

Saint Hill
Site 1 Car Park,
Saint Hill Road,
East Grinstead, West Sussex, RH19 4JY
Flood Risk Assessment & Surface Water Drainage

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Appendix I – Architect Plans and topographical survey

Appendix II – Proposed Drainage strategy

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1 Introduction

QED have prepared a site-specific Flood Risk Assessment (FRA) and Surface Water Drainage Strategy (SWDS) for the development at Saint Hill, Saint Hill Road, East Grinstead, West Sussex, RH19 4JY.

The wider proposed development is for permanent infrastructure to facilitate annual events, permanent road access with hardstanding areas and additional toilet block. A permanent parking surface to be installed to allow year-round parking and a report for this area has been provided separately for convenience.

The site is located on the south-western side of Saint Hill Road and consists of a field to the west of a group of buildings centred around The Castle and Saint Hill Manor that comprise the UK headquarters of the International Association of Scientologists.

The site of the currently consists of a field which is used as an informal car park. It is understood the proposed development is for the transition of this field into a car park, to provide 750 parking spaces, as well as a new toilet block and extended access road.

The site is shown located within EA defined Flood Zone 1, within an area at low risk from surface water flooding and development proposed would be at low risk from other sources.

The British Geological Survey (BGS) Geology of Britain Viewer indicates that the bedrock underlying the site consists of Cuckfield Stone Bed – Sandstone, Calcareous. As well as Upper Grinstead Clay - Mudstone (Source: BGS). There are no superficial deposits underlying the site. Initial infiltration testing indicated poor results; however, further testing in the areas of the proposed attenuation should be undertaken to maximise infiltration and reduce runoff where feasible.

Due to the limited flood risk to the site, residual risks are limited to overland flow from an exceedance event and potential surface water risk. In the event of heavy rainfall the low permeability of the sub soil would limit infiltration, and overland flow could occur.

The drainage strategy proposes managing surface water runoff from the car park by providing attenuation before discharging into an existing watercourse at the west of the site.

Due to the site's existing slope, the car park is divided into two sections, each controlled by complex flow controls consisting of a hydro-brake and an orifice plate, to restrict discharge to pre-development greenfield rates for the 2, 30 and 100yr rainfall events. The western section is attenuated using an attenuation tank beneath the grassed car park, while the eastern section is attenuated via the permeable subbase of the formal car park.

The western attenuation limits discharge to 20.6 l/s during the 1 in 100-year event + 40% climate change (1%AEP+CC), while the eastern attenuation restricts flow to 32.4 l/s under the same conditions. These rates align with predevelopment greenfield runoff rates calculated using the UKSUDS greenfield runoff estimation tool. All surfacing is proposed to be unlined to promote infiltration where feasible.

All foul water drainage is to be drained to the closest existing foul water drainage connection. Details of the capacity of the existing foul system is to be confirmed at the later design stage.

This report has been produced broadly in accordance with the National Planning Policy Framework (NPPF), West Sussex County Council relevant guidance, West Sussex County Council SFRA

2 Policy Compliance

The purpose of this assessment is to demonstrate that the proposals outlined above can be satisfactorily accommodated without increasing the risk of flooding for the area or without placing the development itself at risk of flooding, as per the:

- National Planning Policy Framework
- West Sussex County Council relevant guidance
- West Sussex County Council SFRA

3 Site Location

The site is located at Saint Hill Manor, Saint Hill Road, East Grinstead, West Sussex, RH19 4JY. The site is currently a field used for car parking during events.



Figure: Site location plan

The site currently consists of a field which is used as a car park. It is understood the proposed development is for the transition of this field into a car park, to provide 750 parking spaces, as well as a new toilet block and extended access road.

Architectural site plans detailing the proposed development are included Appendix I and are extracted below for reference.



Figure 2: Proposed Site Plan

4 Geology and Infiltration Potential

The British Geological Survey (BGS) Geology of Britain Viewer indicates that the bedrock underlying the site consists of Cuckfield Stone Bed – Sandstone, Calcareous. As well as Upper Grinstead Clay - Mudstone (Source: BGS). There are no superficial deposits underlying the site. Preliminary site investigations, including infiltration testing, have been conducted, and the results indicate that infiltration is not viable.

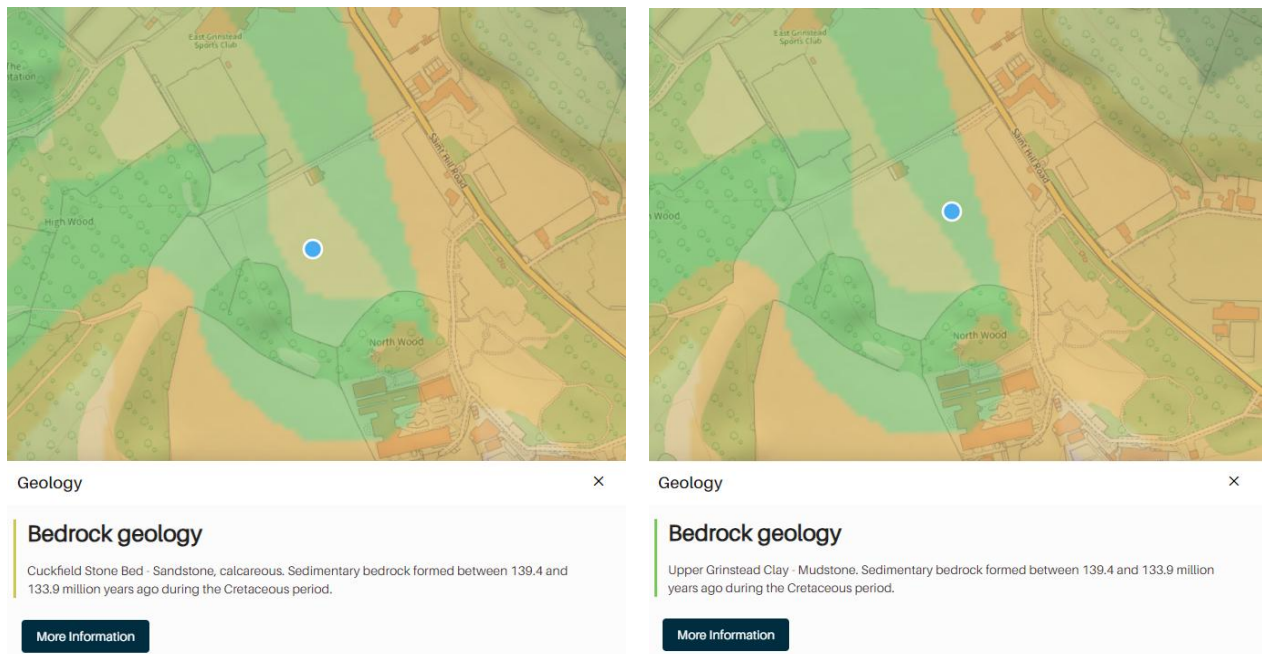


Figure 3: BGS geology map extracts

5 Sequential Test/Exception Test

Under the NPPF, all new planning applications should undergo a Sequential Test. This test should be implemented by local planning authorities with a view to locating vulnerable new developments outside of the floodplain.

No sequential test is deemed necessary as proposed development can be located outside of risk areas and safe access/egress can be provided, as outlined by guidance published in the NPPF.

The NPPF Sequential Test: Flood Risk Vulnerability and Flood Zone 'Compatibility' Table is reproduced below;

Flood Zones	Flood Risk Vulnerability Classification				
	Essential infrastructure	Highly vulnerable	More vulnerable	Less vulnerable	Water compatible
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception Test required	✓	✓	✓
Zone 3a †	Exception Test required †	X	Exception Test required	✓	✓
Zone 3b *	Exception Test required *	X	X	X	✓ *

Key:
 ✓ Exception test is not required
 X Development should not be permitted

Table 1: NPPF Sequential Test: Flood Risk Vulnerability and Flood Zone 'Compatibility' Table

Using the principles of the Sequential Test outlined above, the proposed car park development is classed as 'Less Vulnerable' under the NPPF. Therefore, in accordance with the Flood Risk and Coastal Change Planning Policy Guidance (PPG), the proposed development does not require the application of the Exception Test as is appropriate in Flood Zone 1.

The development would still need to apply the principles of flood risk assessment to the development and the standing advice published by the Environment Agency as part of the NPPF for Flood Risk assessment for Minor Development is applicable to this proposed development.

6 Climate Change Allowances

Making an allowance for climate change in the design of surface water drainage systems will help to minimise vulnerability and provide resilience to flooding and coastal change in the future. Climate Change allowances vary across the UK subject to catchment conditions and are based on climate change projections and different scenarios of carbon dioxide (CO₂) emissions to the atmosphere.

Climate change allowances were recently updated by the EA and the climate change allowances are now defined by River Catchment peak rainfall allowances. The data published on the DEFRA database shows the site located within the Medway Management Catchment and for commercial development (lifespan 100yrs), in accordance with NPPF guidance, the central allowance of 40% should be applied to rainfall events as the climate change allowance within this region.

1% annual exceedance rainfall event

Epoch		
	Central allowance	Upper end allowance
2050s	20%	45%
2070s	20%	40%

Figure 4: Peak Rainfall allowances

The EA have updated the River Flow climate change allowances in July 21. Where previously England had been split into ten River Basin Districts, the new approach divides these ten River Basin Districts into 92 smaller Catchment Management areas, each of which has its own climate change allowance value. The proposed development is located in the Medway Management Catchment and the new river flow allowances for the London Management Catchment as extracted below.

Medway Management Catchment peak river flow allowances ⊗

	Central	Higher	Upper
2020s	14%	19%	29%
2050s	15%	21%	37%
2080s	27%	37%	62%

Figure 5: River Flow Catchment allowances

The guidance published within the NPPF for using peak river flow allowances for flood risk assessments is extracted below:

In flood zones 2 or 3a for:

- essential infrastructure – use the higher central allowance
- highly vulnerable – use central allowance (development should not be permitted in flood zone 3a)
- more vulnerable – use the central allowance
- less vulnerable – use the central allowance
- water compatible – use the central allowance

On the basis of the climate change table above the river flow climate change allowance should therefore be 27%.

7 Sources of Flooding

7.1 Tidal and Fluvial

The Environment Agency (EA) mapping for Flood Risk, shows the site located within Flood Zone 1 (Low risk of fluvial or tidal flooding). Locations in flood zone 1 have a low probability of flooding. This means that in any year land has a less than 0.1% chance of flooding from rivers or the sea.

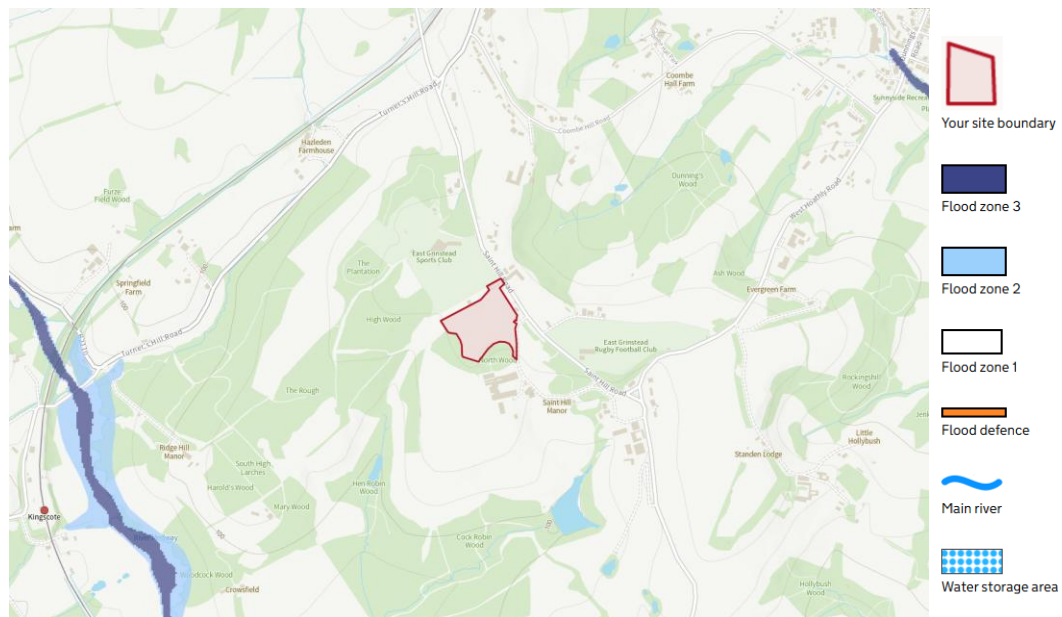


Figure 6: Flood map for planning extract with site boundary shown in red

7.2 EA Long Term Flood Risk map

According to the EA Long Term Flood Risk map, the site is at a very low risk of tidal and fluvial flooding. Very low risk means that each year this area has a chance of flooding less than 0.1%. It is understood that the EA Long Term Flood Risk map uses a 15m boundary to determine the flood risk to a development.

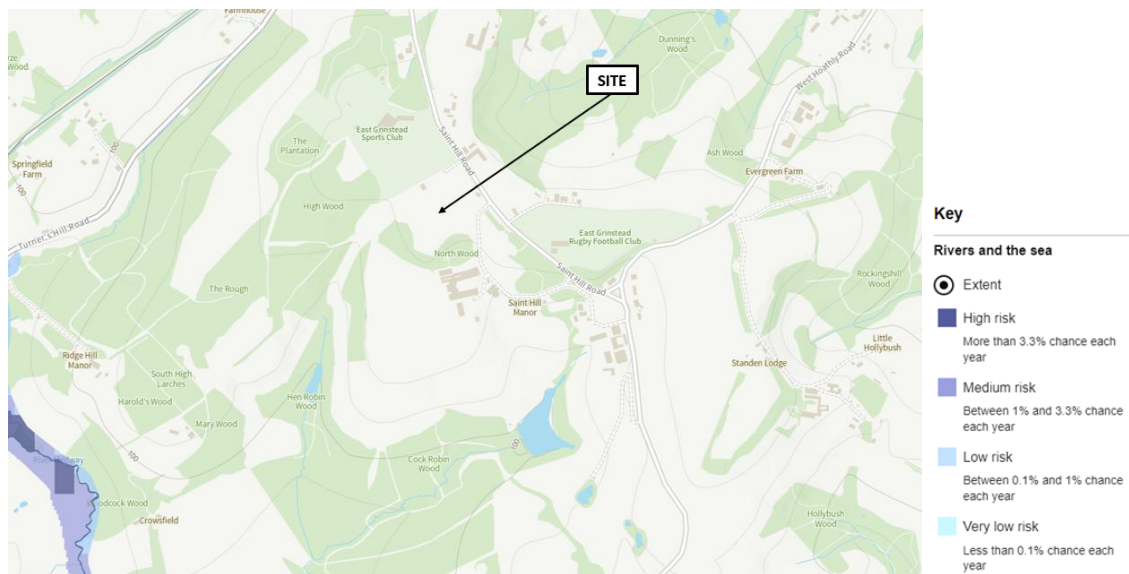


Figure 7: EA Long term risk of flooding map extract - Rivers and the Sea

According to the EA Long Term Flood Risk map for planning the site is at a very low risk of surface water flooding. Very low risk means that each year this area has a chance of flooding less than 0.1%.

