

Saint Hill
Site 1 Car Park,
Saint Hill Road,
East Grinstead, West Sussex, RH19 4JY
FLOOD RISK ASSESSMENT & SURFACE WATER DRAINAGE
STRATEGY

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Appendix I – Architect Plans and topographical survey

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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.

Discharge from the Western Area of the carpark is proposed to be limited to 21.3l/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 25.8/s during the 1 in 100-year event + 40% climate change (1%AEP+CC).

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.

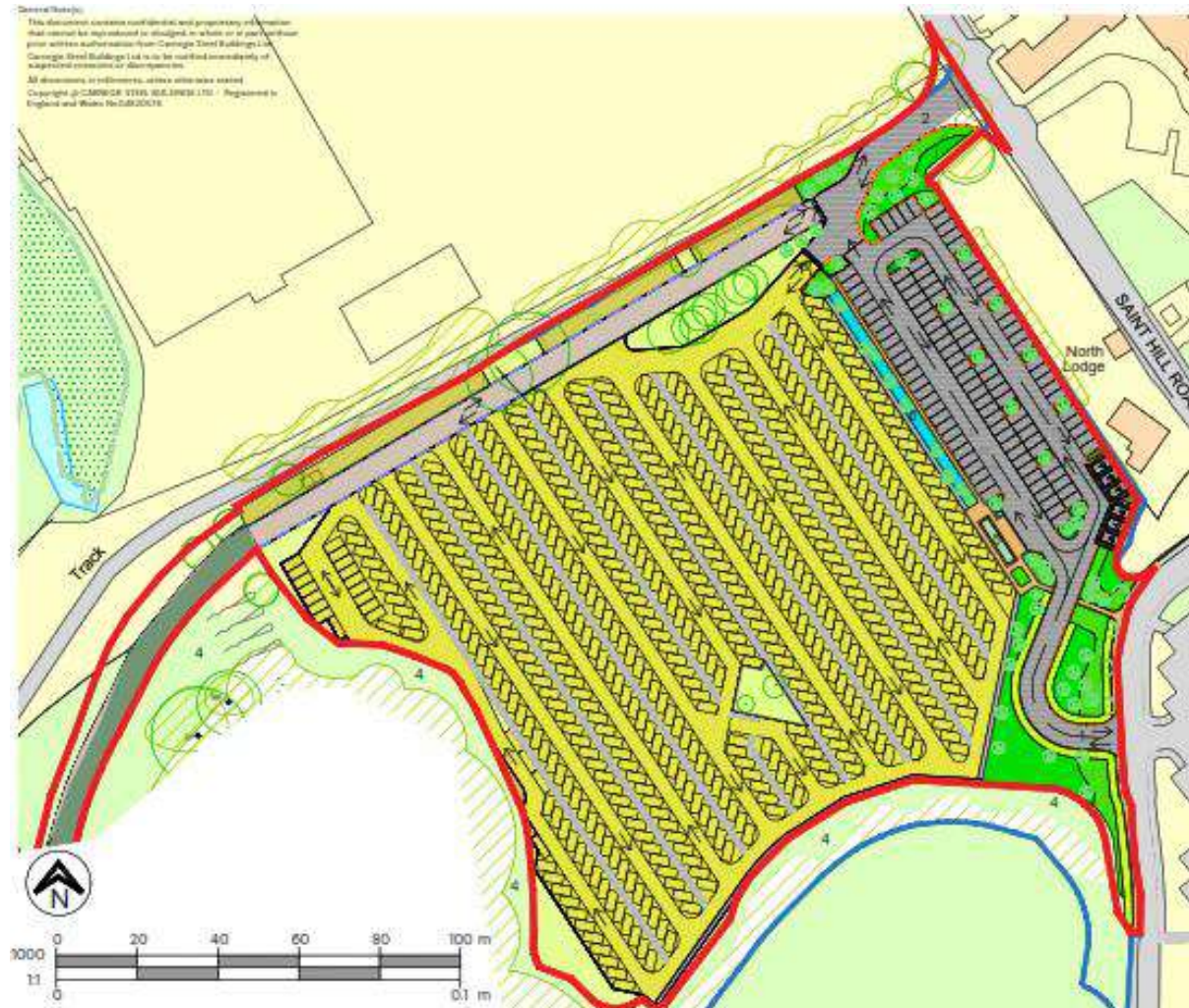


Figure 2: Proposed Site Plan

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.

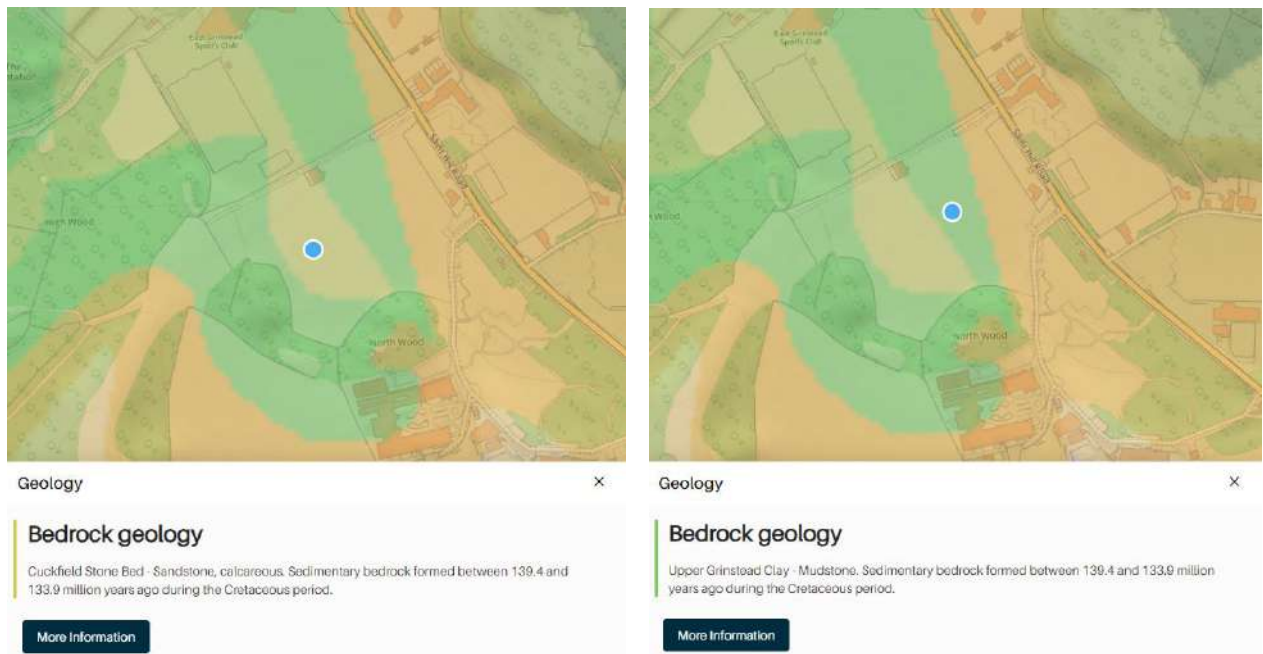


Figure 3: BGS geology map extracts

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.

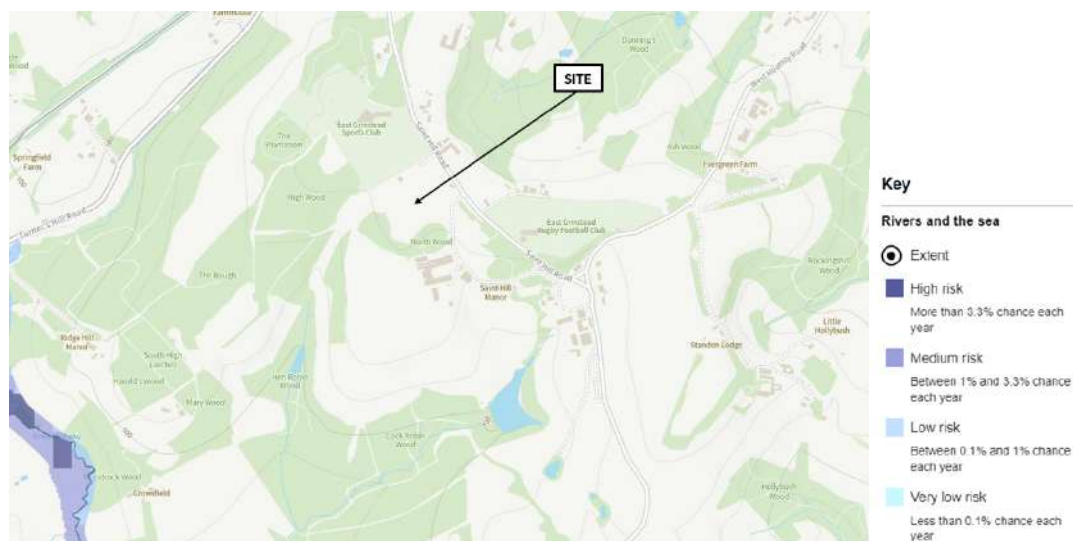


Figure 7: EA Long term risk of flooding map extract - Rivers and the Sea

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

It can be seen a potential flow path is shown across the car park area but it is not clear from the topographic survey where this would be located.

Proposals are to limit level changes/earthworks as much as feasible so any flow path can be maintained as part of the proposed work.

The introduction of sustainable drainage features in the car park areas and additional surface water storage would lessen the impact of any overland flow.



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 less than 200mm flooding for the most part for the 2040 to 2060 event. Local areas are shown at 200mm.

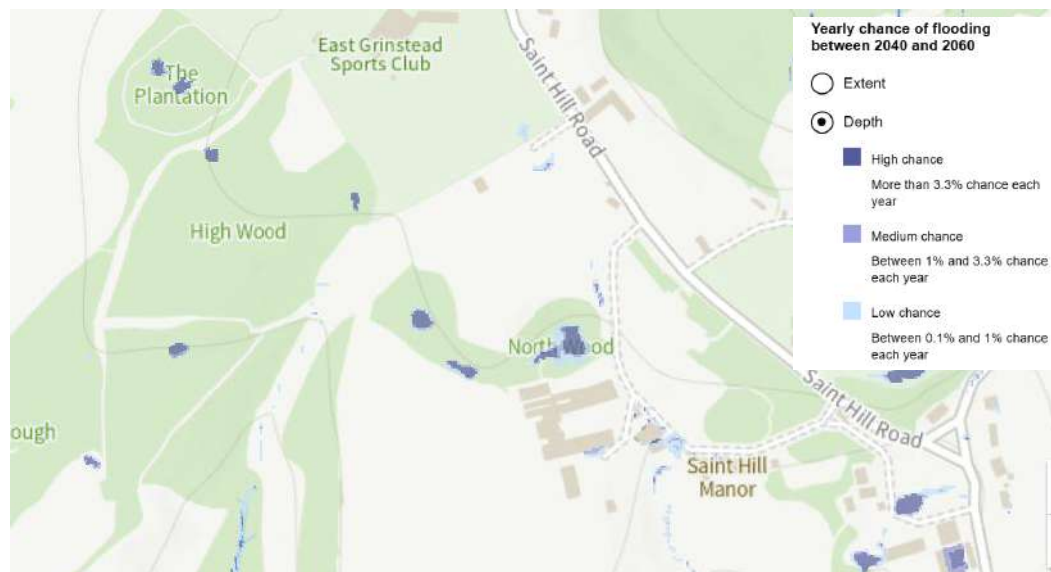


Figure 9: EA RoFSW data showing chance of 200mm water depth during the 2040-2060 surface water event

