

Saint Hill – Site 2,
Marquee area
Saint Hill Road,
East Grinstead, West Sussex, RH19 4JY
Flood Risk Assessment & Surface Water Drainage
Strategy

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Table of Contents

1	Introduction	4
2	Policy Compliance	5
3	Site Location	5
4	Geology and Infiltration Potential	8
5	Sequential Test/Exception Test	9
6	Climate Change Allowances	10
7	Sources of Flooding	11
8	Residual Risks	20
9	Flood Risk Management Measures	20
10	Sustainable Urban Drainage (SuDS) Assessment	21
11	Surface Water Drainage Strategy	23
12	Foul Water Drainage Strategy	24
13	Water Quality	24
14	Schedule of Maintenance	25
15	Conclusion	27

Appendices

Appendix I – Architect Plans

Appendix II – Proposed Drainage strategy

Appendix III – Calculations

1 Introduction

QED Structures has been instructed to prepare 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 proposed development includes permanent infrastructure to facilitate annual events, permanent road access with permeable access areas, and an additional toilet block.

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 marquee development site currently consists of a field at a steep incline. It is understood that the proposed development is for levelling and grading of the existing land to facilitate the erection of marquees during events held on-site. This requires the construction of crane pads to the south of this area, as well as a new toilet block to the southeast. Land to the immediate west is to be rewilded following construction.

The British Geological Survey (BGS) Geology of Britain Viewer indicates that the bedrock underlying the majority of the site is Ardingly Sandstone Member – Sandstone. The northern boundary of the site contains bedrock geology consisting of Lower Grinstead Clay-Mudstone (Source: BGS). There are no superficial deposits underlying the site.

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.

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.

All new external surfaces should be permeable, where feasible, to promote infiltration and mitigate the potential to generate additional surface water runoff.

The drainage strategy proposes that runoff from the turning area, marquee pad and toilet block (22930m²) will discharge to a watercourse west of the site via an unlined permeable surface (Type B), with storage in the sub-base of the marquee pad, and supplemental Geocellular storage in the turning area.

Infiltration testing has been undertaken and results indicate impeded infiltration and therefore full infiltration may not be a feasible method of discharge for all surface water; however, the sub-base attenuation is to be unlined (Type B) to promote infiltration as much as possible.

Discharge rates for the marquee pad storage are restricted to a maximum of 9.4l/s for the marquee pad area via an orifice plate at invert level, which is less than the Q_{bar} (10.5/s) for this area. The Marquee pad is to have a storage area of a minimum 10000sqm and 0.65m deep sub base with 30% void ratio offering 1950m³ storage.

This flows downstream to additional storage for the access track and turning area where a secondary orifice plates complex flow control limits flow to a maximum of 30.8l/s in the 1in100yr+CC rainfall event.

The turning area will be attenuated with an Aco HD geocellular crate. It has a contributing attenuation area of 5513m², with the tank plan area being 400m², 910mm deep and 97% porosity. The storage volume needed is 345.6m³.

It can be seen proposed outflow rates are no greater than the pre-development greenfield runoff rates for the Marquee pad and access track drained areas.

It can be seen from the calculations all half drains times are less than 1440mins so are acceptable offering greater than 50% storage after 24hrs.

All foul water drainage is to be routed to the existing foul water network and thereby to the existing water treatment plant south of the site – the capacity of which 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.

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
- River Medway CFMP

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 as an event space.



Figure 1: Site location plan

The marquee development site currently consists of field at a steep incline. It is understood the proposed development is for levelling and grading of the existing land to facilitate the erection of marquees during events held on site. This requires the construction of crane pads to the south of this area, as well as a new toilet block to the southeast. Land to the immediate west is to be rewilded following construction. Architectural plans are included in Appendix I.

Architectural site plans detailing the proposed development are included below.



Figure 2: Proposed Site Plan

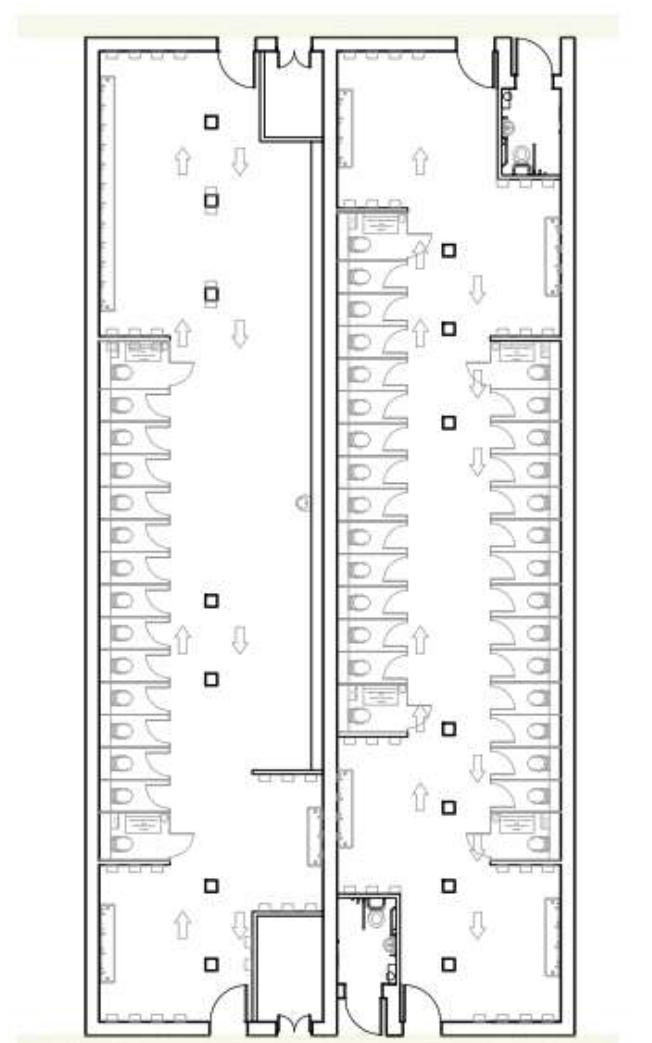


Figure 3: Proposed Toilet Block Plan

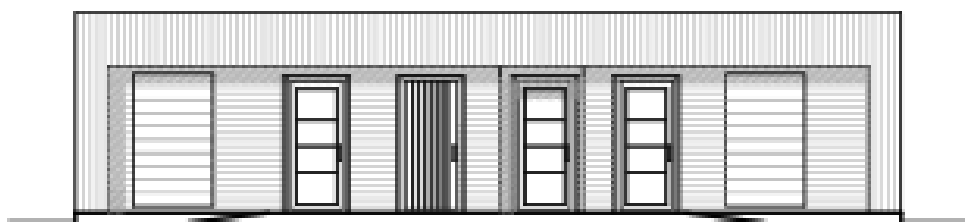


Figure 4: Proposed Toilet Block Elevation

4 Geology and Infiltration Potential

The British Geological Survey (BGS) Geology of Britain Viewer indicates that the bedrock underlying the majority of the site is Ardingly Sandstone Member – Sandstone. The northern boundary of the site contains bedrock geology consisting of Lower Grinstead Clay - Mudstone (Source: BGS). There are no superficial deposits underlying the site. Initial infiltration testing has been undertaken and is shown not to be a feasible method of discharge for all surface water; however, the sub-base attenuation is to be unlined (Type B) to promote infiltration as much as possible.

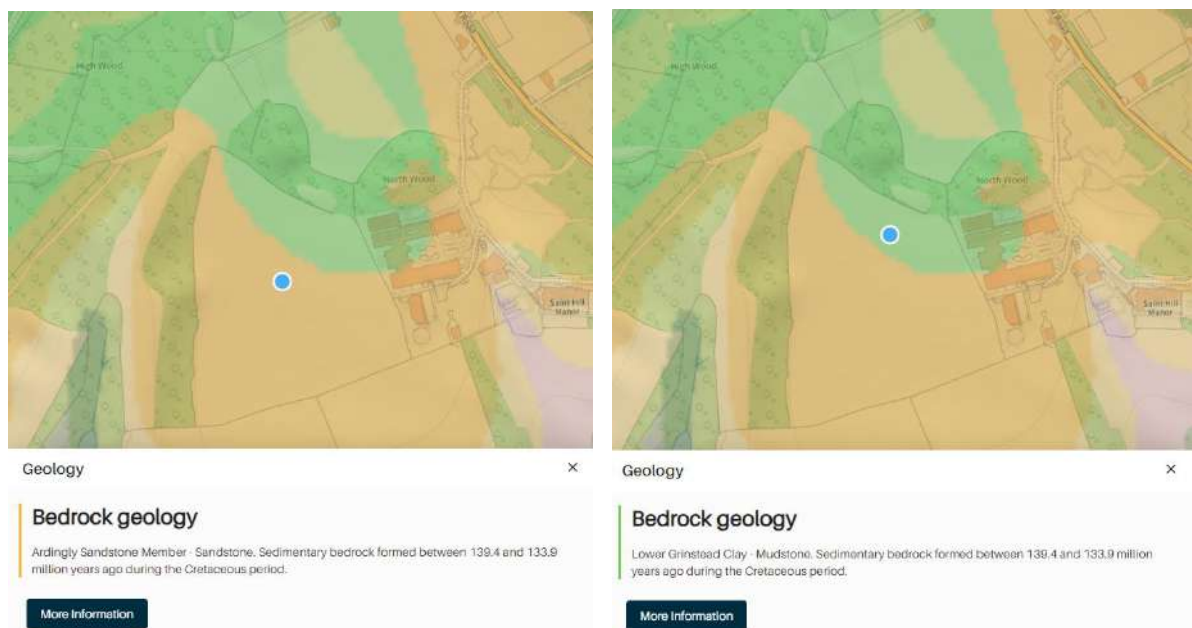


Figure 5: BGS geology map extracts

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 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 residential development (lifespan 100yrs), in accordance with NPPF guidance, the upper end 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 6: 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 7: 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

As the proposals are within flood zone 1, the peak river flow allowances do not apply to the site.

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.

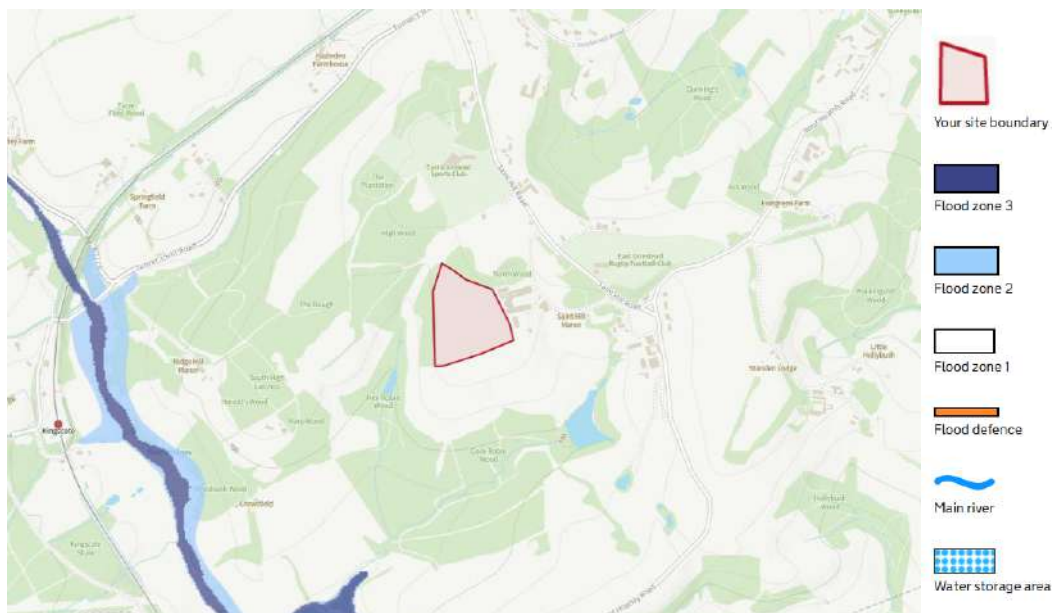


Figure 8: Flood map for planning extract with site boundary shown in red

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.

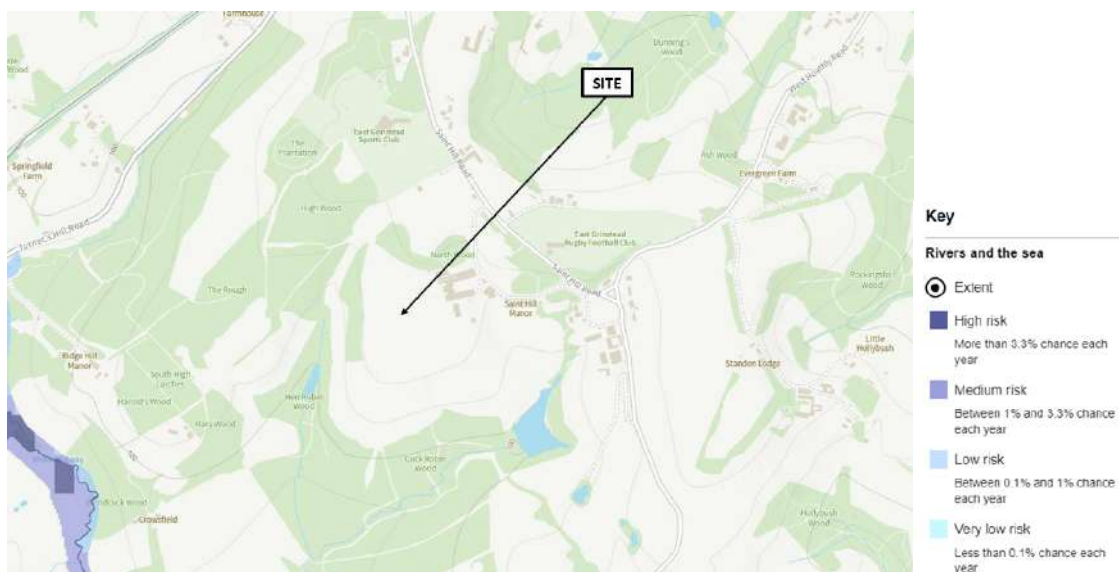


Figure 9: EA Long term risk of flooding map extract - Rivers and the Sea

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.

