LONDON'S GLOBAL UNIVERSITY

UCL

Civil, Environmental & Geomatic Engineering Department

CEGE0039: Urban Flooding and Drainage

Proposed SuDS Scheme for Brunel University (Zone E)

Preliminary Design Report

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1. Introduction

The site selected for this design scheme is Brunel University Zone E. A site plan of the existing conditions of the site is shown in Figure 1. The site's master plan was devised in the 1990s by Shepard Robson architects and therefore is assumed to not currently have any SuDS installed on site[1]. This study's objective is to assess the current runoff conditions of the site and accordingly propose a SuDS scheme that will restore the site to its greenfield pre-development conditions. Data on the existing drainage design of the site is not available, and so the scheme will be designed making educated assumptions on the existing drainage conditions. To simplify the design and calculation process, the site was split into 7 sub-catchments, generally separated by major road networks and water bodies.

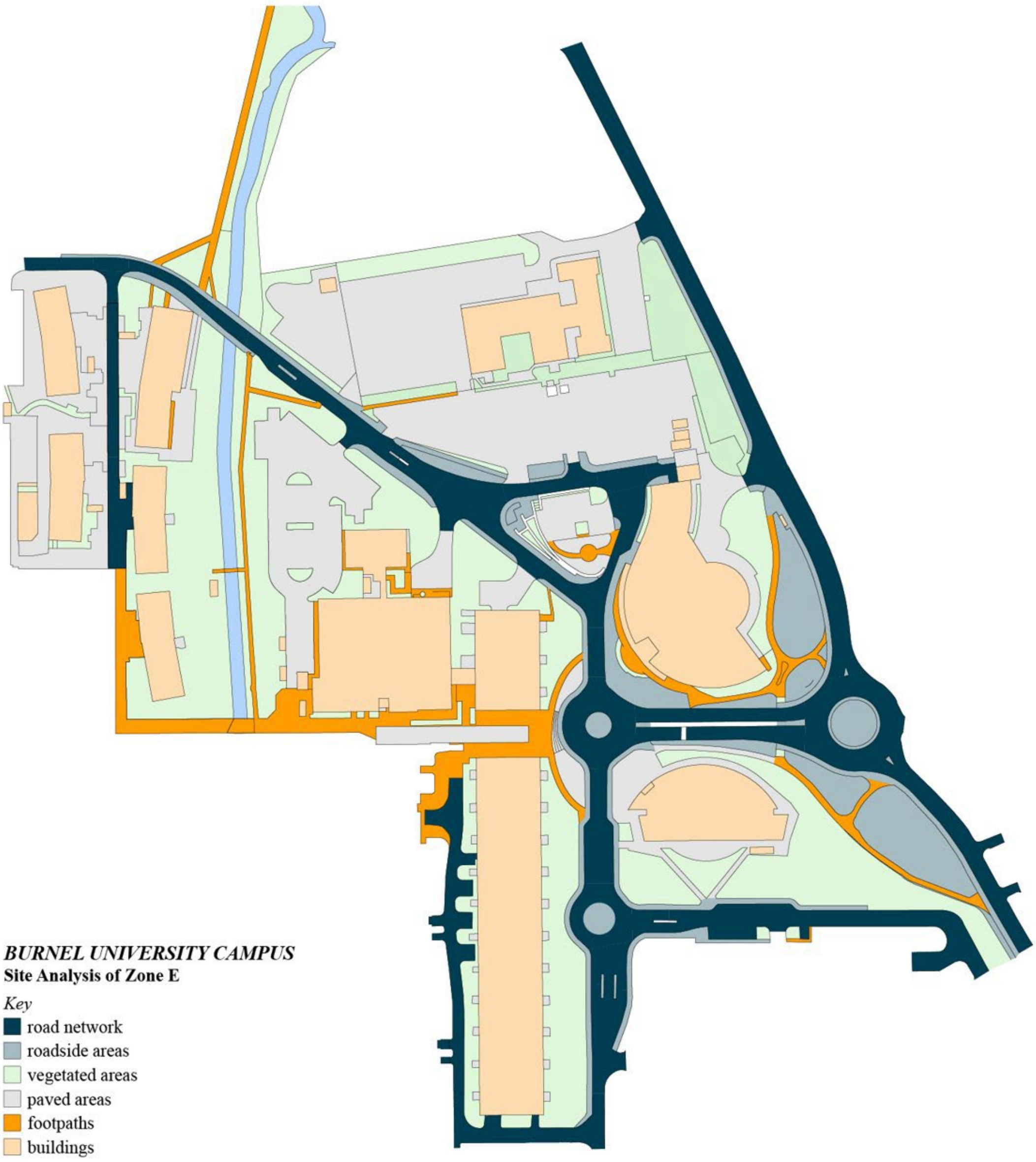

Figure 1: Original site plan

2. *Site Information*

Figure 2: Site soil characteristics

Figure 2 highlights a few important site characteristics that to a large extent determined the SuDS scheme design to follow. As is shown, the site is crossed by a river in the north-south direction which creates a high flood risk zone adjacent to it (zone 3) and a medium flood risk zone (zone 2) in some inhabited areas alongside it. The fact that there already exists a high risk of flooding on the site means that the river can't be used as a direct outflow for the site's water management - and instead any water injected into the river after a storm must have its flow rate carefully controlled.

As a second comment, the second and third figures (in figure 2) show the soil conditions of the site. Geological maps of the area revealed that most of the site is composed of silt and clay soils (impermeable soils), while zone 3 is predominantly composed of gravel, making the south east side of the site the only permeable area. Borehole data taken in the proximity of the site show that the first 8 meters of soil in the general area are actually gravel, meaning that some permeability may be allowed even in the rest of the zones. Nevertheless, in the design of the Suds layout, only zone 3 was assumed to be permeable, with all other zones taken as completely non-permeable. This was done in order to account for the worst case scenario, given that borehole data of the exact site are not available. In the case that some permeability is allowed in the rest of the site, that would only improve the performance of the suds scheme.

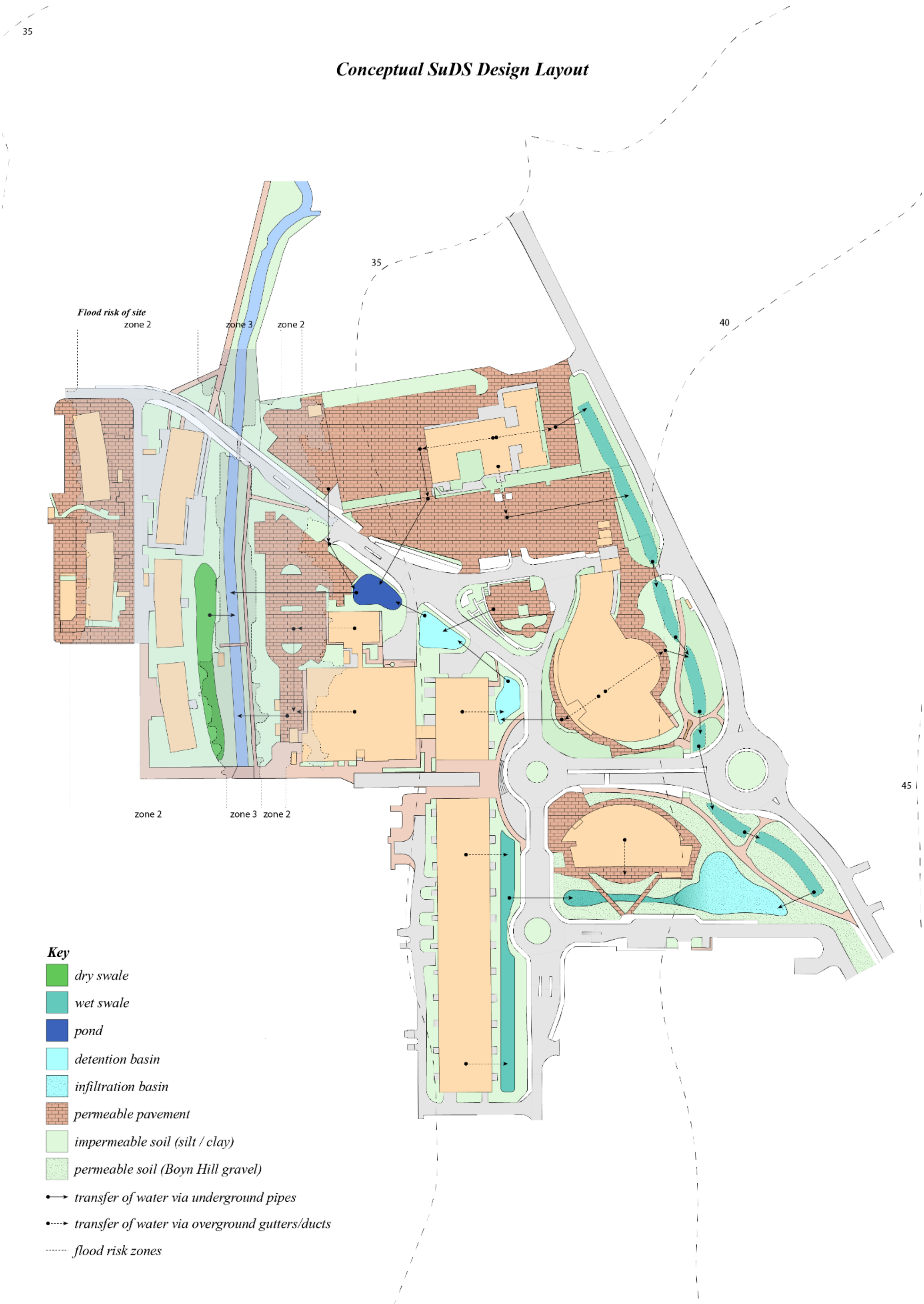

Figure 3: Initial concept SUDS layout

Figure 3 shows the concept design layout for the proposed SuDS scheme. The design was conducted following a downstream strategy, where the site's two key outlets were identified: 1) the river and 2) the permeable soil of zone 3. Given this, the selected design attempts to redirect all runoff from each zone to one of the two outlets using the ground's natural north-south sloping. As seen, conveyance swales are largely used to transfer water from the impermeable zones to zone 3 and then into an infiltration basin that allows the runoff the slowly infiltrate into the soil. On the west side of the site, detention basins and ponds are used to collect runoff and slow down its outflow rate before injecting it into the river. Permeable pavement is largely used to collect runoff from the buildings, redirecting it to one of the basins. All buildings are fitted with a rainwater harvesting system that partially provides for the water needs of the university and concurrently decreases the total runoff volumes to be managed (to approximately 30% of their original value). Finally, a catch basin is integrated into the permeable pavement adjacent to the river in order to again provide some flow control. The figure above shows only a first iteration of the suds design that will be revised once the relevant sizing calculations have been completed.

3. Hydrological Calculations

Preliminary hydrologic calculations were undertaken to determine the peak runoff rate and runoff volume estimation for a return period of 5, 30, and 100 years. As a first measure, the site's size was determined to be approximately 7 hectares. However, by examining the existing layout of zone E, it was conceivable to divide the total area into seven sub-catchments for the utilization of different Suds implementation:

Table 1: Sub-Catchment Areas

Catchment Area	***Area (m2)***
1	*13,176*
2	*6,554*
3	*11,349*
4	*10,423*
5	*21,325*
6	*2,022*
7	*4,658*
Total	*69,507*

The first check was for greenfield conditions. For this purpose, the 'IH 124' and the Rational methods were used and compared. These two methods are commonly used to estimate peak runoff rates and runoff volumes for small catchment areas[2] (<~100 ha for the rational method and <~50 ha for the IH 124) and, therefore, are deemed adequate for this analysis.

3.1. IH 124 Runoff Estimation Approach:

This method is applied only to greenfield conditions. Hence, by using the following three-stage process for predicting flood frequency (as specified in the CIRIA SuDS manual), the greenfield runoff could be obtained:

1. Estimation of the mean annual flood (Q_{bar}) - This estimation was based on the following equation:

$$Q_{bar(rural)} = 0.00108 AREA^{0.89} \times SAAR^{1.17} \times SOIL^{2.17}$$

Where:

$Q_{bar, rural}$ = Mean Annual Flow (m3 /s).