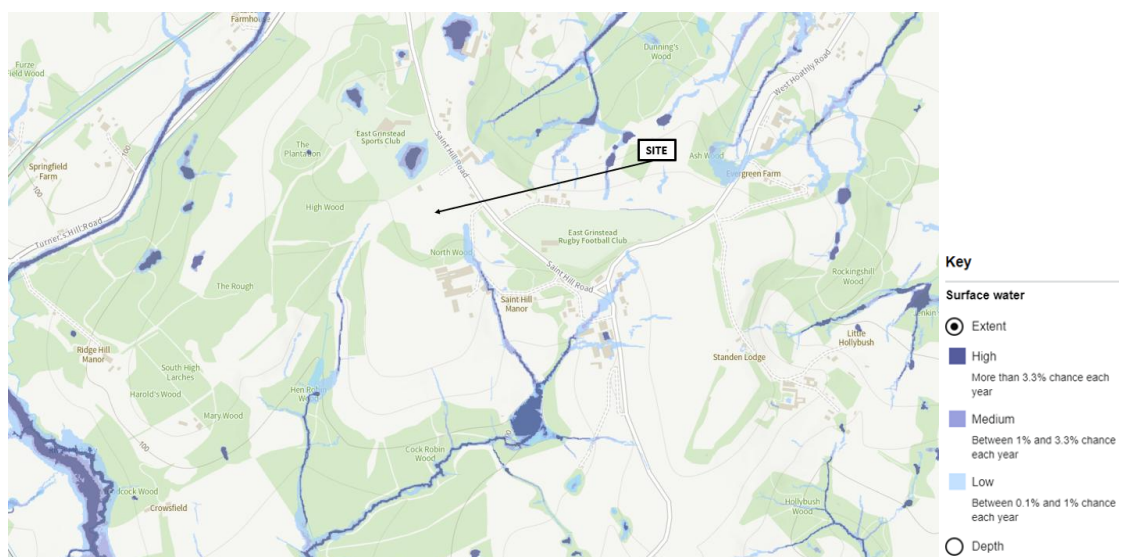


Figure 8: EA Long term risk of flooding map extract – Surface Water

7.3 Surface Water

Due to the presence of risk nearby to the proposed developments, QED Structures have obtained RoFSW data to better understand the risk of flooding from surface water.

The figure below shows the site to experience no flooding during the 1 in 30-year storm event according to EA RoFSW data.



Figure 9: EA RoFSW data showing 1in30yr surface water event

The figure below shows the site to be unaffected during the 1 in 100-year storm event according to EA RoFSW data.

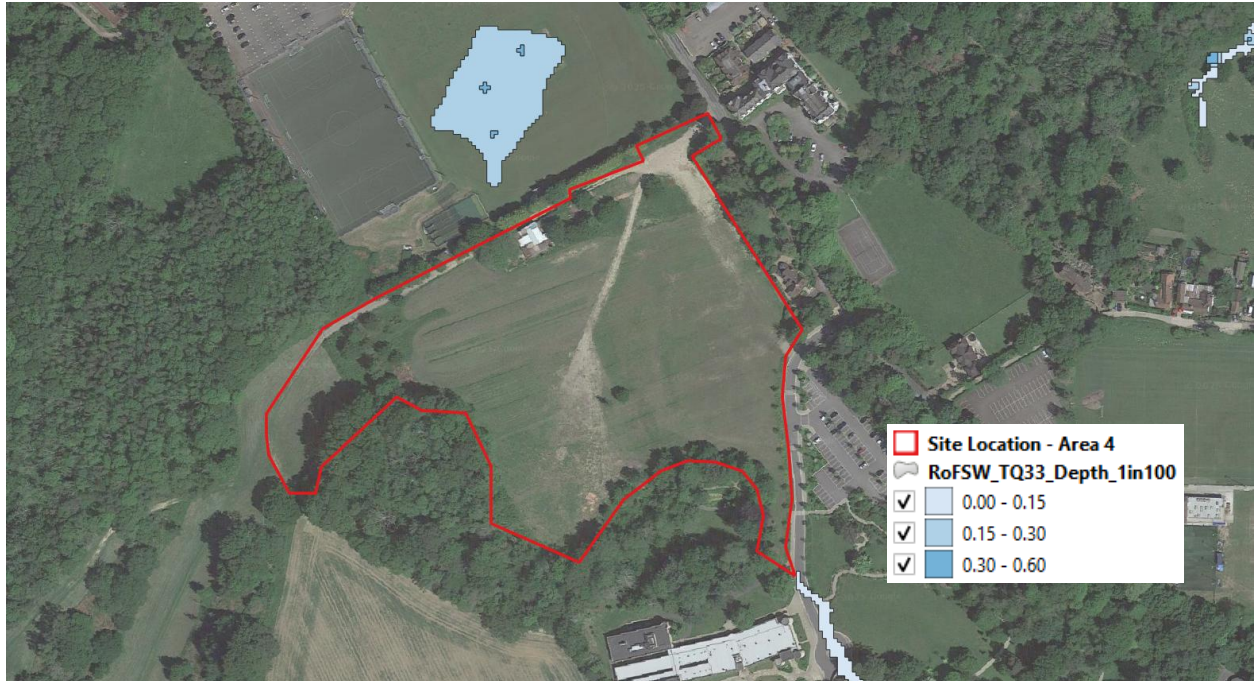


Figure 10: EA RoFSW data showing 1in100yr surface water event

The figure below shows the site to be inundated during the 1 in 1000-year storm event according to EA RoFSW data, reaching depths of up to 0.3m at the southeast of the site, and a minor encroachment of up to 0.15m at the west of the site.

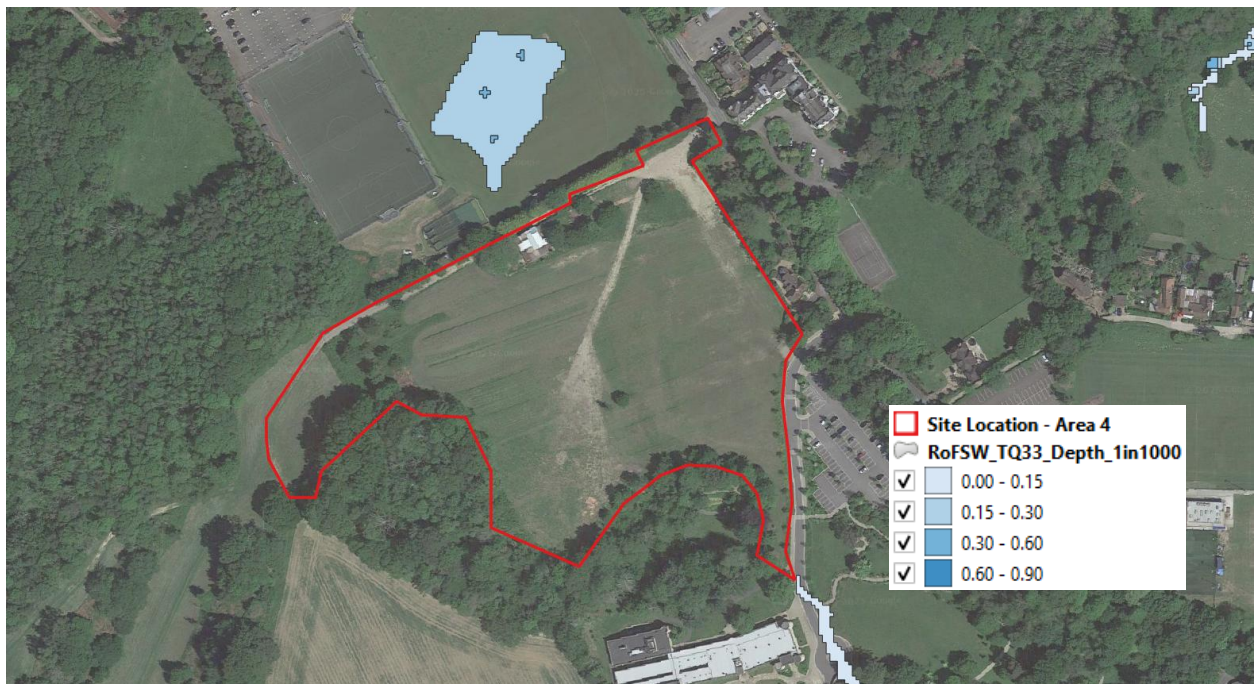


Figure 11: EA RoFSW data showing 1in1000yr surface water event

Following a detailed review of the GIS data, it is understood the risk to the proposed development from surface water flooding is generally very low. Inundation only occurs in the 1in1000yr event which far exceeds the design life of the proposed development.

7.4 Catchment Flood Management Plan

Mapping published within the West Sussex County Council SFRA shows the site to be located in the Medway CFMP (Catchment Flood Management Plan). A CFMP is a strategic planning document developed by the Environment Agency within a river catchment to set policies for managing flood risk.

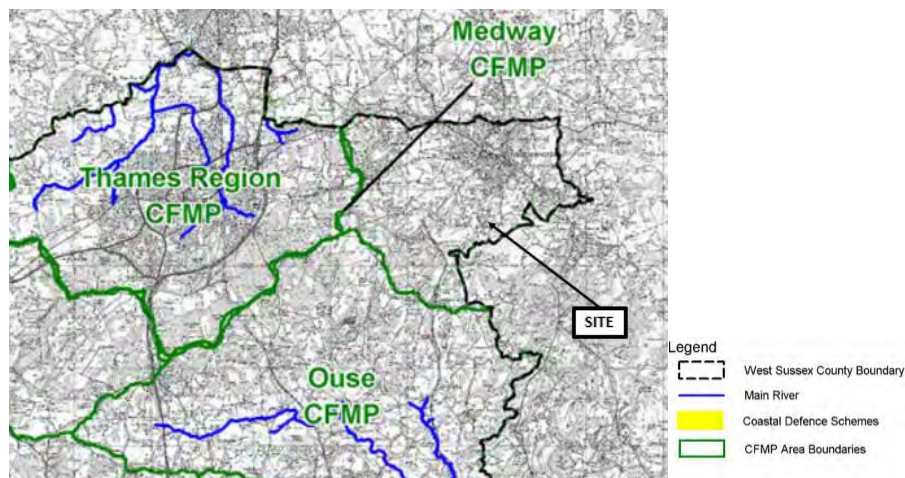


Figure 12: West Sussex County Council SFRA – Flood Management Plans and Strategies map extract

Mapping published within the River Medway Catchment Flood Management Plan shows the site to be located in the Upper Catchment, for which Policy 3 applies.

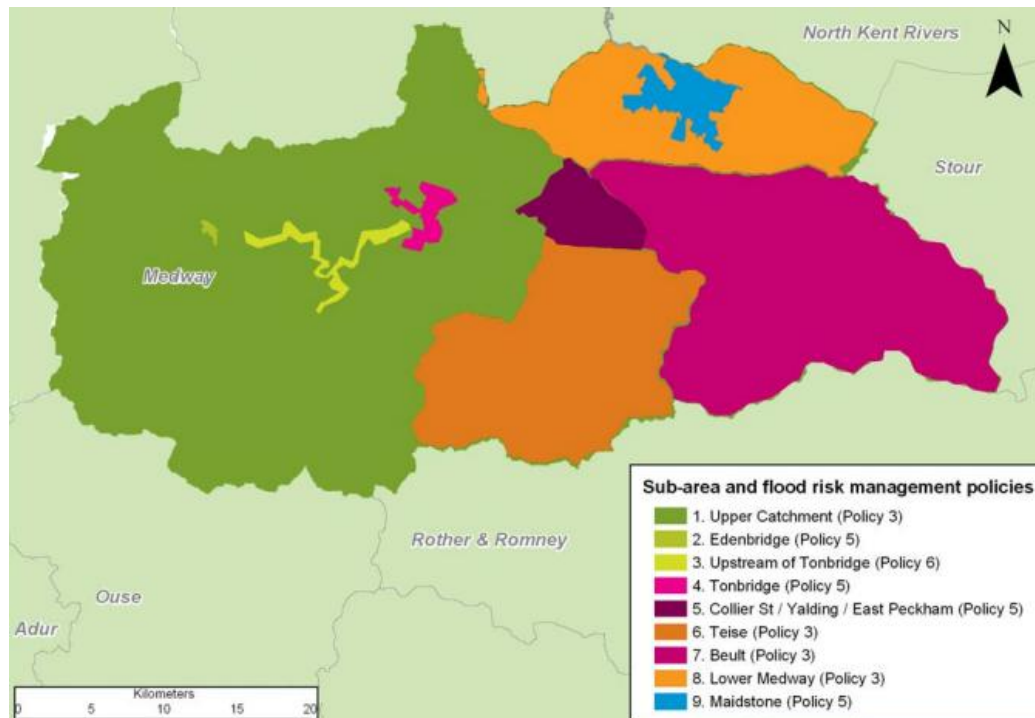


Figure 13: River Medway Catchment Flood Management Plan – Sub-areas and Flood Risk Management Policies map extract

Policy 3 for the River Medway CFMP is as follows:

Areas of low to moderate flood risk where we are generally managing existing flood risk effectively

This policy will tend to be applied where the risks are currently appropriately managed and where the risk of flooding is not expected to increase significantly in the future. However, we keep our approach under review, looking for improvements and responding to new challenges or information as they emerge. We may review our approach to managing flood defences and other flood risk management actions, to ensure that we are managing efficiently and taking the best approach to managing flood risk in the longer term.

7.5 Groundwater

Mapping published within the West Sussex County Council SFRA shows the site to be located in an area with medium potential for groundwater flooding and no recorded groundwater flooding events.