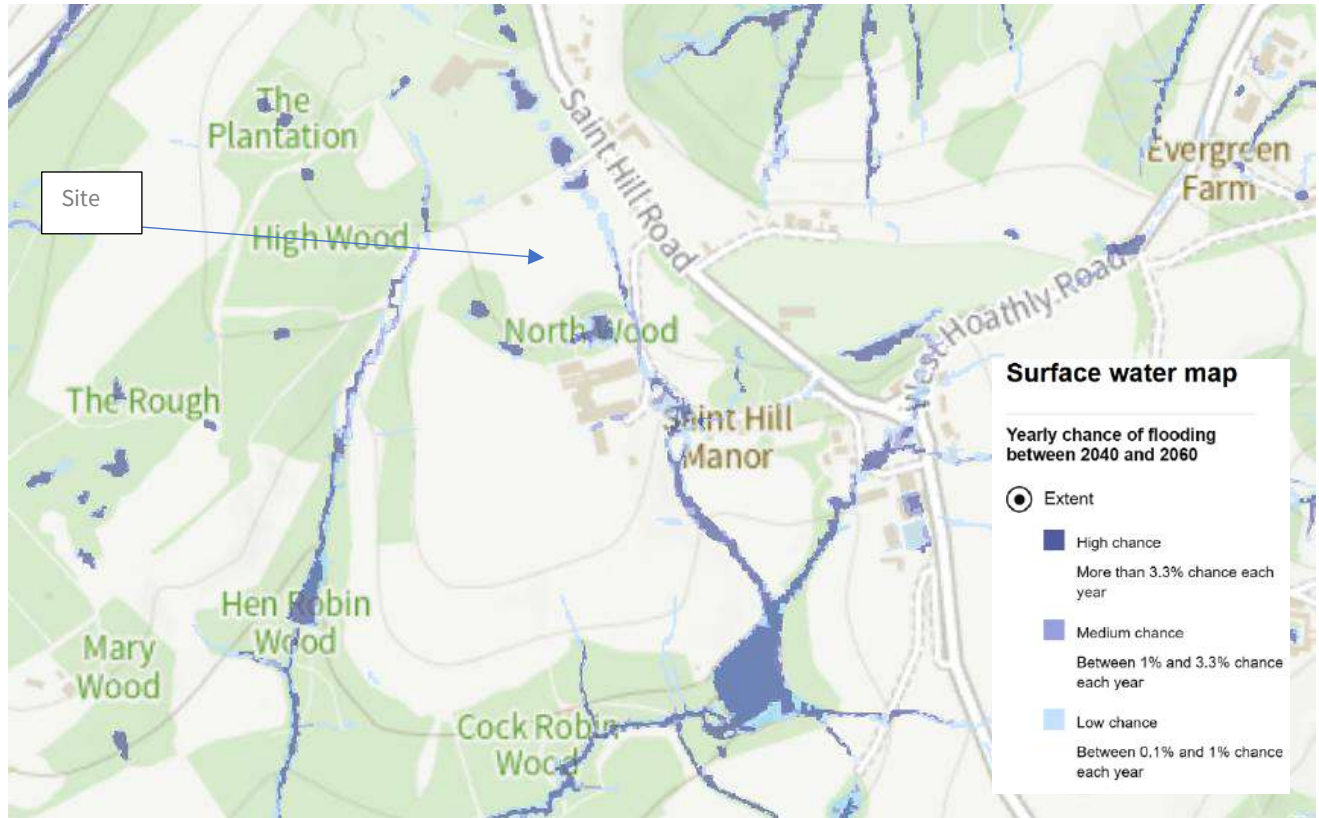


Figure 10: EA Long term risk of flooding map extract – Surface Water

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.

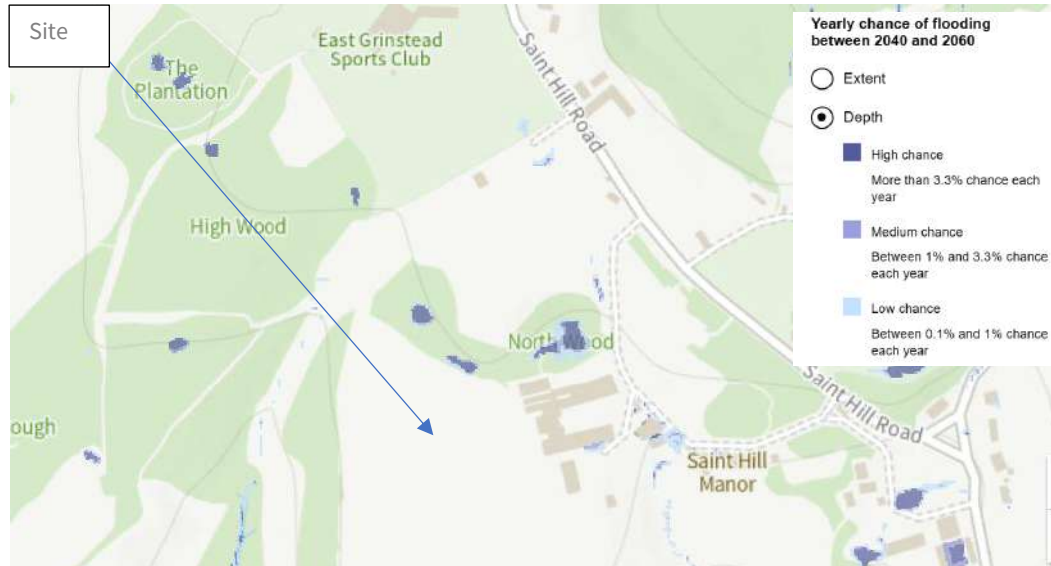


Figure 11: EA RoFSW data showing chance of 200mm water depth during the 2040-2060 surface water event

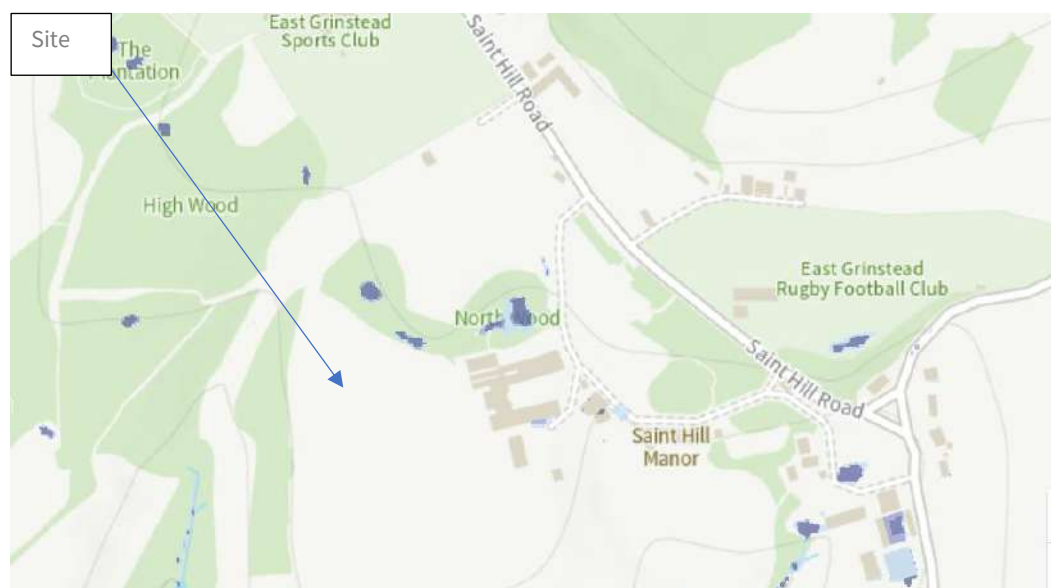


Figure 12: EA RoFSW data showing chance of 300mm water depth during the 2040-2060 surface water event

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.

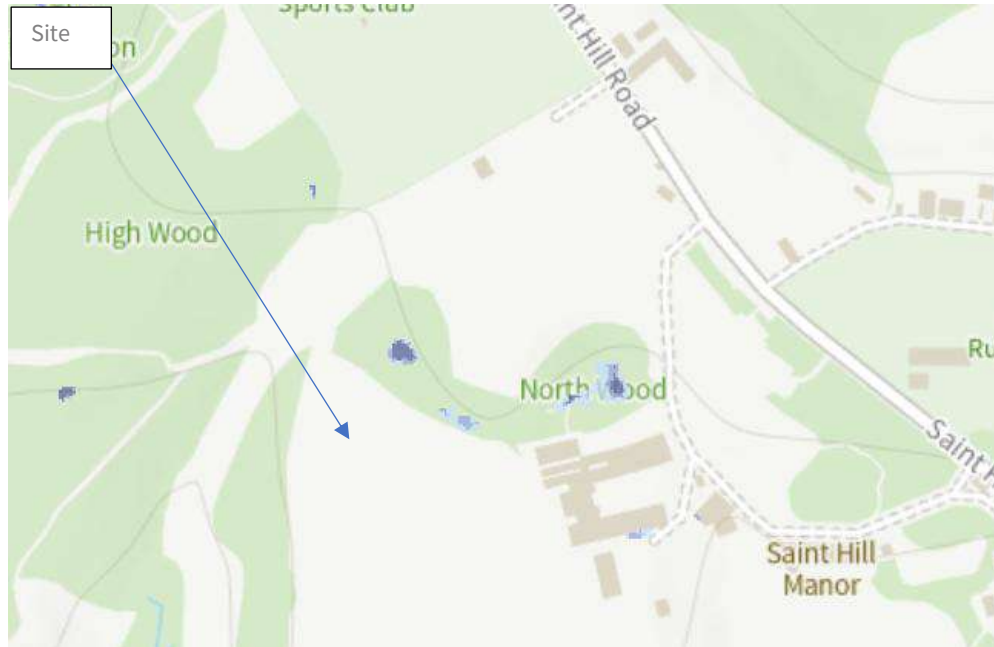


Figure 13: EA RoFSW data showing chance of 600mm water depth during the 2040-2060 surface water event

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.

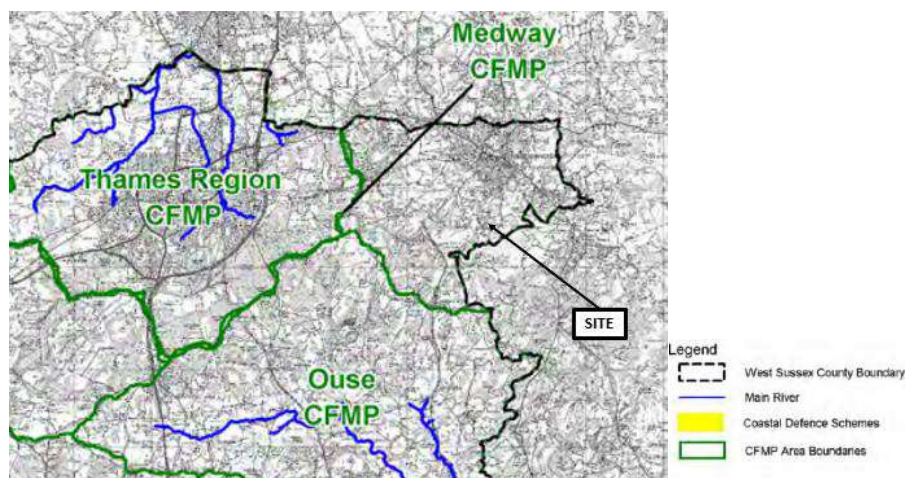


Figure 14: West Sussex County Council SFRA – Flood Management Plans and Strategies map extract

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.

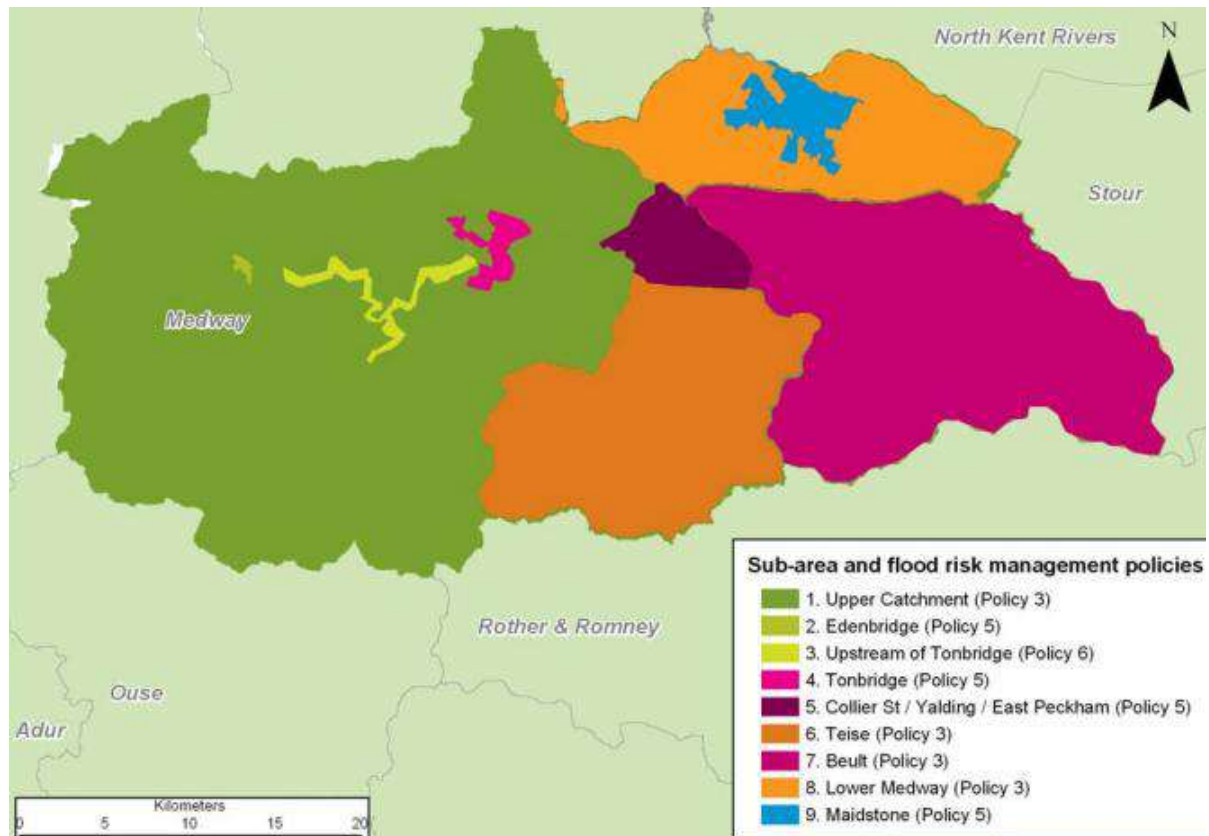


Figure 15: River Medway Catchment Flood Management Plan – Sub-areas and Flood Risk Management Policies map extract

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.