AREA = Catchment Area (km^2) - Based on common practice, since the development is smaller than 50 ha, the greenfield discharge rate should be calculated using 50 ha in the equation.

SAAR = Standard Average Annual Rainfall for the period 1941 to 1970 (mm). The site was determined to be within location 6 using the hydrological region map (*Appendix 1*); hence, 700 mm was picked as the SAAR value (*Appendix 2*).

SOIL = Soil Index from Wallingford Procedure Volume 3. Soil type 4 = 0.47 was used (*Appendix 3*).

2. For the 5, 30, and 100-year return periods, Q_{bar} is factored in using the UK FSR regional appropriate growth curves (*Appendix 4*)

3. Peak flow rates were obtained as the product of Q_{bar} and the relevant growth curve factor. Results are then derived by dividing the size of the actual site (of each sub-catchment) by the applied area (50 ha). The table below depicts each peak flow rate for a 100 years return period:

Table 2: IH 124 Greenfield Runoff Rates

Sub catchment	***Flow rate***	***(l/s)***
Zone 1	Q_{bar} Qg_{100}	*6.36* ***20.29***
Zone 2	Q_{bar} Qg_{100}	*3.16* ***10.09***
Zone 3	Q_{bar} Qg_{100}	*5.48* ***17.48***
Zone 4	Q_{bar} Qg_{100}	*5.03* ***16.05***
Zone 5	Q_{bar} Qg_{100}	*10.30* ***32.84***
Zone 6	Q_{bar} Qg_{100}	*0.98* ***3.11***
Zone 7	Q_{bar} Qg_{100}	*2.25* ***7.17***

3.2. Modified Rational Method:

The rational method is widely used and offers a simple and easy-to-understand design tool[3]. In this approach, the runoff rate is directly derived using the relevant rainfall intensity acquired from the IDF curves. Moreover, it is also dependent on the runoff coefficient C, which may be regarded as a function of the soil type. In this analysis, the runoff coefficient is set (from *Appendix 5*) as cp=0.1 for pervious areas (greenfield) and ci=0.9 for impervious areas (developed). As opposed to the previous method, this approach is applicable to both greenfield and developed conditions. Calculations for this method used the following equation:

$$Q = 2.78\, C\, i\, A$$

Where:

Q = design event peak rate of runoff (l/s)

C = non-dimensional runoff coefficient

i = rainfall intensity for the design return period (mm/hr) – (*Appendix 6)*

A = total catchment area being drained (ha)

The table below depicts each peak flow rate for both greenfield and developed conditions for a 100 years return period:

Table 3: Modified Rational Method Runoff Rates

Sub catchment	***Flow rate***	***(l/s)***
Zone 1	Qg_{100} Qd_{100}	***16.47*** *116.14*
Zone 2	Qg_{100} Qd_{100}	***8.19*** *65.60*
Zone 3	Qg_{100} Qd_{100}	***14.19*** *101.00*
Zone 4	Qg_{100} Qd_{100}	***13.03*** *92.74*
Zone 5	Qg_{100} Qd_{100}	***26.66*** *168.64*
Zone 6	Qg_{100} Qd_{100}	***2.53*** *21.52*
Zone 7	Qg_{100} Qd_{100}	***5.82*** *50.50*

The comparison between the two methods for greenfield states illustrates that the IH 124 approach is on the conservative side, resulting in a potential overdesign. However, there are certain drawbacks to the rational method as well, such as the necessity to define values for the runoff coefficient and period of concentration, or oversimplification, which may neglect certain crucial complications[4]. However, as the rational approach analysis was performed for the worst-case scenario, the discharge rates for greenfield appear more sensible.

3.3. Runoff Volume estimation

To defend downstream areas from increasing flood risk caused by the development, it is critical to assess greenfield or developed runoff volume and determine the maximum runoff volume that can be discharged from the development site. Volume runoff estimations were made for the worst-case scenario, predicated on a 100-year, 6-hour storm event[5], adding a 40% of climate change and a 10% tolerance for urban creep. For that, the following equation was used:

$$Runoff\ Volume = (SPR\ or\ PR) \times Catchment\ Area\ \times Rainfall\ Depth$$

Where PR stands for the Variable UK runoff model and can be computed from the below equation:

$$PR = IF\ \times PIMP + (100 - IF \times PIMP) \times \frac{NAPI}{PF}$$

Where:

PR = percentage runoff

IF = effective paved area factor (0.7 suggested according to CIRIA manual)

PIMP = percentage impermeability (0-100)

PF = soil moisture depth (normally be set at 200 mm)

NAPI = 30-day antecedent precipitation index (for soil type 4, 25 mm is recommended)

A summary table of the computed Peak discharge Volume runoff for each sub-catchment produced from the rational method is shown below:

Table 4: Rational Method Volume Runoff for a 100 years, 6 hours event

Sub catchment	***Volume runoff (m3)***
Zone 1	*567*
Zone 2	*323*
Zone 3	*494*
Zone 4	*453*
Zone 5	*815*
Zone 6	*107*
Zone 7	*251*

However, as the IH 124 method consider only greenfield condition, for assessing the influence of the existing developed runoff volume, the Percentage Runoff model (NERC, 1985) is used as follow:

$$PR_{RURAL} = SPR + DPR_{CWI} + DPR_{RAIN}$$

Where:

PR_{RURAL} = total percentage runoff for the greenfield catchment for a particular event.

SPR = standard percentage runoff.

DPR_{CWI} the dynamic component of the percentage runoff.

DPR_{RAIN} = the second dynamic component that increases the percentage runoff from large rainfall events.

Below are the obtained values for the computed Peak discharge Volume runoff for each sub-catchment in accordance with the IH 124 approach (for greenfield volume runoff) and the Percentage Runoff model (considering developed conditions):

Table 5: PR Model Volume Runoff for a 100 years, 6 hours event

Sub catchment	***PR_{RURAL} Volume runoff (m3)***
Zone 1	*625*
Zone 2	*311*
Zone 3	*538*
Zone 4	*494*
Zone 5	*1012*
Zone 6	*96*
Zone 7	*221*

Overall, the outcomes of both analyses are relatively similar, although the rational method appears to be slightly less conservative and more sensible to employ. For the sake of consistency and adherence to one approach, it can be concluded that the results generated by the rational method will be utilized in the subsequent SUDS design.

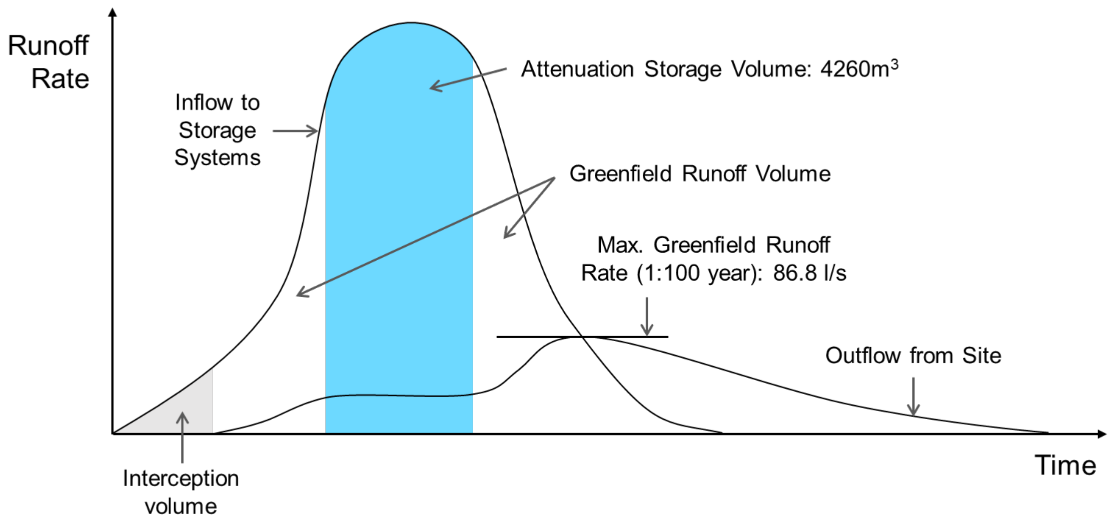

Figure 4: Interception, attenuation, and volume control [6] (after Kellagher, 2013)

- *(The complete calculation process of this segment can be found in Appendix 7).*

4. *Pervious Pavement*

Given that more than 80% of the site is covered with impervious paved surfaces, installation of pervious pavements can significantly reduce the amount of surface runoff in the catchment area during rainfall events. In particular, two types of pervious pavements are proposed as part of the SuDS scheme. These include pervious pavements designed for the carparks as well as the pedestrian footpaths/driveways around some of the buildings.

4.1. *Carpark*

The coverage of pervious pavement in the four carparks are illustrated in Figure 5. The base area (A_b) and drained area (A_D) of each carpark is tabulated in Table 6. Flow control points will be included to manage the discharge rate of stored water into the River Pinn or adjacent swales. This ensures that the discharge of stored water into the River Pinn will not exceed the limiting greenfield rates.

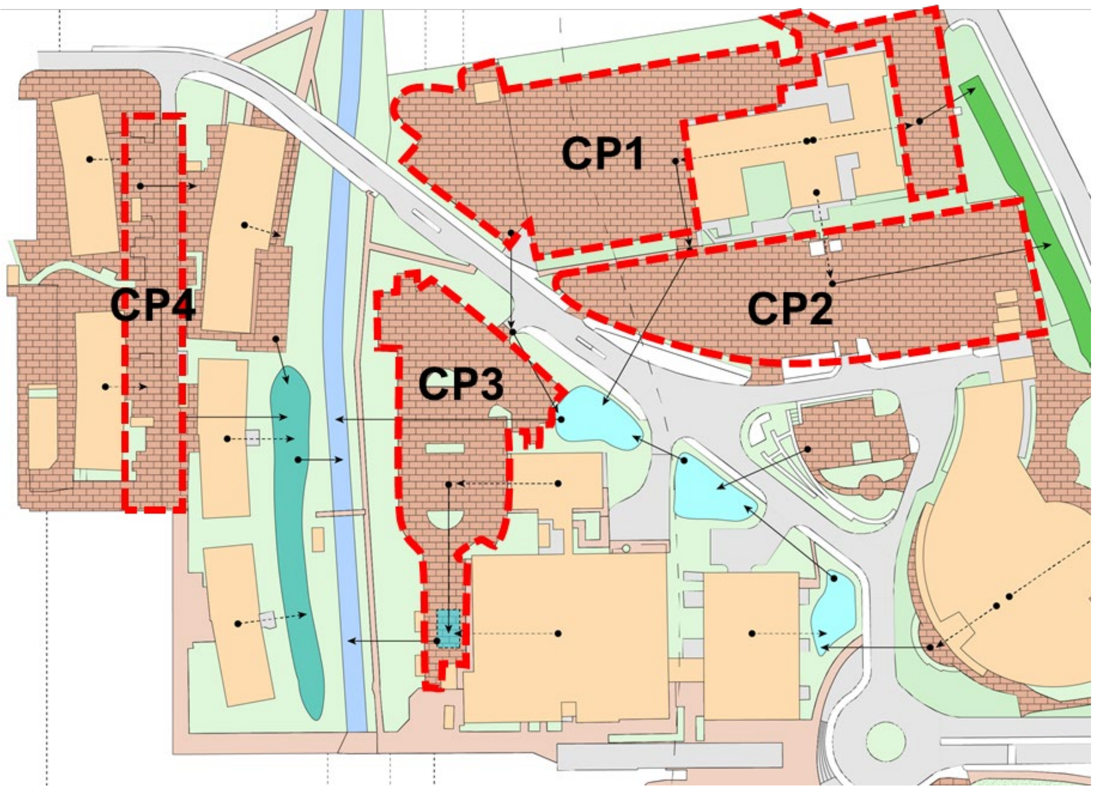

Figure 5: Proposed pervious pavement scheme for carparks

Table 6: Summary of base area and drained area (carparks)

	A_b (m^2)	A_D (m^2)	$R = A_D/A_b$
CP-1	2701	3892	1.5
CP-2	2127	3190	1.5
CP-3	1572	2358	1.5
CP-4	819	1228	1.5

4.1.1. Structural Design

Langley silt is found at the formation level of all four carparks. As such, the equilibrium CBR of the subgrade is estimated to be 3% for preliminary design. On the other hand, the school zone carparks are mainly trafficked by passenger cars and occasional light commercial vehicles, therefore, the loading category shall be classified as Traffic Category 4 (CIRIA, 2015). Following the recommendations from CIRIA (2015), the design thickness of each pavement layer is illustrated in Figure 6. Since the subgrade CBR is less than 5%, an additional capping layer of 225*mm* thick coarse-graded aggregate (CGA) is required to improve the foundation strength (CIRIA, 2015).

