

RIDGE

PROPERTY & CONSTRUCTION CONSULTANTS

**DRAINAGE IMPACT ASSESSMENT & DRAINAGE
STRATEGY, HARTS FARM WAY, HAVANT, PO9 1FA**

06 March 2018

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EXECUTIVE SUMMARY

This report explores the drainage impact of the proposed development and outlines the surface water management strategy that can be implemented.

The surface water drainage network has been designed to accommodate the surface water runoff generated by a critical storm from a 1 in 100 year return period. Allowance for climate change has been added which equates to an additional 20%. The foul water will be discharged to the existing foul sewer network.

The Local Authority SUDS Officer Organization (LASOO) through its publication “Non-Statutory Technical Standards for Sustainable Drainage” has set out guidance together with a number of technical standards that can be used to “judge” whether a drainage scheme submitted through the planning process complies with current policy (both national and local).

The table below sets out how this development proposals drainage strategy complies with this guidance and meets the technical standards expected.

Technical Standard	LASOO Strategy Comment	Report Comment
S1	Where the drainage system discharges to a surface water body that can accommodate uncontrolled surface water discharges without any impact on flood risk from that surface water body (e.g. the sea or a large estuary) the peak flow control Standards (S2 and S3) and volume control Standards (S4 to S5) need not apply.	Discharge is proposed to an EA main river that is located approximately 350 m from the tidal estuary. A 50% reduction on the existing discharge rates is offered by the proposed strategy. Refer to Chapter 4.3.
S2	For greenfield developments, the peak runoff rate from the development to any highway drain, sewer or surface water body for the 1 in 1-year rainfall event and the 1 in 100-year rainfall event should never exceed the peak greenfield runoff rate for the same event.	N/A (Brownfield site)
S3	For developments which were previously developed, the peak runoff rate from the development to any drain, sewer or surface water body for the 1 in 1-year rainfall event and the 1 in 100-year rainfall event should be as close as reasonably practicable to the greenfield runoff rate from the development for the same rainfall event, but should never exceed the rate of discharge from the development prior to redevelopment for that event	A 50% reduction on the existing discharge rates is offered by the proposed strategy. Refer to Chapter 4.3.
S4	Where reasonably practicable, for greenfield development, the runoff volume from the development to any highway drain, sewer or surface water body in the 1 in 100 years, 6-hour rainfall event should never exceed the greenfield runoff volume for the same event.	N/A (Brownfield site)
S5	Where reasonably practicable, for developments which have been previously developed, the runoff volume from the development to any highway drain, sewer or surface water body in the 1 in 100 years, 6-hour rainfall event must be constrained to a value as close as is reasonably practicable to the greenfield runoff volume for the same event, but should never exceed the runoff volume from the development site prior to redevelopment for that event.	Storage volume associated with a 50% reduction on the existing discharge rates is provided by the proposed strategy. Refer to Chapter 4.3.
S6	Where it is not reasonably practicable to constrain the volume of runoff to any drain, sewer or surface water body in accordance with S4 or S5 above, the runoff volume must be discharged at a rate that does not adversely affect flood risk.	N/A

Technical Standard	LASOO Strategy Comment	Report Comment
S7	The drainage system must be designed so that, unless an area is designated to hold and/or convey water as part of the design, flooding does not occur on any part of the site for a 1 in 30-year rainfall event.	Below ground storage volume for the 1 in 30 year annual probability event is provided for the whole site. Above ground storage is incorporated in the drainage design of Phase 2. Drainage design ensures no flooding to any building or to the pump station. Refer to Chapter 4.3.
S8	The drainage system must be designed so that, unless an area is designated to hold and/or convey water as part of the design, flooding does not occur during a 1 in 100-year rainfall event in any part of: a building (including a basement) or in any utility plant susceptible to water (e.g. pumping station or electricity substation within the development).	Drainage design ensures no flooding to any building or to the pump station. Refer to Chapter 4.3.
S9	The design of the site must ensure that so far as is reasonably practicable, flows resulting from rainfall in excess of a 1 in 100-year rainfall event are managed in exceedance routes that minimise the risks to people and property.	Flood flow route for events in excess of the 1 in 100 year annual probability event to be provided along the south boundary of the site. Levels design to promote such flood flow route and be developed at detailed design stage. Refer to Chapter 4.3.
S10	Components must be designed to ensure structural integrity of the drainage system and any adjacent structures or infrastructure under anticipated loading conditions over the design life of the development taking into account the requirement for reasonable levels of maintenance.	General maintenance requirements are highlighted in this report. Refer to Chapter 4.5.
S11	The materials, including products, components, fittings or naturally occurring materials, which are specified by the designer must be of a suitable nature and quality for their intended use.	To be confirmed at detailed design stage.
S12	Pumping should only be used to facilitate drainage for those parts of the site where it is not reasonably practicable to drain water by gravity	Pumping is required due to the shallow depth of the adjacent receiving watercourse. The pump stations are located as close as reasonably practicable to the outfall. Refer to Chapter 4.3.

1. INTRODUCTION

1.1.1. This Drainage Impact Assessment and Drainage Strategy report has been prepared by Ridge on behalf of Salmon Property in support of a planning submission for a commercial development on a parcel of land located in Harts Farm Way, Havant, PO9 1FA.

1.1.2. This report has been prepared to provide the following information on drainage to satisfy planning requirements:

- Assessing the current infrastructure;
- Assessing the capacity and suitability of existing drainage to accommodate additional flows;
- Preliminary assessment of storm water attenuation (SuDS) requirements and volumes;
- Determining the foul and surface water runoff generated by the proposed development;
- Preliminary concept drainage design.