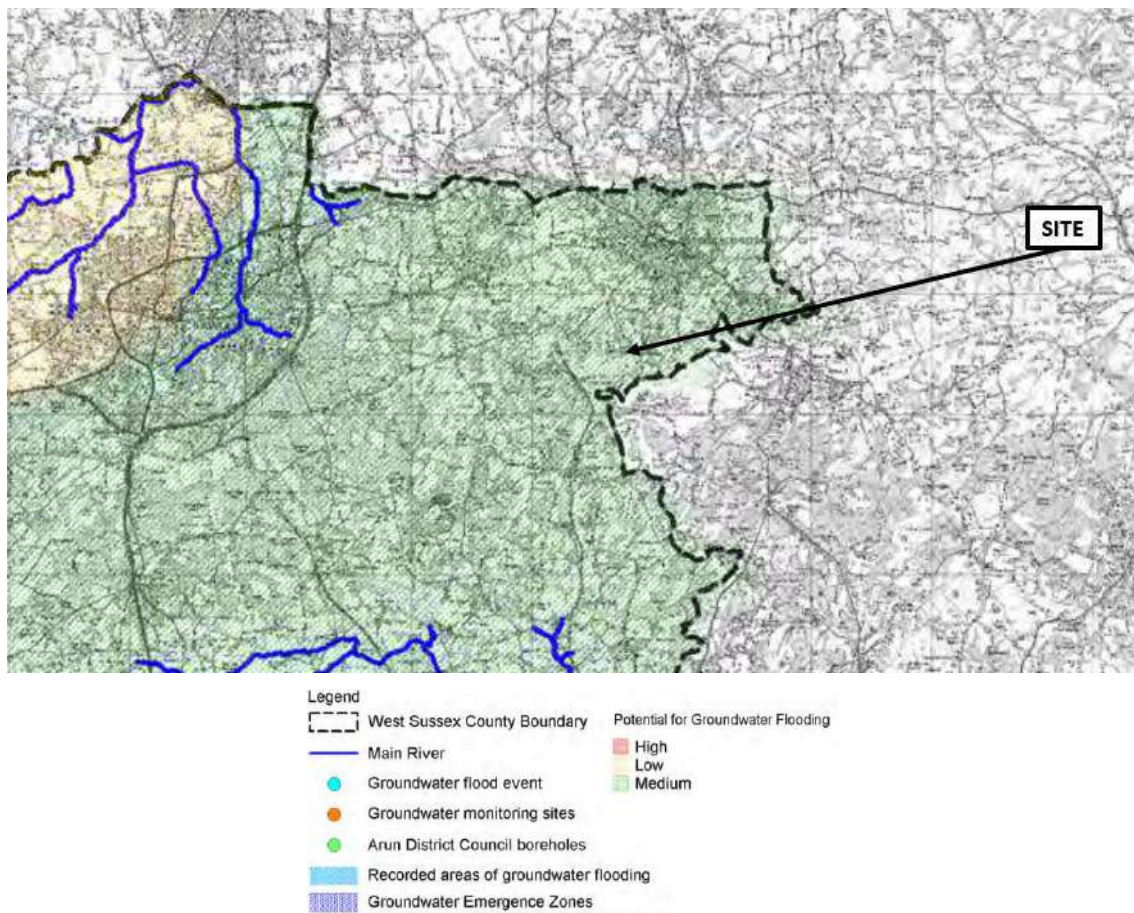


Figure 16: West Sussex County Council SFRA – Areas Prone to Groundwater Flooding map extract

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 are no recorded surface water flooding events recorded on or near to the development site.

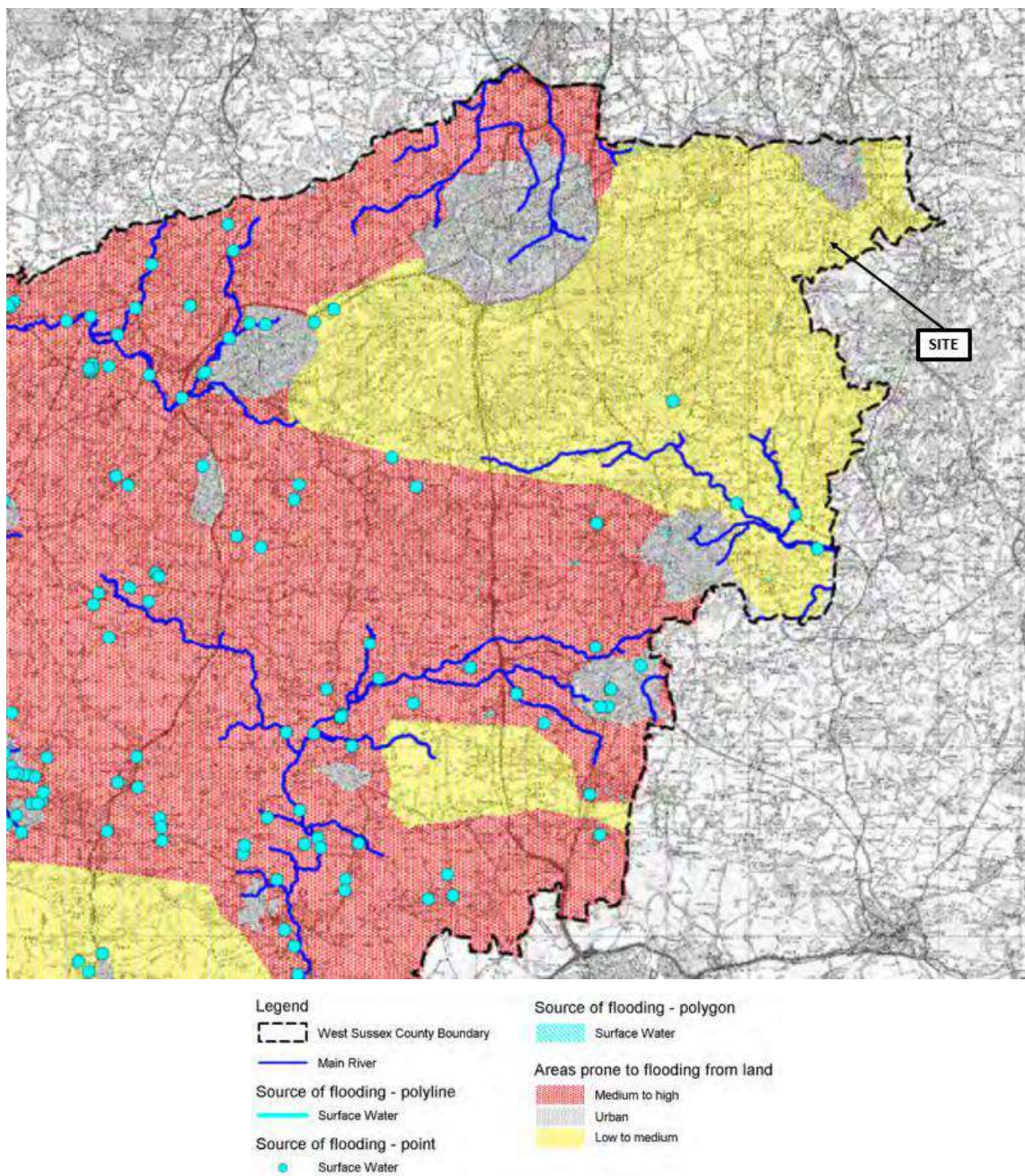


Figure 17: West Sussex County Council SFRA – Areas Prone to Flooding from the Land map extract

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.

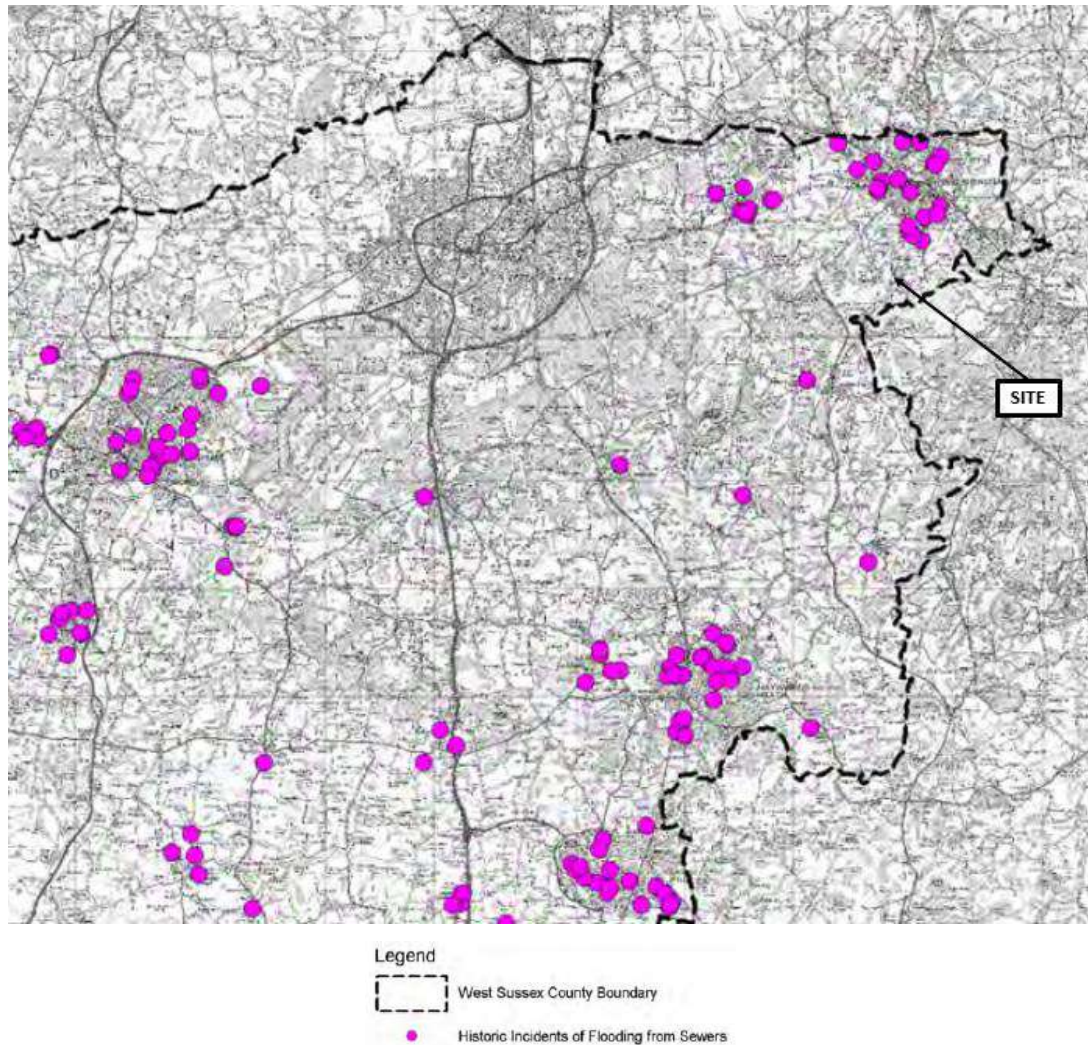


Figure 18: West Sussex County Council SFRA – Areas Prone to Flooding from Sewers map extract

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 marquee pad 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 grading of the site in preparation for temporary structures is undertaken in a flood resilient manner in case of exceedance events.

The proposed ground levels for the Marquee pad and access road should be directed to fall towards the south and west to direct overland flow towards the existing water course rather than cause on site flooding or direct water to existing infrastructure.

All new external surfaces should be permeable in nature, where feasible, to promote infiltration and mitigate the potential to generate additional surface water runoff.

Provision of sustainable storm water drainage measures should be considered by the developer and are outlined within these reports proposals in order to limit runoff from the site, to greenfield runoff rates, or as close to as practical, to ensure that risk to others does not increase when this proposal is complete.

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.
	2.	Use infiltration techniques, such as porous surfaces in non-clay areas	✓	Infiltration testing has revealed that only partial infiltration is feasible.
	3.	Attenuate rainwater in ponds or open water features for gradual release	x	Not proposed due to gradient of land.
	4.	Attenuate rainwater by storing in tanks or sealed water features for gradual release	✓	It is necessary to provide attenuation in order to keep runoff rates from the proposed development as close as feasible to greenfield runoff rates.
	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	x	
	7.	Discharge rainwater to Combined Sewer	-	

Table 2: SuDS Drainage Hierarchy

Following the SuDS drainage hierarchy, surface water discharge via infiltration was initially proposed due to the presence of the Ardingly Sandstone Member – Sandstone underlying most of the site. However, infiltration testing conducted in a limited area of the site indicated poor infiltration rates. As a result, full infiltration is not considered a feasible discharge method, though unlined permeable surfaces will be used to encourage infiltration where possible.