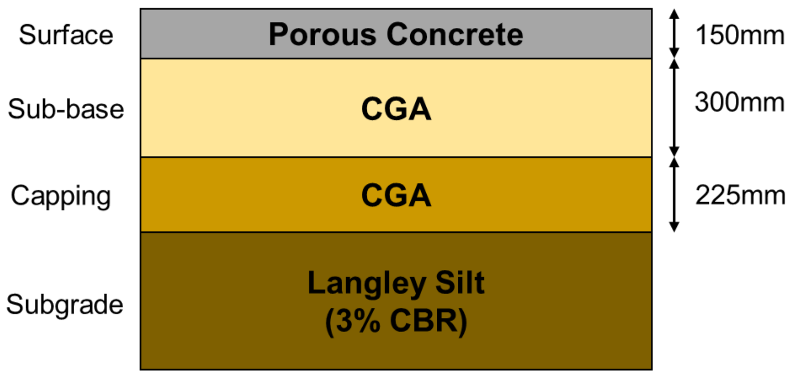

Figure 6: Proposed structural design of pervious pavement at carparks

4.1.2. *Hydraulic Design*

The estimated infiltration rate of Langley silt is estimated to be 1×10^{-6} *m/s*. Given that the drained areas of all four carparks are in excess of 1,000m^2, a safety factor of <u>5</u> shall be applied to obtain the design infiltration rate (q), as follows (CIRIA, 2015):

$$q = 1\times10^{-6} \div 5 = \underline{2\times10^{-7} m/s}$$

Due to the low permeability of Langley silt, the "*No Infiltration* (*Type C*)" pavement system was selected. The maximum depth of water in the storage layer (h_{max}) for various combinations of storm duration and rainfall intensity are estimated based on the following equation, and the results are tabulated in Table 7.

$$h_{max} = \frac{D(Ri - q)}{n}$$

where, D = storm duration (*hr*), R = drainage ratio, i = rainfall intensity for 100-year rainfall event (*mm/hr*), q = design infiltration rate (*mm/hr*), and n = porosity. For coarse-graded aggregate, the porosity n is taken as 0.3, while the drainage ratio R is 1.5 for all four carparks.

Table 7: h_{max} for various storm duration and rainfall intensity

Duration	D (*hr*)	i (*mm/hr*)	h_{max} (*mm*)
15 *min*	0.25	105.2	130.9
30 *min*	0.5	68.0	168.7
45 *min*	0.75	51.4	190.8
60 *min*	1	41.8	206.4
2 *hr*	2	24.8	243.0
6 *hr*	6	10.3	294.9
24 *hr*	24	3.3	**342.3**

For a 100-year rainfall event, the highest value of h_{max} is 342*mm*, therefore, the storage depth of 525*mm* thick CGA layer will be sufficient (Figure 7).

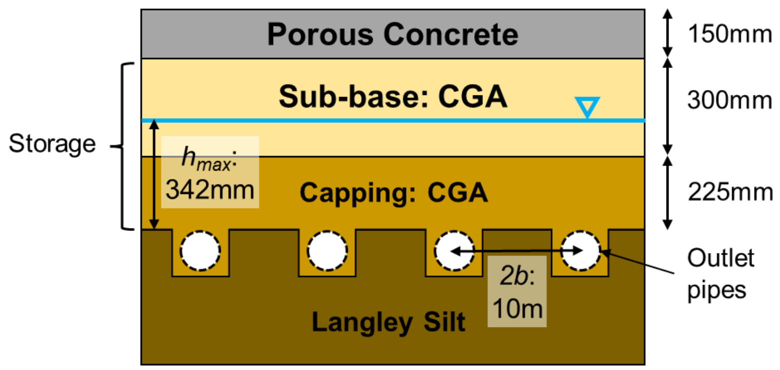

Figure 7: h_{max} for 100-year rainfall event; 24 hours duration

Considering an outflow pipe spacing ($2b$) of $10m$, as well as a sub-base permeability coefficient (k_{sb}) of $1\times10^{-3}m/s$, the pipe outflow rate (q_{out}) is estimated as follows:

$$q_{out} = k_{sb}(h_{max}/b)^2 = 1\times10^{-3} \times (0.342/5)^2 = 4.7 \times 10^{-6} m/s$$

Subsequently, the time to half-empty is determined as follows:

$$t_{1/2} = (n \times h_{max})/2q_{out} = 1/3600 \times (0.3 \times 0.342)/9.4\times10^{-6} = \mathbf{3.0\ hr} < 24 \text{ hr} \Rightarrow \text{Acceptable}$$

Overall, the available storage volume for each carpark is summarized in Table 8. As part of exceedance flow management, gullies that elevated slightly above the pavement will be installed to enable some ponding above pavement surface to be used as additional storage (CIRIA, 2015).

Table 8: Available storage volume

	Base area (m^2)	Storage volume (m^3)
CP-1	2701	1418
CP-2	2127	1117
CP-3	1572	825
CP-4	819	430

4.2. Pedestrian Footpath and Driveway

The coverage of pervious pavement for pedestrian footpath and driveway are illustrated in *Figure 8*. The base area (A_b) and drained area (A_D) of each carpark is tabulated in Table 9.

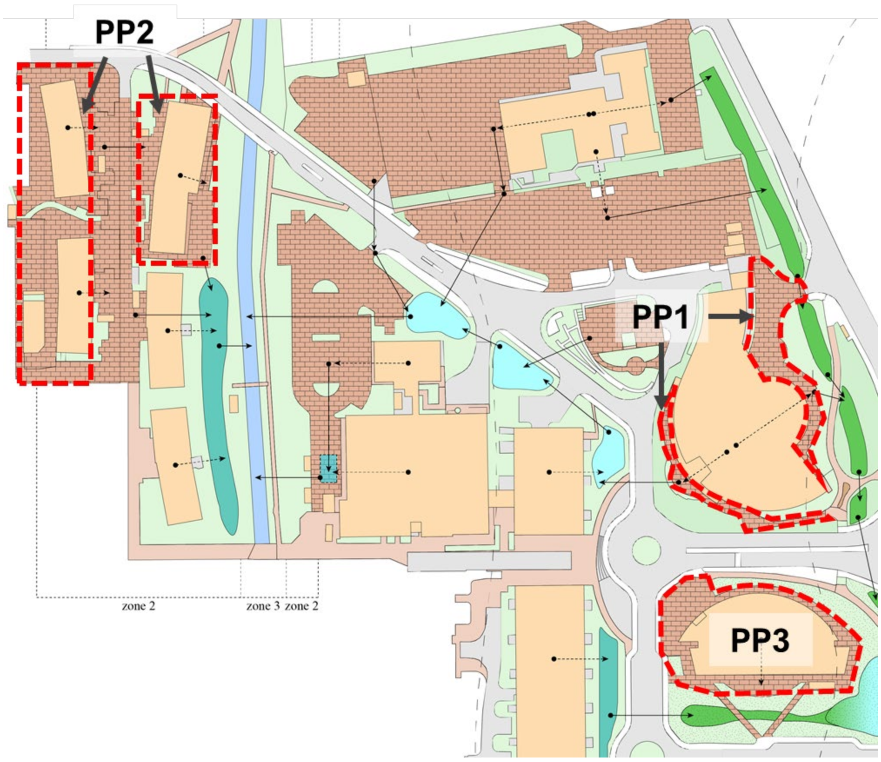

Figure 8:Proposed pervious pavement scheme for pedestrian footpaths

Table 9:Summary of base area and drained area (carparks)

	A_b (m^2)	A_D (m^2)	$R = A_D/A_b$
PP-1	873	873	1.0
PP-2	2324	2324	1.0
PP-3	1052	1052	1.0

4.2.1. Structural Design

The subgrade of PP-1 and PP-2 consist of Langley silt, while the subgrade of PP-3 consists of Boyn Hill gravel. As such, the subgrade CBR of PP-1 and PP-2 is estimated to be 3%, while the subgrade CBR of PP-3 is approximately 15%. The pervious pavements at these areas are classified as Traffic Category 2 (CIRIA, 2015). The recommended pavement thickness for PP-1 and PP-2 is shown in Figure 9, while the pavement thickness of PP-3 is illustrated in Figure 10 (CIRIA, 2015).

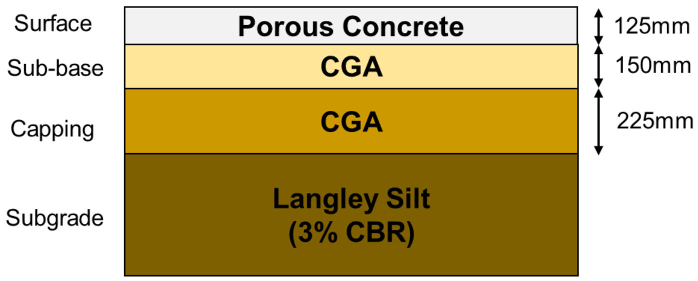

Figure 9:Proposed structural design for PP-1 & PP-2

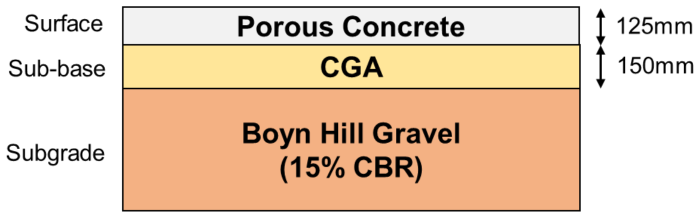

Figure 10:Proposed structural design for PP-3

4.2.2. Hydraulic Design

For pedestrians pavements and driveways, consequences of failure are relatively minor. Therefore, a safety factor of 5 is applied to the infiltration rate, as follows:

PP-1 & PP-2 (Langley Silt): $q = 1.0\times10^{-6} \div 5 = \underline{2.0\times10^{-7} m/s}$

PP-3 (Boyn Hill Gravel): $q = 9.5\times10^{-4} \div 5 = \underline{1.9\times10^{-4} m/s}$

Given the low permeability of Langley silt, PP-1 and PP-2 will adopt a "*No Infiltration* (*Type C*)" system type. On the other hand, PP-3 will adopt the "*Full Infiltration* (*Type A*)" system type. The maximum depth of water in the storage layer of PP-1 and PP-2 for various duration and intensity of the 100-year rainfall are tabulated in Table 10.

Table 10: h_{max} for various storm duration and rainfall intensity (PP-1 & PP-2)

Duration	D (hr)	i (mm/hr)	h_{max} (mm)
15 *min*	0.25	105.2	87.1
30 *min*	0.5	68.0	112.1
45 *min*	0.75	51.4	126.6
60 *min*	1	41.8	136.8
2 *hr*	2	24.8	160.4
6 *hr*	6	10.3	191.8
24 *hr*	24	3.3	**209.0**

The maximum water depth for the 100-year rainfall is 209*mm*, therefore, the storage depth of 375*mm* thick CGA will be sufficient for PP-1 & PP-2 (Table 10). For PP-3, due to the high permeability of the gravel subgrade, infiltrated rainwater will readily percolate into the subgrade (i.e., $Ri < q$). As such, provision of a porous surface is sufficient for PP-3, and no storage requirement is needed.