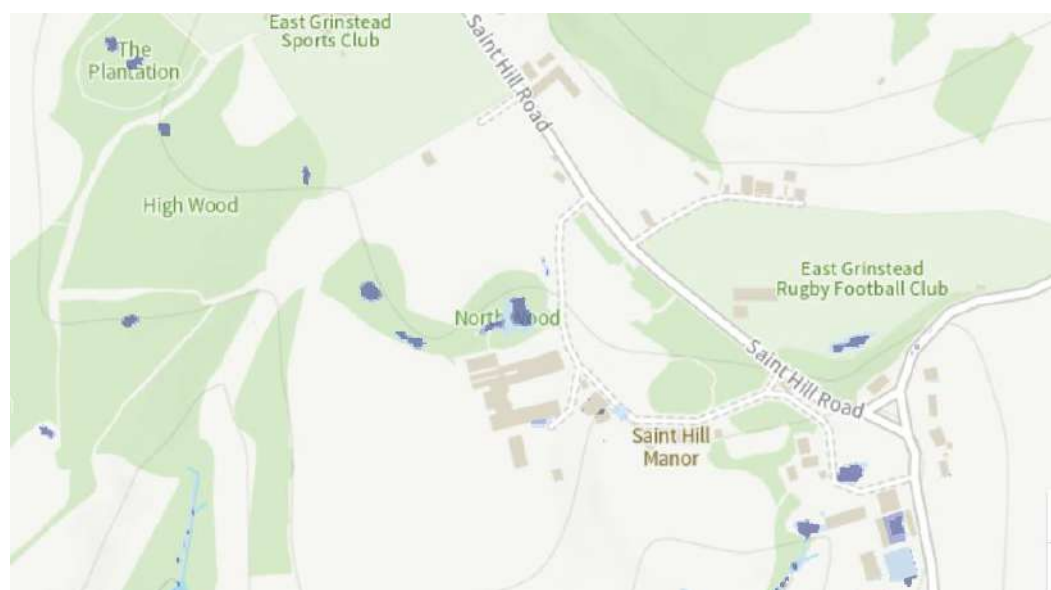


Figure 10: EA RoFSW data showing chance of 300mm water depth during the 2040-2060 surface water event

Figure 10 above shows only very localised flooding risk to a 300mm depth. Figure 11 below shown no water depths up to 600mm are expected on the site.



Figure 11: EA RoFSW data showing chance of 600mm water depth during the 2040-2060 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. The introduction of sustainable drainage features in the car park areas and additional surface water storage would lessen the impact of any overland flow and reduce potential water depths.

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.

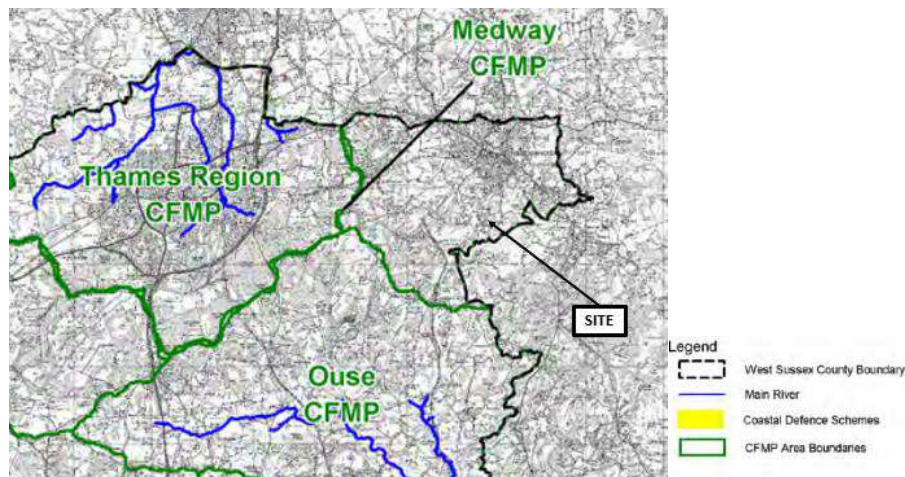


Figure 12: West Sussex County Council SFRA – Flood Management Plans and Strategies map extract

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.

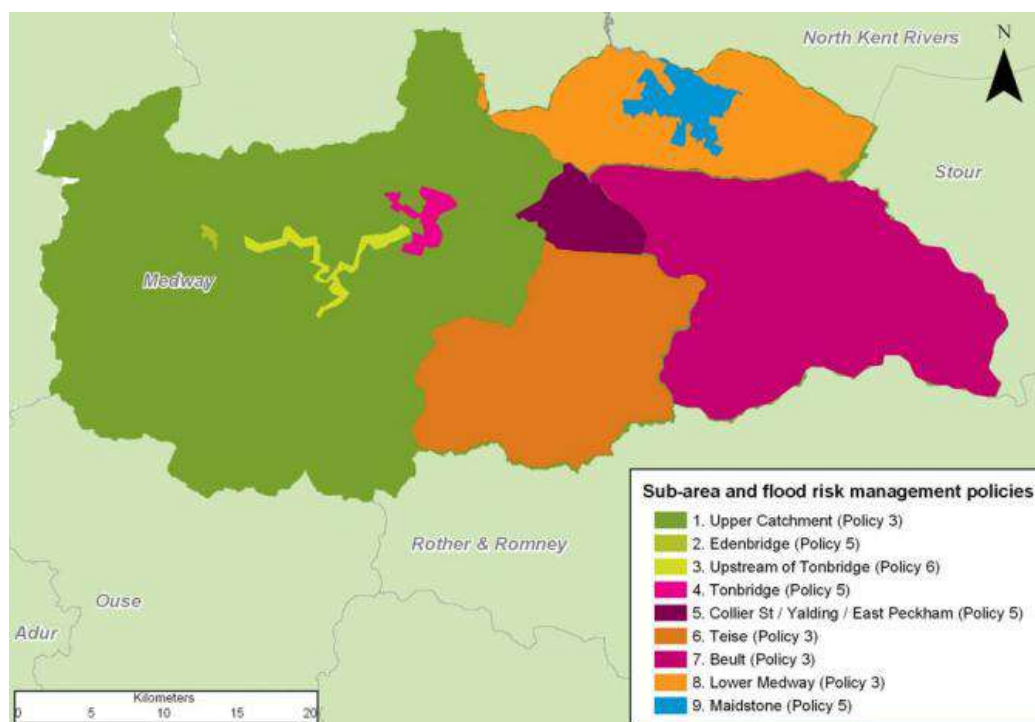


Figure 13: River Medway Catchment Flood Management Plan – Sub-areas and Flood Risk Management Policies map extract

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.

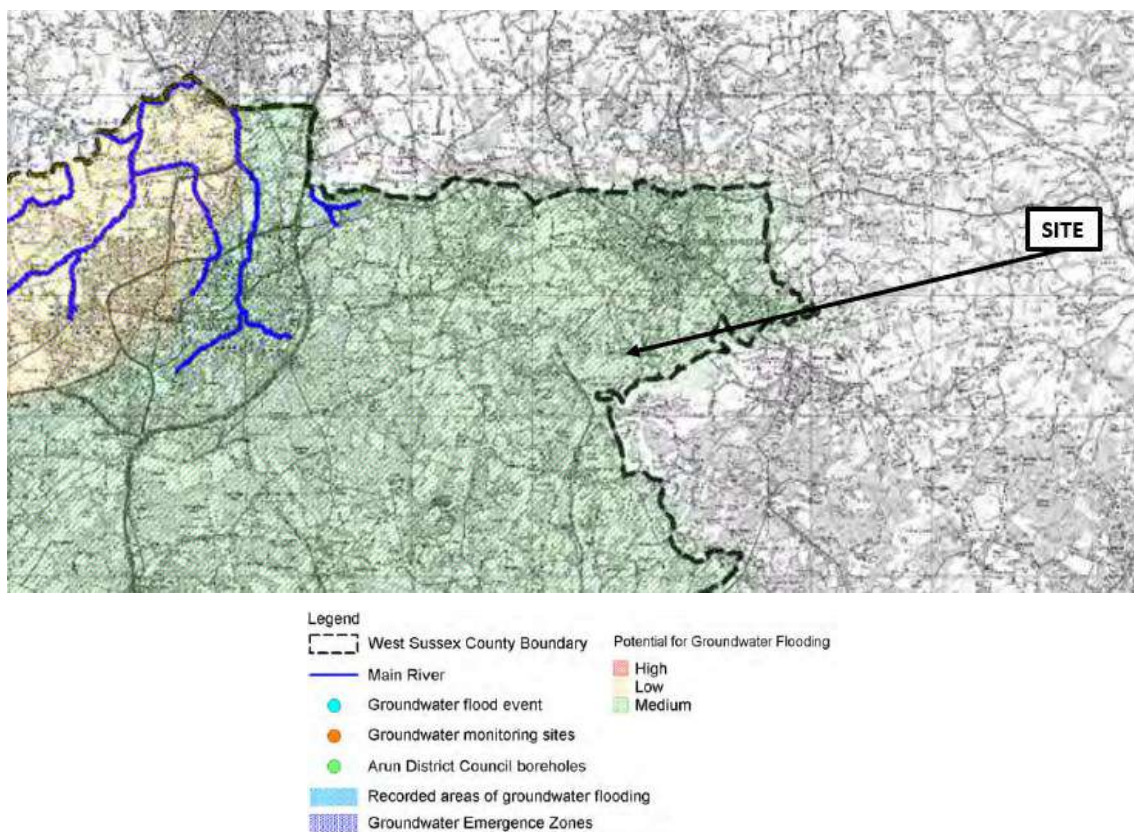


Figure 14: West Sussex County Council SFRA – Areas Prone to Groundwater Flooding map extract

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.

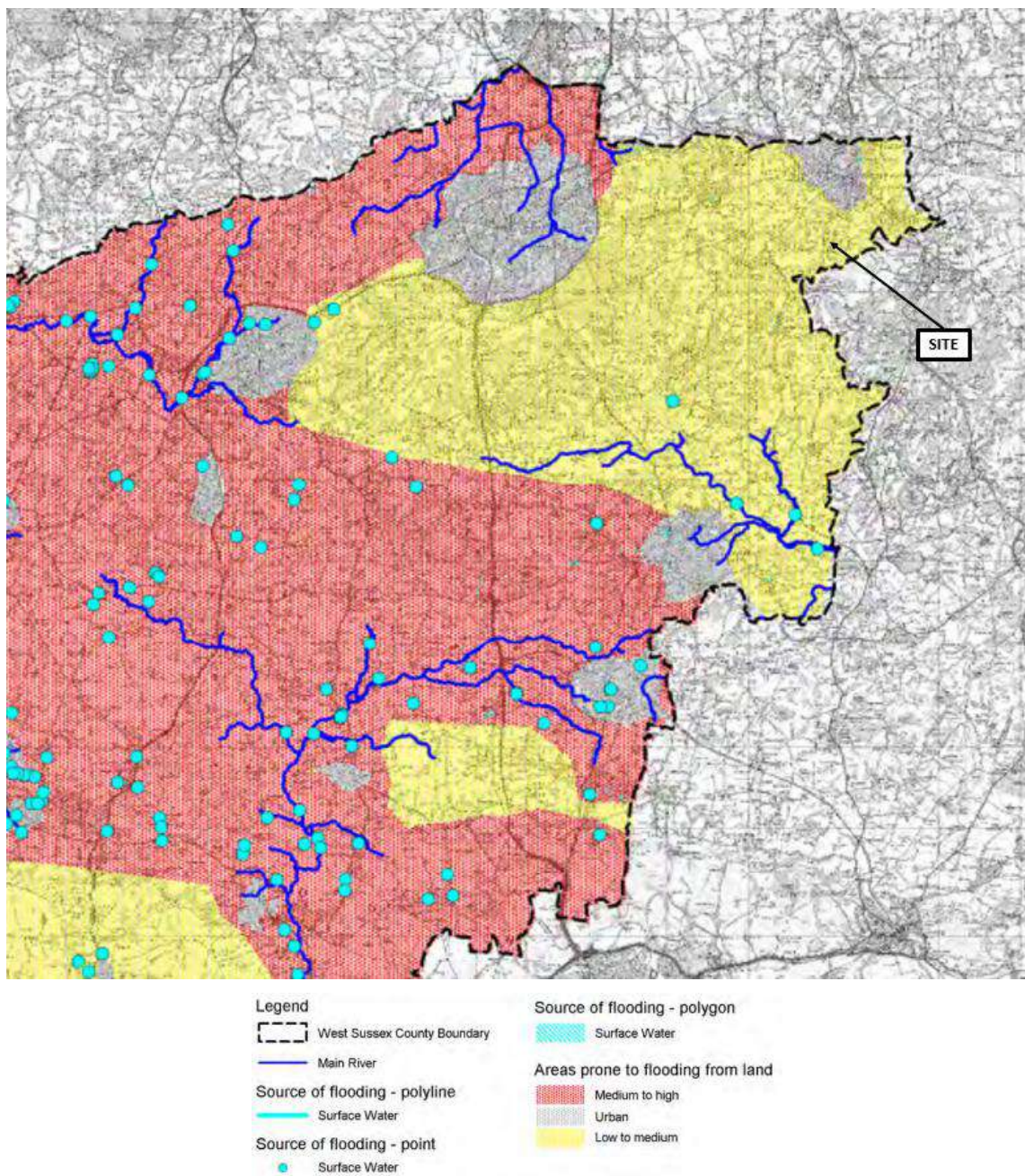


Figure 15: West Sussex County Council SFRA – Areas Prone to Flooding from the Land map extract

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.