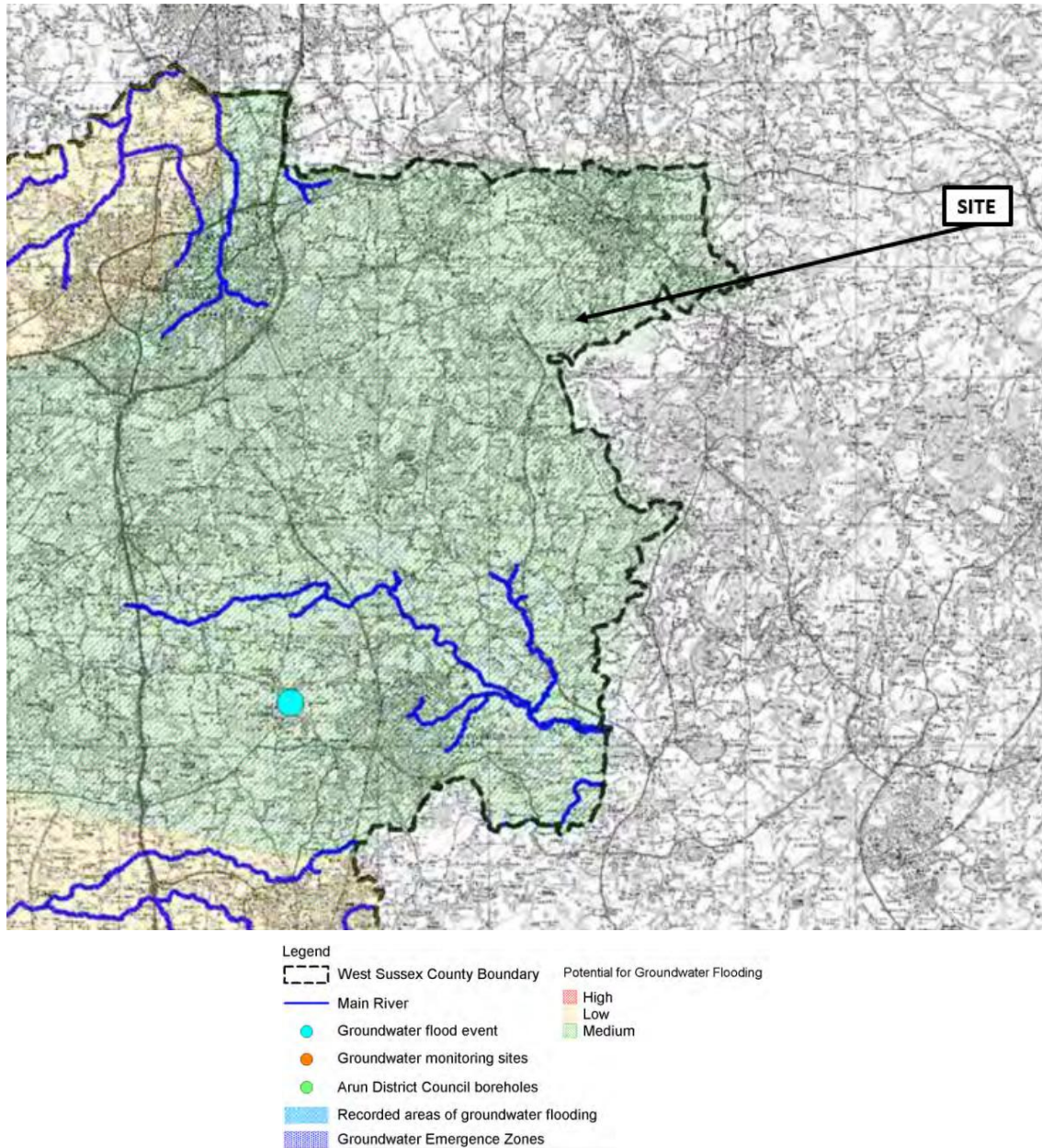


Figure 14: West Sussex County Council SFRA – Areas Prone to Groundwater Flooding map extract

7.6 Historic flood

Mapping published within the West Sussex County Council SFRA shows the site to be located in an area with low to medium proneness to flooding from the land. There is no recorded surface water flooding events recorded on or near to the development site.

■ Hove ■ London ■ Manchester ■ Leeds ■ Barcelona

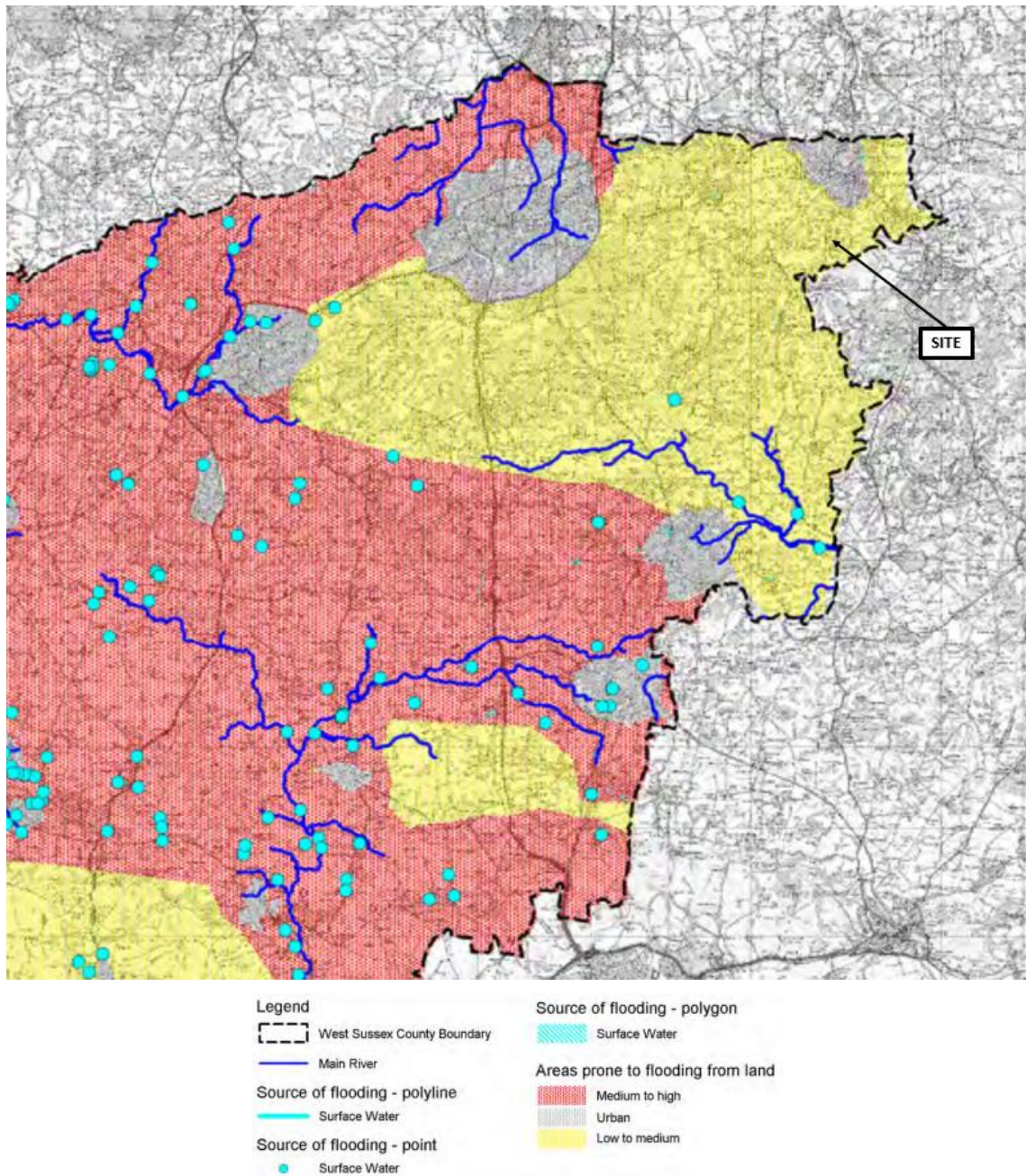


Figure 15: West Sussex County Council SFRA – Areas Prone to Flooding from the Land map extract

7.7 Sewer

Mapping published within the West Sussex County Council SFRA shows the site to be located in an area in close proximity to a number of recorded sewer-based flooding events. However, these occurred in the urban area of East Grinstead, and the proposed development is on unaffected rural land.

■ Hove ■ London ■ Manchester ■ Leeds ■ Barcelona

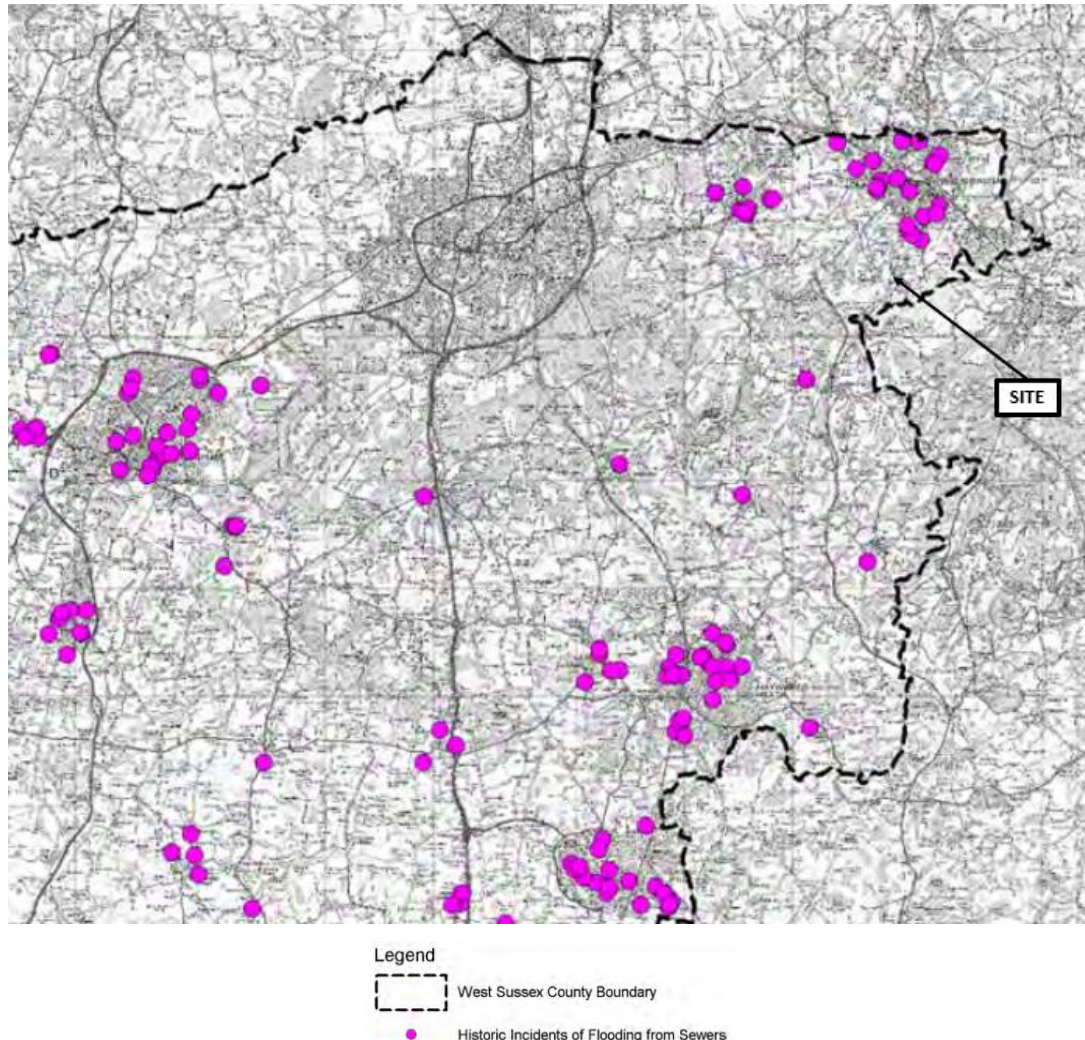


Figure 16: West Sussex County Council SFRA – Areas Prone to Flooding from Sewers map extract

7.8 Reservoir

The EA Long Term Flood Risk map shows that the development is located within an area where flooding from reservoirs is unlikely.

In the unlikely event that a reservoir dam fails, a large volume of water would escape at once, with little or no warning. According to the EA, there has been no loss of life in the UK from reservoir flooding since 1925. All large reservoirs must be inspected and supervised by reservoir panel engineers as detailed by the enforcement authority for the Reservoirs Act 1975 in England. The EA are responsible to ensure that reservoirs are inspected regularly, and essential safety work carried out.

8 Residual Risks

Due to the limited flood risk to the site, residual risks are limited to overland flow from an exceedance event and potential surface water risk. In the event of heavy rainfall the low permeability of the sub soil would limit infiltration, and overland flow could occur.

Proposed ground levels should be designed to contain surface water runoff within the car park extents to direct surface runoff away from existing adjacent development.

Permeable surfacing is proposed to allow infiltration, as existing, where feasible. A filter trench with a perforated pipe has been proposed to capture runoff and direct excess flow to areas for attenuation and storage prior to controlled release.

Consideration is also needed of the proposed development site being located within an area adhering to Policy 3 of the River Medway CFMP. This policy is applied to areas of low to moderate flood risk where there is a general management of existing flood risk effectively. Developments within this area could have an influence upon the risk of flooding, and so mitigating sustainable drainage measures should be considered by the developer to limit runoff from the site, to ensure that risk to others does not increase when this proposal is complete.

9 Flood Risk Management Measures

It should be ensured the construction of the car park is undertaken in a flood resilient manner in case of exceedance events. The proposals employ infiltration and offsite flow management strategies which match or achieve close to the existing greenfield runoff rates, so to not increase flood risk on site or elsewhere.

Proposed ground levels should be designed to contain surface water runoff within the car park extents to direct surface runoff away from adjacent exiting development where feasible

Permeable surfacing is proposed to allow infiltration, as existing, where feasible. Filter trenches and permeable surfacing are proposed to capture, and treat, runoff and direct excess flow to areas for attenuation and storage prior to controlled release at pre-development greenfield runoff rates.

10 Sustainable Urban Drainage (SuDS) Assessment

In accordance with the SuDS management train approach, the use of various SuDS measures to reduce and control surface water flows have been considered in detail for the development.

The management of surface water has been considered in respect to the SuDS hierarchy below as detailed in the CIRIA 753 'The SUDS Manual', Section 3.2.3:

■ Hove ■ London ■ Manchester ■ Leeds ■ Barcelona

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SUDS DRAINAGE HIERARCHY				
			Suitability	Comment
	1.	Store rainwater for later use	✓	Rainwater harvesting should be considered by the developer and could be used for the proposed toilet block.
	2.	Use infiltration techniques, such as porous surfaces in non-clay areas	✓	Infiltration testing shows infiltration may not be a feasible means of managing all surface water discharge. Partial infiltration is likely to be feasible
	3.	Attenuate rainwater in ponds or open water features for gradual release	x	We have identified a swale feature between the permanent car park and overflow field parking area. please refer to landscape GA plan and detail drawing 1232-ENP-SI-00-DR-L-90431
	4.	Attenuate rainwater by storing in tanks or sealed water features for gradual release	✓	May be necessary as full infiltration may not be feasible.
	5.	Discharge rainwater direct to a watercourse	✓	There is an existing watercourse in close proximity to the site
	6.	Discharge rainwater to a surface water sewer/drain	-	
	7.	Discharge rainwater to Combined Sewer	-	

Table 2:

SuDS Drainage Hierarchy

Following the SuDS drainage hierarchy, surface water flows from the site are proposed to discharge, as close as feasible to greenfield runoff rates, into the wider surface water drainage system on-site, which utilises waterbodies at lower levels. Exact routes and discharge locations will be confirmed pending further site investigations, ensuring compliance with LLFA policy and the CIRIA SuDS Manual C753

The suitability of SuDS components has been assessed in order to provide a sustainable means of providing the required attenuation volumes. The following components have been assessed as follows in Table 3, below.