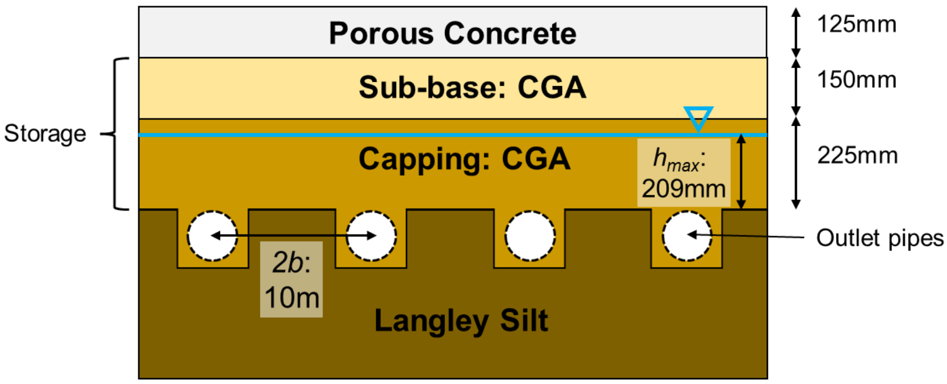

Figure 11: h_{max} for 100-year rainfall event; 24 hours duration

For PP-1 and PP-2, considering an outflow pipe spacing ($2b$) of 10m and sub-base permeability coefficient (k_{sb}) of $1\times10^{-3} m/s$, the pipe outflow rate (q_{out}) is estimated as follows:

$$q_{out} = k_{sb}(h_{max}/b)^2 = 1\times10^{-3} \times (0.209/5)^2 = 1.7 \times 10^{-6} m/s$$

Subsequently, the time to half-empty is determined as follows:

$$t_{1/2} = (n \times h_{max})/2q_{out} = 1/3600 \times (0.3 \times 0.209)/3.4\times10^{-6} = \mathbf{5.1\ hr} < 24 \text{ hr} \Rightarrow \text{Acceptable}$$

Overall, the available storage volumes for PP-1 and PP-2 are summarised in Table 11.

Table 11: Available storage volume

	Base area (m^2)	Storage volume (m^3)
PP-1	873	327
PP-2	2324	872

5. *Swale Design*

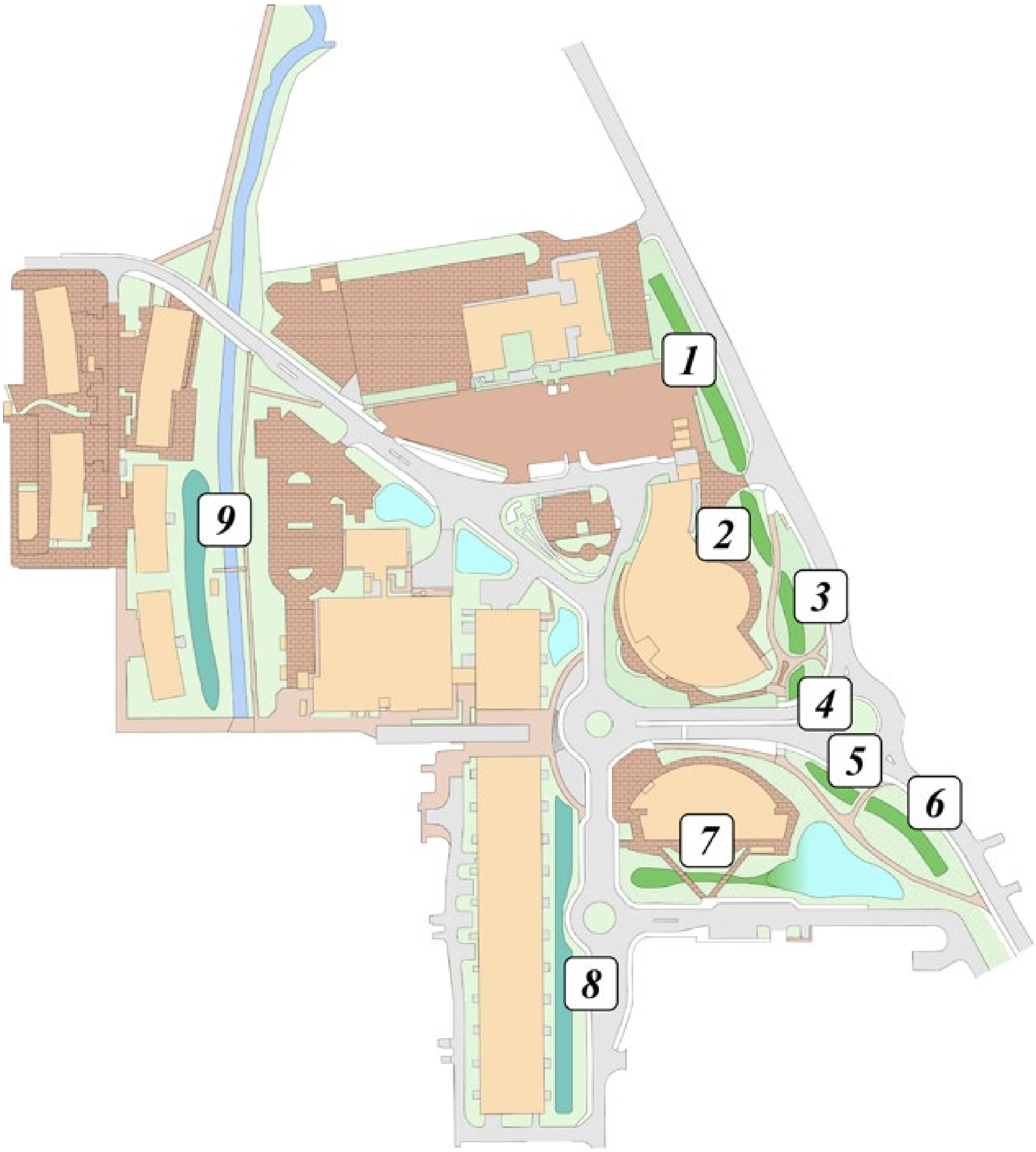

Figure 12:Swale routing proposal

Table 12: Summary of geometrical characteristics of swale

Parameter	Symbol	Value	Source
Length	L	95 m	Measured from site map
Width (at base)	B	1.2 m	Assumed; Recommended 0.5 – 2 m
Height	H	0.4 m	Assumed; Recommended 0.4 - 0.6 m
Slope (side)	-	1:3	Assumed; Recommended 1:3
Longitudinal gradient	S	0.01	Measured from site map
Angle α	*A*	1.25 rad	Calculated as arctan (3/1)
Flow depth	h	0.2 m	Wet swale, so unrestricted

All geometrical characteristics were informed by the site and/or the recommendations given in CIRIA (2015). Considering that swales 1- 6 are continuous in plan, the geometry was kept identical for all. Given the geometrical restrictions and large catchment area of the given swales, it is initially assumed that the swales will operate as *wet swales*, meaning that water depths are *not* restricted to below 100mm. Therefore, an initial flow depth of 0.2m is assumed. Manning's Equation for flow is used to calculate the swale's capacity:

$$Q = \frac{A\, R^{\frac{2}{3}}\, S^{\frac{1}{2}}}{n}$$

Where,

A – cross sectional area of the swale, calculated as $\frac{bh+h^2}{\tan(\alpha)}$

S – longitudinal slope

R – hydraulic radius calculated as *A/P*, where *P* is the wetted perimeter of the swale, calculated as $\frac{b+2h}{\sin(\alpha)}$.

n – Manning's Roughness Coefficient (depends on the soil/ground conditions and on the height of planted grass). A value of 0.05 will be assumed for high grass and 0.03 for low grass (equal to or below the water level).

The calculated components of the Manning's Equation for swale 1 are shown in the table below.

Table 13: Calculation of swale 1 capacity and velocity

Parameter	Symbol	Value	Source
Cross sectional area	A	0.25 m^2	Calculated
Wetted perimeter	P	1.62 m	Calculated
Hydraulic radius	R	0.4 m	Calculated
Manning's Coefficient	n	0.05	For high grass
Swale capacity	Q	146.9 l/s	Calculated
Swale velocity	v	0.58 m/s	< 1m/s, calculated as V = Q/A

The first check to be completed at this stage is that the swale velocity at the given flow depth is below 1m/s in order to minimize erosional effects. Since this check is met under the current design, the swale's capacity can be checked by ensuring that the total inflow into the swale is less than the calculated swale capacity Q. Each swale's contributing catchment area is calculated using the drawn site map, and is used in the equation for the total runoff rate below:

$$Q_R = C \times i \times A_R$$

Where,

C – runoff factor that ranges from 0.1 for green areas to 0.9 for concrete areas

i – rainfall intensity for a 100-year event of 15-minute duration (= 105 mm/hr for London)

A – contributing catchment area [m^2]

The runoff factor for each contributing area is calculated by assuming 0.9 for all paved/built areas and 0.1 for all green areas. For example, 40% of the catchment area for Swale 1 is green, and so the runoff factor was calculated as $C_1 = 0.1 \times 0.4 + 0.9 \times 0.6$. The table shows the calculation of the total inflow into the swale and confirms that the inflow is less than the swale's capacity thus completing the capacity check.

Table 14: Calculation of swale inflow rate

Parameter	Symbol	Value	Source
Runoff area	A_R	6222 m^2	Measured from site map
Runoff factor	C	0.58	Calculated
Rainfall intensity	i	105 mm/hr	100-year event, 15 min duration, London
Runoff rate	Q_R	105.2 l/s	< 146.9 l/s, **check verified**

The final check that needs to be conducted is the swale flow velocity in case of exceedance. In the scenario where the design event is exceeded in intensity and the swale overflows, the velocity must remain under 1 m/s to avoid erosion. Therefore, the previous calculation for flow velocity is redone but this time assuming a flow depth equal to the swale's height. The table below verifies this final check

Table 15: Swale 1 velocity check in case of exceedance

Parameter	Symbol	Value	Source
Height	H	0.4 m	Equal to flow depth
Cross sectional area	A	0.53 m^2	Calculated
Wetted perimeter	P	2.04 m	Calculated
Hydraulic radius	R	0.26 m	Calculated
Manning's Coefficient	n	0.05	For high grass
Swale velocity	v	0.82 m/s	< 1m/s, calculated as V = Q/A

The same process is followed for the rest of the swales. The same geometry is maintained for swales 1-7, since all the relevant checks were verified. Swale 8 is also assumed to function as a wet swale with a 0.2m flow depth, but a slightly larger base width. Swale 9 due to its adjacency to buildings and the river is assumed to function as a dry swale. Its design process is identical to the one shown for Swale 1, except for the fact that the maximum allowable flow depth is 0.1m. Due to this restriction, the swale base width had to be increased to 2m to allow for a larger swale capacity, and the total swale height was decreased to 0.2 m, to better control maximum velocities in the case of exceedance. Figure 13 shows a summary of the calculated swale geometries. The table below summarizes the total swale capacities and velocities for all swales.