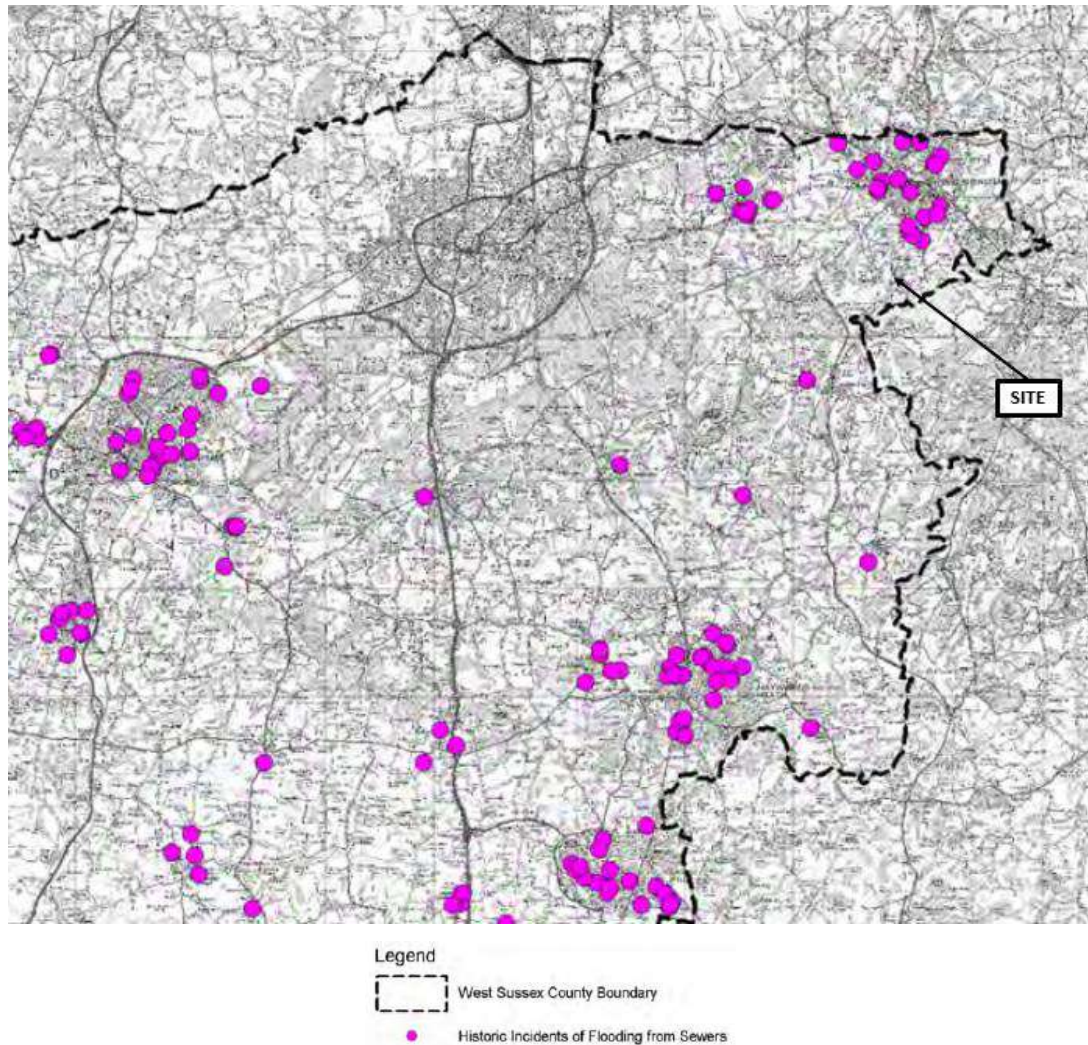


Figure 16: West Sussex County Council SFRA – Areas Prone to Flooding from Sewers map extract

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, filter drains, a swale and attenuation storage is proposed to allow infiltration, as existing, where feasible.

Permeable surfaces, filter trenches, and a swale (all unlined to allow infiltration), have been proposed throughout the car park to capture runoff and allow as much infiltration as feasible prior to directing 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 existing development where feasible

Permeable surfacing, filter drains, a swale and attenuation storage is proposed to allow infiltration, as existing, where feasible. The aim is to provide a more stable surface for the field to allow event parking while maintaining the natural infiltration of runoff and management of excess runoff as required.

Permeable surfaces, filter trenches, and a swale (all unlined to allow infiltration), have been proposed throughout the car park to capture runoff and allow as much infiltration as feasible prior to directing excess flow to areas for attenuation and storage prior to controlled release. These features can be seen on the proposed landscaping plan. The storage and water collection features and sizes are shown on the drainage plans.

Proposed building FFLS should be raised a minimum of 150mm above external ground levels to protect against potential overland or exceedance flows.

A sustainable drainage system for managing runoff that doesn't infiltrate has been provided. To be conservative it has been assumed that 70% of water will drain off the site. Water is managed in underground storage as required prior to controlled release at greenfield runoff rates. The strategy is discussed in more detail in Section 11 of this report.

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:

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 that may potentially be feasible to be used as an open water feature – details to be confirmed. 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 and is specified where feasible.	✓
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	A swale feature is proposed between the permanent car park and overflow field parking area to manage excess runoff from the field parking. Please refer to landscape GA plan	✓
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

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 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.

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).

Surface water models are run using the FEH22 rainfall data and CV values of 1.0 in line with local LLFA standards. 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	4.3	18.5	21.3
Greenfield Rates Eastern Section	1.41	10.95	25.19	34.93
Proposed Runoff Rates Eastern Section+40% CC	1.41	8.4	24.5	25.8
Combined Greenfield Runoff Rates	2.446	19.36	44.54	61.77
Combined Proposed Runoff Rates+40% CC	2.446	12.7	43	47.1

Calculated Runoff rates

Discharge from the Western Area of the carpark is proposed to be limited to 21.3l/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 25.8/s during the 1 in 100-year event + 40% climate change (1%AEP+CC). Runoff rates for both areas 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 (labelled attenuation 1 in the calcs) designed to provide attenuation for an area of 10760m². Runoff from the drained area has been limited to 70% given the largely soft surface nature of the surfacing. 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. The calculations show a water volume of 495.25

in the attenuation storage. 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.

The eastern section provides attenuation for an area of 14,123m². Runoff from the drained area has been limited to 70% given the largely soft surface nature of the surfacing. A flat section of car park (labelled formal car park in the calculations) is proposed retain water within the sub-base, which has a minimum storage plan area of 2100m² and a sub-base depth of 0.875m. With a porosity of 30%, a storage volume of 551.25m³ is needed. Total runoff from the eastern area 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 main formal car park area is proposed to be constructed in permeable pavement to capture, treat and manage runoff from the car park area.

The field adjacent to the formal car park is currently used for parking in events and this is to be stabilised to provide a more robust surface. This will provide initial filtration. Filter trenches and filter drains are provided to capture excess runoff.

Full landscaping plans are included in appendix I which defines the different surfacing areas in more detail.

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
Filter strip	0.4	0.4	0.5
Filter drain	0.4	0.4	0.4

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.
Filter strips/drains	1 year	As required	5 years	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

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 for planning the site is largely at a very low chance of surface water flooding. Very low risk means that each year this area has a chance of flooding less than 0.1%. It can be seen a potential flow path is shown across the car park /field area. The EA surface water depth data shows the site to experience less than 200mm flooding for the most part for the 2040 to 2060 event. Very local areas are shown to a depth of 300mm but higher water depths are indicated.

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.

Discharge from the Western Area of the carpark is proposed to be limited to 21.3l/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 25.8/s during the 1 in 100-year event + 40% climate change (1%AEP+CC). 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.

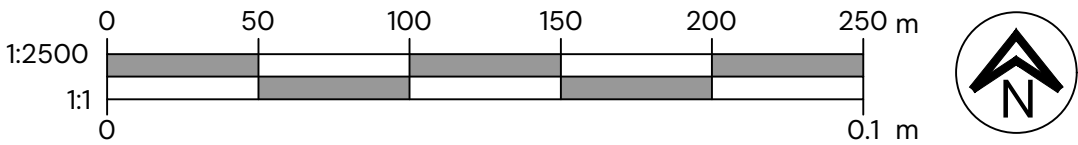
All foul water drainage is to be drained to the closest existing foul water drainage connection. Details of the capacity of the existing foul system is to be confirmed at the later design stage.

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.