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	Slopes and space limit potential for open features.	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 that runoff from the turning area, marquee pad and toilet block (22,900m² including an area for potential future development) will discharge to a watercourse west of the site via an unlined permeable surface (Type B) with storage in the sub-base. This is to be implemented across the graded/reinforced area of land to provide storage within its sub-base. The marquee pad provides initial upstream attenuation flow control with runoff draining downstream to the turning area at the low point where flows are controlled with additional storage to greenfield rates for the combined turning and Marquee pad areas.

Infiltration tests have been conducted, and shown that the results were poor, in which it may not be feasible as the main method of discharge, but the permeable land is to be unlined (Type B) to promote infiltration as much as possible.

Access roads/walkways are proposed to be of permeable construction.

Infiltration testing has been undertaken and is shown not to be a feasible method of discharge for all surface water; however, the sub-base attenuation is to be unlined (Type B) to promote infiltration as much as possible. For the purposes of calculations (Appendix III) sub-base attenuation is assumed to have 0% permeability, providing no infiltration to be conservative pending further infiltration testing.

	SURFACE WATER DISCHARGE RATES SUMMARY				
	Area (ha)	Discharge Rates (l/s)			
		2 year	QBar	30 year	100 year
Greenfield Rates Marquee pad	1.739	9.3	10.5	24.2	33.6
Proposed Runoff Rates Marquee pad+ 40% CC	1.739	7.2		8.5	9.4
Greenfield Rates access track & turning area	0.55	2.7	3.0	7.0	9.7
Combined Greenfield Rates	1.559	12.0		31.2	43.3
Proposed Combined Runoff Rates + 40% CC	2.29	11.6		23.7	30.8

Discharge rates for the marquee pad storage are restricted to a maximum of 9.4l/s for the marquee pad area via an orifice plate at invert level, which is less than the Qbar (10.5/s) for this area. The Marquee pad is to have a storage area of a minimum 10000sqm and 0.65m deep sub base with 30% void ratio offering 1950m³ storage.

This flows downstream to additional storage for the access track and turning area where a secondary orifice plates complex flow control limits flow to a maximum of 30.8l/s in the 1in100yr+CC rainfall event. It can be seen proposed outflow rates are no greater than the pre-development greenfield runoff rates for the Marquee pad and access track drained areas.

The turning area will be attenuated with an Aco HD geocellular crate. It has a contributing attenuation area of 5513m², with the tank plan area being 400m², 910mm deep and 97% porosity. The storage volume needed is 345.6m³.

It can be seen from the calculations all half drains times are less than 1440mins so are acceptable offering greater than 50% storage after 24hrs.

A proposed storm drainage strategy plan layout is included in Appendix II.

■ Hove ■ London ■ Manchester ■ Leeds ■ Barcelona

12 Foul Water Drainage Strategy

All foul water drainage is to be routed to the existing foul water network and thereby to the water treatment plant south of the site – the capacity of which is to be confirmed 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. access 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		0.8	0.6	0.45

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/Bioretentio n	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 Structures has been instructed to prepare 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 proposed development includes permanent infrastructure to facilitate annual events, permanent road access with permeable access areas and an additional toilet block.

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 marquee development site currently consists of a field at a steep incline. It is understood that the proposed development is for levelling and grading of the existing land to facilitate the erection of marquees during events held on-site. This requires the construction of crane pads to the south of this area, as well as a new toilet block to the southeast. Land to the immediate west is to be rewilded following construction.

The British Geological Survey (BGS) Geology of Britain Viewer indicates that the bedrock underlying the majority of the site is Ardingly Sandstone Member – Sandstone. The northern boundary of the site contains bedrock geology consisting of Lower Grinstead Clay-Mudstone (Source: BGS). There are no superficial deposits underlying the site.

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.

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.

All new external surfaces should be permeable, where feasible, to promote infiltration and mitigate the potential to generate additional surface water runoff.

The drainage strategy proposes that runoff from the turning area, marquee pad, toilet block, and potential west extension is proposed to discharge to a watercourse west of the site via an unlined permeable surface (Type B), with storage in the sub-base of the marquee pad, and supplemental Geocellular storage in the turning area.

Infiltration testing has been undertaken and results indicate impeded infiltration and therefore full infiltration may not be a feasible method of discharge for all surface water; however, the sub-base attenuation is to be unlined (Type B) to promote infiltration as much as possible.

Discharge rates for the marquee pad storage are restricted to a maximum of 9.4l/s for the marquee pad area via an orifice plate at invert level, which is less than the Qbar (10.5/s) for this area. The Marquee pad is to have a storage area of a minimum 10000sqm and 0.65m deep sub base with 30% void ratio offering 1950m³ storage.

This flows downstream to additional storage for the access track and turning area where a secondary orifice plates complex flow control limits flow to a maximum of 30.8l/s in the 1in100yr+CC rainfall event.

The turning area will be attenuated with an Aco HD geocellular crate. It has a contributing attenuation area of 5513m², with the tank plan area being 400m², 910mm deep and 97% porosity. The storage volume needed is 345.6m³.

It can be seen proposed outflow rates are no greater than the pre-development greenfield runoff rates for the Marquee pad and access track drained areas.

It can be seen from the calculations all half drains times are less than 1440mins so are acceptable offering greater than 50% storage after 24hrs.

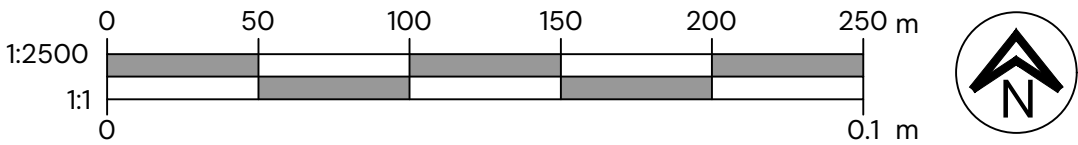
All foul water drainage is to be routed to the existing foul water network and thereby to the existing water treatment plant south of the site – the capacity of which is to be confirmed at the later design stage.

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.

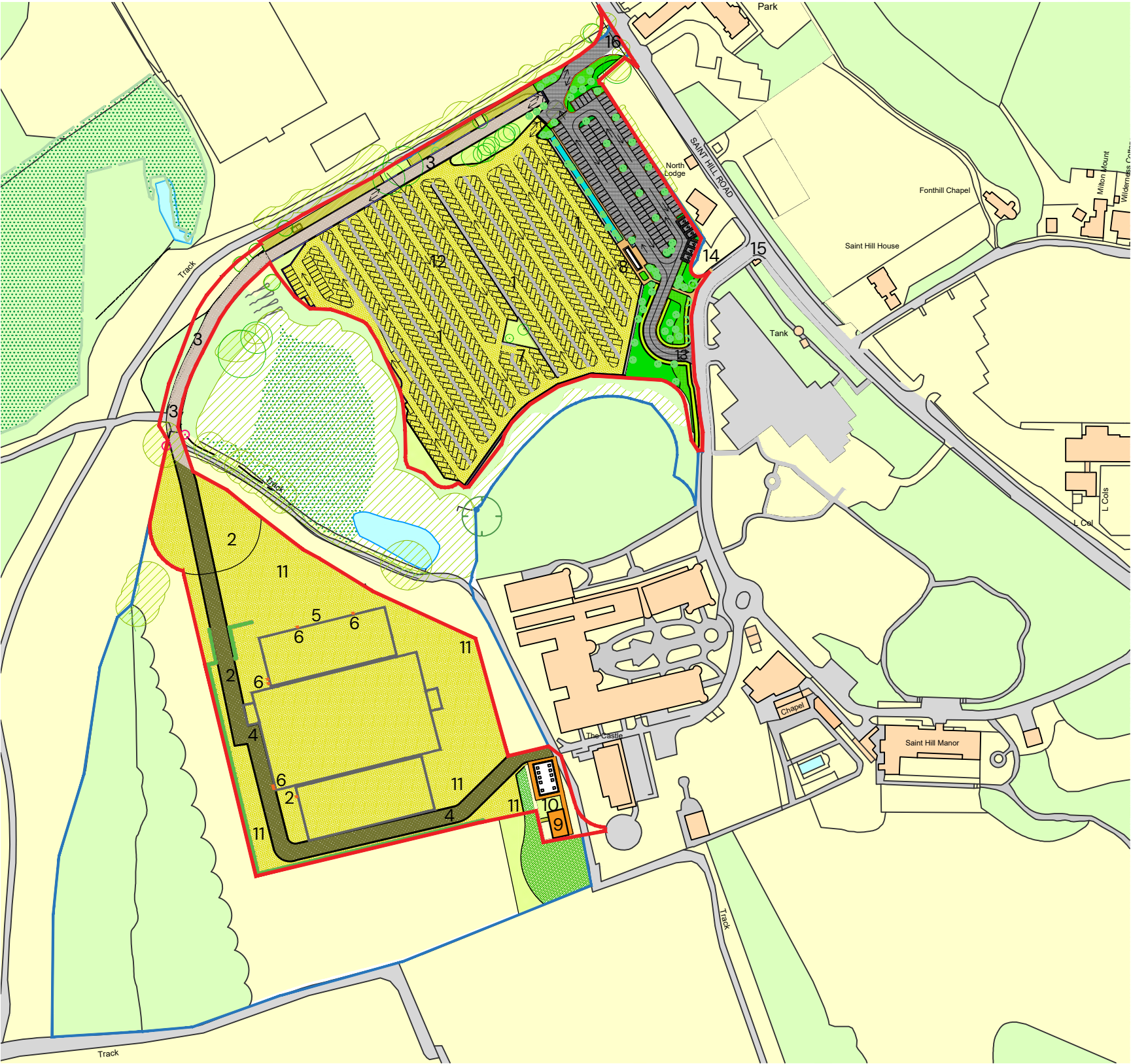


APPENDIX I – ARCHITECT PLANS

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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

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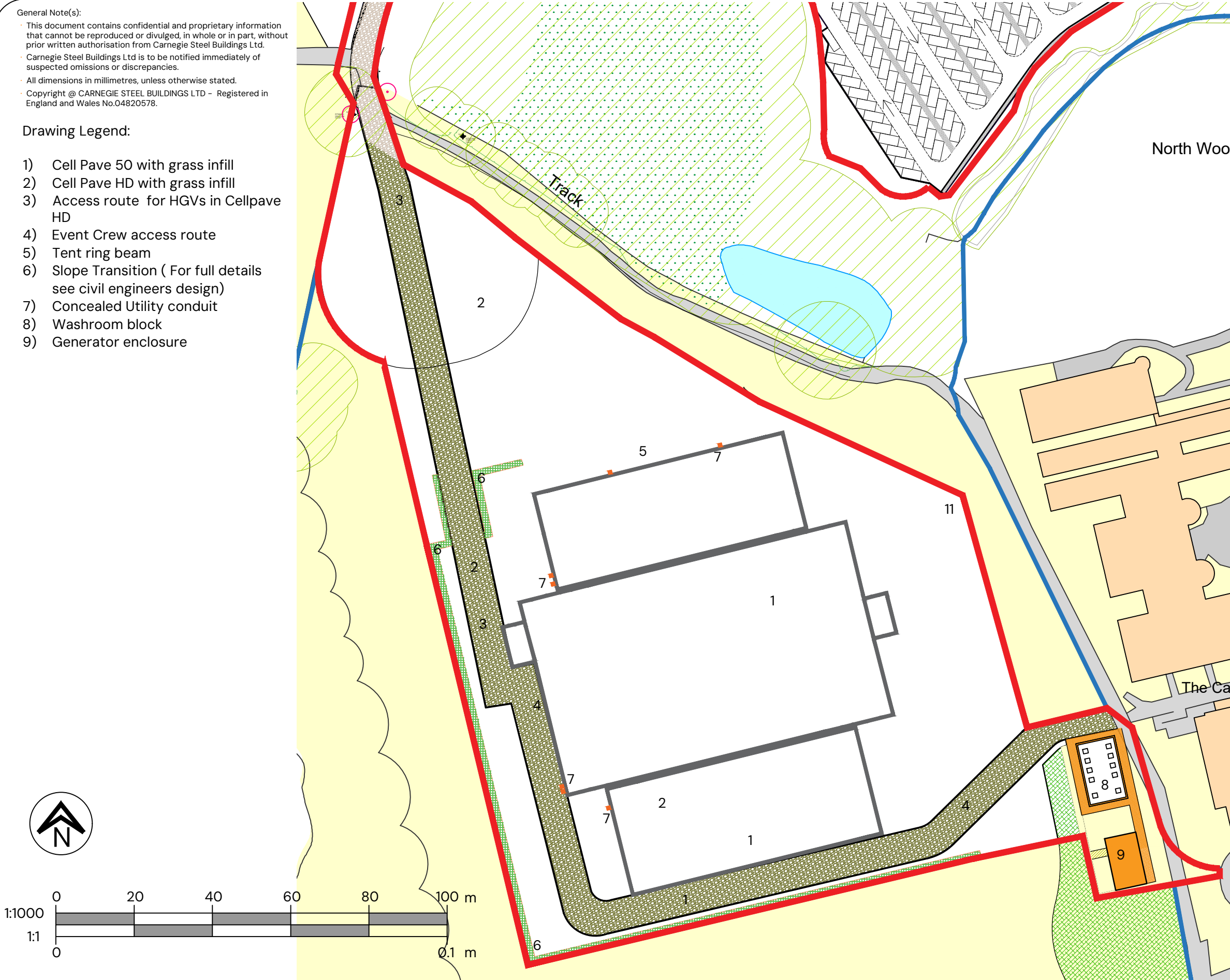
Drawing Legend:

- 1) Cell Pave 50 with grass infill
- 2) Cell Pave HD with grass infill
- 3) Access route for HGVs in Cellpave HD
- 4) Event Crew access route
- 5) Tent ring beam
- 6) Slope Transition (For full details see civil engineers design)
- 7) Concealed Utility conduit
- 8) Washroom block
- 9) Generator enclosure

FOR PLANNING

Hobbs Field Event Space

- Mar-Key ringbeam designed to be made from concrete and laid into grassland area. Intention is that tall grass will hide these in off peak use and then the field will be cut and strimmed for event season.
- Cellpave products by Groundtrax is to be used on surfaces to be reinforced. Areas that need more reinforcement will require Cellpave HD. This will include the access road, staging area and some of the construction area. Some parts can use Cellpave 50 which is a lower strength alternative.
- Infill of the Cellpave areas will be predominately fast growing grass mix with wildflowers.
- Due to low use, the grass and flowers will grow and conceal the foundations for the tent in off peak use and then be trimmed in back prior to the large event each year.