Table 16: Summary of the total swale capacities and velocities

Swale Number	Capacity (l/s)	Velocity (m/s)
1	147	0.58
2	*147*	0.58
3	*147*	0.58
4	*147*	0.58
5	*147*	0.58
6	*147*	0.58
7	*49*	0.39
8	*187*	0.59
9	118	0.58

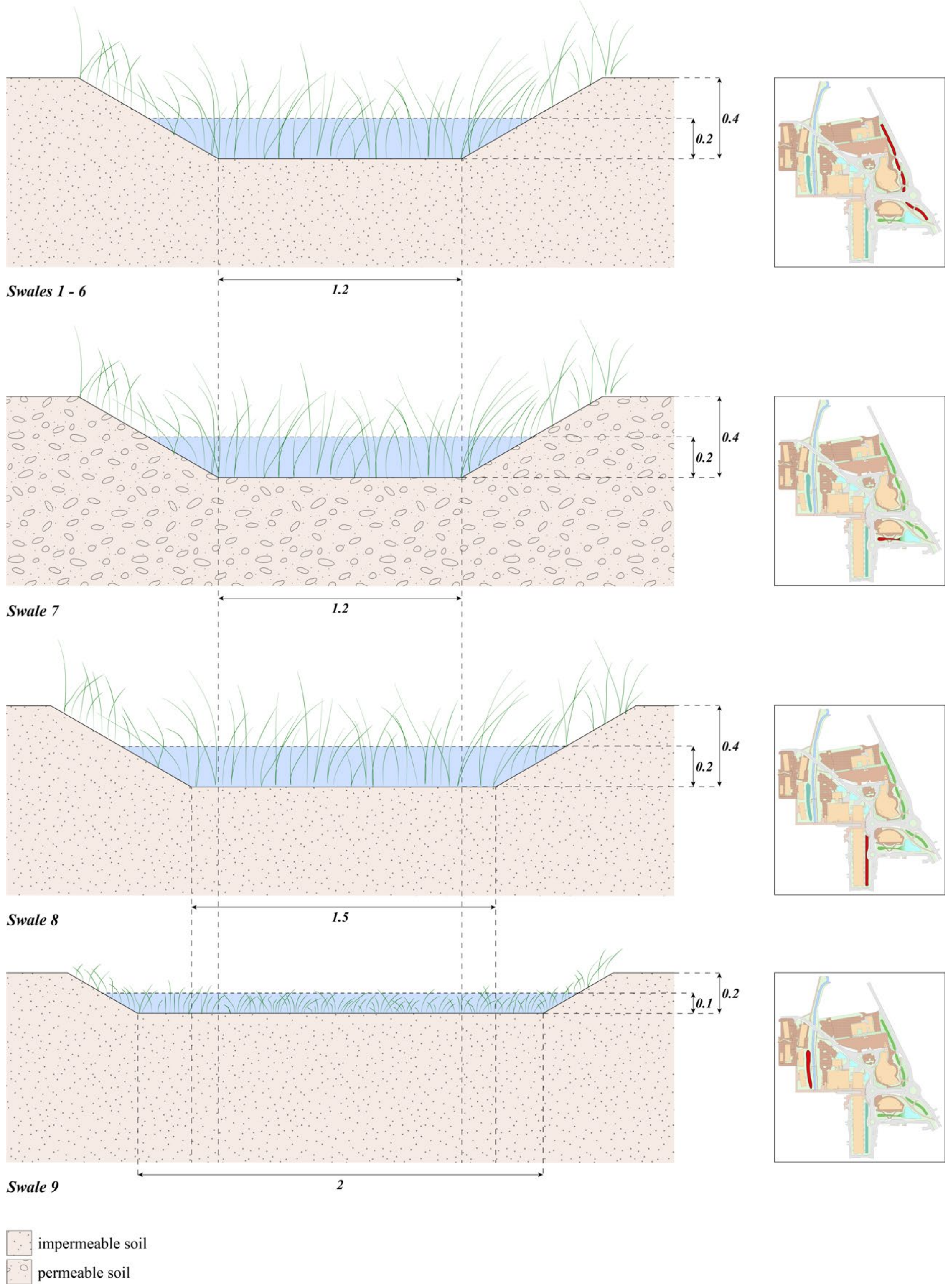

Figure 13: Swale sizing diagrams

6. *Detention Basin and Pond*

At the centre of the site, the water is conveyed through two detention basins and a pond, which then discharges the stored water into the river. These systems will have an inflow of surface run off from the appropriate catchment zones as illustrated in figure 14.

Figure 14: Catchment areas for basins and pond

Basins and ponds are to be designed for 1:100-year return period for a 6-hour duration rainfall event. The modified rational method is utilised for calculations of the greenfield and developed flow rates (as shown in *Appendix 8)*.

6.1. *Detention Basin*

The depressed areas of land, allow for the water to be stored and conveyed to the pond. The detention basins will discharge at a controlled flow rate of 0.84l/s with an orifice of 20mm diameter.

6.1.1. *Key Design Parameters*

The pond is designed, as per the SuDs manual with the average length to width ratio of the basin to be between 3:1 and 5:1 and the side slopes of a vegetated basin to have a maximum ratio of 1V:3H. The depth of basin is designed not to exceed 2m.

6.1.2. *Hydraulic Design Calculations*

<u>*Sizing of Basin*</u>

Initially, the storage volume required considers the inflow of catchment area (accounting for climate change and creep allowance of 1.5) of developed run off including: 30% of roof area, green space, and pavement. For detention basin 1, the outfall from the previous pavements is also considered to discharge at a controlled flow rate, considering a 30mm diameter orifice. Table 17 summarises the storage calculations with volumes calculated using a Δt of 6 hours.

Table 17: Storage calculation for basin

	Inflow Flow Rate, I (l/s)		Outflow Flow Rate, O (l/s)	Storage Volume (m^3) = (I-O)Δt	Required Design Storage Volume (m^3)
Detention Basin 1	Developed Run Off	1.95	0.84	44.93	56.17
	Permeable Pavement	0.97			
Detention Basin 2	Developed Run Off	2.35	0.84	50.70	63.38
	Detention Basin 1	0.84			

The outflow flow rates shown in Table 17 are calculated using the equation with values of $C_d = 0.6$ and $g = 9.81ms^{-2}$:

$$Q = C_d \times A_0 \times \sqrt{2gh} = 0.6 \times \frac{\pi * 0.02^2}{4} \times \sqrt{2 \times 9.81 \times depth\ of\ water\ level}$$

The total volume is increased by factor of 1.25, shown as the design storage in Table 17. In addition to the required depth of the pond, a freeboard height of 300mm is added to account for additional storage in the case for a more severe rainfall event. Accounting for these safe design practices, the basins are of following dimensions as shown in figure 15 and 16.

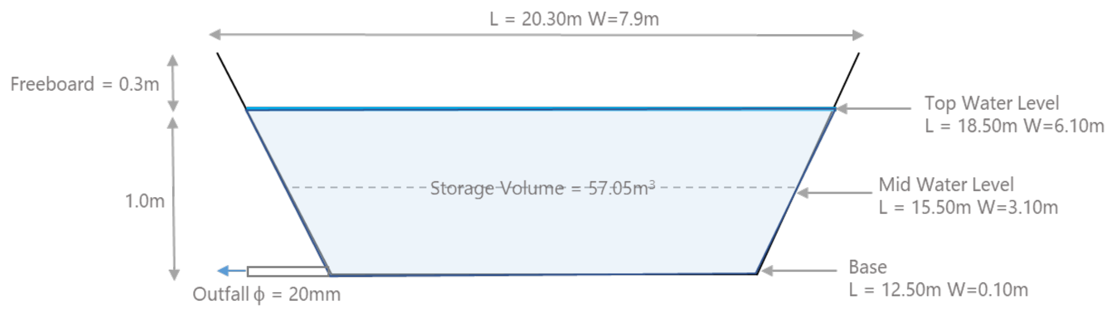

Figure 15: Sizing of detention basin 1

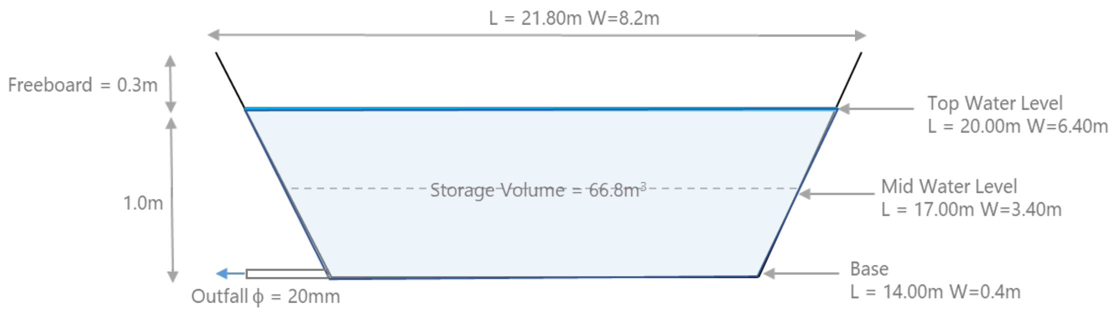

Figure 16: Sizing of detention basin 2

6.2. Pond

A pond will be placed within the central region of the plot and conveys water from the adjacent detention basin as well the catchment to northeast of the plot. As this catchment area includes car parking spaces, the runoff will need to be treated which is achievable with the temporary storage of water. The pond will enhance the area as a key landscape amenity to increase biodiversity with the plantation of native flora on the base and within the dry zone of the pond. The sides of the pond will not be planted due to the lack of site space for vegetation zones. The water stored is then directly discharged to the river

6.2.1. Key Design Parameters

The pond is designed in accordance with the SuDs manual which suggests a maximum average length to width ratio between 3:1 to 5:1 and side slopes of 1V:3H. The maximum depth of temporary storage is limited to 0.5m and permanent pool should not exceed 1.2m to avoid stratification and anoxic conditions. However, some depth of minimum 0.6m should be included to counteract the risk of the pond drying up when rainfall is low.

Safety benches and maintenance access routes should be provided at an appropriate level. Suitable width for a safety bench is 3.5m, dependent on land availability and with a slope less than 1:15. However, due to the site size constraint, a limitation of the design is that only a 3m safety bench width is provided as shown in figure 17.

6.2.2. Hydraulic Design Calculations

<u>*Treatment Volume*</u>

The first 15mm of rainfall depth falling on 3647m^2 of the catchment area (car park), will require treatment and assuming an average run off factor of 0.5:

$$Treatment\ Volume = 0.015 \times 3647 \times 0.5 = 27.35m^3$$

Therefore, the base area of the pond is calculated to be 12.5m and 1.2m in length and width with a depth of the permanent water level is at 0.6m. This allows the pond to have a permanent water volume of 27.69m^3.

Sizing of Pond

The temporary water level above the permanent water, allows for the attenuation of surface run off. To find the required storage for the pond, the methodology outlined in Section 6.1.2 will be used. The outflow flow rate of the pond will be at the greenfield run off rate for the entirety of the catchment for detention basin 1, detention basin 2 and the pond, as all of surface run off will be discharged through the pond. The developed run off will only consider the pond catchment area as other surface run off are conveyed at a controlled flow rate. Breakdown of storage volume can be seen in table 18:

Table 18: Storage calculation for pond

	Inflow Flow Rate, I (l/s)		**Greenfield Flow Rate, O (l/s)**	**Storage Volume (m^3) (I-O)Δt**	**Required Design Storage Volume (m^3)**
Pond	Developed Run Off	1.15	2.23	15.90	19.80
	Permeable Pavement	0.97			
	Detention Basin 2	0.84			

Dimension of the pond and key features are highlighted in figure 17:

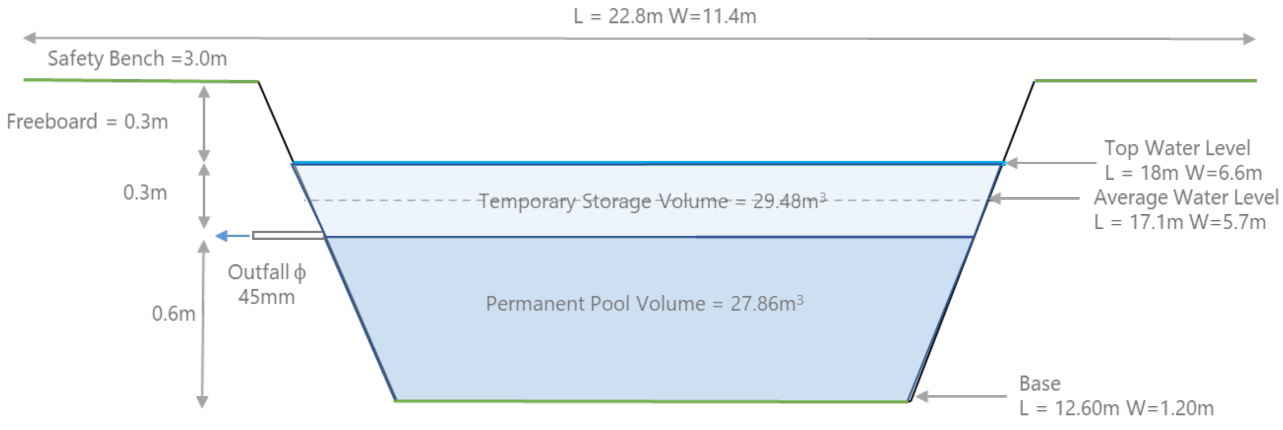

Figure 17: Sizing of pond

Orifice Size

The pond is designed to outfall at greenfield rate, Q =2.23l/s, and the diameter of the orifice is calculated using the values of $C_d = 0.6$. $g = 9.81ms^{-2}$ and $h = 0.3m$:

$$D = \sqrt{\frac{4Q}{\pi C_d \sqrt{2gh}}}$$

Thus, the diameter of the orifice is suggested to be 44mm and the design diameter of the orifice is 45mm for the pond respectively.

