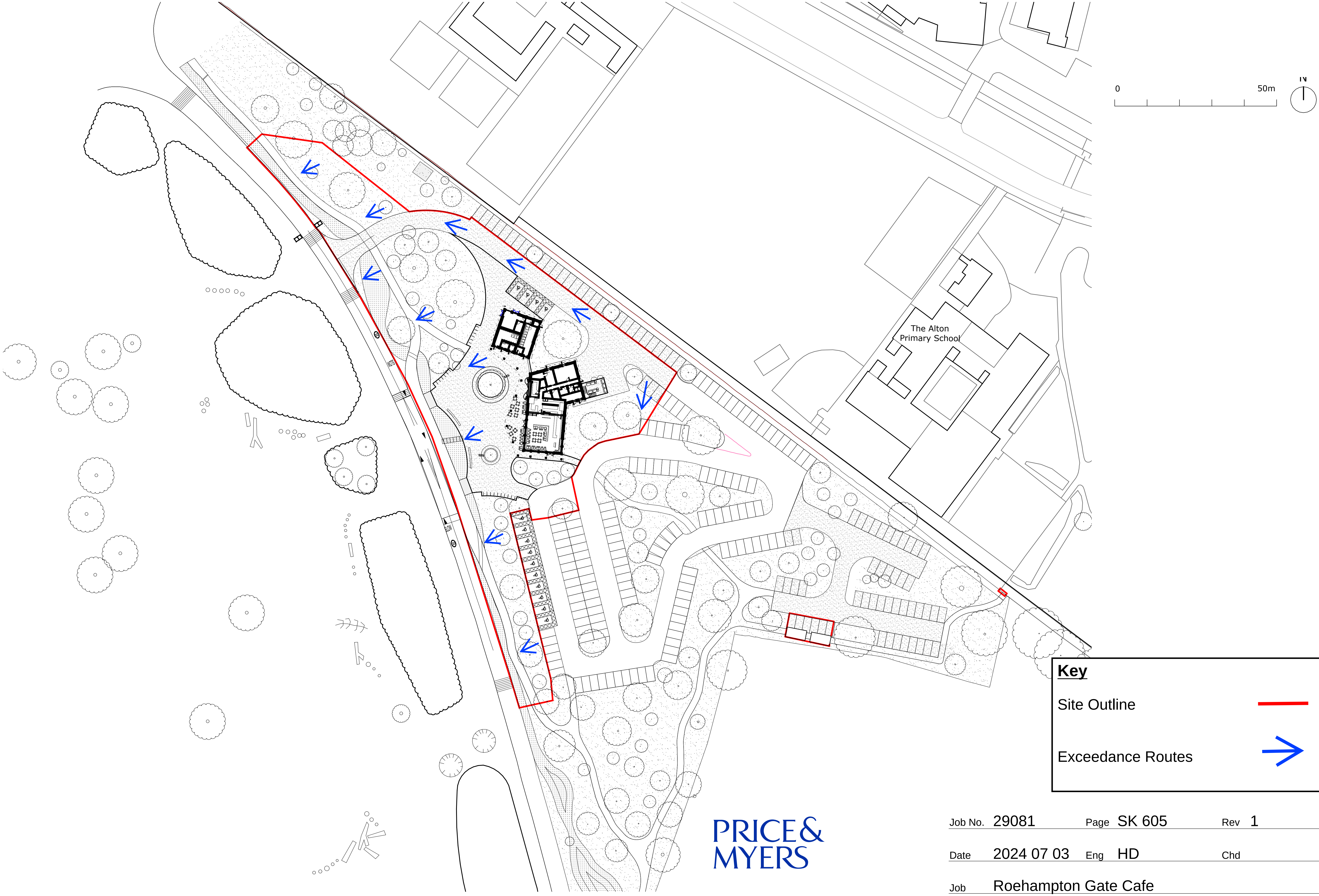
Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
30 minute winter	S3(Swale)	S3-S4	S4 (PP)	-19.6	0.563	-0.494	0.3812	
30 minute winter	S4 (PP)	S4-S8	S8 (FC)	48.5	1.949	0.485	1.3045	
15 minute winter	S8 (FC)	Outfall	S9 (Outfall)	2.9	1.327	0.068	0.0117	42.7
30 minute winter	S7	S7-S8	S8 (FC)	21.6	1.426	0.221	0.5493	
30 minute winter	S6	S6-S7	S7	20.4	1.611	0.287	0.4889	
30 minute winter	S5	S5-S6	S6	10.5	1.081	0.527	0.2152	
15 minute winter	S1	S1-S2	S2	27.8	0.849	0.657	0.4789	
15 minute winter	S2	S2-S3	S3(Swale)	40.2	1.409	1.069	0.6568	

Results for 100 year +25% CC Critical Storm Duration. Lowest mass balance: 96.86%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
30 minute winter	S3(Swale)	34	9.481	0.331	85.9	48.4986	0.0000	FLOOD RISK
30 minute winter	S4 (PP)	34	9.481	0.331	76.8	33.0846	0.0000	FLOOD RISK
30 minute winter	S8 (FC)	17	9.515	1.656	73.8	21.0552	0.0000	FLOOD RISK
15 minute winter	S7	12	9.567	1.225	35.0	0.3467	0.0000	FLOOD RISK
15 minute winter	S6	12	9.626	1.055	35.1	0.2987	0.0000	FLOOD RISK
15 minute winter	S5	12	9.746	0.947	18.5	0.2680	0.0000	FLOOD RISK
15 minute winter	S1	10	9.695	0.345	38.1	0.0978	0.0000	FLOOD RISK
15 minute winter	S2	10	9.593	0.341	55.1	0.0965	0.0000	FLOOD RISK
15 minute winter	S9 (Outfall)	12	7.662	0.027	2.9	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
30 minute winter	S3(Swale)	S3-S4	S4 (PP)	-17.7	0.482	-0.445	0.3812	
30 minute winter	S4 (PP)	S4-S8	S8 (FC)	53.7	1.904	0.536	1.3045	
30 minute winter	S8 (FC)	Outfall	S9 (Outfall)	2.9	1.329	0.069	0.0117	61.9
15 minute winter	S7	S7-S8	S8 (FC)	32.5	1.643	0.333	0.5493	
15 minute winter	S6	S6-S7	S7	35.0	1.846	0.492	0.4889	
15 minute winter	S5	S5-S6	S6	17.1	1.208	0.862	0.2152	
15 minute winter	S1	S1-S2	S2	37.1	0.933	0.876	0.5858	
15 minute winter	S2	S2-S3	S3(Swale)	53.9	1.449	1.432	0.7685	

Appendix H
Exceedance Routes



Key

Site Outline

Exceedance Routes

PRICE&MYERS

Job No.	29081	Page	SK 605	Rev	1
Date	2024 07 03	Eng	HD	Chd	
Job	Roehampton Gate Cafe				