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Address: The Beeches, Old Ln, Tatsfield, Westerham TN16 2LH Phone: 01959 572501

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

DRAWING TITLE:
Event field Site Plan A3

CLIENT:
COSRECI

DRAWN: FH	CHECKED: JA	DATE: 09.01.2025	SCALE: 1:1000 @A3
PROJECT NO: 1093	DRAWING NO: 62	REV: C	SHEET NO: 1/1



APPENDIX II – PROPOSED DRAINAGE STRATEGY

FOR CONTINUATION SEE DRAWING 101

Car Park

North Wood

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LEGEND

- PERMEABLE ASPHALT TO PERMANENT CAR PARK ONLY
- COMPACTED GRAVEL
- GROUND REINFORCEMENT GRID SYSTEM WITH GRAS 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

P06	Updated layout and LLFA comments	29/10/25	MN	JR
P05	For comment	20/03/25	KB	MN
P04	Issued for information	25/02/25	KB	MN
P03	Issued for information	12/02/25	KB	MN
P02	Issued for information	10/12/24	KB	MN
REV	DESCRIPTION	DATE	BY	APP

QED
structures

CONSULTING CIVIL AND
STRUCTURAL ENGINEERS
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QED PROJECT No.
24-164

CLIENT
International Association of
Scientists

PROJECT
Saint Hill Site 2 - New temporary
structure + North & West extensions
and new toilet block

TITLE
Proposed Drainage Layout

DRAWN	ENGINEER	CHECKED BY	APPROVED BY
IW	KB	MN	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-201

REVISION
P06

FLOW CONTROL MANHOLE
ORIFICE PLATE AT INVERT - 85mm DIA
SECONDARY ORIFICE AT 120.65mAOD
- 94mm DIA

FLOW RATE:
1IN2YR+CC = 11.6L/S
1IN30YR+CC = 23.7L/S
1IN100YR+CC =30.8L/S
MINIMUM OUTLET DIAMETER: 0.225m

TURNING AREA ATTENUATION

CONTRIBUTING AREA:5500m²
ATTENUATION TANK PLAN AREA:400m²
ACO HD GEOCELLULAR CRATE DEPTH: 0.92m
POROSITY: 97%
STORAGE VOLUME NEEDED:345.6m3

LEVELS AND SUBSEQUENT PIPE ROUTES ARE TO BE
CONFIRMED AT LATER DESIGN STAGE

OUTFALL TO
WATERCOURSE IN
WOODED AREA IN
AGREEMENT WITH
ADJACENT LANDOWNER.

DETAILS OF DISCHARGE
LOCATION TO BE
CONFIRMED AT LATER
DESIGN STAGE

RELEVANT PERMISSIONS
TO BE ASCERTAINED PRIOR
TO CONSTRUCTION

MARQUEE PAD FLOW CONTROL

ORIFICE PLATE AT INVERT - 69mm DIA

FLOW RATE:
1IN2YR+CC = 7.1L/S
1IN30YR+CC = 8.5L/S
1IN100YR+CC =9.4L/S
MINIMUM OUTLET DIAMETER: 0.225m
FOR OVERFLOW/EXCEEDANCE

RESURFACED/GRADED AREA TO BE OF PERMEABLE
CONSTRUCTION INCLUDING A SUB-BASE AS
ATTENUATION
MARQUEE PAD 14 in calcs

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

650mm SUB-BASE DEPTH NEEDED ACCORDING TO
CAUSEWAY FLOW CALCULATIONS DURING THE 1 IN
100 YEAR +40%CC RAINFALL EVENT

SUB-BASE TO BE 30% POROSITY
SUB BASE TO BE LAID FLAT TO MAXIMISE STORAGE

STORAGE DESIGNED TO ACCOMMODATE EXISTING
BUILDING RUNOFF (THAT CURRENTLY DRAINS TO
SOAKAWAY IN MARQUEE PAD AREA) AND AN
ALLOWANCE FOR POTENTIAL EXTENSIONS

STORAGE PROVIDED 1950m3

LEVELS TO BE CONFIRMED AT
DETAILED DESIGN STAGE.

PATHWAY TO HAVE NOMINAL
SUB-BASE TO CAPTURE AND
ATTENUATE SURFACE WATER.

DEPTH TO BE CONFIRMED AT
DETAILED DESIGN STAGE.

EXTENDED CATCHMENT
INCLUDED IN NETWORK
CALCULATIONS TO MAKE
ALLOWANCE FOR POTENTIAL
FUTURE DEVELOPMENT

NEW FOUL WATER CONNECTION
TO BE MADE FROM PROPOSED
WEST EXTENSION TO EXISTING
FOUL WATER DRAINAGE ON SITE

CAPACITY OF EXISTING DRAINAGE
NETWORK TO BE CONFIRMED AT
LATER DESIGN STAGE

SITE 2 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 WEST. 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 THE 1IN2yr, 1IN30yr and 1IN100yr
PREDEVELOPMENT RAINFALL EVENTS) VIA GRAVITY.

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

NO INFILTRATION HAS BEEN ALLOWED FOR IN THE STORAGE
CALCULATIONS AT THIS TIME DUE TO FAILED INITIAL INFILTRATION
TESTING. FURTHER INFILTRATION TESTING SHOULD BE UNDERTAKEN
ACROSS THE AREA AT FORMATION LEVEL ONCE WORKS COMMENCE
TO CONFIRM IF ANY INFILTRATION IS VIABLE.

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.

AN AREA OF EXISTING MAIN BUILDING ROOF RUNOFF AND POTENTIAL
FUTURE EXTENSION IS PROPOSED TO DRAIN INTO THE SITE 2 MARQUEE
SUB-BASE ATTENUATION. FEASIBILITY AND CAPACITY OF SURFACE
WATER DRAINAGE TO BE CONFIRMED AT LATER DESIGN STAGE.

NEW TOILET BLOCK ASSUMED TO
ALSO SERVE TEMPORARY
STRUCTURE

NEW TOILET BLOCK TO ROUTE
INTO CLOSEST FEASIBLE FOUL
WATER MANHOLE

EXISTING FOUL SEWAGE SYSTEM
CAPACITY TO BE CONFIRMED AT
LATER DESIGN STAGE

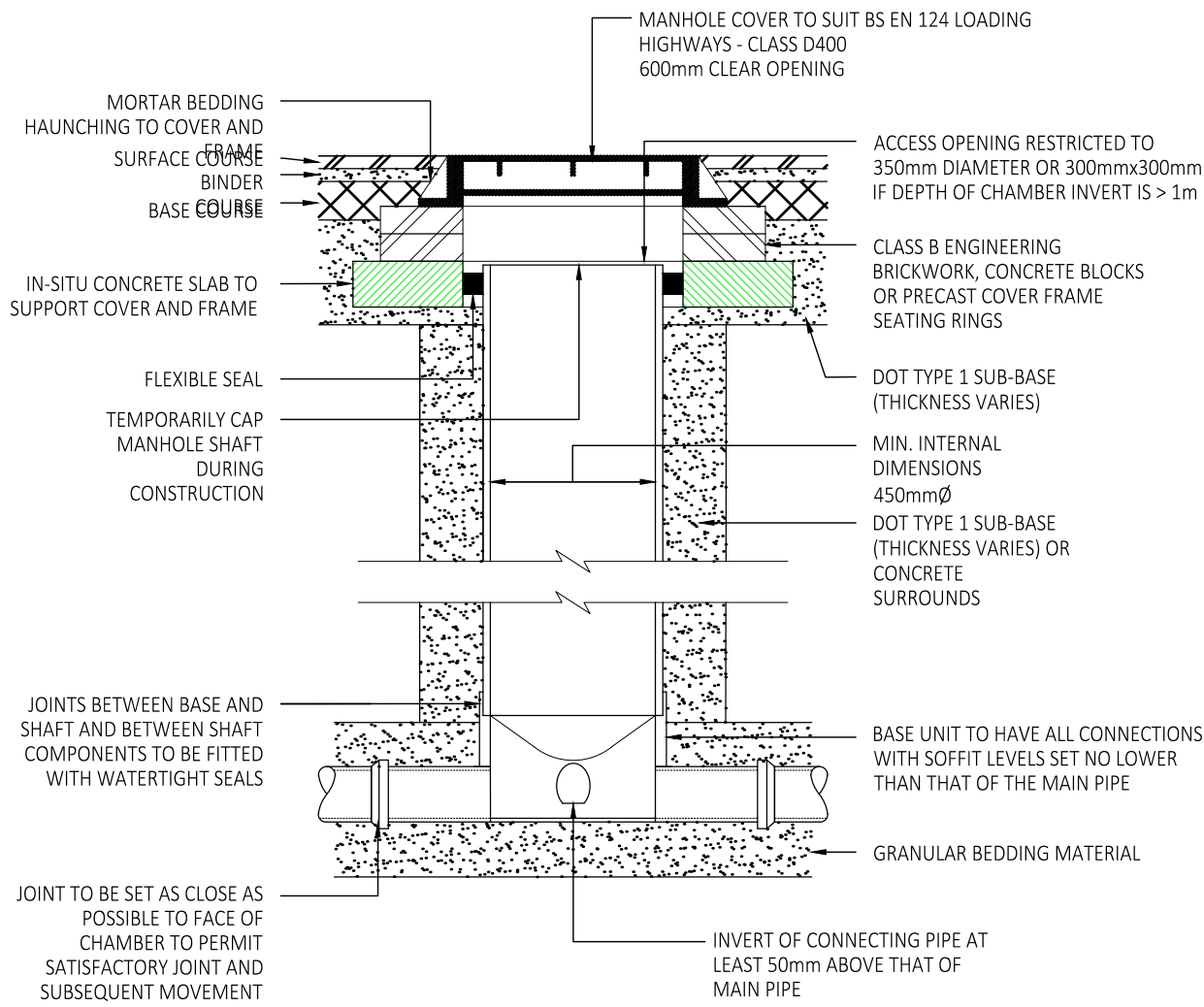
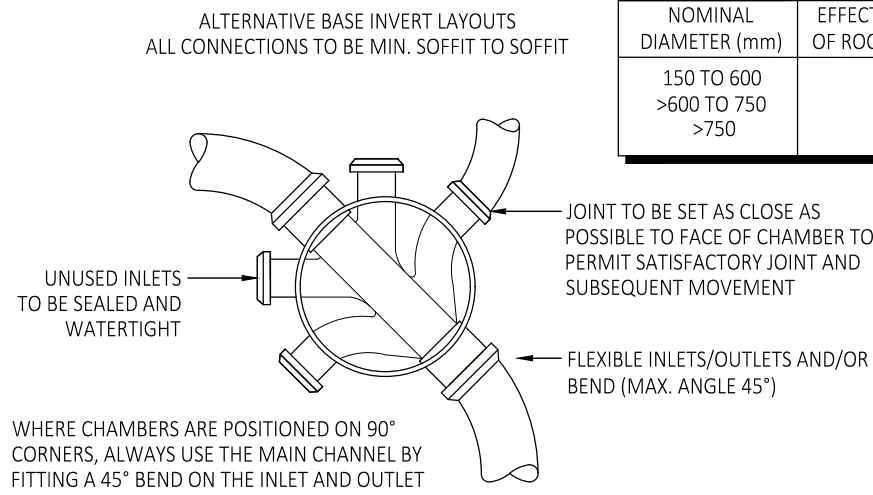
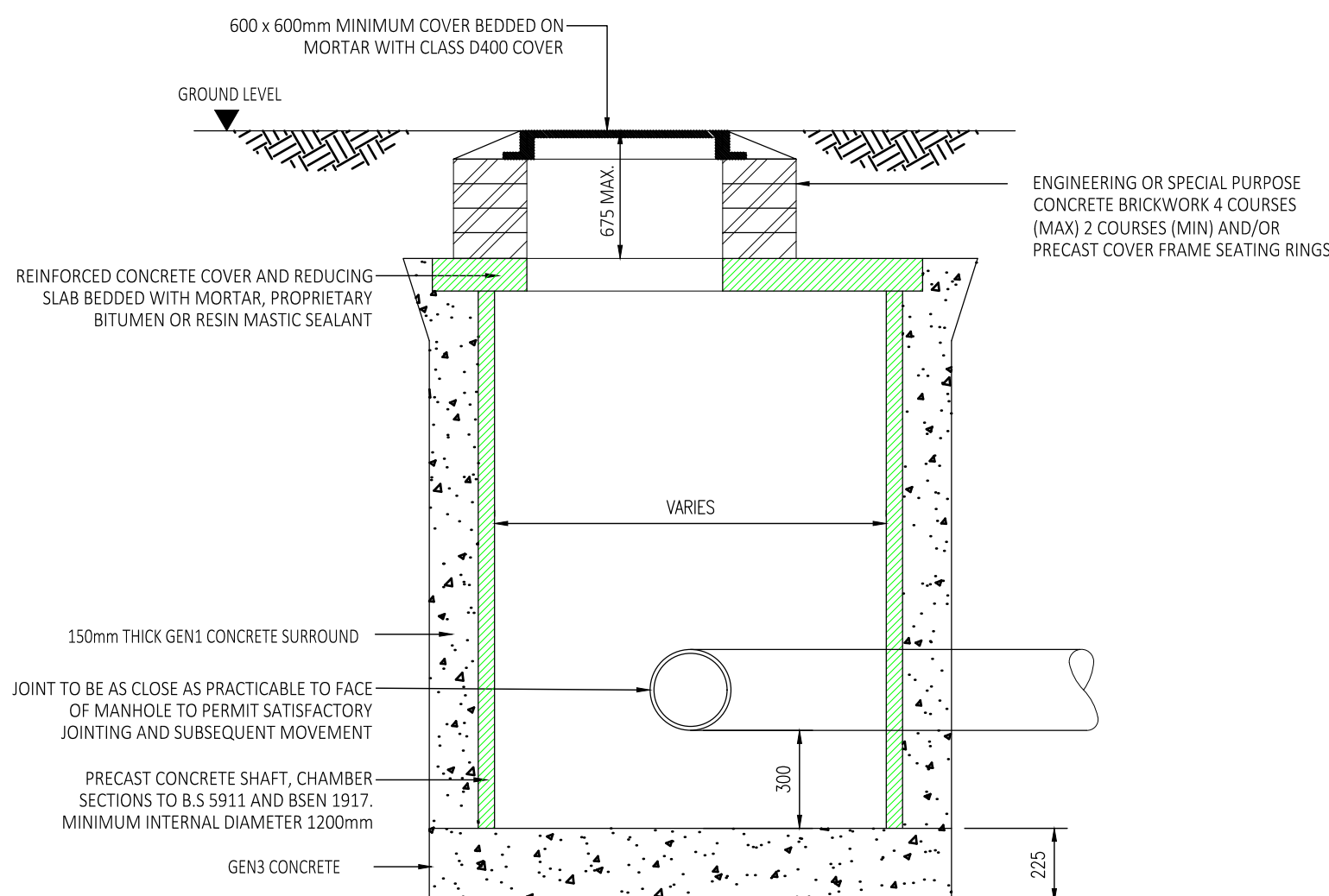