7. *Infiltration Basin*

An infiltration basin is utilised in the southeast corner of the plot, shown in figure 18, as the underlying soil profile consists of Boyn Hill Gravel which is permeable (see figure 2). These are landscape depressions that store surface water runoff and subsequently allow the water to percolate through the voids of permeable soil.

7.1. *Key Design Parameters*

Infiltration basins are required to have a side slope of 1V:3H and ensure that the depth does not exceed 2m. Due to the proximity of the basin to buildings, a factor of safety of 10 will be placed on the infiltration coefficient.

7.2. *Hydraulic Design Calculations*

The infiltration basin is to be considered as a 3D infiltration system and to find the maximum storage height the following equation is used:

$$h_{max} = \alpha\left[e^{(-bD)} - 1\right] \text{ where } \alpha = \frac{A_b}{P} - i\frac{A_D}{Pq},\ b = \frac{Pq}{nA_b}$$

The input parameters can be found in Table 19 and summary of the results in Table 20.

Table 19: Input design parameters

Base Perimeter of Infiltration System, P	146m
Base Area of Infiltration System, Ab	600m^2
Area to be drained, Ad	17211m^2
Ratio of drained area to infiltration area, R	28.69
Infiltration coefficient. q	9.4×10^{-5} m/s
Porosity of fill mater, n	1 (open structure)

Table 20:Maximum storage depth

Duration, D (h)	Intensity, I (mm/h)	α	b	hmax (m)
0.2500	150	-47.59	0.0832	0.98
0.5000	86	-25.53	0.0832	1.04
0.7500	62	-17.26	0.0832	1.04
1.0000	**50**	**-13.12**	**0.0832**	**1.05**
2.0000	28	-5.54	0.0832	0.85
6.0000	12	-0.03	0.0832	0.01
24.0000	4	2.73	0.0832	-2.36

From Table 20, the 1-hour duration rainfall for 1:100-year return period, obtains the maximum height required for the infiltration basin to be 1.05m, hence the depth of the basin is suggested to be 1.1m

The time take for the infiltration basin to half empty was also checked using the equation stated below and is equal to 1.3 hours which is less than the threshold time of 24 hours

$$Half\ Emptying\ Time = \frac{nA_b}{qP} log_e \left[\frac{h_{max} + \frac{A_b}{P}}{\frac{h_{max}}{2} + \frac{A_b}{P}} \right]$$

Subsequent to this verification, the basin is designed to have sufficient capacity to store water for 1:100-year return period and 6-hour duration rainfall event:

$$Volume\ of\ Infiltration = P \times q \times h \times dt = 329$$
$$Volume\ of\ Rainfall = A_D \times i \times dt = 1239$$
$$Required\ Storage\ Volume = 909m^3$$

Table 21: Summary of infiltration basin sizing

Base Plan Area (m^2)	600
Top Plan Area (m^2)	1145
Depth (m)	1.1
Storage Volume (m^3)	943

8. Rainwater Management System

A passive rainwater harvesting surface water management system with water conservation is implemented across the site. It accommodates the water demand for the buildings as well as a specific depth of rainfall during a large event. As the buildings are large we can be more certain of their demand, this increases the effectiveness of passive surface water control during a rainfall event. RWH will be implemented for all the building surfaces and they are assumed to be flat roofed, these are estimations given the data, so the resulting values will be general and indicative[7].

The simple method of calculating RWH storage capacity is as follows:

$$Y_R = A\ e\ AAR\eta \times 0.05$$

Table 22:Runoff yield Calculation

Parameter	Symbol	Value	Source
Collecting runoff area (m^2)	A	14338 m^2	Equal to flow depth
Runoff (yield) coefficient	e	0.8	Assuming flat roof without gravel for all buildings
Average annual rainfall depth (mm)	AAR	600	Taken from FEH map for site
Hydraulic filter efficiency (ratio)	η	0.9	Standard hydraulic filter value
Runoff volume (yield) litres	Y_R	309700.8	Calculated

$$D_N = P_d n \times 365 \times 0.05$$

Table 23: Non-Potable Water Demand Calculation

Parameter	Symbol	Value	Source
Daily demand per person (l)	P_d	26	Dm^3 per full time student, taken from a study in Warsaw, Poland[8]
Number of occupants	n	1000	calculation
Non potable water demand (l)	D_n	474500	Standard hydraulic filter value

$$V_{SC} = \frac{AR_d\beta\eta}{1000}$$

Table 24:Tank Storage volume calculation, simple method

Parameter	Symbol	Value	Source
Design storm rainfall depth (mm)	R_d	26.25	100 year event: 105mm/hr rainfall for 15minutes
Contributing runoff area (m^2)	A	14338	Taken from FEH map for site
Design storm event runoff coefficient	β	0.9	CIRIA SuDS recommendation
Hydraulic filter efficiency (ratio)	η	0.9	Standard hydraulic filter value
Storage volume (m^3)	V_{SC}	304862	Calculated

When:

$$\frac{Y_R}{D_N} < 0.7 \qquad Total\ Storage = V_{SC} + Y_R$$

Table 25:Total Tank Storage volume, simple method, water conservation + surface water management, passive control

Parameter	Symbol	Value	Source
Y_R/D_N	*Ratio*	0.653	Calculated
Total Storage (litres)		614,563	Calculated
Total Storage (m3)		615	calculated
Rainfall storage capacity (mm)		43	calculated

The rainfall storage capacity is larger than the 26.5mm demanded for a 100 year event. As the rainwater harvesting system is passive, a 70% harvesting rate during large events is implemented to account for the uncertainty. Consequently the other SuDS will take this reduction of building rainfall runoff into account.

9. Revised SUDS Layout Proposal

The sizing and calculation for each SUDS component revealed that some components, specifically the previous pavements, were oversized. Figure 18 shows the revised SUDS layout for the selected site. This proposal includes the resize of pervious pavements and the addition of three flow controls on the inlets into the river.

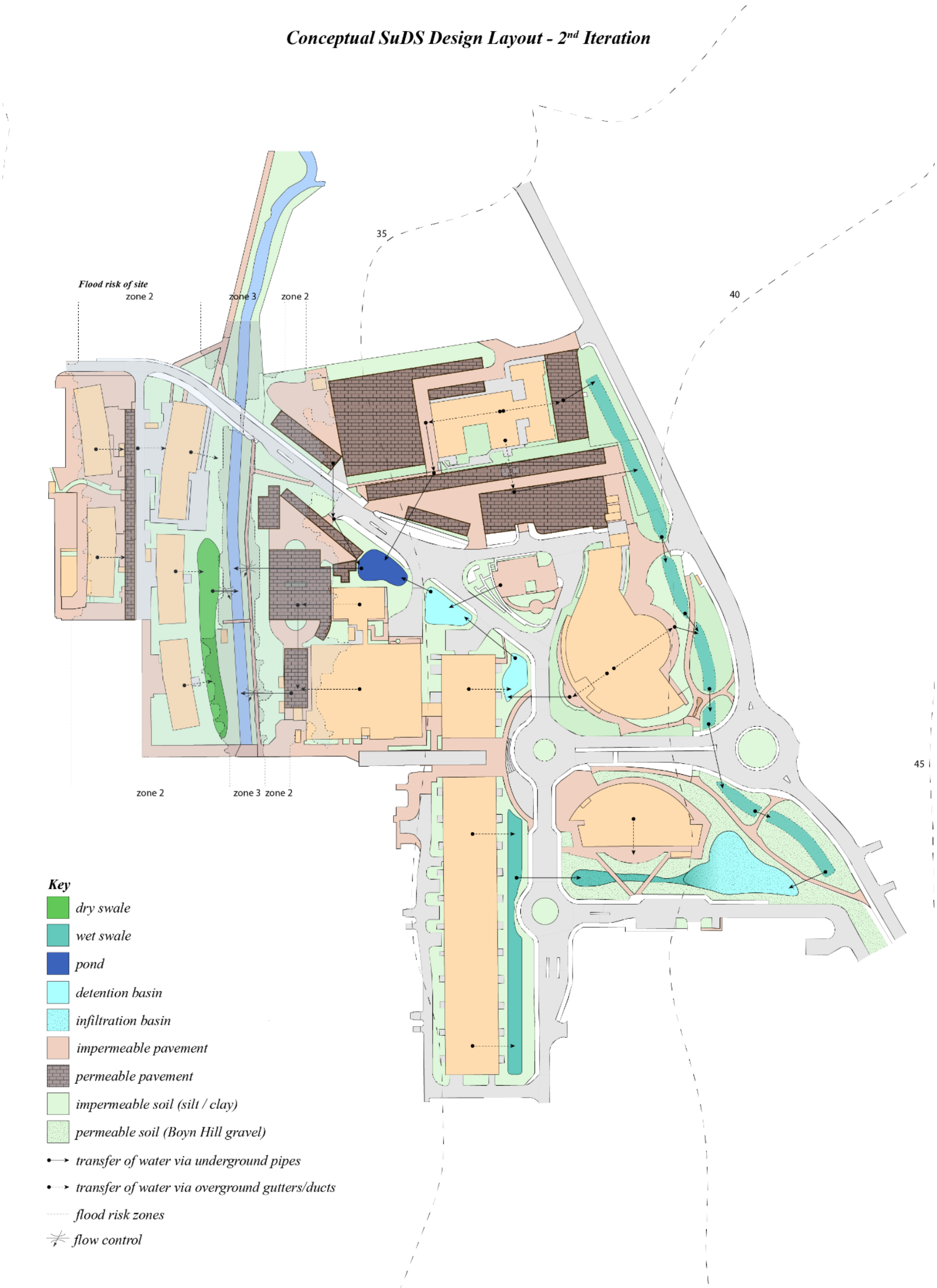

Figure 18 – 2nd Iteration layout

10. Cost Analysis

10.1. Expenditure

A cost analysis of a SuDS must consider the design life of the components specified. HM Treasury recommends to discount projects at 3.5% for 30 years when calculating expenditures, as such our SuDS will be designed for a 30 year life.

Table 26:the notional design life of the SuDS we have implemented

Component	***Replacement Needed After 30 years?***	***Component Life***[9]
permeable paving	*No*	*Replacement of filter material (20-25 yrs)*
infiltration basin	*No*	*Deep tilling and replacement of infiltration surface (5-10 yrs)*
retention pond	*Yes*	*N/A*
detention basin	*Yes*	*Sediment disposal (10-15yrs)*
Swale	*No*	*deep tilling - if infiltration swale replacement of infiltration surfaces required (5-20yrs)*
rainwater harvesting	*No*	*No reliable information*

Table 27: Capital Expenditure of components

		Capital Costs		
Component	***Unit***	***Capital Cost [£/unit]***[10]	***Quantity [unit]***	***Capital Cost [£]***
permeable paving	*m3 (stored volume)*	*375*	*3,137*	*1,176,375*
infiltration basin	*m3 (stored volume)*	*25*	*943*	*23,575*
retention pond	*m3 (stored volume)*	*50*	*29*	*1,467*
detention basin	*m3 (stored volume)*	*25*	*93*	*2,334*
Swale	*m2*	*20*	*1,112*	*22,239*
rainwater harvesting	*m2*	*50*	*14,338*	*716,900*
Total				*1,942,889*

Table 28:Operational Expenditure of components

Operational Costs				
Component	***Unit***	***Operational Cost (£ p.a./unit)[11]***	***Quantity [unit]***	***Operational Cost [£ p.a] present value***
permeable paving	*m3 (stored volume)*	*1.1*	*3137*	*3450.7*
infiltration basin	*m3 (stored volume)*	*0.3*	*943*	*282.9*
retention pond	*m3 (stored volume)*	*1.4*	*29.33*	*41.062*
detention basin	*m3 (stored volume)*	*0.3*	*93.34*	*28.002*
Swale	*m2*	*0.1*	*1111.96*	*111.196*
rainwater harvesting	*m2*	*0.4*	*14338*	*5735.2*

Table 29: Present Value Expenditure

Discounted OPEX sum @3.5% over 30 years [£]	***Undiscounted OPEX sum [£]***	***Present Value OPEX + CAPEX [£]***
187115	*241226.5*	*2,130,004.21*

30 years operational expenditure costs – discounted at the treasury's recommended interest rate of 3.5%. The total present value estimate is £2,130,000.