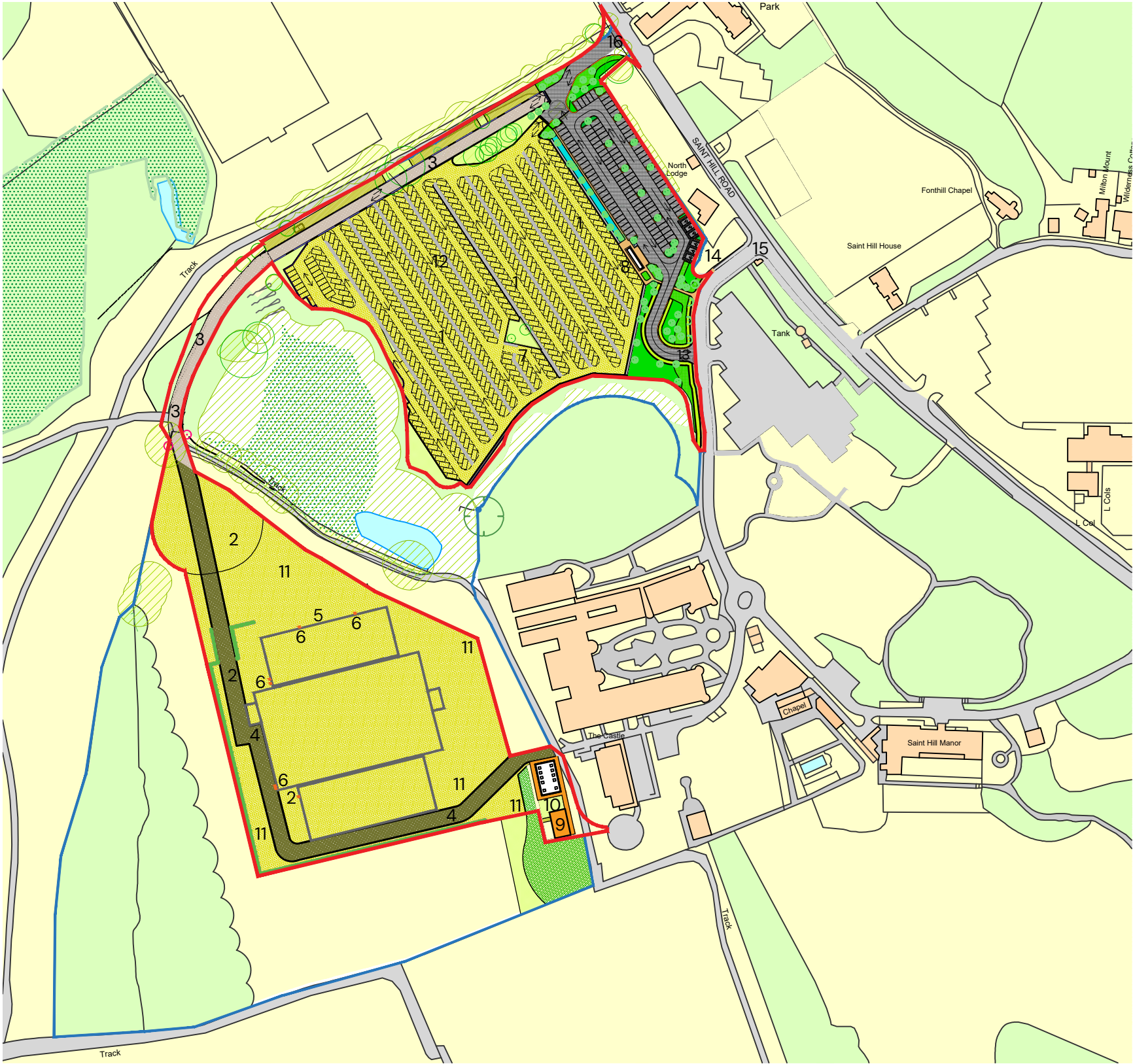


APPENDIX I – ARCHITECT PLANS

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FOR PLANNING



Drawing Legend:

1. Cell Pave 50 filled with Grass
2. Cell Pave HD filled with Grass
3. Cell Pave HD filled with Gravel
4. Event Crew Access Route
5. Tent ring beam
6. Utility Connection point for event power
7. Trees and Water main valve
8. Waiting area
9. Generator enclosure
10. Washroom
11. Area to be graded
12. Carpark design by Enplan (SEE LANDSCAPE DESIGN)
13. New Access Route - 50m 1 in 20 incline
14. Accessible parking
15. (Existing) Security hut
16. Increased Safety Vision Splay



Carnegie Steel Buildings Ltd T/A BM Steels Email: info@bmsteels.co.uk
Address: The Beeches, Old Ln, Tatsfield, Westerham TN16 2LH Phone: 01959 572501

PROJECT TITLE:
Proposal on Juhring &
Hobbs Field, Saint Hill
Manor

DRAWING TITLE:
Site Plan A3

CLIENT:
COSRECI

DRAWN: FH	CHECKED: JA	DATE: 09.01.2025	SCALE: 1:2500 @A3
PROJECT NO: 1093	DRAWING NO: 060	REV: K	SHEET NO: 1/1

General Note(s):

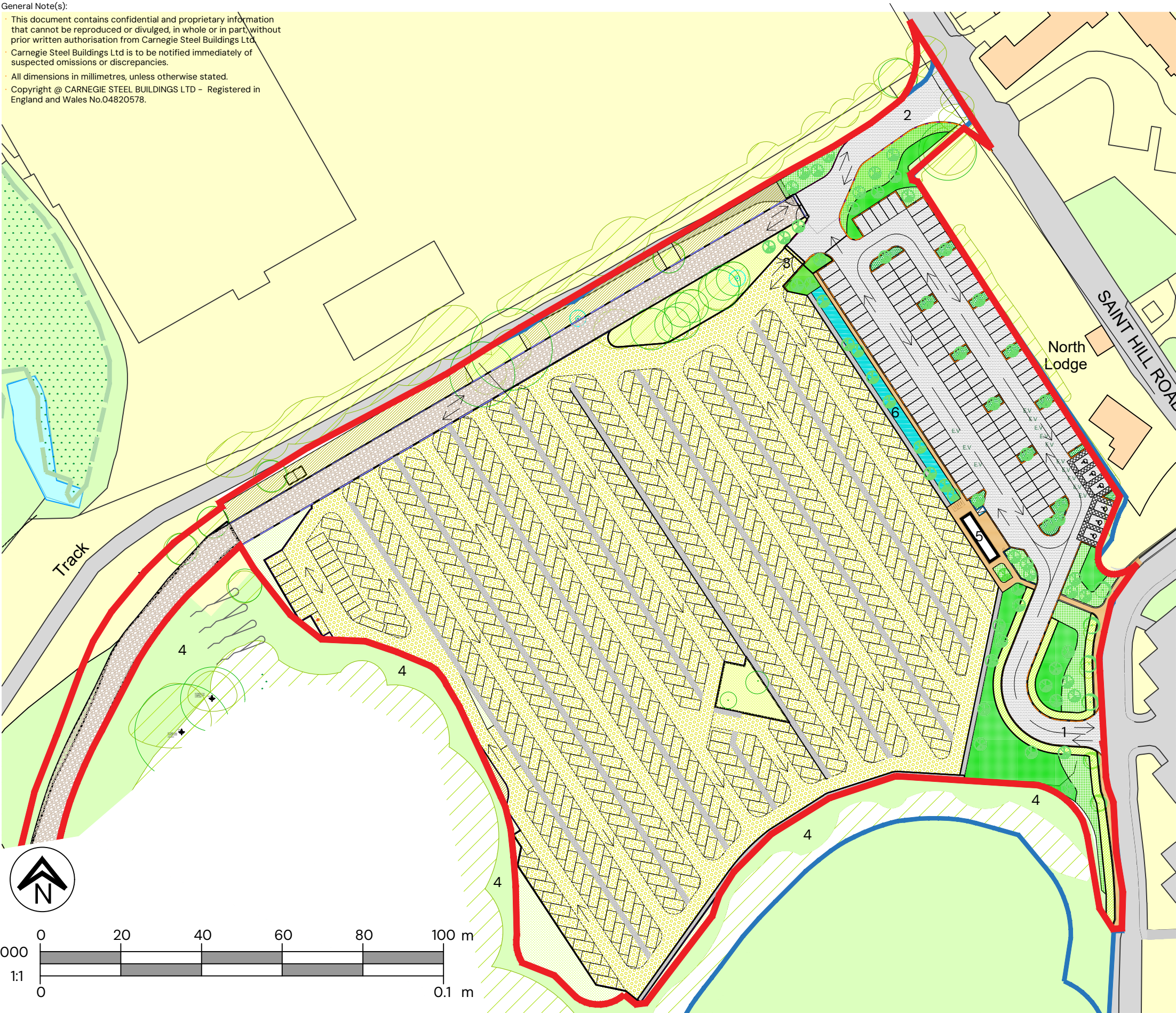
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FOR PLANNING

- Drawing Legend:
- 1) Main Access for Permanent Car Park
 - 2) Secondary access and exit for overflow parking and events.
 - 3) Main access for overflow Parking.
 - 4) Rewilding area
 - 5) Waiting area block
 - 6) Planted Bund

Areas

Permanent Car Park	5500m²
Overflow Parking	20000m²
Access Road	750m²
(Gravel Filled Cellpave HD)	





CARNEGIE
ARCHITECTURE

Carnegie Steel Buildings Ltd T/A BM Steels Email: info@bmsteels.co.uk
Address: The Beeches, Old Ln, Tatsfield, Westerham TN16 2LH Phone: 01959 572501

PROJECT TITLE:

Proposal on Juhring Field,
Saint Hill Manor

DRAWING TITLE:

Carpark Site Plan A3

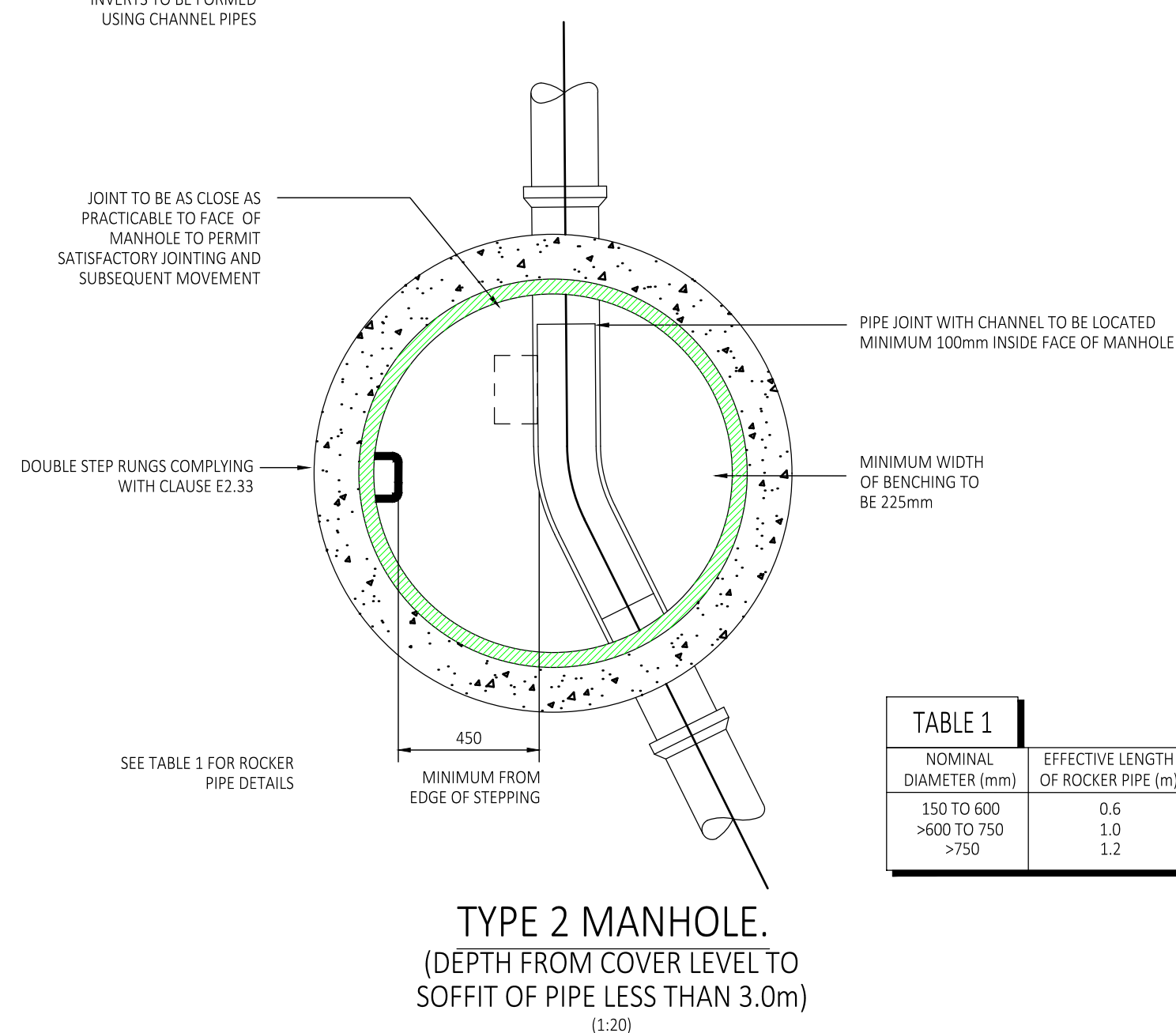
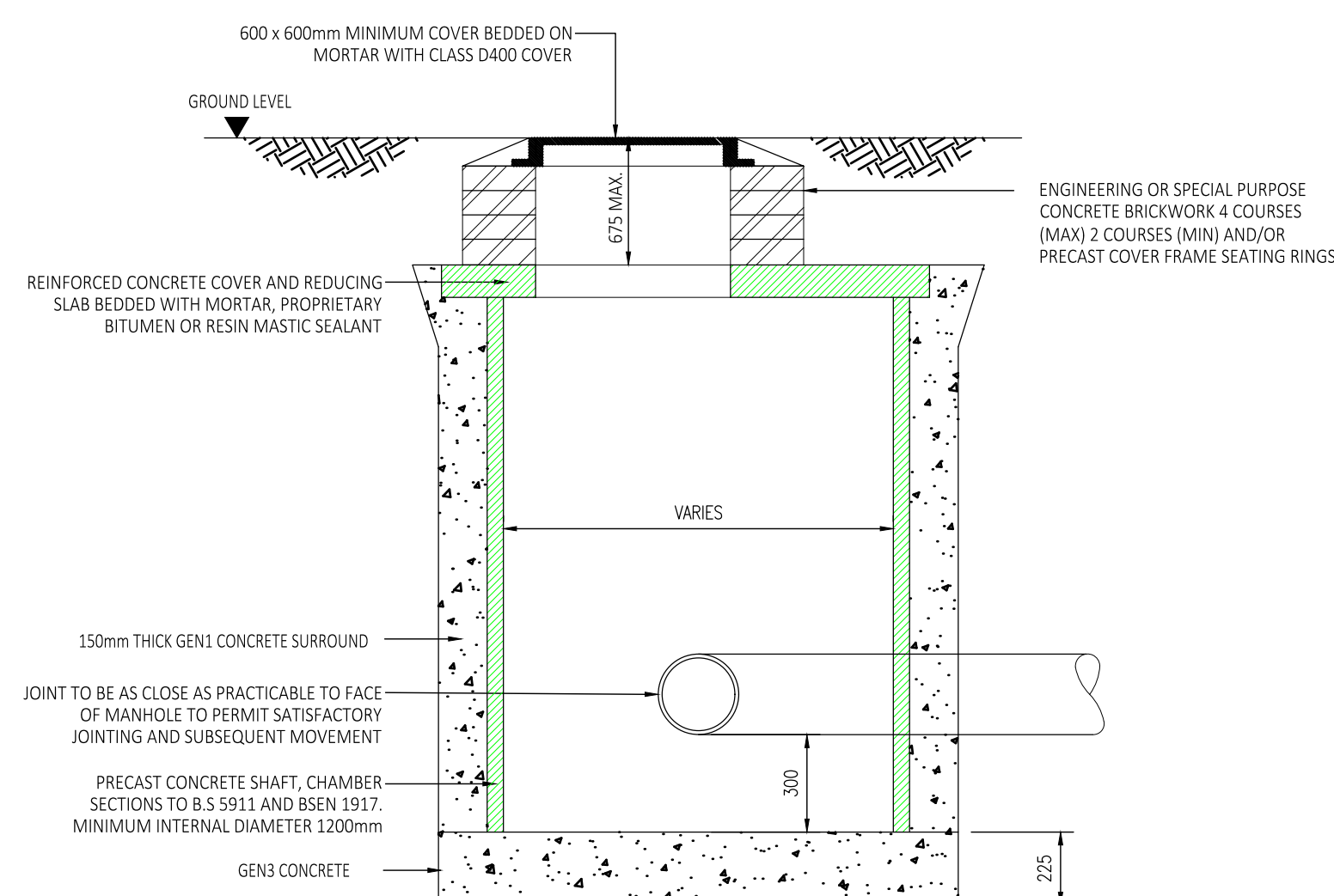
CLIENT:

COSRECI

DRAWN: FH	CHECKED: JA	DATE: 09.01.2025	SCALE: 1:1000 @A3
PROJECT NO: 1093	DRAWING NO: 061	REV: K	SHEET NO: 1/1



APPENDIX II – PROPOSED DRAINAGE STRATEGY



QED
structures

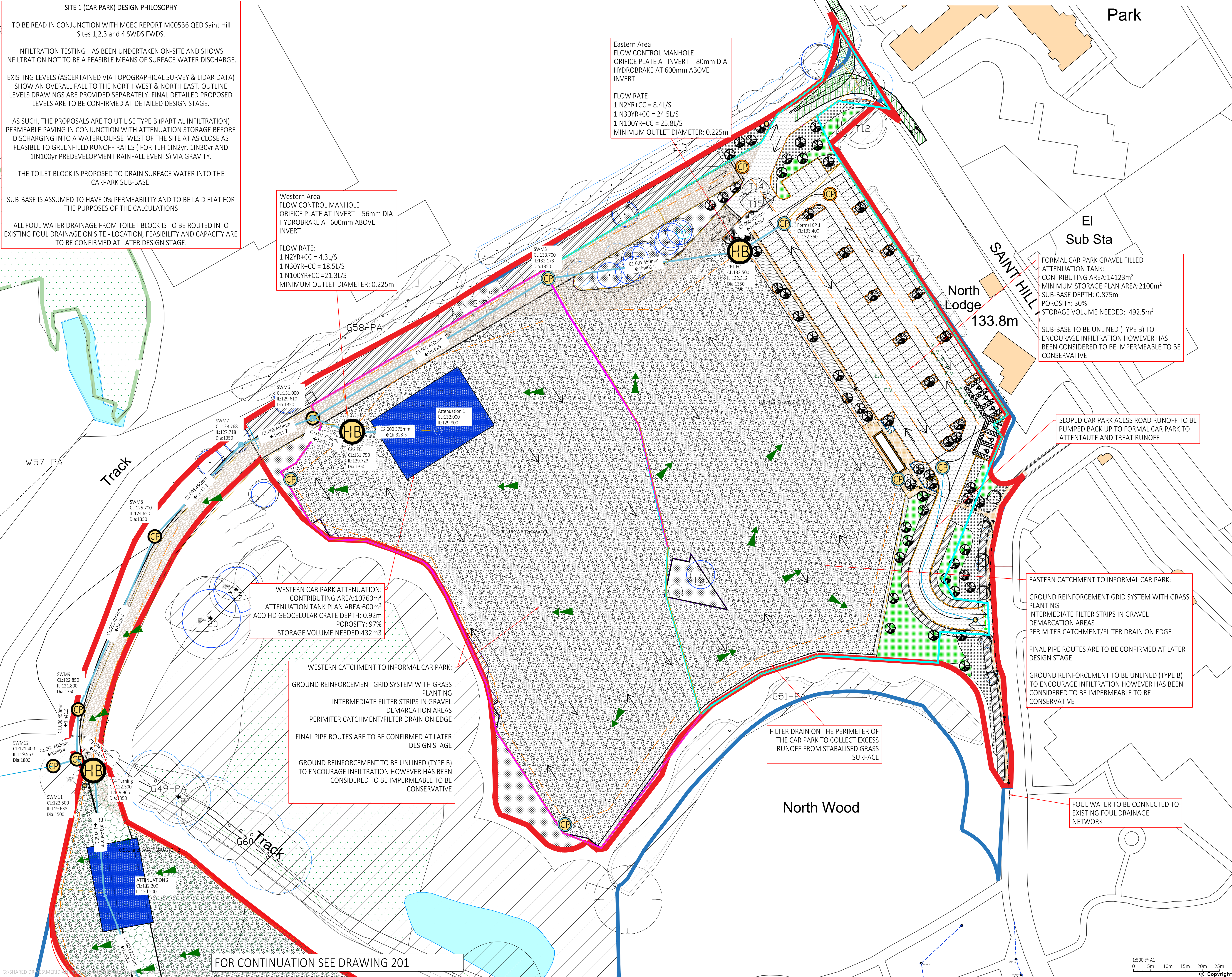
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CLIENT
International Association of
Scientologists

TITLE
TYPICAL DRAINAGE DETAILS
SHEET 1

STATUS	S3 - Suitable for Review and Comment
--------	--------------------------------------

DRAWING No. 24-164-QED-XX-XX-DR-C-401	REVISION P01
--	-----------------



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.

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

FLOW RATE:
1IN2YR+CC = 4.3L/S
1IN30YR+CC = 18.5L/S
1IN100YR+CC = 21.3L/S
MINIMUM OUTLET DIAMETER: 0.225m

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

FLOW RATE:
1IN2YR+CC = 8.4L/S
1IN30YR+CC = 24.5L/S
1IN100YR+CC = 25.8L/S
MINIMUM OUTLET DIAMETER: 0.225m

FORMAL CAR PARK GRAVEL FILLED ATTENUATION TANK:
CONTRIBUTING AREA:14123m²
MINIMUM STORAGE PLAN AREA:2100m²
SUB-BASE DEPTH: 0.875m
POROSITY: 30%
STORAGE VOLUME NEEDED: 492.5m³

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

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

WESTERN CATCHMENT TO INFORMAL CAR PARK:
GROUND REINFORCEMENT GRID SYSTEM WITH GRASS PLANTING
INTERMEDIATE FILTER STRIPS IN GRAVEL
DEMARICATION 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

EASTERN CATCHMENT TO INFORMAL CAR PARK:
GROUND REINFORCEMENT GRID SYSTEM WITH GRASS PLANTING
INTERMEDIATE FILTER STRIPS IN GRAVEL
DEMARICATION 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 SD SURFACING
 - PROPOSED SW PUMP

P05	Architects plan updated	23/10/25	KB	MN
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

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TEL +44 (0) 1273 033680

QED PROJECT No.
24-164

CLIENT
International Association of
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	
DRAWING No. 24-164-QED-XX-XX-DR-C-101	REVISION P05		



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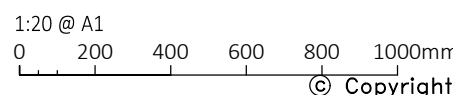
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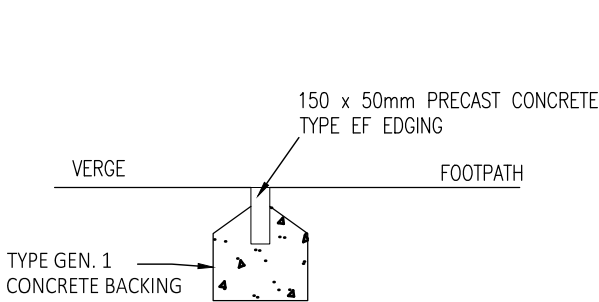
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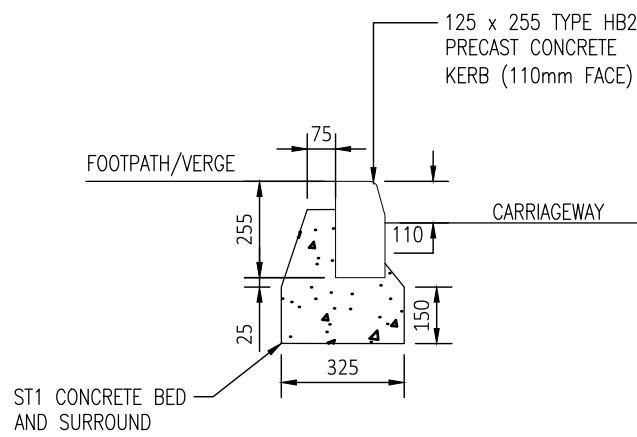
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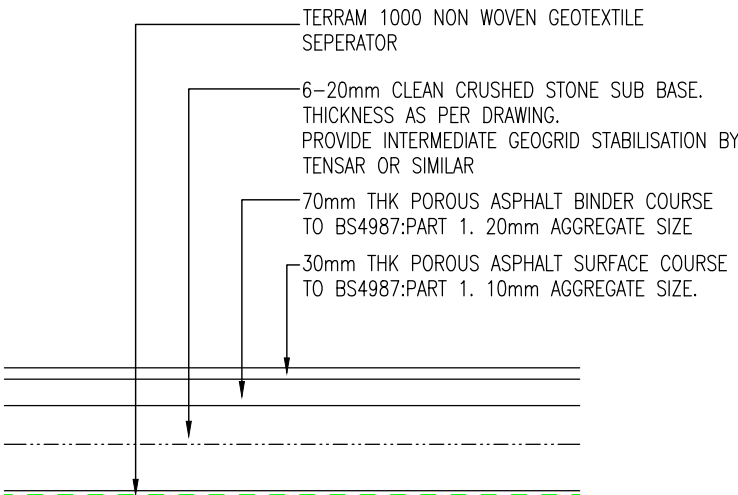
FOOTPATH EDGING
DETAIL.