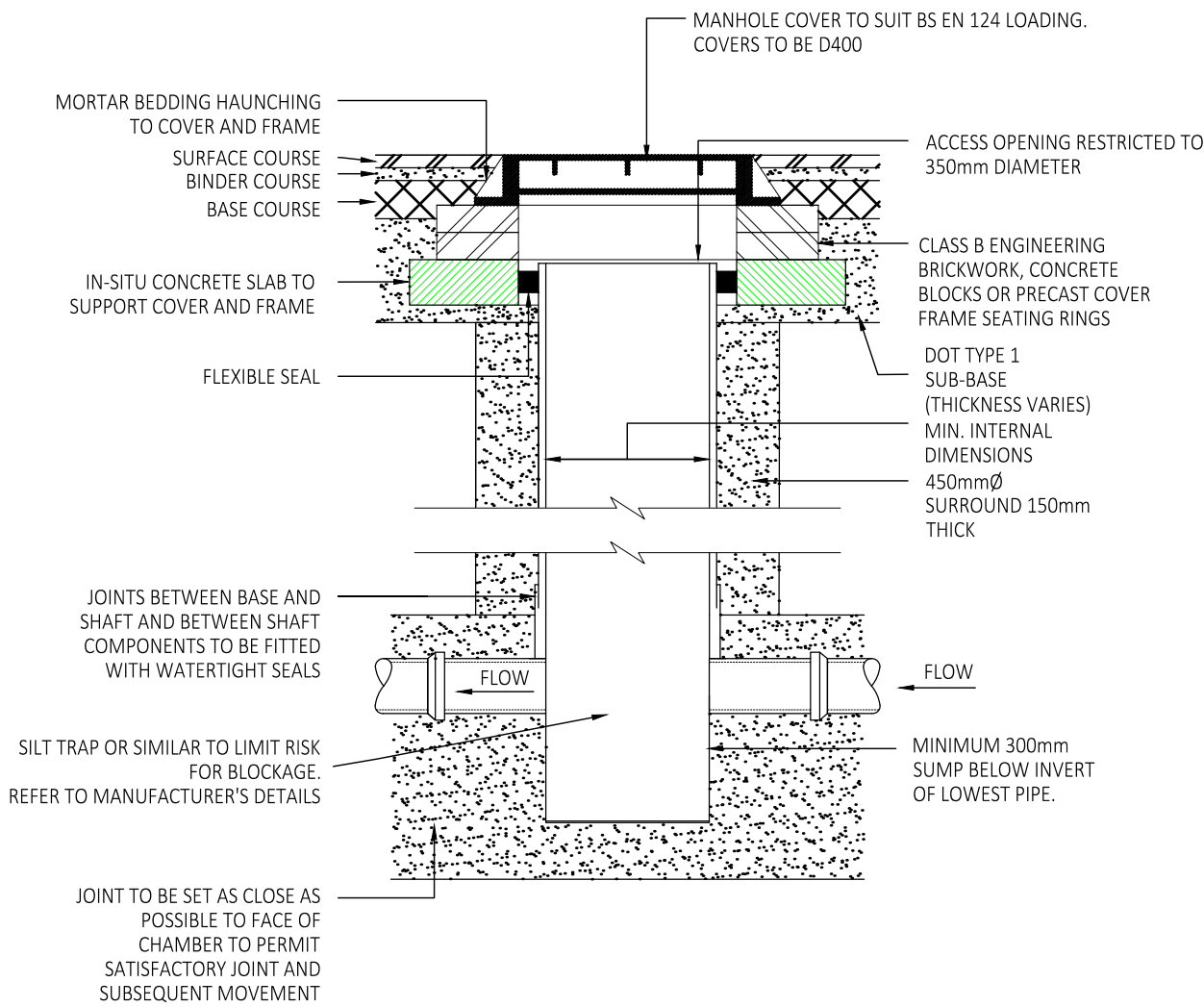
TABLE 1	
NOMINAL DIAMETER (mm)	EFFECTIVE LENGTH OF ROCKER PIPE (m)
150 TO 600	0.6
>600 TO 750	1.0
>750	1.2



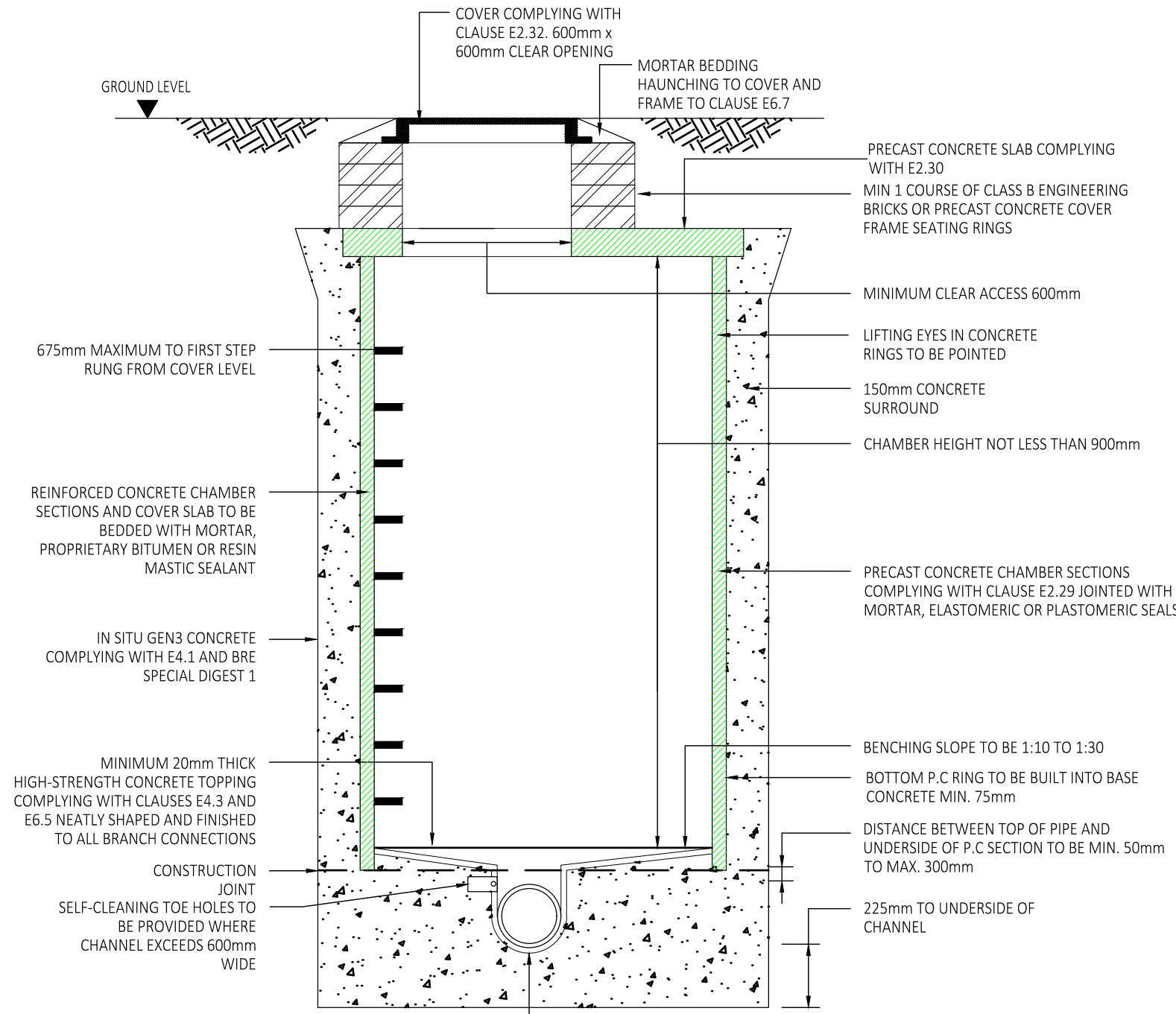
TYPE 3 ACCESS CHAMBER - FLEXIBLE MATERIAL.
(SUBJECT TO VEHICLE LOADING,
MAX. DEPTH 3m, NON-ENTRY)
(1:20)



CATCHPIT TO INVERT 0.9m TO 2.7m.
(1:20)



CATCHPIT INSPECTION CHAMBER.
(PREFORMED POLYPIPE RIDGISTORM 450-1500Ø CATCHPIT MANHOLE OR SIMILAR),
MAX. DEPTH 3m, NON-ENTRY)
(1:20)



TYPE 2 MANHOLE.
(DEPTH FROM COVER LEVEL TO
SOFFIT OF PIPE LESS THAN 3.0m)
(1:20)

TABLE 1	
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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REV	DESCRIPTION	DATE	BY	APP
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QED PROJECT No.
24-164

CLIENT
International Association of
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PROJECT
Saint Hill

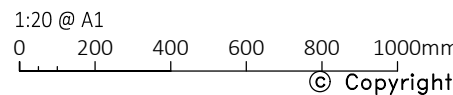
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TYPICAL DRAINAGE DETAILS
SHEET 1

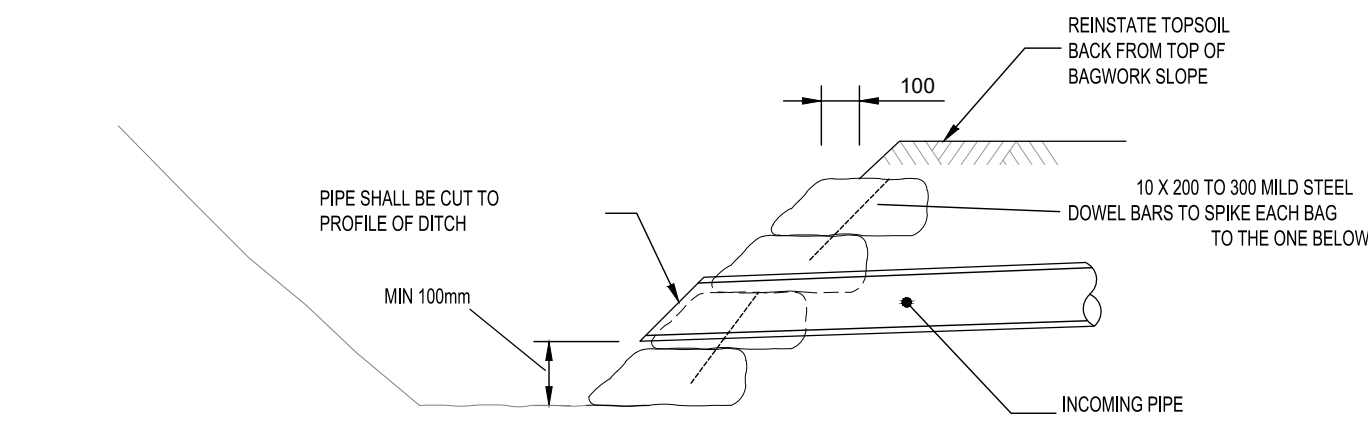
DRAWN IW	ENGINEER KB	CHECKED BY MN	APPROVED BY MN
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DATE APPROVED 10/12/24 SCALE @ A1 1:20

STATUS
S3 - Suitable for Review and Comment

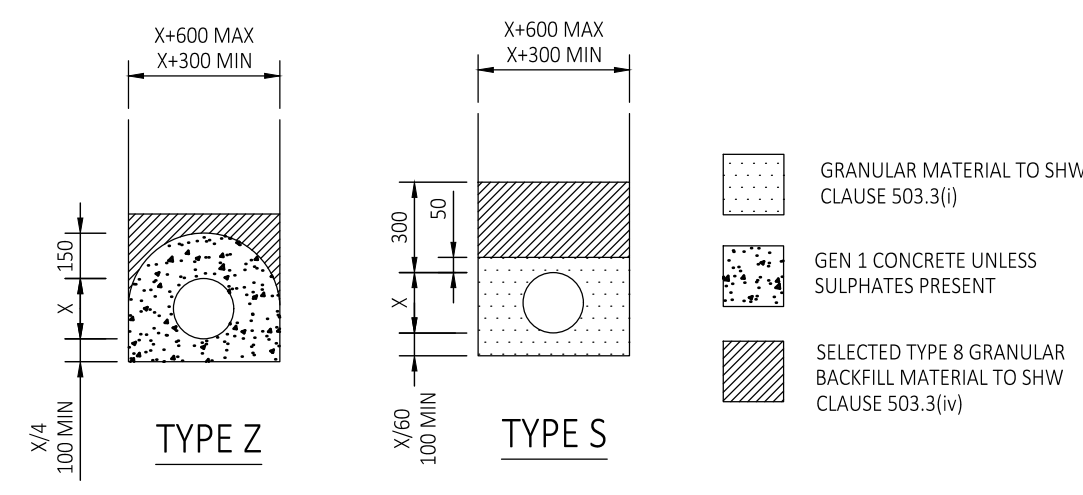
DRAWING No. 24-164-QED-XX-XX-DR-C-401	REVISION P01
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DITCH OUTFALL BAGWALL DETAIL.