10.2. Estimating the benefits of the SuDS scheme

CIRIA provides the B£ST workflow to evaluate the benefits of an appraisal. A numerical estimation of the benefits can only ever be rough approximations as the returns are unlikely to be seen as steady cashflows. CIRIA, however, does assist in making a coarse assessment of the scheme. It asks us to take into account the following:

Our site has 16,800 m2 of green space wherein we plan to put 1 tree per 100 m2 amounting to 168 new trees. We expect the SuDS to improve the 175m of the river Pinn flowing through our site. Three buildings are currently within flood risk zone 2, the SuDS will be mitigating this risk (note: B£ST asks for number of properties and does not discriminate between scales). There are 15,000 students attending Brunel University, the site is 15% of the total campus area and has the river Pinn running through it, it is the principal entrance to the campus and contains important educational and fitness facilities. For these reasons we expect 30% (4500 people) of the student body to benefit from the improvements to the green space. For all these reasons the coarse assessment in B£ST estimates that the benefits may be £2 million.

Table 30:Coarse Assessment results summary table (B£ST)

Benefit category	Present Value Lower Bound Estimate (£)	Present Value Central Estimate (£)	Present Value Upper Bound Estimate (£)
Air quality	£ 20,046	£ 27,486	£ 34,926
Amenity	£ 749,623	£ 1,249,372	£ 1,749,120
Biodiversity and ecology	£ 344	£ 1,443	£ 2,542
Carbon sequestration	£ 1,276	£ 5,207	£ 8,513
Education	£ 102,983	£ 130,243	£ 157,503
Flood Risk	£ 81,967	£ 81,967	£ 81,967
Flows in watercourses	£ 1,635	£ 1,986	£ 2,346
Health	£ 271,937	£ 452,897	£ 627,908
Recreation	£ 48,898	£ 97,917	£ 146,937
Water quality in watercourse	£ 4,905	£ 5,958	£ 7,037
TOTAL	**£ 1,283,613**	**£ 2,054,476**	**£ 2,818,798**

10.3. Critical Analysis

The present value expenditure (£2.1million) is similar to the benefits estimation (£2.05 million). From Table 27. we see that "Amenity" is the main benefit, the flooding benefits could be higher if the property figure included scale of building within the estimation e.g. 3 commercial that are the size of 30 houses would increase the flood risk benefit 10x and improve the present value central estimate accordingly. Permeable paving will be the largest expenditure followed by rainwater harvesting (RWH). Rainwater harvesting is a SuDS and a non-potable water supply system, although it is not considered in the B£ST estimation there will be significant savings from including a RWH. By considering the benefits that the B£ST misses and the expected expenditure of the selected SuDS, we can be confident that the value from the specified scheme will match the investment and enhance the community.

11. Conclusion

The specified sustainable urban drainage scheme reduces the outflow rate compared to the current development. On average the SuDS outflow rates are 37% below the developed rates (Table 31.), this is not enough to comply with certain planning requirements, which state that new SuDS schemes must be 50% below the brownfield rates[12].

Table 31: Performance of SUDS scheme evaluation

Flow Rate	Q_G – greenfield condition (l/s)	Q_D – Initial calculation (l/s)	Q_{SUDS} (l/s)	(%) Difference Q_{SUDS} to Q_D
Zone 1	16.47	116.14	66.26	43
Zone 2	8.19	65.60	38.82	41
Zone 3	14.19	101.00	87.04	14
Zone 4	13.13	92.74	73.59	21
Zone 5	26.66	168.64	46.59	72
Zone 6	2.53	21.52	(No SUDS in zone 6)	n/a
Zone 7	5.82	50.5	33.96	33
Average				37

The third iteration would require further thinking of the SuDS design, this may mean that a new drainage system or that more of the same systems are introduced as part of the scheme. More pervious pavements could be implemented. In the second iteration they were reduced due to their oversized storage capacity, however, this overdesign could provide the difference in outflow rate that is necessary to comply with for planning purposes.

12. References

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*13. All calculations were based on and adhered to***:**

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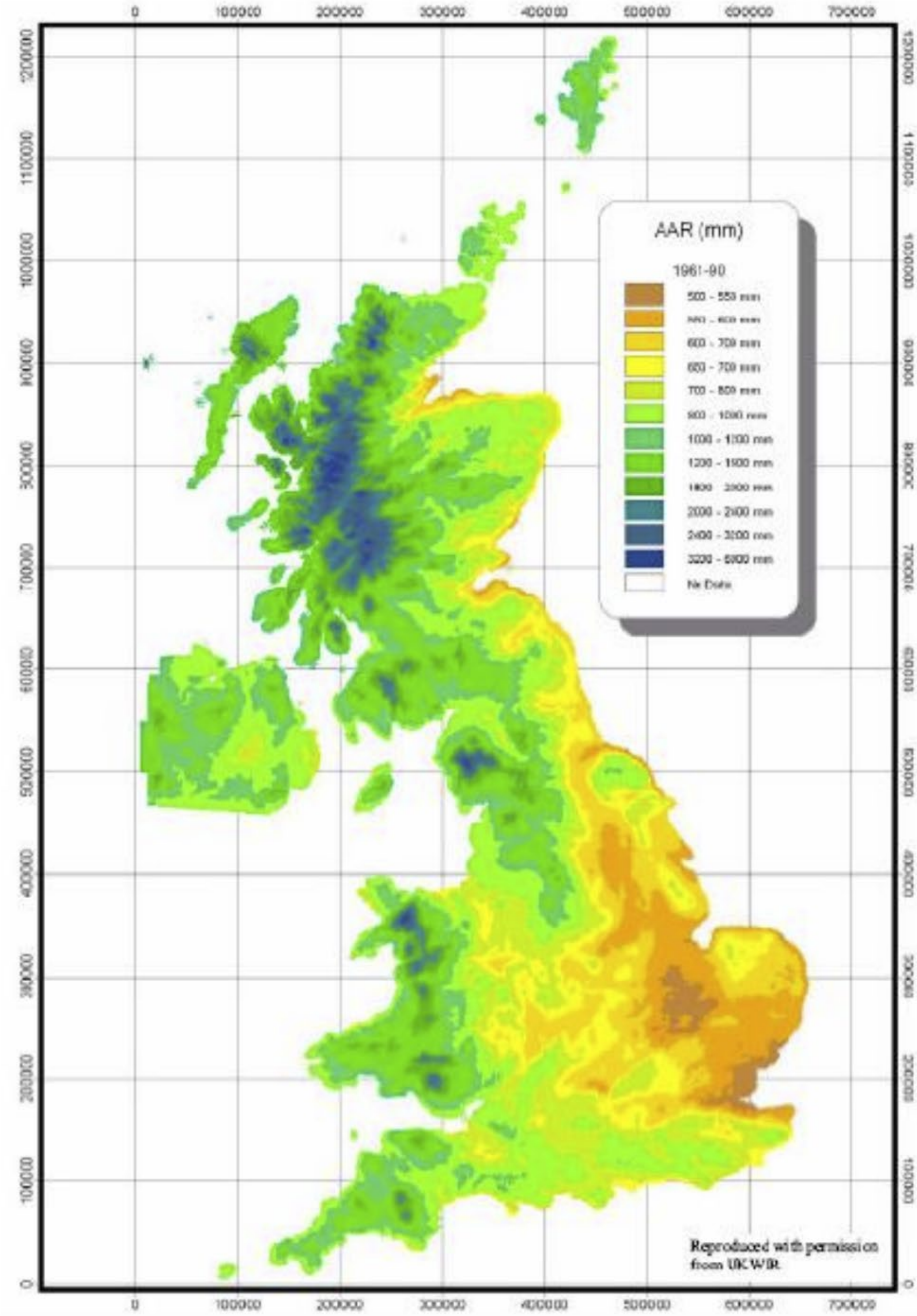
13. Appendix

13.1. Appendix 1 - UK rainfall regions

13.2. *Appendix 2 - Standard Average Annual Rainfall (SAAR) across the UK*



13.3. *Appendix 3 - Soil types across the UK*

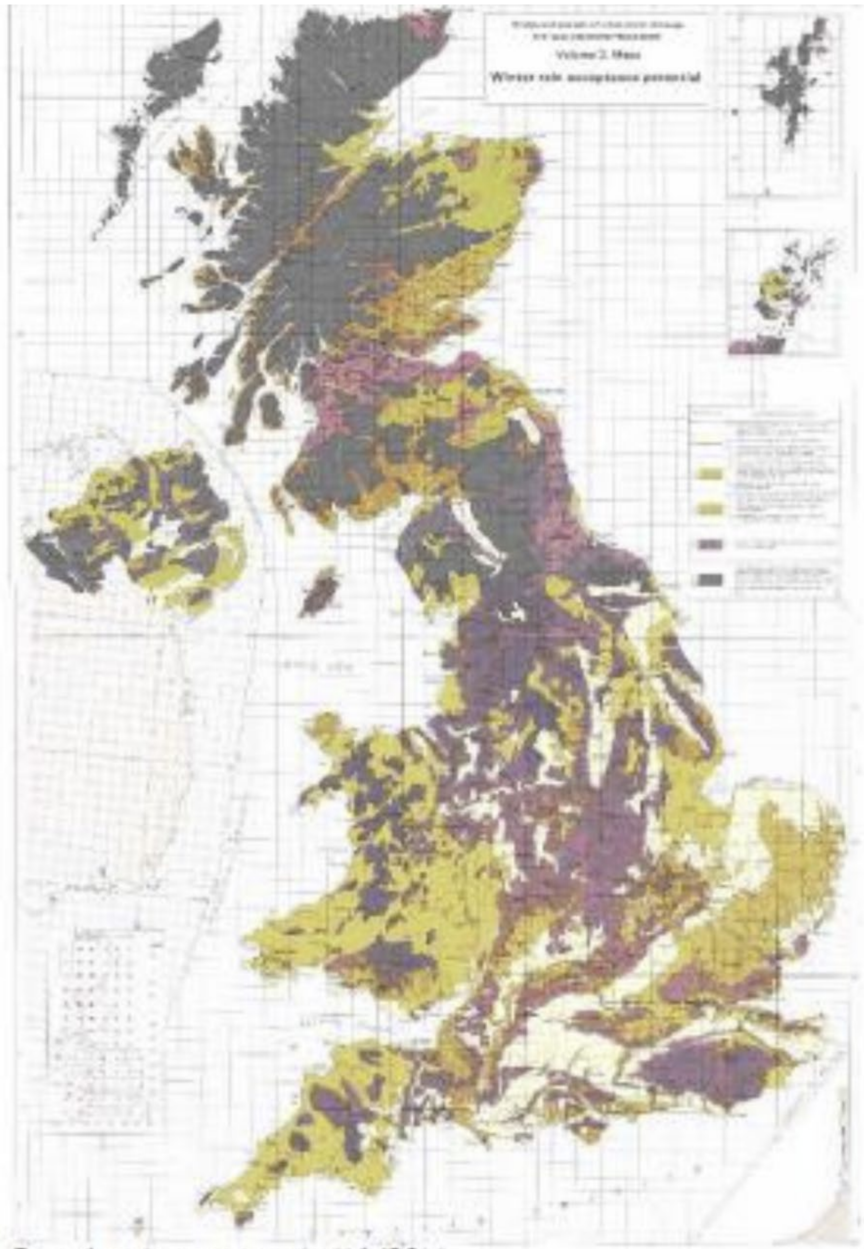


Figure 3 soil types across the UK (SOIL)

Figure 2 Soil types across the UK

SOIL2 - . - very permeable soils with shallow groundwater = 0.3

SOIL3 - . - permeable soils with shallow groundwater in low lying areas = 0.37

SOIL4 - . - Clayey soils with an impermeable layer at shallow depth = 0.47

SOIL5 - . - Soils of the wet uplands = 0.53

13.4. *Appendix 4 – UK growth curve factors*

TABLE 24.2 UK and Ireland growth curve factors (after NERC, 1993)