SUITABILITY OF SUDS COMPONENTS		
SuDS Component	Comment	Suitability
Infiltrating SuDS	Infiltration testing shows infiltration as a sole means of discharge not to be feasible. However partial infiltration via type B permeable paving could be utilised to promote infiltration where possible.	✓
Permeable Pavement	Permeable pavement is deemed suitable for this site.	✓
Green / Blue Roofs	May be possible to include on the proposed toilet block as proposals show it to have a flat roof.	✓
Rainwater Harvesting	It is recommended that a rainwater harvesting tank is considered in order to reduce the clean water usage of the site.	✓
Swales	We have identified a swale feature between the permanent car park and overflow field parking area. please refer to landscape GA plan and detail drawing 1232-ENP-SI-00-DR-L-90431	x
Rills and Channels	Filter drains are used instead of rill and channels	x
Bioretention Systems	Provide little additional storage and not needed as all external surfaces are permeable and the majority are grassed.	x
Retention Ponds and Wetlands	Slopes and space limit potential for open features.	x
Detention Basins	Slopes and space limit potential for open features.	x
Geocellular Systems	Proposed to offer additional storage where needed	✓
Proprietary Treatment Systems	Not needed as proposed permeable surfacing can treat car park areas.	x
Filter Drains and Filter Strips	Included on the car park extent to capture, treat and manage runoff in the permeable paving sub base	✓

Table 3: Suitability of SuDS Components

Permeable Pavement

Permeable pavements are efficient in treating oils and metals but also in intercepting debris and silt. Type B (partial infiltration) permeable paving is proposed in order to promote infiltration where possible.

Filter Drain

A filter drain is a trench filled with permeable material, such as gravel and a perforated pipe at the base. They can be used to filter and convey stormwater to downstream SUDS components.

11 Surface Water Drainage Strategy

In accordance with the West Sussex County Council and Local Sustainable Urban Drainage and Flood Risk Policies, developments are required to use SuDS to reduce both the volume and runoff rates to the drainage system. Current local Sustainable drainage guidance is to achieve as close as feasible to greenfield runoff rates for storm runoff from new developments.

The drainage strategy proposes to drain the car park (24883m²) surface water. Type B permeable paving/surface is proposed to provide attenuation, whilst promoting infiltration where feasible, before discharging into an existing watercourse west of the site. the car park is split into two sections due to the slope of the existing ground

The western section is attenuated with a Type B gravel tank, and the eastern section is attenuated within the formal parking area with Type B permeable subbase prior to discharging at as close as feasible to greenfield runoff rates via complex flow controls (combined orifice plate and hydro-brake manholes).

Proposed Surface water discharge rates have been calculated below and supporting calculations are included in Appendix III.

SURFACE WATER DISCHARGE RATES SUMMARY				
	Area (ha)	Discharge Rates (l/s)		
		2 year/ Q_{BAR}	30 year	100 year
Greenfield Rates Western Section	1.036	8.41	19.35	26.84
Proposed Runoff Rates Western Section +40% CC	1.036	8.3	11.9	20.6
Greenfield Rates Eastern Section	1.41	10.95	25.19	34.93
Proposed Runoff Rates Eastern Section+40% CC	1.41	8.3	30.6	32.4
Combined Greenfield Runoff Rates	2.446	19.36	44.54	61.77
Combined Proposed Runoff Rates+40% CC	2.446	16.6	42.5	53

Calculated Runoff rates

Discharge from the Western Area of the carpark is proposed to be limited to 20.6l/s during the 1 in 100-year event + 40% climate change (1%AEP+CC). Discharge from the Eastern Area of the carpark is proposed to be limited to 32.4l/s during the 1 in 100-year event + 40% climate change (1%AEP+CC). Both rates are shown to be less than, the pre-development greenfield runoff rates for the pre-development scenario.

At this stage infiltration has not been included in the calculations as initial infiltration test results were poor. Further infiltration in the area of the proposed attenuation and across the car park area should be undertaken to understand the potential infiltration potential and allow a calculation of the potential reduction of runoff feasible due to infiltration.

The surface water drainage strategy for the western section of the car park includes a supplementary attenuation storage designed to provide attenuation for an area of 10760m². Based on calculations from Causeway Flow, the attenuation tank plan area is 600m², a porosity of 97%, and the geocellular crate depth of 0.92m (based on the ACO HD crate system). This results in a total attenuation volume of 535m³, calculated using the formula: area × depth × porosity. A complex flow control device, consisting of an orifice plate and a hydro-brake, is proposed to limit the discharge to the flowrates shown above. Total car park runoff can be seen to be less than existing pre-development greenfield runoff rates.

The eastern section requires attenuation for 14,123m² of surface water. A flat section of the formal car park is proposed retain water within the sub-base, which has a minimum storage plan area of 2000m² and a sub-base depth of 0.875m. With a porosity of 30%, a storage volume of 492.5m³ is needed. Total car park runoff can be seen to be less than existing pre-development greenfield runoff rates.

A proposed storm drainage strategy plan layout, proposed levels plan and typical construction details are included in Appendix II.

Greenfield surface water discharge rates have been calculated via the use of UKSuDS greenfield rate runoff tool and supporting calculations utilising causeway flow are included in Appendix III.

12 Foul Water Drainage Strategy

All foul water drainage from the new toilet block is to be routed to the existing foul water network and therefore to the existing foul water treatment plant at south of the site – the capacity of the treatment plant is to be confirmed at the later design stage by specialist.

Given the capacity of the existing water treatment plant is unknown and the large volume of toilets proposed, if the overall population of the site is increased as a result of development then an upgrade in capacity to the existing water treatment plant may be necessary. Any upgrades would need to be confirmed by a treatment specialist at the later design stage.

A route for foul drainage connectivity is shown on the layout plan included in Appendix II.

13 Water Quality

The primary risk to water quality is from the trafficked areas (i.e. low traffic road). This can be treated through the implementation of permeable paving across trafficked areas.

The plans included in Appendix IV shows permeable pavement is proposed to be provided for the external trafficked areas to treat the runoff from the surrounding catchments.

The Pollution Hazard Indices are summarised in Table 4 – Summary of Pollution Hazard Indices for different Land Use below (based on Table 26.2 of The SuDS Manual):

POLLUTION HAZARD INDICES FOR DIFFERENT LAND USE CLASSIFICATIONS				
LAND USE	Pollution Hazard Level	Total Suspended Solids	Metals	Hydrocarbons
Low Traffic Roads	Low	0.5	0.4	0.4
TOTAL POLLUTION HAZARD INDEX		0.5	0.4	0.4

Table 5: Summary of Pollution hazard Indices for different Land Use

When designing suitable treatment trains to deliver adequate treatment prior to discharging to sewers, watercourse or groundwater, consideration of the treatment properties of each SuDS component specified should be reviewed. With the total mitigation potential of a SuDS treatment train determined by:

$$SuDS_1 + 0.5 * SuDS_2 + 0.5 * SuDS_3 \dots = Total\ SuDS\ Mitigation\ Index$$

The Mitigation Indices of the proposed SuDS techniques are summarised in Table 6 below.

INDICATIVE SuDS MITIGATION INDICES FOR DISCHARGES TO SURFACE WATER			
SuDS Component	Total Suspended Solids	Metals	Hydrocarbons
Permeable Paving	0.7	0.6	0.7
TOTAL SUDS MITIGATION INDEX	0.7	0.6	0.7

Table 6: Indicative SuDS Mitigation Indices.

It can be seen that the Total SuDS Mitigation Index $\geq$ Pollution Hazard Index therefore the water treatment provided by this SuDS train is enough to remove the potential pollutants.

14 Schedule of Maintenance

All onsite SuDS and drainage systems will be privately maintained. A long-term maintenance regime should be agreed with the site owners before adoption.

In addition to a long-term maintenance regime, it is recommended that all drainage elements implemented on site should be inspected following the first rainfall event post-construction and monthly for the first quarter following construction.

The property owner will be responsible for the management and maintenance of SuDS devices.

Typical key suds components operation and maintenance activities are provided below.

TYPICAL KEY SUDS COMPONENTS OPERATION AND MAINTENANCE ACITIVIES													
Operation and Maintenance Activity	SuDS Component												
	Pond	Wetlands	Detention Basin	Infiltration Basin	Soakaway	Infiltration Trench	Filter Drain	Modular Storage	Pervious pavement	Swale/Bio retention	Filter Strip	Green Roofs	Proprietary Treatment
Regular Maintenance													
Inspection	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Litter/debris removal	✓	✓	✓	✓	-	✓	✓	-	✓	✓	✓	x	-
Grass cutting	✓	✓	✓	✓	-	✓	✓	-	-	✓	✓	x	x
Weed/invasive plant control	-	-	-	-	x	-	-	x	-	x	-	✓	x
Shrub management	-	-	-	-	x	x	x	x	-	-	-	x	x
Shoreline vegetation management	✓	✓	-	x	x	x	x	x	x	x	x	x	x
Aquatic vegetation management	✓	✓	-	x	x	x	x	x	x	x	x	x	x
Occasional Maintenance													
Sediment management	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Vegetation replacement	-	-	-	-	x	x	x	x	x	-	-	✓	x
Vacuum sweeping and brushing	x	x	x	x	x	x	x	x	✓	x	x	x	x
Remedial Maintenance													
Structure rehabilitation/repair	-	-	-	-	-	-	-	-	-	-	-	-	x
Infiltration surface reconditioning	x	x	x	-	-	-	-	x	-	-	-	x	x
Key:													
Will be Required		May be Required						Not Normally Required					

Table 7: General Maintenance Requirements (Source: CIRIA SuDS Manual C753)

General maintenance of key SuDS components are provided below.

PROPOSED SCHEDULE OF MAINTENANCE FOR BELOW GROUND DRAINAGE				
Item	Visual Inspection	Cleanse / De-sludge	CCTV Survey	Comments
Foul Drainage System (pipework, chambers etc.)	5 years	10 years	10 years	Cleansing to be carried as necessary
Surface Water Drainage System (pipework, chambers etc.)	5 years	10 years	10 years	Cleansing to be carried as necessary
Gullies/Channels	1 year	1 year	N/A	Cleansing to be carried as necessary
Attenuation tanks and catchpits	1 year	As required	N/A	Refer to manufacturers guidance for long term maintenance requirements.
Flow Controls	1 year	As required		Inspect after heavy rainfall and in autumn when leaf fall begins.
Permeable Paving	1 year	Clear debris every 2 years.	N/A	Refer to manufacturers guidance for long term maintenance requirements.

Table 8: Maintenance for Below Ground Drainage

15 Conclusion

QED have prepared a site-specific Flood Risk Assessment (FRA) and Surface Water Drainage Strategy (SWDS) for the development at Saint Hill, Saint Hill Road, East Grinstead, West Sussex, RH19 4JY.

The wider proposed development is for permanent infrastructure to facilitate annual events, permanent road access with hardstanding areas and an additional toilet block. A permanent parking surface to be installed to allow year-round parking and a report for this area has been provided separately for convenience.

The site is located on the south-western side of Saint Hill Road and consists of a field to the west of a group of buildings centred around The Castle and Saint Hill Manor that comprise the UK headquarters of the International Association of Scientologists.

The site currently consists of a field which is used as a car park. It is understood the proposed development is for the transition of this field into a car park, to provide 750 parking spaces, as well as a new toilet block and extended access road.

The British Geological Survey (BGS) Geology of Britain Viewer indicates that the bedrock underlying the site consists of Cuckfield Stone Bed – Sandstone, Calcareous. As well as Upper Grinstead Clay - Mudstone (Source: BGS). There are no superficial deposits underlying the site. Preliminary site investigations, including testing, have been conducted and the results indicate that infiltration is not viable

■ Hove ■ London ■ Manchester ■ Leeds ■ Barcelona

QED Structures Limited. Reg. address: 7 Hove Manor Parade, Hove Street, Hove, East Sussex, BN3 2DF. No: 5559060 England and Wales.

The Environment Agency (EA) mapping for Flood Risk, shows the site located within Flood Zone 1 (Low risk of fluvial or tidal flooding). Locations in flood zone 1 have a low probability of flooding. This means that in any year land has a less than 0.1% chance of flooding from rivers or the sea.

According to the EA Long Term Flood Risk map, the site is at a very low risk of tidal and fluvial flooding. Very low risk means that each year this area has a chance of flooding less than 0.1%. It is understood that the EA Long Term Flood Risk map uses a 15m boundary to determine the flood risk to a development.

According to the EA Long Term Flood Risk map for planning the site is at a very low risk of surface water flooding. Very low risk means that each year this area has a chance of flooding less than 0.1%.

Due to the presence of risk nearby to the proposed developments, Meridian Civils have obtained RoFSW data to better understand the risk of flooding from surface water.

Mapping published within the West Sussex County Council SFRA shows the site to be located in an area with medium potential for groundwater flooding and no recorded groundwater flooding events. The nature of the proposed development means risk from groundwater flooding would be low.

Mapping published within the West Sussex County Council SFRA shows the site to be located in an area with low to medium proneness to flooding from the land. There are no recorded surface water flooding events recorded on or near to the development site. Runoff from the proposed development is to be managed to pre-development greenfield runoff rates to mitigate potential risk to others as a result of development.

Mapping published within the West Sussex County Council SFRA shows the site to be located in an area unaffected by sewer flooding.

Due to the limited flood risk to the site, residual risks are limited to overland flow from an exceedance event and potential surface water risk. In the event of heavy rainfall, the low permeability of the sub soil would limit infiltration, and overland flow could occur.

The drainage strategy proposes managing surface water runoff from the car park by providing attenuation before discharging into an existing watercourse at the west of the site. Outfall to watercourse in wooded area in agreement with adjacent landowner. Relevant permissions to be ascertained prior to construction.