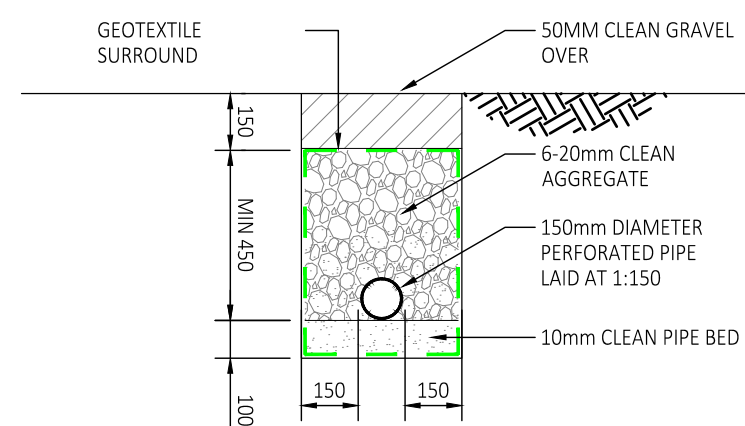
(1:20)



TRENCH AND BEDDING DETAILS

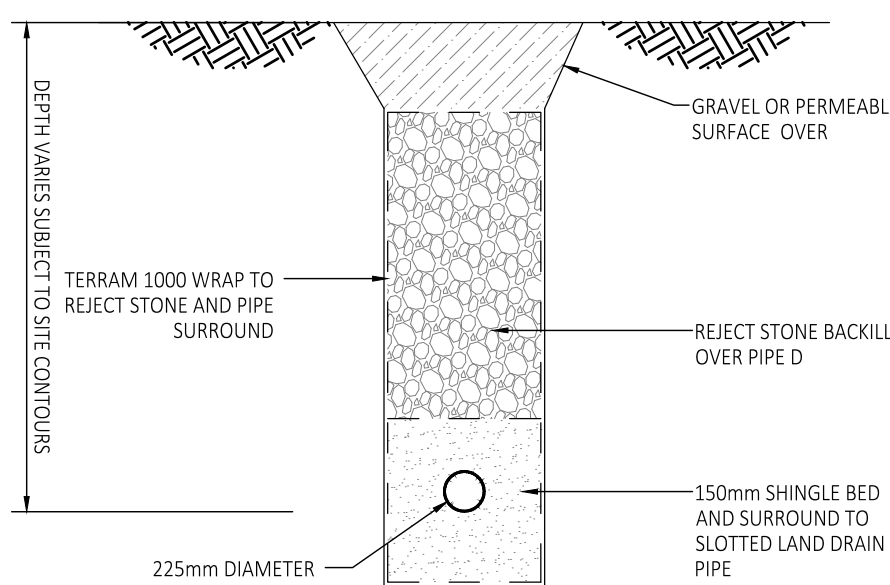
IN TRAFFICKED AREAS:
TYPE S WHERE COVER >900mm
TYPE Z WHERE COVER <900mm

IN NON-TRAFFICKED AREAS:
TYPE S WHERE COVER >600mm
TYPE Z WHERE COVER <600mm



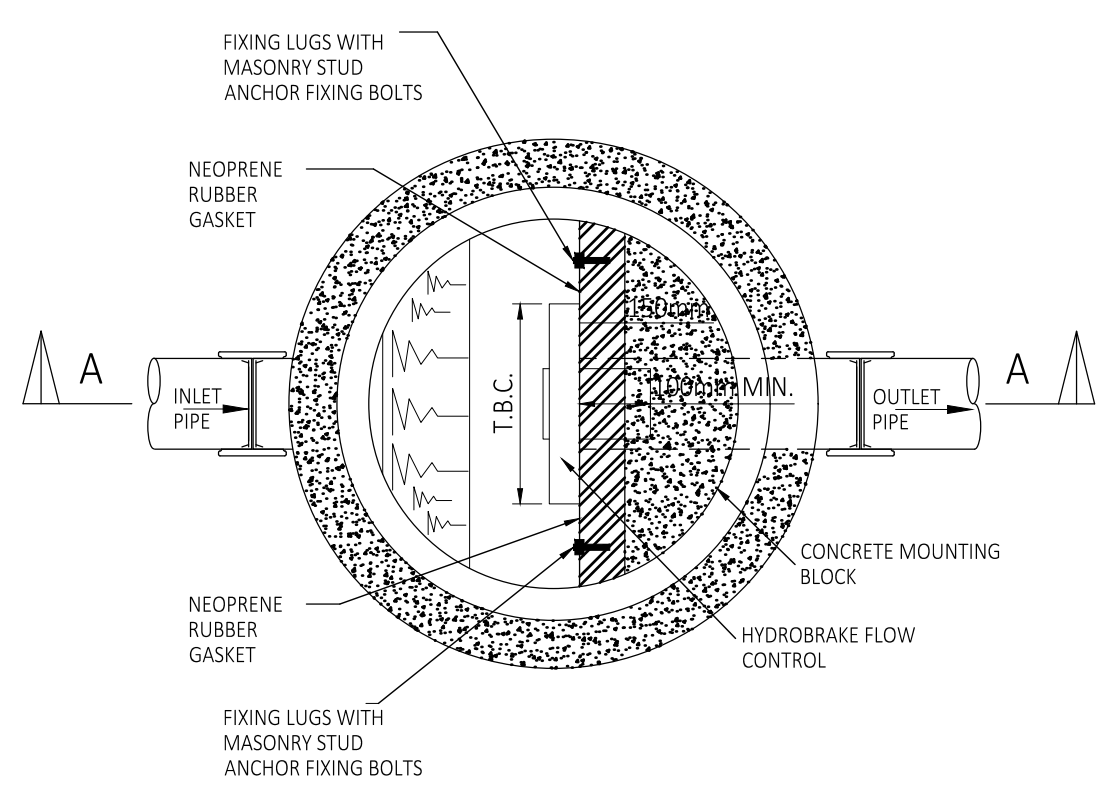
GRAVEL DRAIN

(1:20)



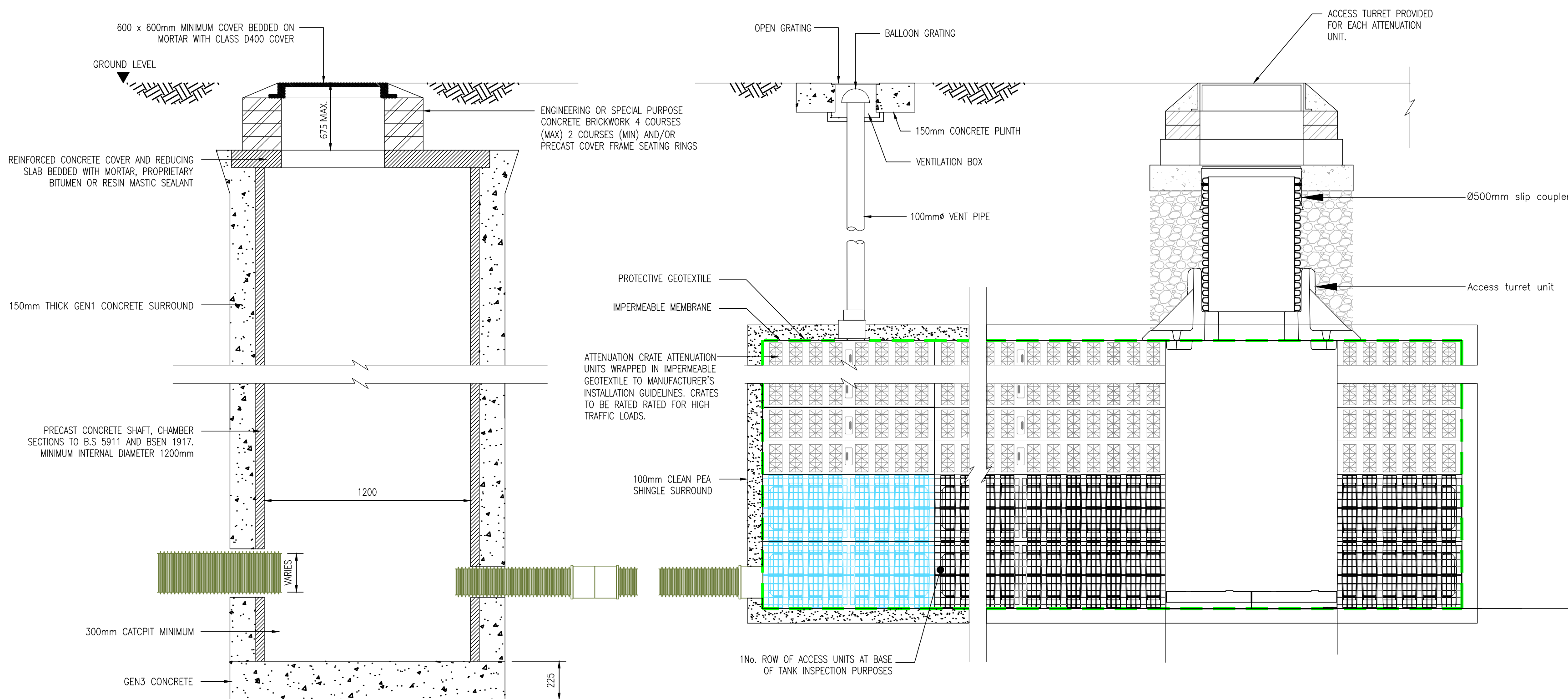
COLLECTOR/FILTER DRAIN

(1:20)



MANHOLE WITH HYDROBRAKE FLOW CONTROL PLAN

N.T.S.

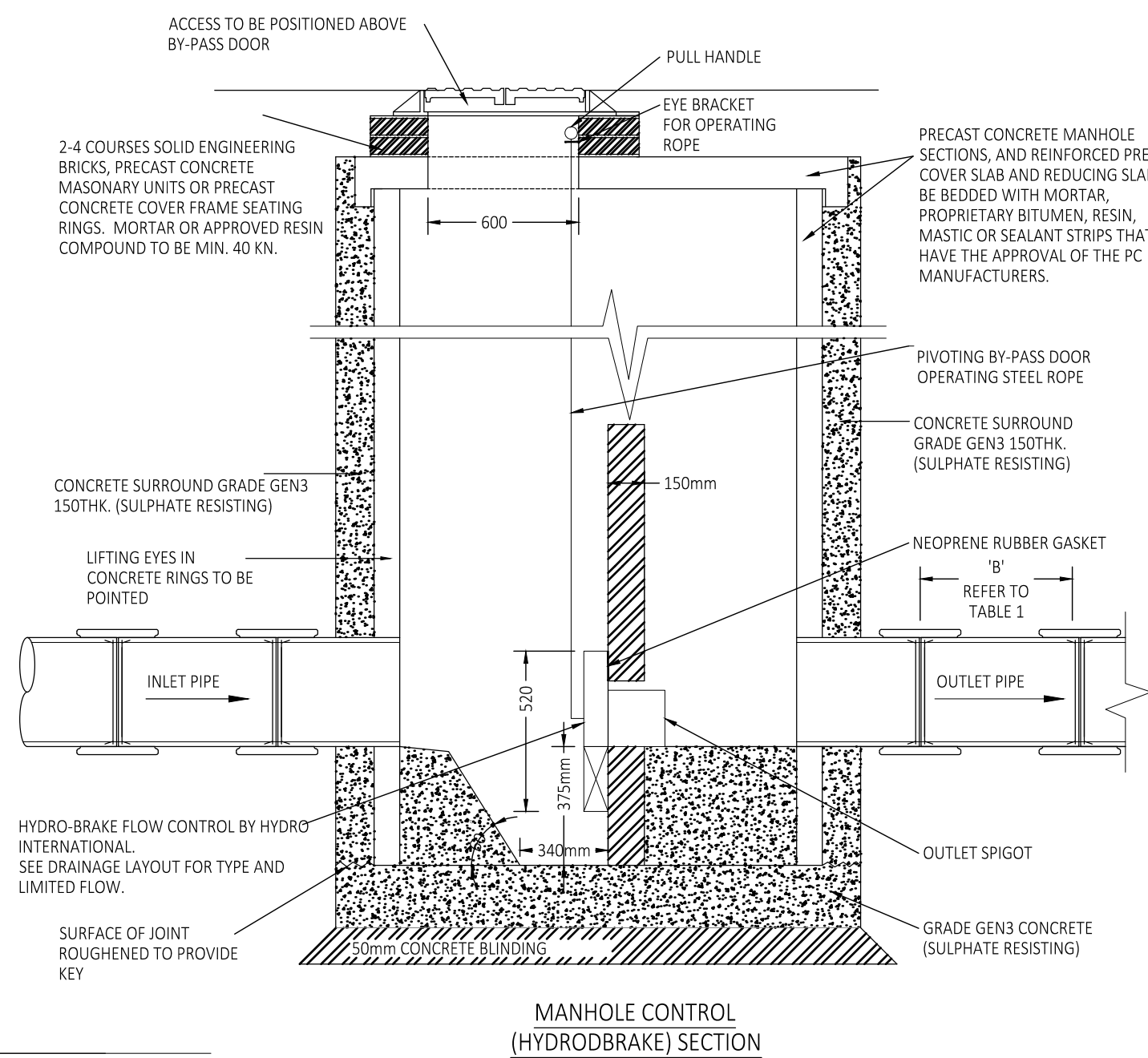


TYPE 2 MANHOLE.
(DEPTH FROM COVER LEVEL TO
SOFFIT OF PIPE LESS THAN 3.0m)

(1:20)

ATTENUATION TANK DETAIL.