Hydrometric area	Return period								
	1[1]	2	5	10	25	30[2]	50	100	500
1	0.85	0.90	1.20	1.45	1.81	1.99	2.12	2.48	3.25
2	0.87	0.91	1.11	1.42	1.81	1.99	2.17	2.63	3.45
3	0.86	0.94	1.25	1.45	1.70	1.75	1.90	2.08	2.73
9	0.88	0.93	1.21	1.42	1.71	1.80	1.94	2.18	2.86
10	0.87	0.93	1.19	1.38	1.64	1.70	1.85	2.08	2.73
4	0.83	0.89	1.23	1.49	1.87	1.99	2.20	2.57	3.62
5	0.87	0.89	1.29	1.65	2.25	2.55	2.83	3.56	5.02
6/7	0.85	0.88	1.28	1.62	2.14	2.40	2.62	3.19	4.49
8	0.78	0.88	1.23	1.49	1.84	1.98	2.12	2.42	3.41
Ireland	0.83[2]	0.95	1.20	1.37	1.60	1.65	1.77	1.96	2.40

13.5. *Appendix 5 – Runoff coefficient tables*

DESCRIPTION OF AREA	RUNOFF COEFFICIENT
Business	
Downtown	0.70 - 0.95
Neighborhood	0.50 - 0.70
Residential	
Single-family	0.30 - 0.50
Multiunits, detached	0.40 - 0.60
Multiunits, attached	0.60 - 0.75
Residential (suburban)	0.25 - 0.40
Apartment	0.50 - 0.70
Industrial	
Light	0.50 - 0.80
Heavy	0.60 - 0.90
Parks, Cemeteries	0.10 - 0.25
Playgrounds	0.20 - 0.35
Railroad yard	0.20 - 0.35
Unimproved	0.10 - 0.30
CHARACTER OF SURFACE	
Pavement	
Asphaltic and concrete	0.70 - 0.95
Brick	0.70 - 0.85
Roofs	0.75 - 0.95
Lawns, sandy soil	
Flat, 2%	0.05 - 0.10
Average, 2-7 %	0.10 - 0.15
Steep, 7%	0.15 - 0.20
Lawns, heavy soil	
Flat, 2%	0.13 - 0.17
Average, 2-7 %	0.18 - 0.22
Steep, 7%	0.25 - 0.35

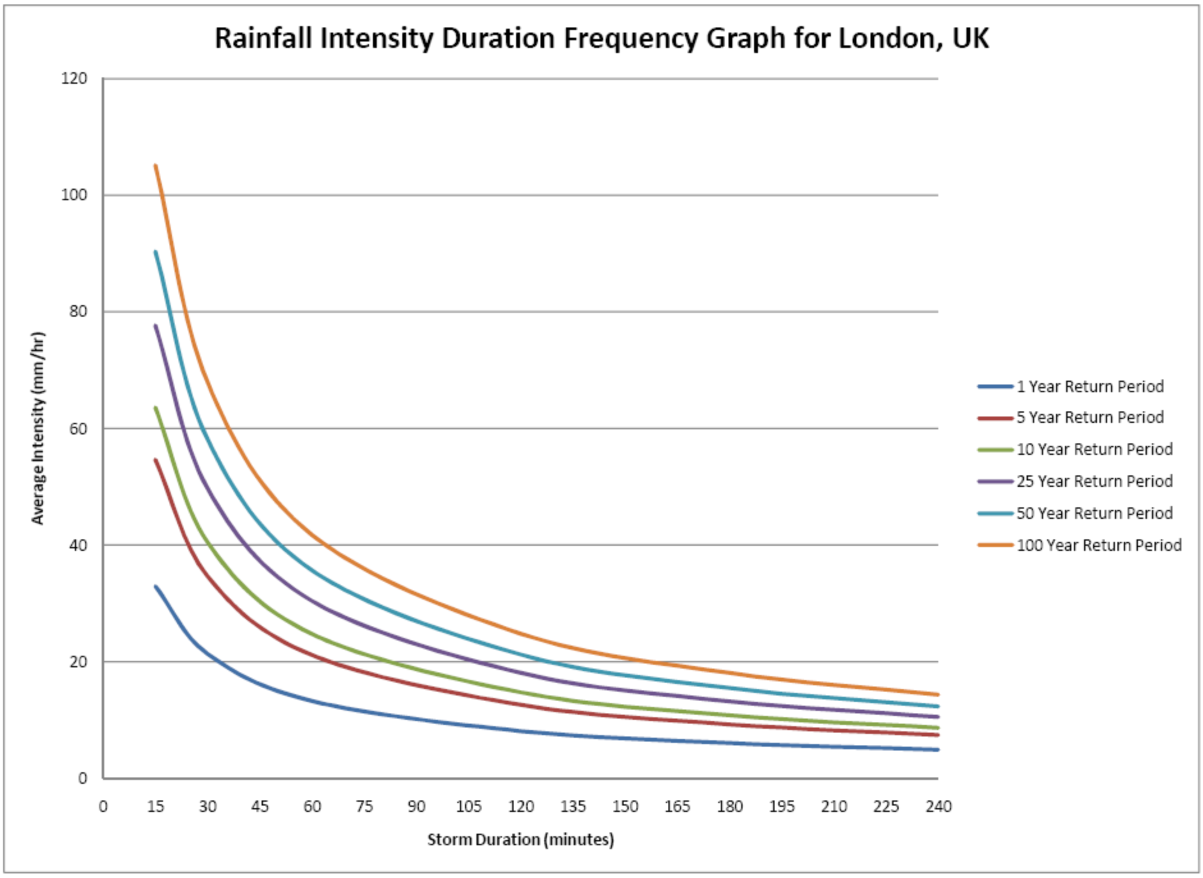
https://swmm5.org/2017/11/06/runoff-coef 1

Reference	Types of pavement	Runoff coefficient range
[9]	Grid	0.00–0.35
[12]	PICP	0.37–0.45
[22]	Grid, PICP	0.00–0.03
[24]	Grid	0.00–0.10
[27]	Grid	0.00–0.26

PICP: permeable interlocking concrete pavement.

https://www.researchgate.net/publication/287405436_Experimental_results_on_permeable_pavements_in_Urban_areas_A_synthetic_review

13.6. Appendix 6– IDF curves



13.7. Appendix 7 – Example calculation process of a sub catchment

Example of the complete process of calculation for zone 1:

SUDS Area	Total	**Ac =**	7493	m2
			0.75	ha
			0.007	km2
	Impervious	**Ai =**	4284	m2
			0.43	ha
			0.004	km2
	Pervious	**ApGreenery =**	3209	m2
			0.32	ha
			0.0032	km2
		ApPavement =	4828	m2
			0.48	ha
			0.0048	km2

Area	Total	**Ac =**	13176	m2
			1.32	ha
			0.013	km2
	Impervious	**Ai =**	9967	m2
			1.00	ha
			0.010	km2
	Pervious	**Ap =**	3209	m2
			0.32	ha
			0.0032	km2

			IDF curve London			**IDF equation, i = coeff1/(D+coeff2)**		
IDF			Duration (min)	Duration (min)	Duration (min)	Duration (min)	Duration (min)	Duration (min)
			15	60	360	15	60	360
	Return period	Rainfall intensity						
	T = 5 yr	i (mm/hr) =	55	22	7	53.48	20.40	4.08
	T = 30 yr	i (mm/hr) =	80	32	9	77.39	29.87	5.97
	T = 100 yr	i (mm/hr) =	105	45	12	100.83	39.34	7.95

Parameters	Soil		0.47
	SAAR		700
	BFIHOST		0.22
	Runoff coeff	C-Greenfield:	0.1
		C-after developn	0.9
	Growth curve factor	5y:	1.28
		30y:	2.4
		100y:	3.19

Rational method (1 hr)

Runoff	Flow rate	m3/s	l/s	
	Qg05	0.01	8.05	Greenfield
	Qd05	0.06	**56.78**	**After Development**
	Qg30	0.01	11.71	Greenfield
	Qd30	0.08	**82.59**	**After Development**
	Qg100	0.02	16.47	Greenfield
	Qd100	0.12	**116.14**	**After Development**

IH124

	for 50 ha		for 6.9 ha	
Flow rate	m3/s	l/s	m3/s	l/s
Qbar	0.24	241.39	0.01	6.36
Qg05	0.31	308.98	0.01	**8.14**
Qg30	0.58	579.34	0.02	**15.27**
Q100g	0.77	770.04	0.02	**20.29**

Volumes	T (yr)		30	
	Duration (h)		1	6
	Volumes Rational Method			
	Volumes	m3		m3
	Volg30		42	71
	Vold30		297	502
	DV30		255	431
	Volg100		59	95
	Vold100		418	669
	DV100		359	**574**
	IF		0.7	
	PIMP		76	
	PF		200	
	NAPI		25	
	PR		59	
	RuV		558	
SUMMARY	**Peak discharge (1hr)**			
	Qg100		16.47	l/s
	Qd100		116.14	l/s
	Climate Change & Urban creep allowances (40% + 10%)			
	Qg100		16.47	l/s
	Qd100		174.21	l/s
	Volg100		59.29	m3
	Vold100		627.16	m3
	Storage volume		567.86	m3

T (yr)	100						
Duration (h)	6						
Greenfield PR			**Proposed development site**				
Percentage Runoff (NERC, 1985)			*FEH-Uk variable*				
SPR	70.8		IF	0.75			
CWI	100		PIMP	87		100	
DPRcwi	-6.25		PF	30		40	
P	45		NAPI	-15		65	100
DPRrain	1.39		PR1summer	47.5		65	100
PR	65.94		PR2winter	79			
Runoff volume (m3)	**625.5384378**						
			Runoff Vol winter (m	749			
IH124			Runoff Vol summer (i	450.6192			
Volg100 (m3)	**438.3107617**		Runoff Vol aveg (m3)	**616.6368**			
			FEH				
			SOIL	0.47			
			UCWI	30		300	
			PRdry	65.24			
			PRwet	86.30			
			Runoff Vol (m3)	618.8819904			
			Runoff Vol (m3)	**818.6723136**			

IDF			**IDF curve London**				**IDF equation, i = coeff1/(D+coeff2)**		
			Duration (min)	Duration (min)	Duration (min)		Duration (min)	Duration (min)	Duration (min)
			15	60	360		15	60	360
	Return period	Rainfall intensity							
	T = 5 yr	i (mm/hr) =	55	22	7		53.48	20.40	4.08
	T = 30 yr	i (mm/hr) =	80	32	9		77.39	29.87	5.97
	T = 100 yr	i (mm/hr) =	105	45	12		100.83	39.34	7.95

Parameters	Soil		0.47
	SAAR		700
	BFIHOST		0.22
	Runoff coeff	C-Greenfield:	0.1
		C-pervious	0.3
		C-impervious	0.9
	Growth curve fa	5y:	1.28
		30y:	2.4
		100y:	3.19

	Rational method (1 hr)			
Runoff	Flow rate	m3/s	l/s	
	Qg100	0.02	16.47	Greenfield
	SUDS Qd100	0.07	**66.26**	**After Development**
	Initial Qd100	0.12	116.14	

This process was repeated for the rest of the other 6 sub-catchments

13.8. Appendix 8: Run Off Calculations for Detention Basin and Pond

$$Q_{developed} = (C_{developed} \times i \times A_{impervious}) + (\, C_{greenfield} \times i \times A_{pervious})$$

$$Q_{greenfield} = (C_{greenfield} \times i \times A_{total})$$

	Detention Basin 1	**Detention Basin 2**	**Pond**
$C_{developed}$	0.9		
$C_{greenfield}$	0.1		
i for 1:100 year, 6 hour (mm/hr)	12		
Pervious Area (m^2)	474	474	661
Impervious Area*[1] (m^2)	380	469	183
Total Area (m^2)	854	943	844
Developed Run Off Rate (l/s)	1.95	2.35	1.15

*[1] Impervious area in these case does not include pervious pavement as it has controlled flow rate from the subbase where the water is stored.

	Pond
Pervious Area (m^2)*[2]	1609
Impervious Area (m^2)*[2]	5066
Total Area (m^2)	6675
Greenfield Run Off Rate (l/s)	2.23

*[2] As the pond stores and releases the water for the catchments of detention basin 1 and 2 as well as the pervious pavements, the total area is used so all of the water which is conveyed to the pond outfalls at greenfield rate.