Due to the site's existing slope, the car park is divided into two sections, each controlled by complex flow controls consisting of a hydro-brake and an orifice plate, to restrict discharge to pre-development greenfield rates for the 2, 30 and 100yr rainfall events. The western section is attenuated using an attenuation tank beneath the grassed car park, while the eastern section is attenuated via the permeable subbase of the formal car park.

The western attenuation limits discharge to 20.6 l/s during the 1 in 100-year event + 40% climate change (1%AEP+CC), while the eastern attenuation restricts flow to 32.4 l/s under the same conditions. These rates align with predevelopment greenfield runoff rates calculated using the UKSUDS greenfield runoff estimation tool. All surfacing is proposed to be unlined to promote infiltration where feasible.

All foul water drainage is to be drained to the closest existing foul water drainage connection. Details of the capacity of the existing foul system is to be confirmed at the later design stage.

This report has been produced broadly in accordance with the National Planning Policy Framework (NPPF), West Sussex County Council relevant guidance, West Sussex County Council SFRA and River Medway CFMP.



APPENDIX I – ARCHITECT PLANS

Notes

- All dimensions in millimetres and all levels in metres AOD, unless stated otherwise.
- All drawings to be read in conjunction with relevant details and specification.
- Build up information show indicatively only. Construction details to be provided by Civil Engineer.
- All railing and gate footings/fixings to be Contractor's Design in accordance with Product Supplier's standard and recommendation.
- All works in proximity to existing trees to be carried out by hand.

LANDSCAPE KEY

- Red line boundary
- Site boundary ownership
- P1 - Permeable asphalt to permanent car park only
- P2 - Resin bound gravel to match existing
- P3 - Ground reinforcement grid system with gravel fill - suitable for HGVs
- P4 - Ground reinforcement grid system with grass planting
- P5 - Ground reinforcement grid system with gravel fill - suitable for pedestrians
- K1 - Conservation kerb - upstand
- K1 - Conservation kerb - flush
- K2 - Concrete road kerb - upstand
- K2 - Concrete road kerb - flush
- K3 - Concrete edge
- R1 - Timber rail
- G1 - Automatic barrier
- G2 - Timber gate
- F1 - Cycle stands
- F2 - Litter bin
- Direction of travel

- Existing tree to be retained
- Root Protection Area
- Existing tree to be removed
- Proposed tree planting
- Existing Tulip trees to be relocated or replanted with matching tree size
- PL1 - Ornamental shrub and perennial planting
- PL2 - Swale planting
- PL3 - Low level shrub planting
- PL4 - Native shrub planting
- PL5 - Wildflower meadow planting
- PL6 - Large shrub planting
- PL7 - Wildflower meadow planting

P03 10.03.2025 Issued for planning
P02 28.02.2025 Issued for planning
P01 31.01.2025 Draft Issue for comments
revision / date / description / drawn

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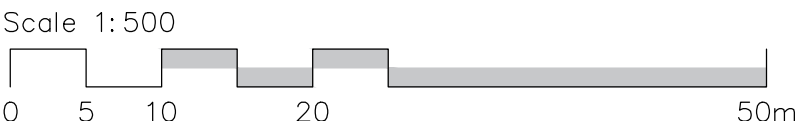
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Saint Hill Manor	1:500@A1
-	date
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title	drawn
Landscape GA Plan	SK
Juhring Field	checked
	MN

drawing number	revision
1232-ENP-SI-00-DR-L-90103	P03

issue For Planning
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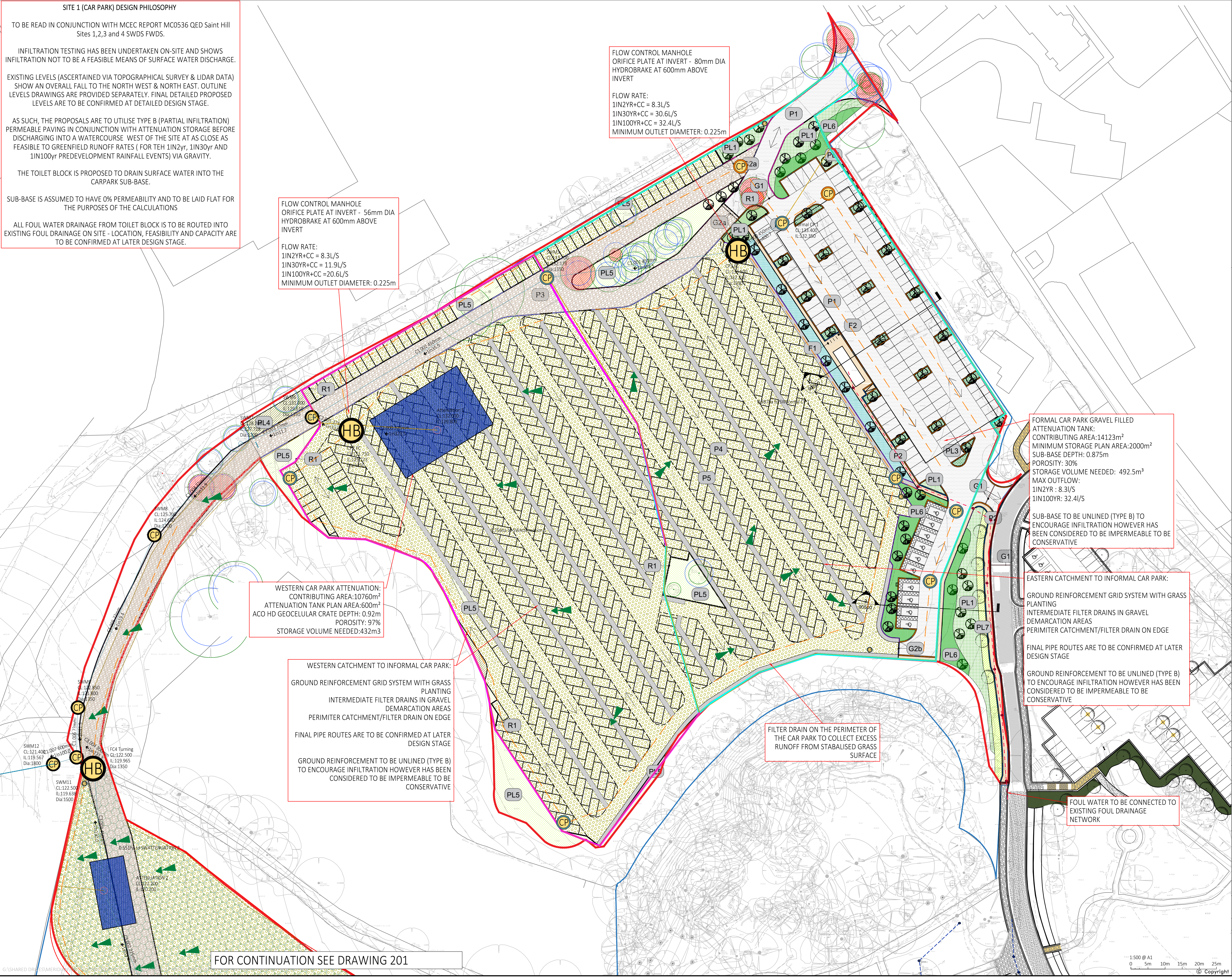
1 Landscape GA Plan - Juhring Field

1 : 500





APPENDIX II – PROPOSED DRAINAGE STRATEGY



SITE 1 (CAR PARK) DESIGN PHILOSOPHY

TO BE READ IN CONJUNCTION WITH MCEC REPORT MC0536 QED Saint Hill Sites 1,2,3 and 4 SWDS FWDS.

INFILTRATION TESTING HAS BEEN UNDERTAKEN ON-SITE AND SHOWS INFILTRATION NOT TO BE A FEASIBLE MEANS OF SURFACE WATER DISCHARGE.

EXISTING LEVELS (ASCERTAINED VIA TOPOGRAPHICAL SURVEY & LIDAR DATA) SHOW AN OVERALL FALL TO THE NORTH WEST & NORTH EAST. OUTLINE LEVELS DRAWINGS ARE PROVIDED SEPARATELY. FINAL DETAILED PROPOSED LEVELS ARE TO BE CONFIRMED AT DETAILED DESIGN STAGE.

AS SUCH, THE PROPOSALS ARE TO UTILISE TYPE B (PARTIAL INFILTRATION) PERMEABLE PAVING IN CONJUNCTION WITH ATTENUATION STORAGE BEFORE DISCHARGING INTO A WATERCOURSE WEST OF THE SITE AT AS CLOSE AS FEASIBLE TO GREENFIELD RUNOFF RATES (FOR TEH 1IN2yr, 1IN30yr AND 1IN100yr PREDEVELOPMENT RAINFALL EVENTS) VIA GRAVITY.

THE TOILET BLOCK IS PROPOSED TO DRAIN SURFACE WATER INTO THE CARPARK SUB-BASE.

SUB-BASE IS ASSUMED TO HAVE 0% PERMEABILITY AND TO BE LAID FLAT FOR THE PURPOSES OF THE CALCULATIONS

ALL FOUL WATER DRAINAGE FROM TOILET BLOCK IS TO BE ROUTED INTO EXISTING FOUL DRAINAGE ON SITE - LOCATION, FEASIBILITY AND CAPACITY ARE TO BE CONFIRMED AT LATER DESIGN STAGE.

FLOW CONTROL MANHOLE
ORIFICE PLATE AT INVERT - 56mm DIA
HYDROBRAKE AT 600mm ABOVE
INVERT

FLOW RATE:
1IN2YR+CC = 8.3L/S
1IN30YR+CC = 11.9L/S
1IN100YR+CC = 20.6L/S
MINIMUM OUTLET DIAMETER: 0.225m

FLOW CONTROL MANHOLE
ORIFICE PLATE AT INVERT - 80mm DIA
HYDROBRAKE AT 600mm ABOVE
INVERT

FLOW RATE:
1IN2YR+CC = 8.3L/S
1IN30YR+CC = 30.6L/S
1IN100YR+CC = 32.4L/S
MINIMUM OUTLET DIAMETER: 0.225m

WESTERN CAR PARK ATTENUATION:
CONTRIBUTING AREA:10760m²
ATTENUATION TANK PLAN AREA:600m²
ACO HD GEOCELLULAR CRATE DEPTH: 0.92m
POROSITY: 97m
STORAGE VOLUME NEEDED:432m³

WESTERN CATCHMENT TO INFORMAL CAR PARK:
GROUND REINFORCEMENT GRID SYSTEM WITH GRASS PLANTING
INTERMEDIATE FILTER DRAINS IN GRAVEL DEMARCATION AREAS
PERIMETER CATCHMENT/FILTER DRAIN ON EDGE

FINAL PIPE ROUTES ARE TO BE CONFIRMED AT LATER DESIGN STAGE

GROUND REINFORCEMENT TO BE UNLINED (TYPE B) TO ENCOURAGE INFILTRATION HOWEVER HAS BEEN CONSIDERED TO BE IMPERMEABLE TO BE CONSERVATIVE

FORMAL CAR PARK GRAVEL FILLED ATTENUATION TANK:
CONTRIBUTING AREA:14123m²
MINIMUM STORAGE PLAN AREA:2000m²
SUB-BASE DEPTH: 0.875m
POROSITY: 30%
STORAGE VOLUME NEEDED: 492.5m³
MAX OUTFLOW:
1IN2YR : 8.3L/S
1IN100YR: 32.4L/S

SUB-BASE TO BE UNLINED (TYPE B) TO ENCOURAGE INFILTRATION HOWEVER HAS BEEN CONSIDERED TO BE IMPERMEABLE TO BE CONSERVATIVE

EASTERN CATCHMENT TO INFORMAL CAR PARK:
GROUND REINFORCEMENT GRID SYSTEM WITH GRASS PLANTING
INTERMEDIATE FILTER DRAINS IN GRAVEL DEMARCATION AREAS
PERIMETER CATCHMENT/FILTER DRAIN ON EDGE

FINAL PIPE ROUTES ARE TO BE CONFIRMED AT LATER DESIGN STAGE

GROUND REINFORCEMENT TO BE UNLINED (TYPE B) TO ENCOURAGE INFILTRATION HOWEVER HAS BEEN CONSIDERED TO BE IMPERMEABLE TO BE CONSERVATIVE

FILTER DRAIN ON THE PERIMETER OF THE CAR PARK TO COLLECT EXCESS RUNOFF FROM STABILISED GRASS SURFACE

FOUL WATER TO BE CONNECTED TO EXISTING FOUL DRAINAGE NETWORK

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LEGEND

- PERMEABLE ASPHALT TO PERMANENT CAR PARK ONLY
- COMPACTED GRAVEL
- GROUND REINFORCEMENT GRID SYSTEM WITH GRASS PLANTING
- PROPOSED PERFORATED SW DRAIN
- SURFACE WATER MANHOLE
- PROPOSED HYDROBRAKE MANHOLE
- CATCHPIT CHAMBER/MANHOLE
- OVERLAND FLOW
- PROPOSED SW PIPE RUN
- EXISTING ROOF RUNOFF CATCHMENT
- PROPOSED FW PIPE RUN
- EXISTING SW PIPE RUN
- EXISTING FW PIPE RUN
- GEOCELLULAR STORAGE
- CELLPAVE HD SURFACING
- CELLPAVE SO SURFACING

P04	For comment	20/03/25	KB	MN
P03	Issued for information	12/02/25	KB	MN
P02	Issued for information	09/12/24	KB	MN
P01	Issued for information	N.Y.I		
REV	DESCRIPTION	DATE	BY	APP

QED structures

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QED PROJECT No.
24-164

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Scientists

PROJECT
Saint Hill Site 1 - Carpark

TITLE
Proposed Drainage Strategy

DRAWN
KB

ENGINEER
KB

CHECKED BY
MN

APPROVED BY
MN

DATE APPROVED
20.03.25

SCALE @ A1
1:500

STATUS
S3 - Suitable for Review and Comment

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24-164-QED-XX-XX-DR-C-101

REVISION
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PROJECT
Saint Hill Site 1 - Carpark