(1:20)



MANHOLE CONTROL (HYDROBRAKE) SECTION

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TEL +44 (0) 1273 033680

QED PROJECT No.
24-164

CLIENT
International Association of
Scientists

PROJECT
Saint Hill

TITLE
TYPICAL DRAINAGE DETAILS
SHEET 2

DRAWN	ENGINEER	CHECKED BY	APPROVED BY
IW	KB	MN	MN

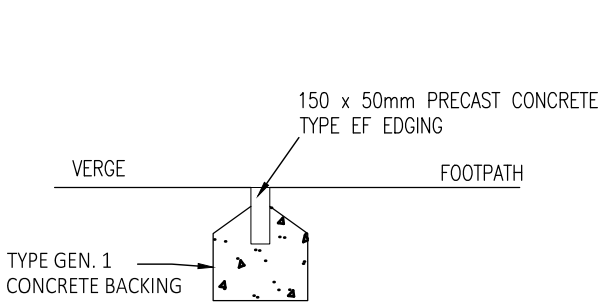
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STATUS
S3 - Suitable for Review and Comment

DRAWING No.
24-164-QED-XX-XX-DR-C-402

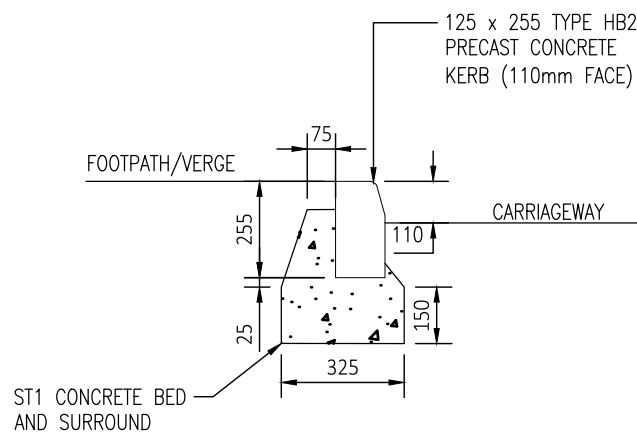
REVISION
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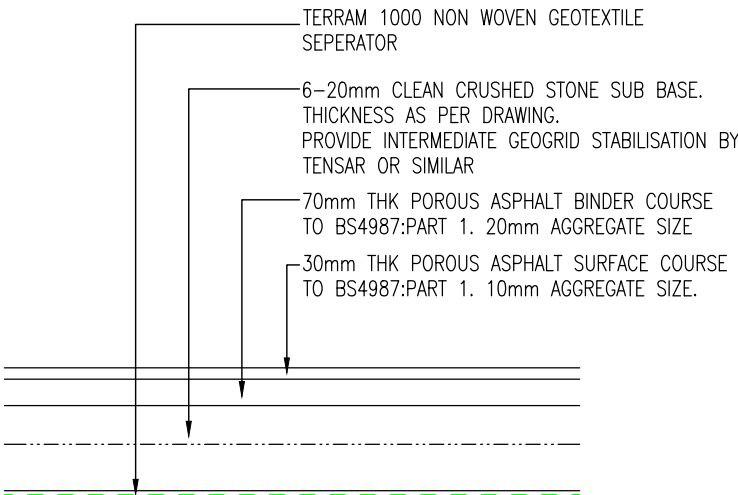
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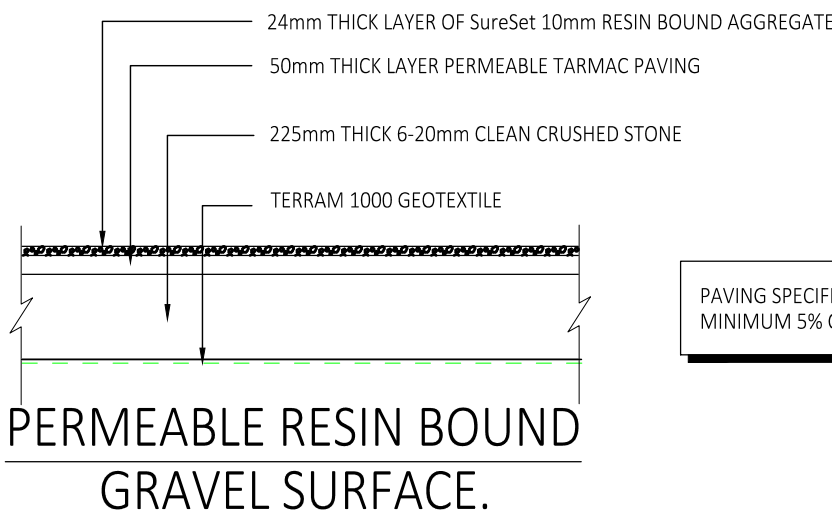
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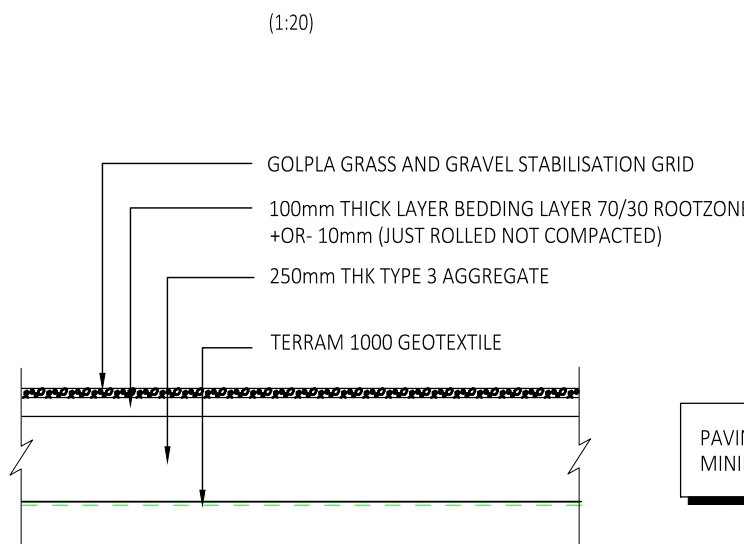


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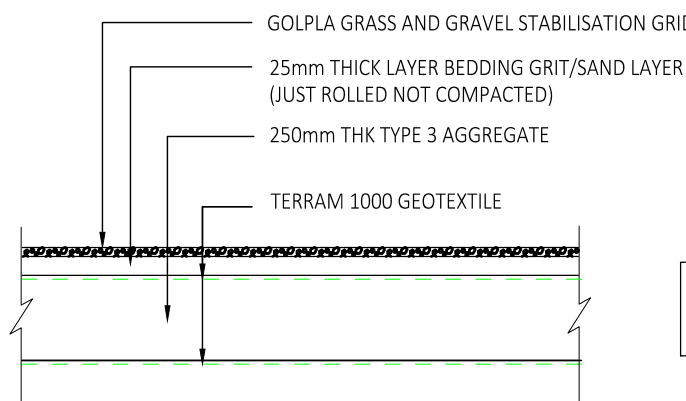
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MINIMUM 5% CBR VALUE



PAVING SPECIFICATIONS ASSUME
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STABILISED GRASS SURFACE.

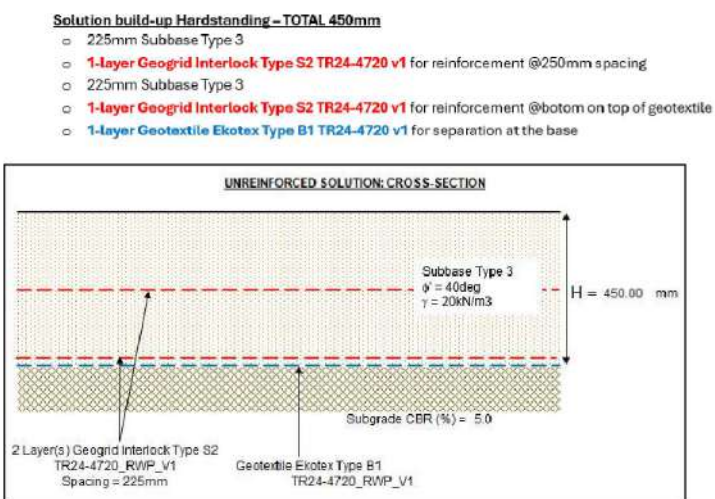
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PAVING SPECIFICATIONS ASSUME
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STABILISED GRAVEL SURFACE.

(1:20)



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

(1:20)

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IW	KB	MN	MN

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STATUS
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24-164-QED-XX-XX-DR-C-450	P01



APPENDIX III – CALCULATIONS

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

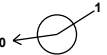
Link	US Node	Dia (mm)	Node Type	MH Type	DS Node	Dia (mm)	Node Type	MH Type
1.000	Formal CP 1		Junction		CP1 FC Eastern	1350	Manhole	SW_HB
1.001	CP1 FC Eastern	1350	Manhole	SW_HB	3	1350	Manhole	SW_Standard
1.002	3	1350	Manhole	SW_Standard	6	1350	Manhole	SW_Standard
2.000	Attenuation 1		Junction		CP2 FC 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

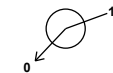
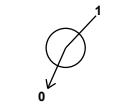
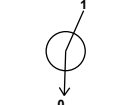
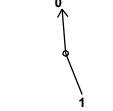
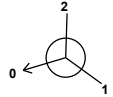
Link	Length (m)	Slope (1:X)	Dia (mm)	Link Type	US CL (m)	US IL (m)	US Depth (m)	DS CL (m)	DS IL (m)	DS Depth (m)
3.000	23.828	80.0	450	Circular	124.400	123.350	0.600	124.200	123.052	0.698
3.001	41.905	34.9	225	Circular	124.200	123.052	0.923	122.900	121.850	0.825
3.002	37.357	53.4	225	Circular	122.900	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					
				0	1.000	132.350	450
CP1 FC Eastern	133.500	1.188	1350		1	1.000	132.312
				0	1.001	132.312	450
3	133.700	1.527	1350		1	1.001	132.173
				0	1.002	132.173	450

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				

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.

Project details

Date	29/10/2025
Calculated by	mn
Reference	Saint Hill Marquee
Model version	2.2.1

Location

Site name	Saint Hill Marquee
Site location	Saint Hill Road, East Grinstead, West Sussex, RH19 4JY



Site easting (British National Grid)	538005
Site northing (British National Grid)	135863

Site details

Total site area (ha)	0.5
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Greenfield runoff

Method

Method	IH124
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IH124

	My value	Map value
SAAR (mm)	849	849
How should SPR be derived?	WRAP soil type	
WRAP soil type	4	4
SPR	0.47	
QBar (IH124) (l/s)	3	

Growth curve factors

	My value	Map value
Hydrological region	7	7
1 year growth factor	0.85	
2 year growth factor	0.88	
10 year growth factor	1.62	
30 year growth factor	2.3	
100 year growth factor	3.19	
200 year growth factor	3.74	

Results

Method	IH124	
Flow rate 1 year (l/s)	2.6	l/s
Flow rate 2 year (l/s)	2.7	l/s
Flow rate 10 years (l/s)	4.9	l/s
Flow rate 30 years (l/s)	7.0	l/s
Flow rate 100 years (l/s)	9.7	l/s
Flow rate 200 years (l/s)	11.3	l/s

Please note runoff estimation is subject to significant uncertainty. Results are therefore normally reported to only 1 decimal place. Where 2 decimal places are provided, this does not indicate accuracy to this level, it has been adopted to prevent 'zero' figures from being reported. Outputs less than 0.01 l/s are reported as 0.01 l/s.

Disclaimer

This report was produced using the Greenfield runoff rate estimation tool (2.2.1) developed by HR Wallingford and available at [uksuks.com](https://www.uksuks.com) (<https://www.uksuks.com/>). The use of this tool is subject to the UK SuDS terms and conditions and licence agreement, which can both be found at [uksuks.com/terms-conditions](https://www.uksuks.com/terms-conditions) (<https://www.uksuks.com/terms-conditions>). The outputs from this tool have been used to estimate 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, Centre for Ecology and Hydrology, Wallingford Hydrosolutions or any other organisation for the use of these data in the design or operational characteristics of any drainage scheme.

hrwallingford

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.

Project details

Date	29/10/2025
Calculated by	mn
Reference	Saint Hill Marquee
Model version	2.2.1

Location

Site name	Saint Hill Marquee
Site location	Saint Hill Road, East Grinstead, West Sussex, RH19 4JY



Site easting (British National Grid)	538005
Site northing (British National Grid)	135863

Site details

Total site area (ha)	1.739	ha
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Greenfield runoff

Method

Method	IH124
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IH124

SAAR (mm)	849	mm	☐	Map value	849
How should SPR be derived?	WRAP soil type				
WRAP soil type	4	☐		4	
SPR	0.47				
QBar (IH124) (l/s)	10.5	l/s			

Growth curve factors

Hydrological region	7	☐	Map value	7
1 year growth factor	0.85			
2 year growth factor	0.88			
10 year growth factor	1.62			
30 year growth factor	2.3			
100 year growth factor	3.19			
200 year growth factor	3.74			

Results

Method	IH124	
Flow rate 1 year (l/s)	8.9	l/s
Flow rate 2 year (l/s)	9.3	l/s
Flow rate 10 years (l/s)	17.0	l/s
Flow rate 30 years (l/s)	24.2	l/s
Flow rate 100 years (l/s)	33.6	l/s
Flow rate 200 years (l/s)	39.4	l/s

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