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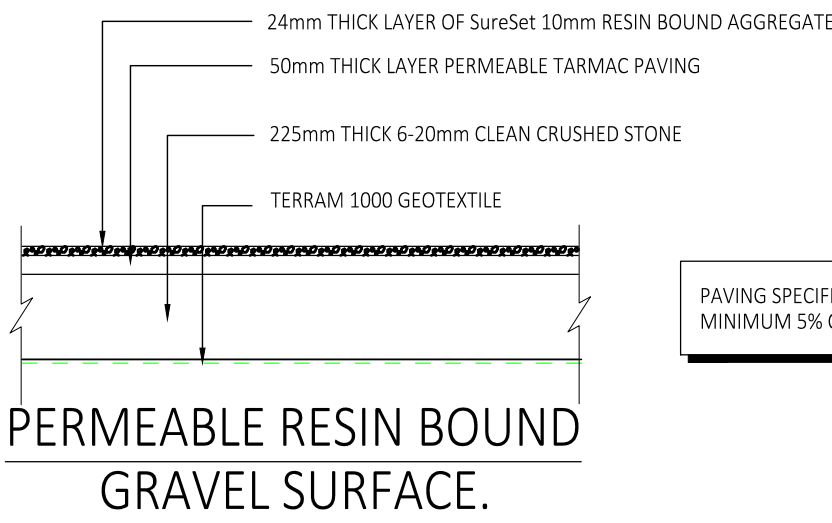
125 x 255 HALF BATTER
KERB TYPE HB2.

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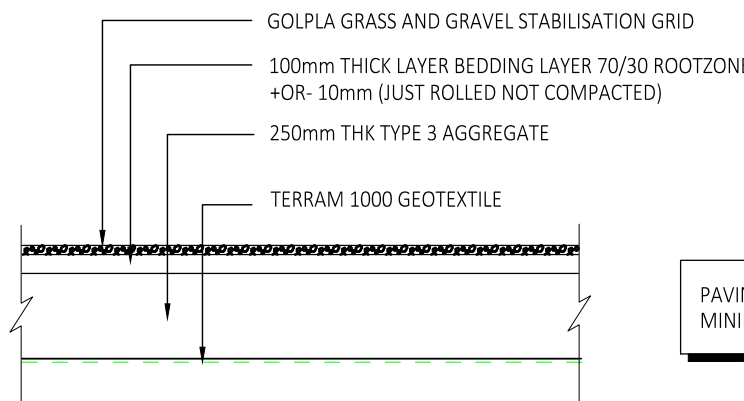
PERMEABLE TARMAC CAR PARK
CONSTRUCTION.

(1:20)



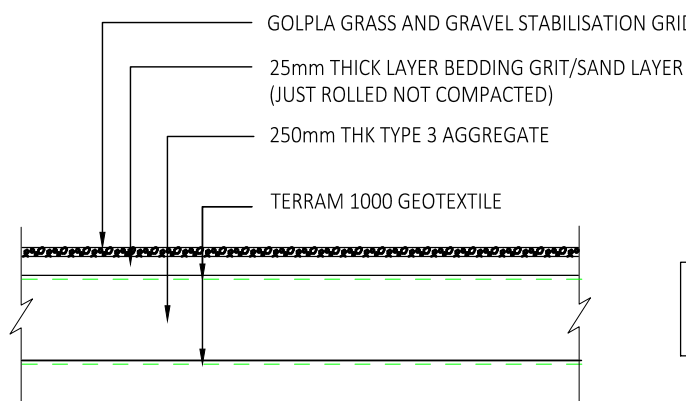
PERMEABLE RESIN BOUND
GRAVEL SURFACE.

(1:20)



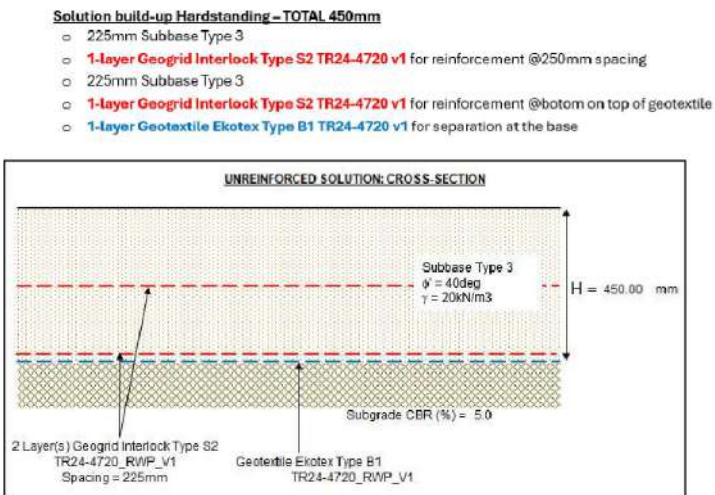
STABILISED GRASS SURFACE.

(1:20)



STABILISED GRAVEL SURFACE.

(1:20)



MARQUEE AREA BASE.
TYPICAL DESIGN BY GEOSYNTHETICS LTD.
MIN 585mm SUB BASE REQUIRED FOR
STORAGE

(1:20)

PAVING SPECIFICATIONS ASSUME
MINIMUM 5% CBR VALUE

PAVING SPECIFICATIONS ASSUME
MINIMUM 5% CBR VALUE

PAVING SPECIFICATIONS ASSUME
MINIMUM 5% CBR VALUE

PAVING SPECIFICATIONS ASSUME
MINIMUM 5% CBR VALUE

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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	ENGINEER	CHECKED BY	APPROVED BY
IW	KB	MN	MN

DATE APPROVED 10/12/24 SCALE @ A1 1:20

STATUS
S3 - Suitable for Review and Comment

DRAWING No.	REVISION
24-164-QED-XX-XX-DR-C-450	P01



APPENDIX III – CALCULATIONS

Calculated by:	Kalvin Bassi
Site name:	Saint hills main car park
Site location:	rh19 4jy

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.

Site Details

Latitude:	51.10764° N
Longitude:	0.03121° W
Reference:	2661284901
Date:	Jan 30 2025 15:54

Runoff estimation approach

IH124

Site characteristics

Total site area (ha):	2.49
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Methodology

Q_{BAR} estimation method:	Calculate from SPR and SAAR
SPR estimation method:	Calculate from SOIL type

Notes

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

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

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 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 $SPR/SPRHOST \leq 0.3$?

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

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	0.873	10.00	133.400		1.050
✓	CP1 FC Eastern			133.500	1350	1.188
✓	3			133.700	1350	1.527
✓	Attenuation 1	0.729	10.00	132.000		2.200
✓	CP2 FC Western			131.750	1350	2.027
✓	6			131.000	1350	1.390
✓	7			128.768	1350	1.050
✓	8			125.700	1350	1.050
✓	9			122.850	1350	1.050
✓	ATTENUATION 2	0.551	10.00	122.200		2.000
✓	11			122.500	1500	2.862
✓	12			121.400	1800	1.833
✓	MARQUEE PAD 14	1.739	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 Eastern	15.227	0.600	132.350	132.312	0.038	400.7	450	10.25	50.0
✓	1.001	CP1 FC Eastern	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 Western	24.907	0.600	129.800	129.723	0.077	323.5	375	10.41	50.0
?	2.001	CP2 FC Western	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	450	11.64	50.0
?	1.004	7	8	36.509	0.600	127.718	124.650	3.068	11.9	450	11.74	50.0
?	1.005	8	9	55.316	0.600	124.650	121.800	2.850	19.4	450	11.94	50.0
?	1.006	9	11	14.510	0.600	121.800	121.450	0.350	41.5	450	12.02	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
?	3.003	ATTENUATION 2	FC4 Turning	35.271	0.600	120.200	119.965	0.235	150.1	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)
	1.000	Formal CP 1	CP1 FC Eastern	1.009	160.5	118.3	0.600	0.738	0.873	0.0	288	1.099
	1.001	CP1 FC Eastern	3	1.003	159.6	118.3	0.738	1.077	0.873	0.0	290	1.095
	1.002	3	6	3.402	541.1	118.3	1.077	0.600	0.873	0.0	142	2.746
	2.000	Attenuation 1	CP2 FC Western	1.002	110.6	98.8	1.825	1.652	0.729	0.0	278	1.127
	2.001	CP2 FC Western	6	1.000	110.5	98.8	1.652	0.940	0.729	0.0	278	1.125
	1.003	6	7	5.958	947.6	217.1	0.940	0.600	1.602	0.0	145	4.872
	1.004	7	8	5.917	941.1	217.1	0.600	0.600	1.602	0.0	146	4.853
	1.005	8	9	4.630	736.4	217.1	0.600	0.600	1.602	0.0	167	4.049
	1.006	9	11	3.164	503.2	217.1	0.600	0.600	1.602	0.0	206	3.051
	3.000	MARQUEE PAD 14	FC3 MARQUEE15	2.275	361.8	235.7	0.600	0.698	1.739	0.0	265	2.415
	3.001	FC3 MARQUEE15	16	2.223	88.4	235.7	0.923	0.825	1.739	0.0	225	2.264
	3.002	16	ATTENUATION 2	1.794	71.3	235.7	0.825	0.825	1.739	0.0	225	1.827
	3.003	ATTENUATION 2	FC4 Turning	1.657	263.5	310.4	1.550	2.085	2.290	0.0	450	1.678
	3.004	FC4 Turning	11	1.016	71.8	310.4	2.235	2.262	2.290	0.0	300	1.029

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)
? 1.007	11	12	7.061	0.600	119.638	119.567	0.071	99.4	600	12.07	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)
1.007	11	12	2.442	690.4	527.5	2.262	1.233	3.892	0.0	395	2.676

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	450	Circular	131.000	129.610	0.940	128.768	127.718	0.600
1.004	36.509	11.9	450	Circular	128.768	127.718	0.600	125.700	124.650	0.600
1.005	55.316	19.4	450	Circular	125.700	124.650	0.600	122.850	121.800	0.600
1.006	14.510	41.5	450	Circular	122.850	121.800	0.600	122.500	121.450	0.600

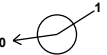
Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	Formal CP 1		Junction		CP1 FC Eastern	1350	Manhole	SW_HB
1.001	CP1 FC Eastern	1350	Manhole	SW_HB	3	1350	Manhole	SW_Standard
1.002	3	1350	Manhole	SW_Standard	6	1350	Manhole	SW_Standard
2.000	Attenuation 1		Junction		CP2 FC Western	1350	Manhole	SW_HB
2.001	CP2 FC Western	1350	Manhole	SW_HB	6	1350	Manhole	SW_Standard
1.003	6	1350	Manhole	SW_Standard	7	1350	Manhole	SW_Standard
1.004	7	1350	Manhole	SW_Standard	8	1350	Manhole	SW_Standard
1.005	8	1350	Manhole	SW_Standard	9	1350	Manhole	SW_Standard
1.006	9	1350	Manhole	SW_Standard	11	1500	Manhole	SW_Standard