TITLE
Proposed Levels Strategy

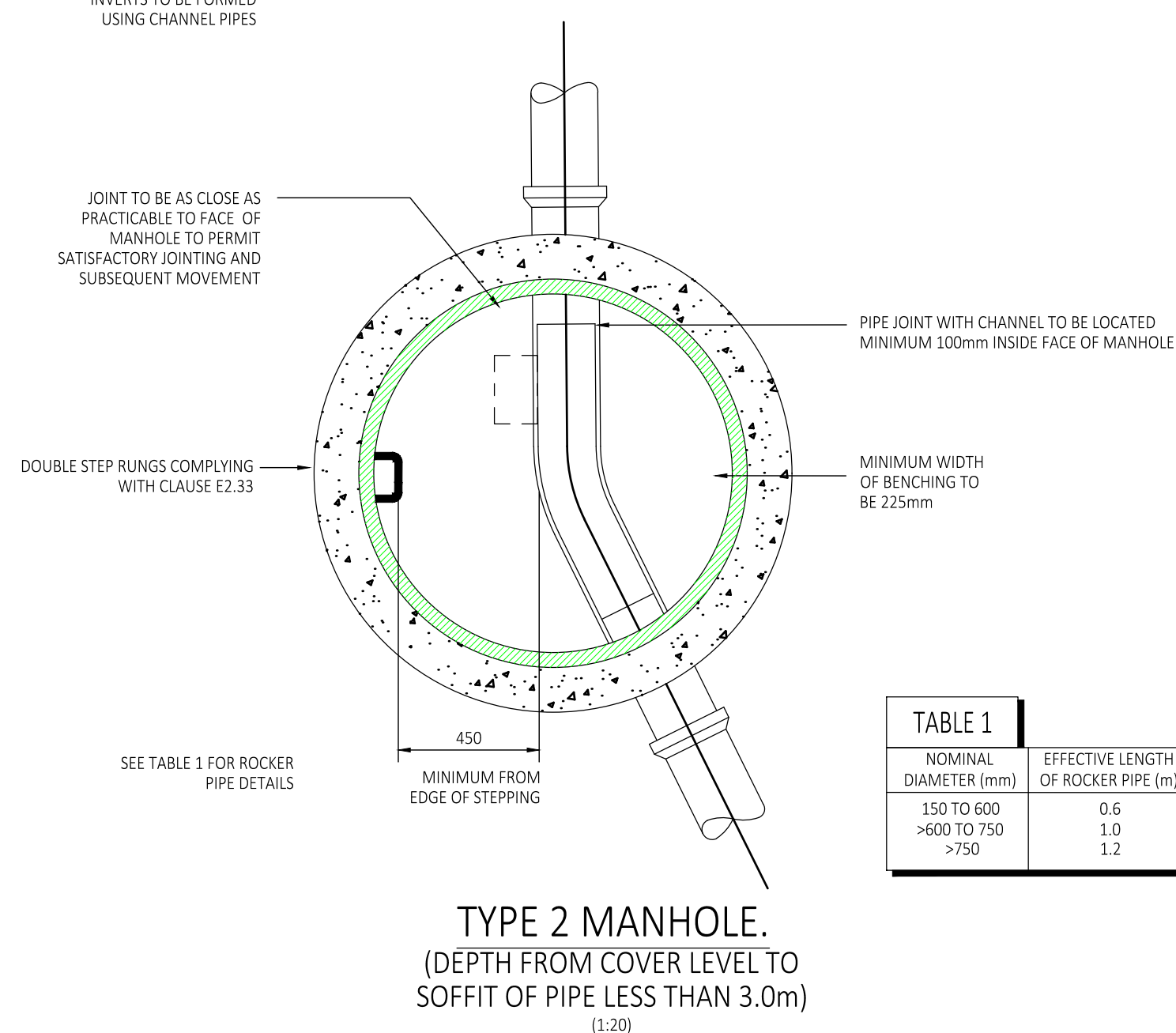
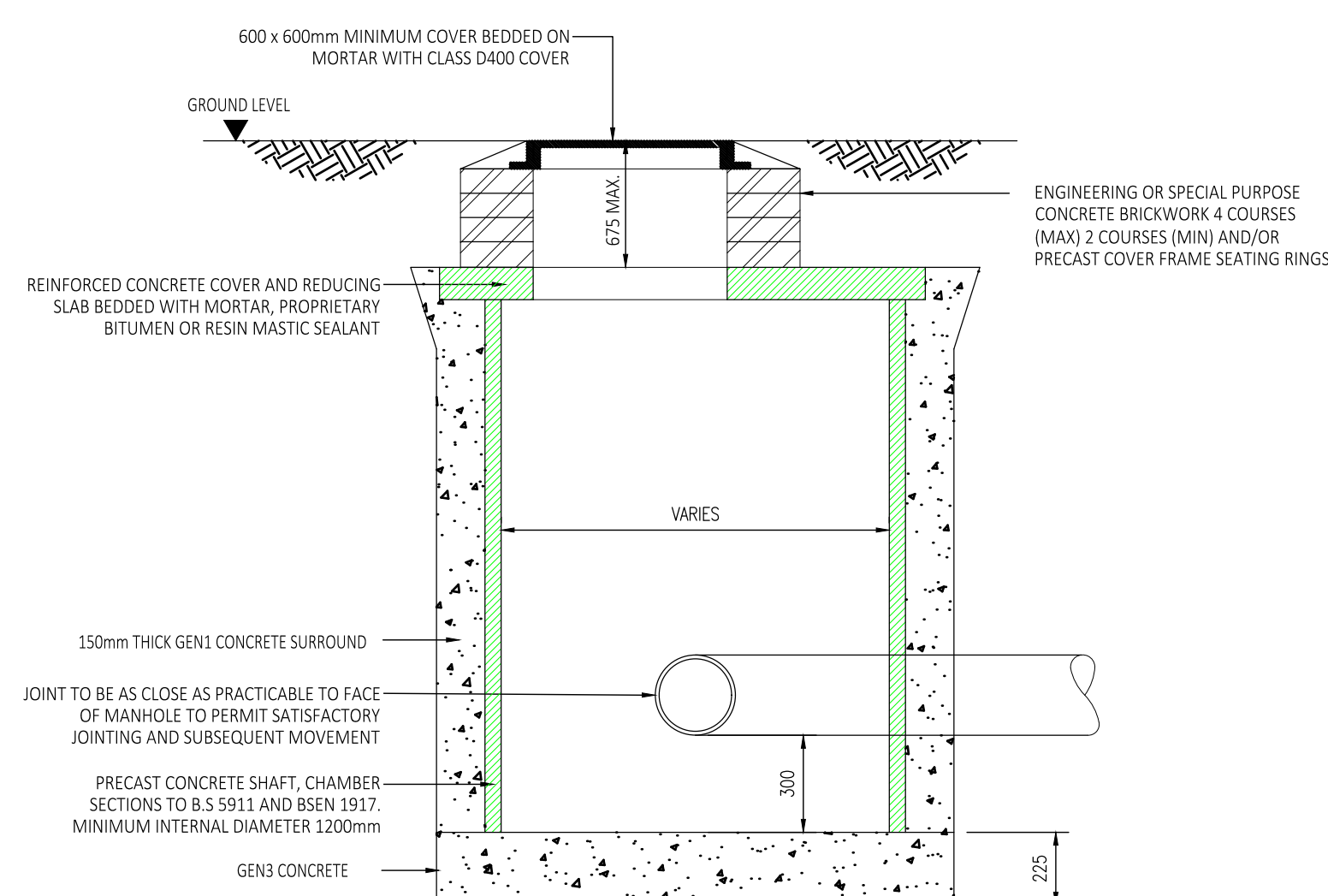
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ALTERNATIVE BASE INVERT LAYOUTS
ALL CONNECTIONS TO BE MIN. SOFFIT TO SOFFIT

NOMINAL DIAMETER (mm)	EFFECT OF RO
150 TO 600	
>600 TO 750	
>750	

UNUSED INLETS TO BE SEALED AND WATERTIGHT

JOINT TO BE SET AS CLOSE AS POSSIBLE TO FACE OF CHAMBER TO PERMIT SATISFACTORY JOINT AND SUBSEQUENT MOVEMENT

FLEXIBLE INLETS/OUTLETS AND/OR BEND (MAX. ANGLE 45°)

WHERE CHAMBERS ARE POSITIONED ON 90° CORNERS, ALWAYS USE THE MAIN CHANNEL BY FITTING A 45° BEND ON THE INLET AND OUTLET

MANHOLE COVER TO SUIT BS EN 124 LOADING. COVERS TO BE D400

MORTAR BEDDING HAUNCHING TO COVER AND FRAME

SURFACE COURSE
BINDER COURSE
BASE COURSE

ACCESS OPENING RESTRICTED TO 350mm DIAMETER

CLASS B ENGINEERING BRICKWORK, CONCRETE BLOCKS OR PRECAST COVER FRAME SEATING RINGS

DOT TYPE 1 SUB-BASE (THICKNESS VARIES) MIN. INTERNAL DIMENSIONS 450mmØ SURROUND 150mm THICK

IN-SITU CONCRETE SLAB TO SUPPORT COVER AND FRAME

FLEXIBLE SEAL

JOINTS BETWEEN BASE AND SHAFT AND BETWEEN SHAFT COMPONENTS TO BE FITTED WITH WATERTIGHT SEALS

FLOW

SILT TRAP OR SIMILAR TO LIMIT RISK FOR BLOCKAGE. REFER TO MANUFACTURER'S DETAILS

MINIMUM 300mm SUMP BELOW INVERT OF LOWEST PIPE

JOINT TO BE SET AS CLOSE AS POSSIBLE TO FACE OF CHAMBER TO PERMIT SATISFACTORY JOINT AND SUBSEQUENT MOVEMENT

NOMINAL DIAMETER (mm)	EFFECTIVE LENGTH OF ROCKER PIPE (m)
150 TO 600	0.6
>600 TO 750	1.0
>750	1.2

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QED PROJECT No.
24-164

CLIENT
International Association of
Scientologists

PROJECT
Saint Hill

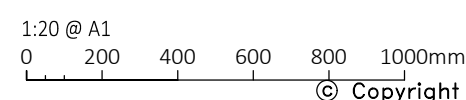
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TYPICAL DRAINAGE DETAILS
SHEET 1

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STATUS	S3 - Suitable for Review and Comment
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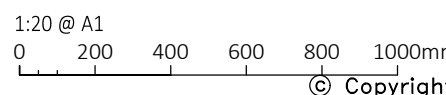
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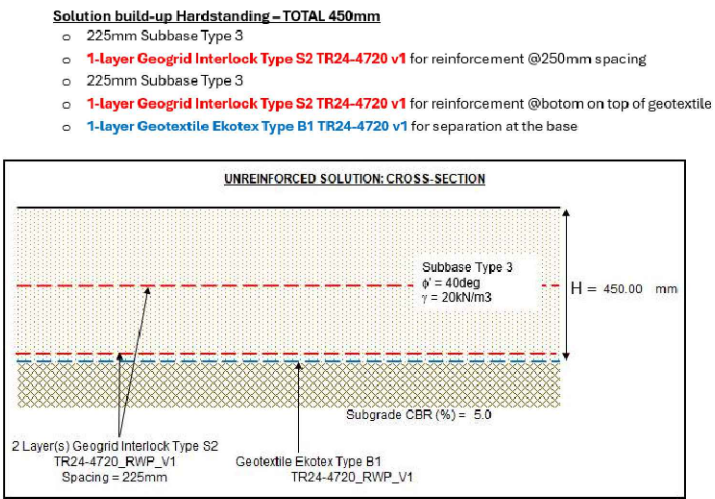
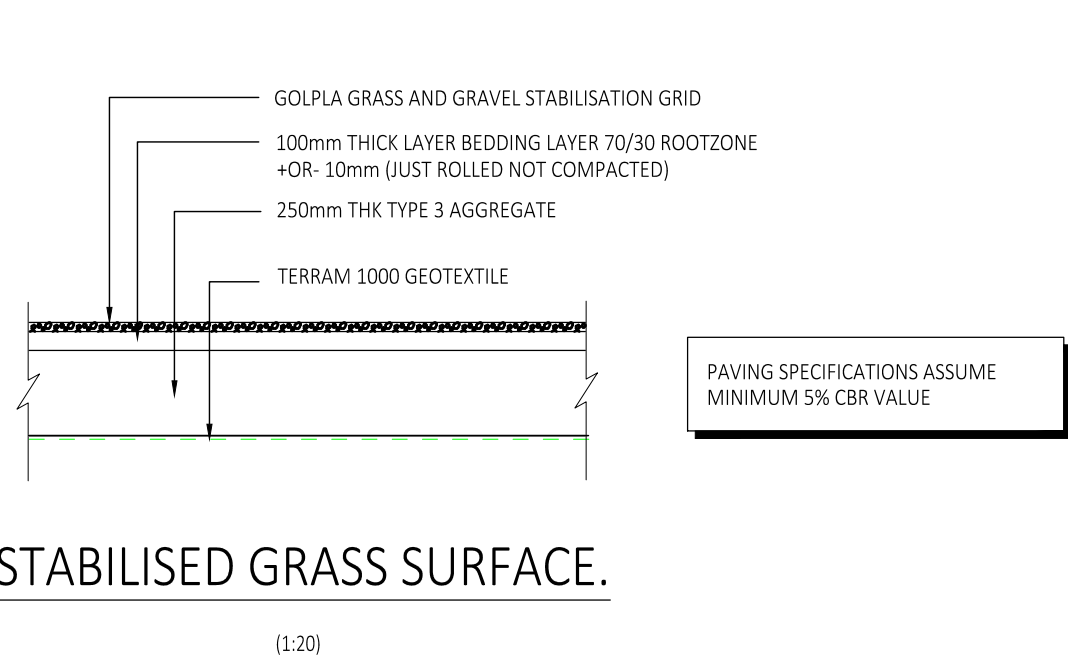
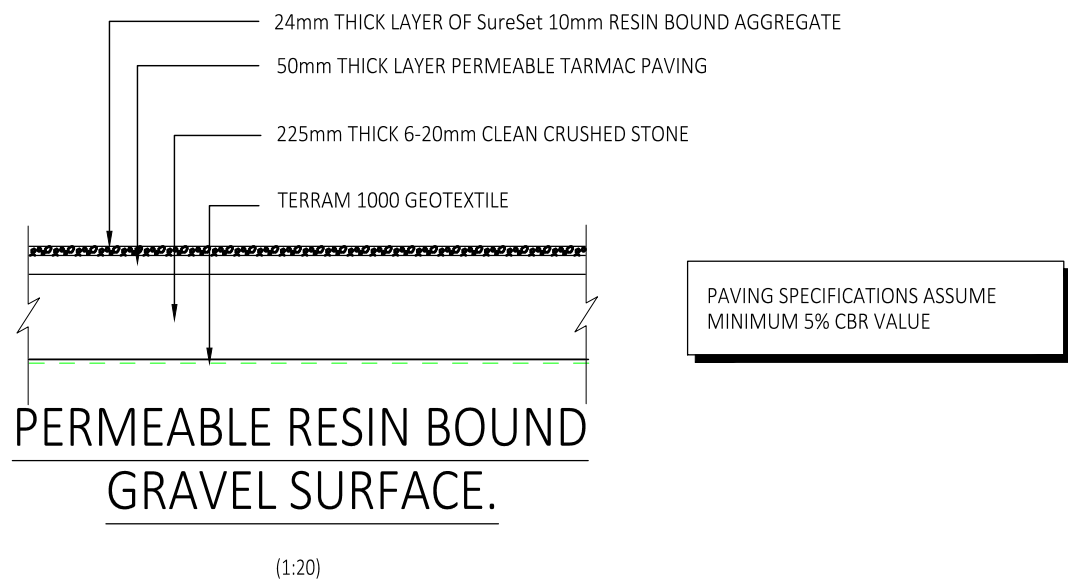
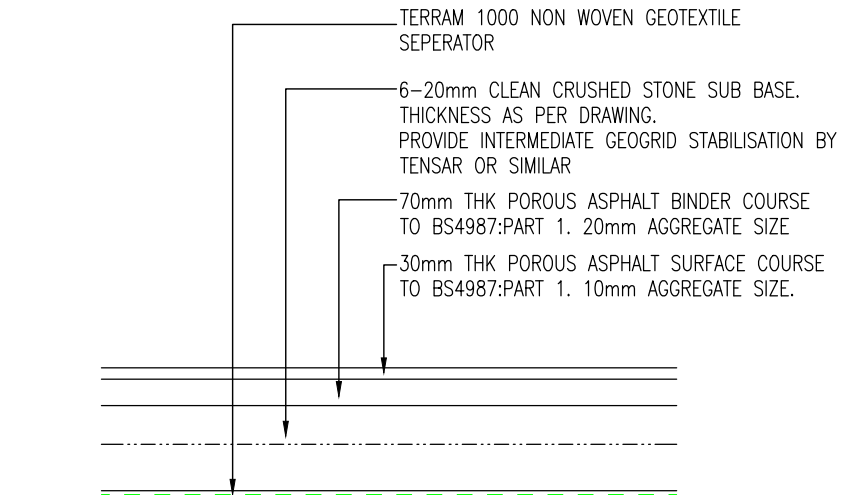
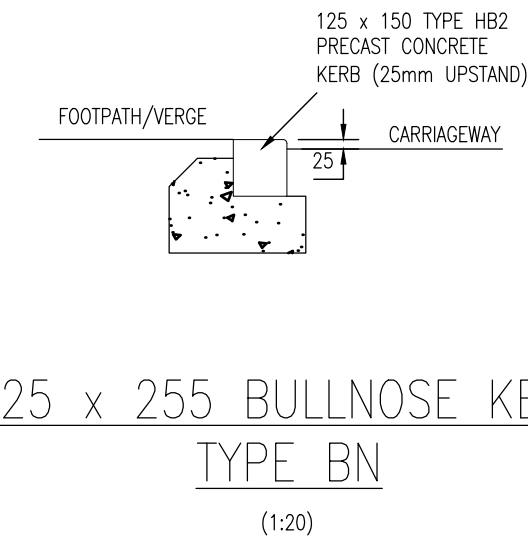
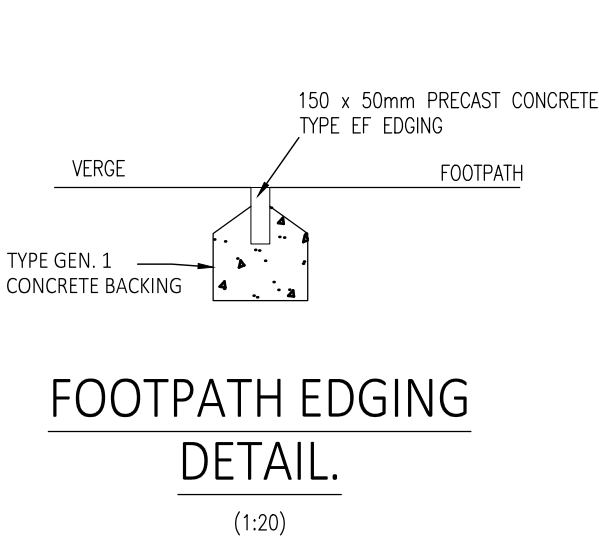




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REVISION
P01





PAVING SPECIFICATIONS ASSUME
MINIMUM 5% CBR VALUE

MARQUEE AREA BASE.
TYPICAL DESIGN BY GEOSYNTHETICS LTD.
MIN 585mm SUB BASE REQUIRED FOR
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REV	DESCRIPTION	DATE	BY	APP
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QED PROJECT No.
24-164

CLIENT
International Association of
Scientologists

PROJECT
Saint Hill

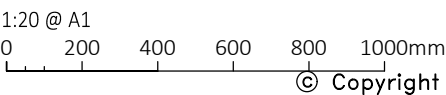
TITLE
TYPICAL PAVING DETAILS

DRAWN IW	ENGINEER KB	CHECKED BY MN	APPROVED BY MN
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DATE APPROVED 10/12/24 SCALE @ A1 1:20

STATUS
S3 - Suitable for Review and Comment

DRAWING No. 24-164-QED-XX-XX-DR-C-450	REVISION P01
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APPENDIX III – CALCULATIONS

Calculated by:

Kalvin Bassi

Site name:

Saint hills main car park

Site location:

rh19 4jy

Site Details

Latitude:

51.10764° N

Longitude:

0.03121° W

Reference:

2661284901

Date:

Jan 30 2025 15:54

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

Runoff estimation approach

IH124

Site characteristics

Total site area (ha):

2.49

Methodology

Q_{BAR} estimation method:

Calculate from SPR and SAAR

SPR estimation method:

Calculate from SOIL type

Notes

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

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

Soil characteristics

	Default	Edited
SOIL type:	4	5
HOST class:	N/A	N/A
SPR/SPRHOST:	0.47	0.53

(2) Are flow rates $< 5.0 \text{ l/s}$?

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

Hydrological characteristics

	Default	Edited
SAAR (mm):	846	846
Hydrological region:	7	7
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

(3) Is $\text{SPR/SPRHOST} \leq 0.3$?

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):	15	19.47
1 in 1 year (l/s):	12.75	16.55
1 in 30 years (l/s):	34.51	44.79
1 in 100 year (l/s):	47.86	62.12
1 in 200 years (l/s):	56.12	72.83

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.

Nodes

	Name	Area (ha)	T of E (mins)	Cover Level (m)	Diameter (mm)	Depth (m)
✓	Formal CP 1	0.987	10.00	133.400		1.050
✓	CP1 FC			133.500	1350	1.188
✓	3			133.700	1350	1.527
✓	Attenuation 1	0.756	10.00	132.000		2.200
✓	CP2 FC			131.750	1350	2.027
✓	6			131.000	1350	1.390
✓	7			128.768	1200	1.050
✓	8			125.700	1200	1.050
✓	9			122.850	1350	1.050
✓	ATTENUATION 2	0.343	10.00	122.200		2.000
✓	11			122.500	1500	2.862
✓	12			121.400	1800	1.833
✓	MARQUEE PAD 14	1.691	10.00	124.400		1.050
✓	FC3 MARQUEE15			124.200	1350	1.148
✓	16			122.900	600	1.050
✓	FC4 Turning			122.500	1350	2.535

Links

	Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
✓	1.000	Formal CP 1	CP1 FC	15.227	0.600	132.350	132.312	0.038	400.7	450	10.25	50.0
✓	1.001	CP1 FC	3	56.360	0.600	132.312	132.173	0.139	405.5	450	11.19	50.0
?	1.002	3	6	79.761	0.600	132.173	129.950	2.223	35.9	450	11.58	50.0
?	2.000	Attenuation 1	CP2 FC	24.907	0.600	129.800	129.723	0.077	323.5	375	10.41	50.0
?	2.001	CP2 FC	6	12.323	0.600	129.723	129.685	0.038	324.3	375	10.62	50.0
?	1.003	6	7	22.210	0.600	129.610	127.718	1.892	11.7	300	11.66	50.0
?	1.004	7	8	36.509	0.600	127.718	124.650	3.068	11.9	300	11.79	50.0
?	1.005	8	9	55.316	0.600	124.650	121.800	2.850	19.4	300	12.05	50.0
✓	1.006	9	11	14.510	0.600	121.800	121.450	0.350	41.5	375	12.13	50.0
✓	3.000	MARQUEE PAD 14	FC3 MARQUEE15	23.828	0.600	123.350	123.052	0.298	80.0	450	10.17	50.0
?	3.001	FC3 MARQUEE15	16	41.905	0.600	123.052	121.850	1.202	34.9	225	10.49	50.0
?	3.002	16	ATTENUATION 2	37.357	0.600	121.850	121.150	0.700	53.4	225	10.84	50.0
?	1.007	11	12	7.061	0.600	119.638	119.567	0.071	100.0	600	12.18	50.0

	Name	US Node	DS Node	Vel (m/s)	Cap (l/s)	Flow (l/s)	US Depth (m)	DS Depth (m)	Σ Area (ha)	Σ Add Inflow (l/s)	Pro Depth (mm)	Pro Velocity (m/s)
	1.000	Formal CP 1	CP1 FC	1.009	160.5	133.8	0.600	0.738	0.987	0.0	315	1.124
	1.001	CP1 FC	3	1.003	159.6	133.8	0.738	1.077	0.987	0.0	317	1.118
	1.002	3	6	3.402	541.1	133.8	1.077	0.600	0.987	0.0	152	2.840
	2.000	Attenuation 1	CP2 FC	1.002	110.6	102.5	1.825	1.652	0.756	0.0	287	1.131
	2.001	CP2 FC	6	1.000	110.5	102.5	1.652	0.940	0.756	0.0	287	1.130
	1.003	6	7	4.613	326.1	236.3	1.090	0.750	1.743	0.0	190	5.012
	1.004	7	8	4.581	323.8	236.3	0.750	0.750	1.743	0.0	191	4.982
	1.005	8	9	3.584	253.4	236.3	0.750	0.750	1.743	0.0	231	4.049
	1.006	9	11	2.821	311.5	236.3	0.675	0.675	1.743	0.0	245	3.089
	3.000	MARQUEE PAD 14	FC3 MARQUEE15	2.275	361.8	229.2	0.600	0.698	1.691	0.0	261	2.402
	3.001	FC3 MARQUEE15	16	2.223	88.4	229.2	0.923	0.825	1.691	0.0	225	2.264
	3.002	16	ATTENUATION 2	1.794	71.3	229.2	0.825	0.825	1.691	0.0	225	1.827
	1.007	11	12	2.435	688.5	512.0	2.262	1.233	3.778	0.0	387	2.655

Links

Name	US Node	DS Node	Length (m)	ks (mm) / n	US IL (m)	DS IL (m)	Fall (m)	Slope (1:X)	Dia (mm)	T of C (mins)	Rain (mm/hr)
? 3.003	ATTENUATION 2	FC4 Turning	35.271	0.600	120.200	119.965	0.235	150.0	450	11.19	50.0
? 3.004	FC4 Turning	11	6.406	0.600	119.965	119.938	0.027	237.3	300	11.30	50.0

Name	US Node	DS Node	Vel (m/s)	Cap (l/s)	Flow (l/s)	US Depth (m)	DS Depth (m)	Σ Area (ha)	Σ Add Inflow (l/s)	Pro Depth (mm)	Pro Velocity (m/s)
3.003	ATTENUATION 2	FC4 Turning	1.657	263.6	275.7	1.550	2.085	2.034	0.0	395	1.863
3.004	FC4 Turning	11	1.016	71.8	275.7	2.235	2.262	2.034	0.0	300	1.029

Pipeline Schedule

Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
1.000	15.227	400.7	450	Circular	133.400	132.350	0.600	133.500	132.312	0.738
1.001	56.360	405.5	450	Circular	133.500	132.312	0.738	133.700	132.173	1.077
1.002	79.761	35.9	450	Circular	133.700	132.173	1.077	131.000	129.950	0.600
2.000	24.907	323.5	375	Circular	132.000	129.800	1.825	131.750	129.723	1.652
2.001	12.323	324.3	375	Circular	131.750	129.723	1.652	131.000	129.685	0.940
1.003	22.210	11.7	300	SW_Perforated	131.000	129.610	1.090	128.768	127.718	0.750
1.004	36.509	11.9	300	SW_Perforated	128.768	127.718	0.750	125.700	124.650	0.750
1.005	55.316	19.4	300	SW_Perforated	125.700	124.650	0.750	122.850	121.800	0.750

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	Formal CP 1		Junction		CP1 FC	1350	Manhole	SW_HB
1.001	CP1 FC	1350	Manhole	SW_HB	3	1350	Manhole	SW_Standard
1.002	3	1350	Manhole	SW_Standard	6	1350	Manhole	SW_Standard
2.000	Attenuation 1		Junction		CP2 FC	1350	Manhole	SW_HB
2.001	CP2 FC	1350	Manhole	SW_HB	6	1350	Manhole	SW_Standard
1.003	6	1350	Manhole	SW_Standard	7	1200	Manhole	SW_Standard
1.004	7	1200	Manhole	SW_Standard	8	1200	Manhole	SW_Standard
1.005	8	1200	Manhole	SW_Standard	9	1350	Manhole	SW_Standard

Pipeline Schedule

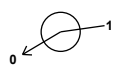
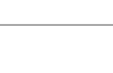
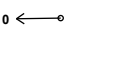
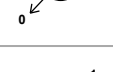
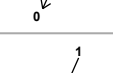
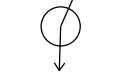
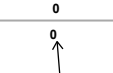
Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
1.006	14.510	41.5	375	SW_Perforated	122.850	121.800	0.675	122.500	121.450	0.675
3.000	23.828	80.0	450	Circular	124.400	123.350	0.600	124.200	123.052	0.698
3.001	41.905	34.9	225	Circular	124.200	123.052	0.923	122.900	121.850	0.825
3.002	37.357	53.4	225	Circular	122.900	121.850	0.825	122.200	121.150	0.825
1.007	7.061	100.0	600	Circular	122.500	119.638	2.262	121.400	119.567	1.233
3.003	35.271	150.0	450	Circular	122.200	120.200	1.550	122.500	119.965	2.085
3.004	6.406	237.3	300	Circular	122.500	119.965	2.235	122.500	119.938	2.262

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.006	9	1350	Manhole	SW_Standard	11	1500	Manhole	SW_Standard
3.000	MARQUEE PAD 14		Junction		FC3 MARQUEE15	1350	Manhole	SW_Standard
3.001	FC3 MARQUEE15	1350	Manhole	SW_Standard	16	600	Manhole	SW_Standard
3.002	16	600	Manhole	SW_Standard	ATTENUATION 2		Junction	
1.007	11	1500	Manhole	SW_Standard	12	1800	Manhole	SW_Standard
3.003	ATTENUATION 2		Junction		FC4 Turning	1350	Manhole	SW_Standard
3.004	FC4 Turning	1350	Manhole	SW_Standard	11	1500	Manhole	SW_Standard

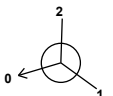
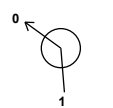
Manhole Schedule