Pipeline Schedule

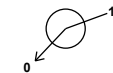
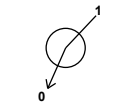
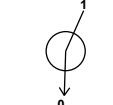
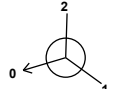
Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
3.000	23.828	80.0	450	Circular	124.400	123.350	0.600	124.200	123.052	0.698
3.001	41.905	34.9	225	Circular	124.200	123.052	0.923	122.900	121.850	0.825
3.002	37.357	53.4	225	Circular	122.900	121.850	0.825	122.200	121.150	0.825
3.003	35.271	150.1	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
1.007	7.061	99.4	600	Circular	122.500	119.638	2.262	121.400	119.567	1.233

Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
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	
3.003	ATTENUATION 2		Junction		FC4 Turning	1350	Manhole	SW_Standard
3.004	FC4 Turning	1350	Manhole	SW_Standard	11	1500	Manhole	SW_Standard
1.007	11	1500	Manhole	SW_Standard	12	1800	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					
CP1 FC Eastern	133.500	1.188	1350		0	1.000	132.350
					1	1.000	132.312
					0	1.001	132.312
3	133.700	1.527	1350		0	1.001	132.173
					1	1.001	132.173
					0	1.002	132.173

Manhole Schedule

Node	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)	
Attenuation 1	132.000	2.200						
				0	2.000	129.800	375	
CP2 FC Western	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	450	
7	128.768	1.050	1350		1	1.003	127.718	450
				0	1.004	127.718	450	
8	125.700	1.050	1350		1	1.004	124.650	450
				0	1.005	124.650	450	
9	122.850	1.050	1350		1	1.005	121.800	450
				0	1.006	121.800	450	
ATTENUATION 2	122.200	2.000			1	3.002	121.150	225
				0	3.003	120.200	450	
11	122.500	2.862	1500		1	3.004	119.938	300
				2	1.006	121.450	450	
				0	1.007	119.638	600	

Manhole Schedule

Node	CL (m)	Depth (m)	Dia (mm)	Connections	Link	IL (m)	Dia (mm)	
12	121.400	1.833	1800	 1	1.007	119.567	600	
MARQUEE PAD 14	124.400	1.050		 0	0	3.000	123.350	450
FC3 MARQUEE15	124.200	1.148	1350	 01	1	3.000	123.052	450
				0	3.001	123.052	225	
16	122.900	1.050	600	 01	1	3.001	121.850	225
				0	3.002	121.850	225	
FC4 Turning	122.500	2.535	1350	 01	1	3.003	119.965	450
				0	3.004	119.965	300	

Simulation Settings

Rainfall Methodology	FEH-22	Winter CV	1.000	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	1.000	Skip Steady State	x	Starting Level (m)			

Storm Durations

15 | 30 | 60 | 120 | 180 | 240 | 360 | 480 | 600 | 720 | 960 | 1440 | 2160

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	0	0	0	30	40	0	0
2	40	0	0	100	0	0	0
30	0	0	0	100	40	0	0

Node CP1 FC Eastern 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 Eastern 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-0169-1300-0600-1300
Design Depth (m)	0.600	Min Outlet Diameter (m)	0.225
Design Flow (l/s)	13.0	Min Node Diameter (mm)	1200

Node CP2 FC Western 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 Western 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)	0.650	Diameter (m)	0.085
Downstream Link	3.004	Invert Level (m)	119.965	Design Flow (l/s)	12.0	Discharge Coefficient	0.600

Node FC4 Turning Online Orifice Control

Flap Valve	x	Invert Level (m)	120.650	Design Flow (l/s)	14.2	Discharge Coefficient	0.600
Replaces Downstream Link	✓	Design Depth (m)	0.620	Diameter (m)	0.094		

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)	21.000	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)	424	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)	536

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)	392

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	400.0	0.0	0.909	400.0	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)	100.000	Inf Depth (m)
Safety Factor	2.0	Time to half empty (mins)	1410	Slope (1:X)	999.0	

Other (defaults)

Entry Loss (manhole)	0.250	Entry Loss (junction)	0.000	Apply Recommended Losses	x
Exit Loss (manhole)	0.250	Exit Loss (junction)	0.000	Flood Risk (m)	0.300

Results for 2 year Critical Storm Duration. Lowest mass balance: 99.58%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
360 minute summer	Formal CP 1	264	132.628	0.277	40.1	143.1724	0.0000	OK
360 minute summer	CP1 FC Eastern	264	132.627	0.315	7.0	0.4513	0.0000	OK
360 minute summer	3	264	132.209	0.036	7.0	0.0513	0.0000	OK
720 minute summer	Attenuation 1	510	130.064	0.264	21.1	150.3598	0.0000	OK
720 minute summer	CP2 FC Western	510	130.064	0.341	3.7	0.4876	0.0000	OK
480 minute summer	6	344	129.644	0.034	10.6	0.0485	0.0000	OK
480 minute summer	7	344	127.752	0.033	10.6	0.0479	0.0000	OK
480 minute summer	8	344	124.688	0.038	10.6	0.0537	0.0000	OK
480 minute summer	9	344	121.846	0.046	10.6	0.0664	0.0000	OK
480 minute summer	ATTENUATION 2	344	120.479	0.279	34.0	105.9043	0.0000	OK
480 minute summer	11	344	119.716	0.077	21.0	0.1369	0.0000	OK
480 minute summer	12	344	119.638	0.071	21.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
360 minute summer	Formal CP 1	1.000	CP1 FC Eastern	7.0	0.229	0.044	1.6844	
360 minute summer	CP1 FC Eastern	Orifice	3	7.0				
360 minute summer	CP1 FC Eastern	Hydro-Brake®	3	0.0				
360 minute summer	3	1.002	6	7.0	1.202	0.013	0.4652	
720 minute summer	Attenuation 1	2.000	CP2 FC Western	3.7	0.171	0.033	2.3422	
720 minute summer	CP2 FC Western	Orifice	6	3.7				
720 minute summer	CP2 FC Western	Hydro-Brake®	6	0.0				
480 minute summer	6	1.003	7	10.6	1.991	0.011	0.1185	
480 minute summer	7	1.004	8	10.6	1.841	0.011	0.2107	
480 minute summer	8	1.005	9	10.6	1.440	0.014	0.4100	
480 minute summer	9	1.006	11	10.6	1.271	0.021	0.1213	
480 minute summer	ATTENUATION 2	3.003	FC4 Turning	36.8	0.385	0.139	4.6124	
480 minute summer	11	1.007	12	21.0	1.056	0.030	0.1403	1160.7

Results for 2 year Critical Storm Duration. Lowest mass balance: 99.58%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
480 minute winter	MARQUEE PAD 14	464	123.522	0.172	68.4	364.5334	0.0000	OK
600 minute winter	FC3 MARQUEE15	480	123.582	0.530	35.1	0.7577	0.0000	SURCHARGED
600 minute summer	16	510	121.897	0.047	6.9	0.0132	0.0000	OK
240 minute summer	FC4 Turning	152	120.485	0.520	40.0	0.7447	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)
480 minute winter	MARQUEE PAD 14	3.000	FC3 MARQUEE15	34.7	0.377	0.096	2.5484	
600 minute winter	FC3 MARQUEE15	Orifice	16	7.0				
600 minute summer	16	3.002	ATTENUATION 2	6.7	1.132	0.094	0.2214	
240 minute summer	FC4 Turning	Orifice	11	10.4				
240 minute summer	FC4 Turning	Orifice	11	0.0				

Results for 2 year +40% CC Critical Storm Duration. Lowest mass balance: 99.58%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
480 minute summer	Formal CP 1	344	132.745	0.395	48.0	217.1385	0.0000	OK
480 minute summer	CP1 FC Eastern	344	132.745	0.433	8.4	0.6194	0.0000	OK
480 minute summer	3	344	132.212	0.039	8.4	0.0558	0.0000	OK
720 minute summer	Attenuation 1	540	130.192	0.392	29.6	223.5238	0.0000	SURCHARGED
720 minute summer	CP2 FC Western	540	130.192	0.469	4.3	0.6712	0.0000	SURCHARGED
480 minute summer	6	360	129.647	0.037	12.7	0.0527	0.0000	OK
480 minute summer	7	360	127.754	0.036	12.7	0.0519	0.0000	OK
480 minute summer	8	360	124.691	0.041	12.7	0.0583	0.0000	OK
480 minute summer	9	360	121.851	0.051	12.7	0.0724	0.0000	OK
600 minute summer	ATTENUATION 2	420	120.604	0.404	43.2	153.4748	0.0000	OK
480 minute summer	11	360	119.722	0.084	24.3	0.1478	0.0000	OK
480 minute summer	12	360	119.643	0.076	24.3	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
480 minute summer	Formal CP 1	1.000	CP1 FC Eastern	8.4	0.226	0.052	2.3141	
480 minute summer	CP1 FC Eastern	Orifice	3	8.4				
480 minute summer	CP1 FC Eastern	Hydro-Brake®	3	0.0				
480 minute summer	3	1.002	6	8.4	1.273	0.015	0.5250	
720 minute summer	Attenuation 1	2.000	CP2 FC Western	4.3	0.175	0.039	2.7472	
720 minute summer	CP2 FC Western	Orifice	6	4.3				
720 minute summer	CP2 FC Western	Hydro-Brake®	6	0.0				
480 minute summer	6	1.003	7	12.7	2.102	0.013	0.1338	
480 minute summer	7	1.004	8	12.7	1.950	0.013	0.2377	
480 minute summer	8	1.005	9	12.7	1.511	0.017	0.4645	
480 minute summer	9	1.006	11	12.7	1.338	0.025	0.1375	
600 minute summer	ATTENUATION 2	3.003	FC4 Turning	28.2	0.311	0.107	5.4390	
480 minute summer	11	1.007	12	24.3	1.101	0.035	0.1562	1589.1

Results for 2 year +40% CC Critical Storm Duration. Lowest mass balance: 99.58%

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	690	123.589	0.239	60.7	566.7402	0.0000	OK
720 minute winter	FC3 MARQUEE15	705	123.604	0.552	32.6	0.7905	0.0000	SURCHARGED
720 minute winter	16	660	121.898	0.048	7.2	0.0136	0.0000	OK
600 minute summer	FC4 Turning	420	120.603	0.638	28.2	0.9136	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	32.6	0.336	0.090	2.9070	
720 minute winter	FC3 MARQUEE15	Orifice	16	7.2				
720 minute winter	16	3.002	ATTENUATION 2	7.1	1.151	0.099	0.2297	
600 minute summer	FC4 Turning	Orifice	11	11.6				
600 minute summer	FC4 Turning	Orifice	11	0.0				

Results for 30 year Critical Storm Duration. Lowest mass balance: 99.59%

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	224	132.915	0.565	75.1	323.9622	0.0000	SURCHARGED
240 minute winter	CP1 FC Eastern	224	132.914	0.602	12.8	0.8619	0.0000	SURCHARGED
240 minute winter	3	224	132.221	0.048	12.8	0.0682	0.0000	OK
480 minute winter	Attenuation 1	456	130.370	0.569	37.2	324.6096	0.0000	SURCHARGED
480 minute winter	CP2 FC Western	456	130.369	0.646	5.4	0.9248	0.0000	SURCHARGED
240 minute winter	6	228	129.653	0.043	17.8	0.0621	0.0000	OK
240 minute winter	7	228	127.761	0.043	17.8	0.0610	0.0000	OK
240 minute winter	8	228	124.698	0.048	17.8	0.0685	0.0000	OK
240 minute winter	9	228	121.860	0.060	17.8	0.0858	0.0000	OK
360 minute winter	ATTENUATION 2	272	120.751	0.551	48.4	209.3851	0.0000	SURCHARGED
360 minute winter	11	280	119.739	0.101	35.0	0.1790	0.0000	OK
360 minute winter	12	280	119.657	0.090	35.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
240 minute winter	Formal CP 1	1.000	CP1 FC Eastern	12.8	0.248	0.080	2.4126	
240 minute winter	CP1 FC Eastern	Orifice	3	10.0				
240 minute winter	CP1 FC Eastern	Hydro-Brake®	3	2.8				
240 minute winter	3	1.002	6	12.8	1.442	0.024	0.7076	
480 minute winter	Attenuation 1	2.000	CP2 FC Western	5.4	0.188	0.049	2.7472	
480 minute winter	CP2 FC Western	Orifice	6	5.1				
480 minute winter	CP2 FC Western	Hydro-Brake®	6	0.3				
240 minute winter	6	1.003	7	17.8	2.327	0.019	0.1702	
240 minute winter	7	1.004	8	17.8	2.157	0.019	0.3018	
240 minute winter	8	1.005	9	17.8	1.665	0.024	0.5937	
240 minute winter	9	1.006	11	17.8	1.475	0.035	0.1755	
360 minute winter	ATTENUATION 2	3.003	FC4 Turning	31.6	0.374	0.120	5.5885	
360 minute winter	11	1.007	12	35.0	1.212	0.051	0.2040	1861.2