Node	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)
Formal CP 1	133.400	1.050					
					0	132.350	450
CP1 FC	133.500	1.188	1350		1	132.312	450
					0	132.312	450

Manhole Schedule

Node	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)	
3	133.700	1.527	1350		1	1.001	132.173	450
					0	1.002	132.173	450
Attenuation 1	132.000	2.200			0	2.000	129.800	375
CP2 FC	131.750	2.027	1350		1	2.000	129.723	375
					0	2.001	129.723	375
6	131.000	1.390	1350		1	2.001	129.685	375
					2	1.002	129.950	450
					0	1.003	129.610	300
7	128.768	1.050	1200		1	1.003	127.718	300
					0	1.004	127.718	300
8	125.700	1.050	1200		1	1.004	124.650	300
					0	1.005	124.650	300
9	122.850	1.050	1350		1	1.005	121.800	300
					0	1.006	121.800	375
ATTENUATION 2	122.200	2.000			1	3.002	121.150	225
					0	3.003	120.200	450

Manhole Schedule

Node	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)	
11	122.500	2.862	1500		1	3.004	119.938	300
					2	1.006	121.450	375
					0	1.007	119.638	600
12	121.400	1.833	1800		1	1.007	119.567	600
MARQUEE PAD 14	124.400	1.050						
FC3 MARQUEE15	124.200	1.148	1350		0	3.000	123.350	450
					1	3.000	123.052	450
					0	3.001	123.052	225
16	122.900	1.050	600		1	3.001	121.850	225
					0	3.002	121.850	225
					1	3.003	119.965	450
FC4 Turning	122.500	2.535	1350		0	3.004	119.965	300

Simulation Settings

Rainfall Methodology	FEH-22	Winter CV	0.840	Drain Down Time (mins)	1440	Check Discharge Rate(s)	x
Rainfall Events	Singular	Analysis Speed	Normal	Additional Storage (m³/ha)	0.0	Check Discharge Volume	x
Summer CV	0.750	Skip Steady State	x	Starting Level (m)			

Storm Durations

15 | 30 | 60 | 120 | 180 | 240 | 360 | 720 | 1440

Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)	Return Period (years)	Climate Change (CC %)	Additional Area (A %)	Additional Flow (Q %)
2	40	0	0	100	40	0	0
30	40	0	0				

Node CP1 FC Online Orifice Control

Flap Valve	x	Replaces Downstream Link	✓	Design Depth (m)	0.600	Diameter (m)	0.080
Downstream Link	1.001	Invert Level (m)	132.312	Design Flow (l/s)	10.0	Discharge Coefficient	0.600

Node CP1 FC Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	132.850	Product Number	CTL-SHE-0203-2000-0600-2000
Design Depth (m)	0.600	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	20.0	Min Node Diameter (mm)	1200

Node CP2 FC Online Orifice Control

Flap Valve	x	Replaces Downstream Link	✓	Design Depth (m)	0.600	Diameter (m)	0.056
Downstream Link	2.001	Invert Level (m)	129.723	Design Flow (l/s)	5.0	Discharge Coefficient	0.600

Node CP2 FC Online Hydro-Brake® Control

Flap Valve	x	Objective	(HE) Minimise upstream storage
Replaces Downstream Link	✓	Sump Available	✓
Invert Level (m)	130.350	Product Number	CTL-SHE-0180-1500-0600-1500
Design Depth (m)	0.600	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	15.0	Min Node Diameter (mm)	1200

Node FC4 Turning Online Orifice Control

Flap Valve	x	Replaces Downstream Link	✓	Design Depth (m)	0.500	Diameter (m)	0.092
Downstream Link	3.004	Invert Level (m)	119.965	Design Flow (l/s)	12.0	Discharge Coefficient	0.600

Node FC3 MARQUEE15 Online Orifice Control

Flap Valve	x	Replaces Downstream Link	✓	Design Depth (m)	1.000	Diameter (m)	0.069
Downstream Link	3.001	Invert Level (m)	123.052	Design Flow (l/s)	10.0	Discharge Coefficient	0.600

Node Formal CP 1 Carpark Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Width (m)	20.000	Depth (m)	
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	132.350	Length (m)	100.000	Inf Depth (m)	
Safety Factor	2.0	Time to half empty (mins)	336	Slope (1:X)	999.0		

Node Attenuation 1 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	129.800
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	456

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	600.0	0.0	0.900	600.0	0.0	0.901	0.0	0.0

Node ATTENUATION 2 Depth/Area Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Safety Factor	2.0	Invert Level (m)	120.200
Side Inf Coefficient (m/hr)	0.00000	Porosity	0.95	Time to half empty (mins)	192

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	200.0	0.0	0.900	200.0	0.0	0.910	0.0	0.0

Node MARQUEE PAD 14 Carpark Storage Structure

Base Inf Coefficient (m/hr)	0.00000	Porosity	0.30	Width (m)	100.000	Depth (m)	
Side Inf Coefficient (m/hr)	0.00000	Invert Level (m)	123.350	Length (m)	82.000	Inf Depth (m)	
Safety Factor	2.0	Time to half empty (mins)	990	Slope (1:X)	999.0		

Results for 2 year +40% CC Critical Storm Duration. Lowest mass balance: 99.55%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
360 minute winter	Formal CP 1	288	132.735	0.385	36.7	200.9975	0.0000	OK
360 minute winter	CP1 FC	288	132.735	0.423	8.3	0.6056	0.0000	OK
360 minute winter	3	288	132.212	0.039	8.3	0.0555	0.0000	OK
720 minute winter	Attenuation 1	555	130.132	0.332	17.3	189.0322	0.0000	OK
720 minute winter	CP2 FC	555	130.132	0.409	4.0	0.5846	0.0000	SURCHARGED
360 minute winter	6	296	129.650	0.040	12.2	0.0577	0.0000	OK
360 minute winter	7	296	127.758	0.040	12.2	0.0449	0.0000	OK
360 minute winter	8	296	124.695	0.045	12.2	0.0505	0.0000	OK
360 minute winter	9	296	121.852	0.052	12.2	0.0748	0.0000	OK
240 minute winter	ATTENUATION 2	180	120.476	0.276	36.7	52.5164	0.0000	OK
360 minute winter	11	248	119.722	0.084	24.2	0.1477	0.0000	OK
360 minute winter	12	248	119.643	0.076	24.2	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	Formal CP 1	1.000	CP1 FC	8.3	0.220	0.052	2.2777	
360 minute winter	CP1 FC	Orifice	3	8.3				
360 minute winter	CP1 FC	Hydro-Brake®	3	0.0				
360 minute winter	3	1.002	6	8.3	1.267	0.015	0.5206	
720 minute winter	Attenuation 1	2.000	CP2 FC	4.0	0.171	0.037	2.6581	
720 minute winter	CP2 FC	Orifice	6	4.0				
720 minute winter	CP2 FC	Hydro-Brake®	6	0.0				
360 minute winter	6	1.003	7	12.2	2.203	0.038	0.1234	
360 minute winter	7	1.004	8	12.2	2.038	0.038	0.2192	
360 minute winter	8	1.005	9	12.2	1.665	0.048	0.4070	
360 minute winter	9	1.006	11	12.2	1.356	0.039	0.1309	
240 minute winter	ATTENUATION 2	3.003	FC4 Turning	35.4	0.372	0.134	4.5948	
360 minute winter	11	1.007	12	24.2	1.099	0.035	0.1560	1211.5

Results for 2 year +40% CC Critical Storm Duration. Lowest mass balance: 99.55%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
720 minute winter	MARQUEE PAD 14	645	123.565	0.215	48.7	428.6726	0.0000	OK
360 minute winter	FC3 MARQUEE15	296	123.581	0.529	30.0	0.7573	0.0000	SURCHARGED
720 minute winter	16	615	121.897	0.047	7.0	0.0134	0.0000	OK
240 minute winter	FC4 Turning	172	120.490	0.525	35.4	0.7507	0.0000	SURCHARGED

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
720 minute winter	MARQUEE PAD 14	3.000	FC3 MARQUEE15	29.2	0.330	0.081	2.7804	
360 minute winter	FC3 MARQUEE15	Orifice	16	7.0				
720 minute winter	16	3.002	ATTENUATION 2	6.9	1.143	0.097	0.2258	
240 minute winter	FC4 Turning	Orifice	11	12.2				

Results for 30 year +40% CC Critical Storm Duration. Lowest mass balance: 99.55%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
240 minute winter	Formal CP 1	180	133.060	0.710	99.8	395.6292	0.0000	SURCHARGED
240 minute winter	CP1 FC	180	133.058	0.746	30.6	1.0675	0.0000	SURCHARGED
240 minute winter	3	180	132.246	0.073	30.6	0.1039	0.0000	OK
720 minute winter	Attenuation 1	510	130.452	0.652	33.1	371.6378	0.0000	SURCHARGED
720 minute winter	CP2 FC	510	130.451	0.728	11.9	1.0417	0.0000	SURCHARGED
360 minute winter	6	272	129.684	0.074	41.0	0.1065	0.0000	OK
360 minute winter	7	272	127.790	0.072	41.0	0.0810	0.0000	OK
360 minute winter	8	272	124.731	0.081	41.1	0.0917	0.0000	OK
360 minute winter	9	272	121.898	0.098	41.1	0.1406	0.0000	OK
180 minute winter	ATTENUATION 2	152	120.863	0.663	49.7	126.0309	0.0000	SURCHARGED
360 minute winter	11	272	119.770	0.132	57.1	0.2327	0.0000	OK
360 minute winter	12	272	119.681	0.114	57.1	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)
240 minute winter	Formal CP 1	1.000	CP1 FC	30.6	0.263	0.191	2.4126	
240 minute winter	CP1 FC	Orifice	3	11.2				
240 minute winter	CP1 FC	Hydro-Brake®	3	19.4				
240 minute winter	3	1.002	6	30.6	1.870	0.057	1.3066	
720 minute winter	Attenuation 1	2.000	CP2 FC	11.9	0.178	0.108	2.7472	
720 minute winter	CP2 FC	Orifice	6	5.5				
720 minute winter	CP2 FC	Hydro-Brake®	6	6.4				
360 minute winter	6	1.003	7	41.0	3.103	0.126	0.2939	
360 minute winter	7	1.004	8	41.1	2.911	0.127	0.5153	
360 minute winter	8	1.005	9	41.1	2.325	0.162	0.9789	
360 minute winter	9	1.006	11	41.1	1.886	0.132	0.3159	
180 minute winter	ATTENUATION 2	3.003	FC4 Turning	41.1	0.451	0.156	5.5885	
360 minute winter	11	1.007	12	57.1	1.377	0.083	0.2932	2075.2

Results for 30 year +40% CC Critical Storm Duration. Lowest mass balance: 99.55%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
720 minute winter	MARQUEE PAD 14	705	123.792	0.442	74.0	985.8224	0.0000	OK
720 minute winter	FC3 MARQUEE15	705	123.792	0.740	31.0	1.0588	0.0000	SURCHARGED
720 minute winter	16	705	121.902	0.052	8.3	0.0148	0.0000	OK
180 minute winter	FC4 Turning	152	120.862	0.897	41.1	1.2840	0.0000	SURCHARGED

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
720 minute winter	MARQUEE PAD 14	3.000	FC3 MARQUEE15	31.0	0.343	0.086	3.7678	
720 minute winter	FC3 MARQUEE15	Orifice	16	8.3				
720 minute winter	16	3.002	ATTENUATION 2	8.3	1.207	0.117	0.2584	
180 minute winter	FC4 Turning	Orifice	11	16.3				

Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 99.55%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
240 minute winter	Formal CP 1	184	133.221	0.871	124.1	492.5193	0.0000	FLOOD RISK
240 minute winter	CP1 FC	184	133.219	0.907	32.2	1.2984	0.0000	FLOOD RISK
240 minute winter	3	184	132.247	0.074	32.2	0.1065	0.0000	OK
360 minute winter	Attenuation 1	272	130.556	0.756	69.7	431.0596	0.0000	SURCHARGED
360 minute winter	CP2 FC	272	130.553	0.830	20.6	1.1879	0.0000	SURCHARGED
360 minute winter	6	272	129.695	0.085	52.7	0.1215	0.0000	OK
360 minute winter	7	272	127.800	0.082	52.7	0.0922	0.0000	OK
360 minute winter	8	272	124.742	0.092	52.7	0.1043	0.0000	OK
360 minute winter	9	272	121.913	0.113	52.8	0.1615	0.0000	OK
180 minute winter	ATTENUATION 2	164	121.056	0.856	60.7	162.5607	0.0000	SURCHARGED
240 minute winter	11	192	119.786	0.148	70.7	0.2614	0.0000	OK
240 minute winter	12	192	119.694	0.127	70.7	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)
240 minute winter	Formal CP 1	1.000	CP1 FC	32.2	0.269	0.201	2.4126	
240 minute winter	CP1 FC	Orifice	3	12.4				
240 minute winter	CP1 FC	Hydro-Brake®	3	20.0				
240 minute winter	3	1.002	6	32.2	1.897	0.059	1.3530	
360 minute winter	Attenuation 1	2.000	CP2 FC	20.6	0.204	0.186	2.7472	
360 minute winter	CP2 FC	Orifice	6	5.9				
360 minute winter	CP2 FC	Hydro-Brake®	6	14.7				
360 minute winter	6	1.003	7	52.7	3.315	0.162	0.3534	
360 minute winter	7	1.004	8	52.7	3.122	0.163	0.6173	
360 minute winter	8	1.005	9	52.8	2.482	0.208	1.1773	
360 minute winter	9	1.006	11	52.8	2.012	0.169	0.3806	
180 minute winter	ATTENUATION 2	3.003	FC4 Turning	37.2	0.409	0.141	5.5885	
240 minute winter	11	1.007	12	70.7	1.455	0.103	0.3435	2232.7

Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 99.55%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
1440 minute winter	MARQUEE PAD 14	1350	123.935	0.585	56.9	1336.4720	0.0000	SURCHARGED
1440 minute winter	FC3 MARQUEE15	1350	123.935	0.882	24.2	1.2628	0.0000	FLOOD RISK
1440 minute winter	16	1350	121.905	0.055	9.2	0.0155	0.0000	OK
180 minute winter	FC4 Turning	164	121.054	1.089	37.2	1.5588	0.0000	SURCHARGED

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
1440 minute winter	MARQUEE PAD 14	3.000	FC3 MARQUEE15	24.2	0.245	0.067	3.7754	
1440 minute winter	FC3 MARQUEE15	Orifice	16	9.2				
1440 minute winter	16	3.002	ATTENUATION 2	9.2	1.239	0.128	0.2759	
180 minute winter	FC4 Turning	Orifice	11	18.0				