Results for 30 year Critical Storm Duration. Lowest mass balance: 99.59%

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	123.681	0.331	81.2	843.0524	0.0000	OK
720 minute winter	FC3 MARQUEE15	705	123.681	0.629	34.3	0.9004	0.0000	SURCHARGED
720 minute winter	16	705	121.900	0.050	7.7	0.0141	0.0000	OK
360 minute winter	FC4 Turning	272	120.750	0.785	31.6	1.1231	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	34.3	0.330	0.095	3.3780	
720 minute winter	FC3 MARQUEE15	Orifice	16	7.7				
720 minute winter	16	3.002	ATTENUATION 2	7.7	1.178	0.107	0.2430	
360 minute winter	FC4 Turning	Orifice	11	13.0				
360 minute winter	FC4 Turning	Orifice	11	4.2				

Results for 30 year +40% CC Critical Storm Duration. Lowest mass balance: 99.68%

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	188	133.088	0.738	105.1	433.4201	0.0000	SURCHARGED
240 minute winter	CP1 FC Eastern	188	133.087	0.775	24.4	1.1094	0.0000	SURCHARGED
240 minute winter	3	188	132.238	0.065	24.4	0.0930	0.0000	OK
480 minute winter	Attenuation 1	344	130.514	0.714	52.1	407.0516	0.0000	SURCHARGED
480 minute winter	CP2 FC Western	344	130.512	0.789	18.6	1.1284	0.0000	SURCHARGED
360 minute winter	6	272	129.677	0.067	42.9	0.0956	0.0000	OK
360 minute winter	7	272	127.783	0.065	42.9	0.0925	0.0000	OK
360 minute winter	8	272	124.723	0.073	42.9	0.1042	0.0000	OK
360 minute winter	9	272	121.895	0.094	42.9	0.1352	0.0000	OK
360 minute summer	ATTENUATION 2	248	120.941	0.741	78.2	281.5094	0.0000	SURCHARGED
360 minute winter	11	272	119.781	0.143	66.4	0.2526	0.0000	OK
360 minute winter	12	272	119.690	0.123	66.4	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 Eastern	24.4	0.263	0.152	2.4126	
240 minute winter	CP1 FC Eastern	Orifice	3	11.5				
240 minute winter	CP1 FC Eastern	Hydro-Brake®	3	13.0				
240 minute winter	3	1.002	6	24.4	1.750	0.045	1.1134	
480 minute winter	Attenuation 1	2.000	CP2 FC Western	18.6	0.191	0.168	2.7472	
480 minute winter	CP2 FC Western	Orifice	6	5.7				
480 minute winter	CP2 FC Western	Hydro-Brake®	6	12.9				
360 minute winter	6	1.003	7	42.9	3.002	0.045	0.3175	
360 minute winter	7	1.004	8	42.9	2.818	0.046	0.5573	
360 minute winter	8	1.005	9	42.9	2.118	0.058	1.1259	
360 minute winter	9	1.006	11	42.9	1.875	0.085	0.3322	
360 minute summer	ATTENUATION 2	3.003	FC4 Turning	37.3	0.405	0.142	5.5885	
360 minute winter	11	1.007	12	66.4	1.432	0.096	0.3279	2383.3

Results for 30 year +40% CC Critical Storm Duration. Lowest mass balance: 99.68%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
960 minute winter	MARQUEE PAD 14	930	123.826	0.476	73.3	1276.0550	0.0000	SURCHARGED
960 minute winter	FC3 MARQUEE15	930	123.826	0.773	36.2	1.1069	0.0000	SURCHARGED
960 minute winter	16	930	121.903	0.053	8.5	0.0149	0.0000	OK
360 minute summer	FC4 Turning	248	120.939	0.974	37.3	1.3932	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)
960 minute winter	MARQUEE PAD 14	3.000	FC3 MARQUEE15	36.2	0.356	0.100	3.7754	
960 minute winter	FC3 MARQUEE15	Orifice	16	8.5				
960 minute winter	16	3.002	ATTENUATION 2	8.5	1.215	0.120	0.2627	
360 minute summer	FC4 Turning	Orifice	11	14.6				
360 minute summer	FC4 Turning	Orifice	11	9.1				

Results for 100 year Critical Storm Duration. Lowest mass balance: 99.67%

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	188	133.015	0.665	93.3	386.9930	0.0000	SURCHARGED
240 minute winter	CP1 FC Eastern	188	133.014	0.702	22.8	1.0041	0.0000	SURCHARGED
240 minute winter	3	188	132.236	0.063	22.8	0.0900	0.0000	OK
960 minute summer	Attenuation 1	615	130.466	0.666	42.0	379.3461	0.0000	SURCHARGED
960 minute summer	CP2 FC Western	615	130.464	0.741	13.4	1.0606	0.0000	SURCHARGED
480 minute winter	6	352	129.670	0.060	35.0	0.0864	0.0000	OK
480 minute winter	7	352	127.777	0.059	35.0	0.0839	0.0000	OK
480 minute winter	8	352	124.716	0.066	35.0	0.0943	0.0000	OK
480 minute winter	9	352	121.885	0.085	35.0	0.1213	0.0000	OK
480 minute summer	ATTENUATION 2	320	120.863	0.663	59.8	251.9986	0.0000	SURCHARGED
480 minute summer	11	328	119.769	0.131	56.2	0.2308	0.0000	OK
480 minute summer	12	328	119.681	0.114	56.2	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)
240 minute winter	Formal CP 1	1.000	CP1 FC Eastern	22.8	0.257	0.142	2.4126	
240 minute winter	CP1 FC Eastern	Orifice	3	10.9				
240 minute winter	CP1 FC Eastern	Hydro-Brake®	3	12.0				
240 minute winter	3	1.002	6	22.8	1.715	0.042	1.0612	
960 minute summer	Attenuation 1	2.000	CP2 FC Western	13.4	0.177	0.121	2.7472	
960 minute summer	CP2 FC Western	Orifice	6	5.5				
960 minute summer	CP2 FC Western	Hydro-Brake®	6	7.9				
480 minute winter	6	1.003	7	35.0	2.833	0.037	0.2742	
480 minute winter	7	1.004	8	35.0	2.646	0.037	0.4826	
480 minute winter	8	1.005	9	35.0	2.006	0.047	0.9685	
480 minute winter	9	1.006	11	35.0	1.774	0.069	0.2860	
480 minute summer	ATTENUATION 2	3.003	FC4 Turning	36.9	0.434	0.140	5.5885	
480 minute summer	11	1.007	12	56.2	1.372	0.081	0.2899	2341.5

Results for 100 year Critical Storm Duration. Lowest mass balance: 99.67%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
960 minute winter	MARQUEE PAD 14	930	123.779	0.429	69.1	1137.1870	0.0000	OK
960 minute winter	FC3 MARQUEE15	930	123.779	0.727	36.3	1.0406	0.0000	SURCHARGED
960 minute winter	16	930	121.902	0.052	8.3	0.0147	0.0000	OK
480 minute summer	FC4 Turning	320	120.861	0.896	36.9	1.2827	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)
960 minute winter	MARQUEE PAD 14	3.000	FC3 MARQUEE15	36.3	0.371	0.100	3.7449	
960 minute winter	FC3 MARQUEE15	Orifice	16	8.3				
960 minute winter	16	3.002	ATTENUATION 2	8.3	1.204	0.116	0.2567	
480 minute summer	FC4 Turning	Orifice	11	13.9				
480 minute summer	FC4 Turning	Orifice	11	7.5				

Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 99.74%

Node Event	US Node	Peak (mins)	Level (m)	Depth (m)	Inflow (l/s)	Node Vol (m³)	Flood (m³)	Status
360 minute summer	Formal CP 1	264	133.278	0.927	139.3	552.4974	0.0000	FLOOD RISK
360 minute summer	CP1 FC Eastern	264	133.276	0.964	25.2	1.3798	0.0000	FLOOD RISK
120 minute summer	3	122	132.239	0.066	25.1	0.0942	0.0000	OK
480 minute summer	Attenuation 1	328	130.669	0.869	97.7	495.2475	0.0000	SURCHARGED
480 minute summer	CP2 FC Western	328	130.666	0.943	21.2	1.3488	0.0000	SURCHARGED
720 minute summer	6	450	129.679	0.069	46.2	0.0993	0.0000	OK
720 minute summer	7	450	127.785	0.067	46.2	0.0959	0.0000	OK
720 minute summer	8	450	124.726	0.075	46.2	0.1080	0.0000	OK
720 minute summer	9	450	121.898	0.098	46.2	0.1407	0.0000	OK
480 minute summer	ATTENUATION 2	312	121.268	1.068	81.4	345.6100	0.0000	SURCHARGED
480 minute summer	11	312	119.793	0.155	77.0	0.2739	0.0000	OK
480 minute summer	12	312	119.700	0.133	77.0	0.0000	0.0000	OK

Link Event (Upstream Depth)	US Node	Link	DS Node	Outflow (l/s)	Velocity (m/s)	Flow/Cap	Link Vol (m³)	Discharge Vol (m³)
360 minute summer	Formal CP 1	1.000	CP1 FC Eastern	25.2	0.245	0.157	2.4126	
360 minute summer	CP1 FC Eastern	Orifice	3	12.8				
360 minute summer	CP1 FC Eastern	Hydro-Brake®	3	13.0				
120 minute summer	3	1.002	6	25.1	1.764	0.046	1.1339	
480 minute summer	Attenuation 1	2.000	CP2 FC Western	21.2	0.200	0.192	2.7472	
480 minute summer	CP2 FC Western	Orifice	6	6.3				
480 minute summer	CP2 FC Western	Hydro-Brake®	6	15.0				
720 minute summer	6	1.003	7	46.2	3.067	0.049	0.3348	
720 minute summer	7	1.004	8	46.2	2.878	0.049	0.5872	
720 minute summer	8	1.005	9	46.2	2.159	0.063	1.1896	
720 minute summer	9	1.006	11	46.2	1.912	0.092	0.3508	
480 minute summer	ATTENUATION 2	3.003	FC4 Turning	37.3	0.416	0.141	5.5885	
480 minute summer	11	1.007	12	77.0	1.488	0.112	0.3661	3027.1

Results for 100 year +40% CC Critical Storm Duration. Lowest mass balance: 99.74%

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	1380	123.985	0.634	69.1	1752.0850	0.0000	SURCHARGED
1440 minute winter	FC3 MARQUEE15	1380	123.984	0.932	36.4	1.3340	0.0000	FLOOD RISK
1440 minute winter	16	1380	121.906	0.055	9.4	0.0157	0.0000	OK
480 minute summer	FC4 Turning	312	121.264	1.299	37.3	1.8585	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	36.4	0.372	0.101	3.7754	
1440 minute winter	FC3 MARQUEE15	Orifice	16	9.4				
1440 minute winter	16	3.002	ATTENUATION 2	9.4	1.249	0.132	0.2816	
480 minute summer	FC4 Turning	Orifice	11	16.9				
480 minute summer	FC4 Turning	Orifice	11	13.9				