2. SITE INFORMATION

2.1 LOCATION

2.1.1. The site is located in Havant, approximately 120 m south of A27. Site Reference information is included in Table 1.

Table 1 Site Reference

ITEM	DESCRIPTION
Site Name	Plot 4, Hermitage Park, Harts Farm Way, Havant, PO9 1FA
County & Local Council Area	Hampshire County Council (Lead Local Flood Authority) Havant Borough Council (Planning Authority) Southern Water (Water Undertaker)
Survey Grid Ref	470653 105851

2.1.2. The site is delineated by Hermitage Park to the north and east, Harts Farm Way to the south, and an unnamed access road to the west (see Figure 1). The site can be accessed both from Harts Farm Way and Hermitage Park.

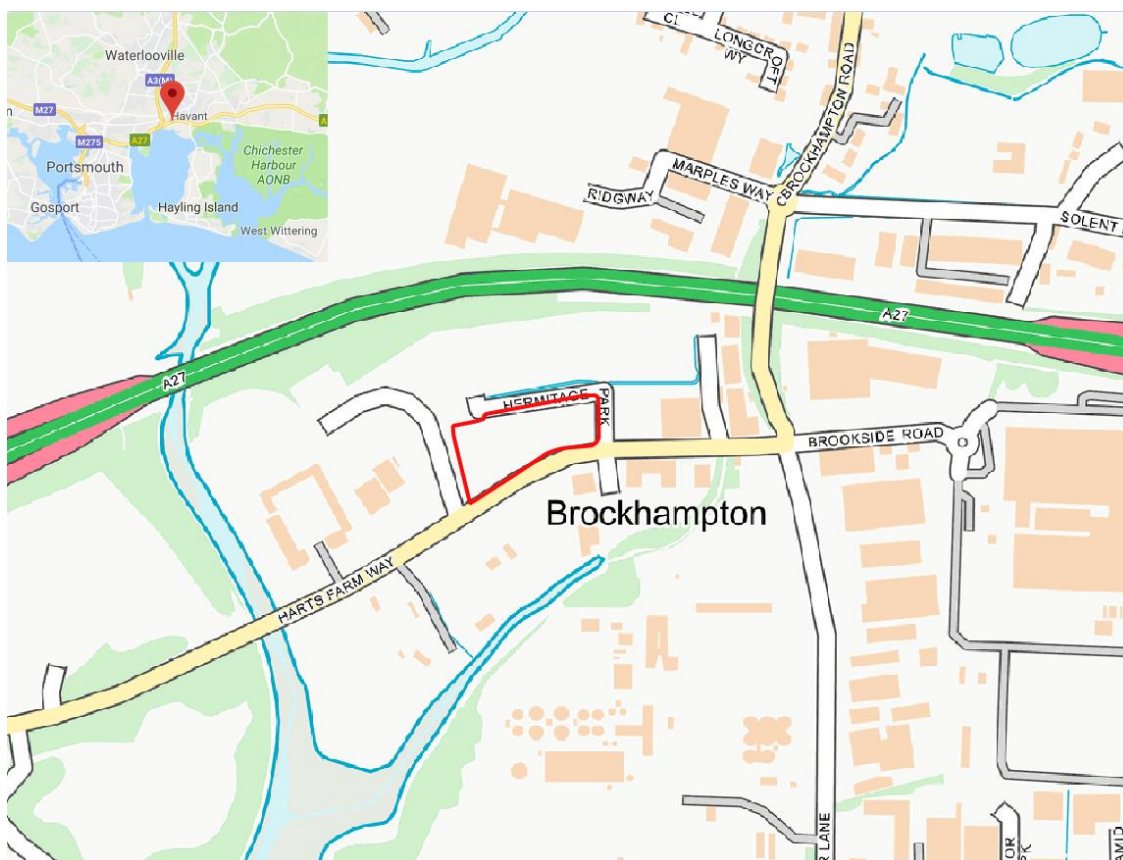


Figure 1 Site Location Plan

2.2 TOPOGRAPHY

- 2.2.1. The topographic Survey¹ indicates that levels generally fall from east to a westerly direction. The highest elevation recorded along the east boundary is 4.00m AOD and along the west boundary is 3.56m AOD. The approximate slope across the site is 1 in 320.
- 2.2.2. The topographic survey indicates the presence of two earth bunds within the southwest and southeast extent of the site. This is understood to be heaped demolition rubble.

2.3 GEOLOGY

- 2.3.1. A ground condition assessment² has been carried out by Ridge in 2017. The report includes a review on all available historical information regarding the site as well as presents the results of recent site investigation data. The description of underlying geology presented in this chapter is based on the information provided by the above assessment.
- 2.3.2. BGS mapping extract indicate the site to be underlain by superficial deposits of Alluvium over River Terrace Deposits overlying bedrock geology of the Lewes Nodular Chalk Formation, Seaford Chalk Formation, Newhaven Chalk Formation, Culver Chalk Formation (undifferentiated).
- 2.3.3. As part of the various ground investigations³ groundwater levels were recorded within the River Terrace Deposits between 1.35m and 2.07mbgl. Given the shallow depth of the groundwater, it is considered that the site is generally unsuitable for the use of soakaways.

2.4 HYDROLOGY

- 2.4.1. The closest watercourses to the site is an unnamed tributary of Brockhampton Stream which flows along the north and west boundary of the site discharging to the stream 160 m south of the site (refer to Figure 2). Both watercourses are classified as Environment Agency (EA) main rivers, therefore any proposed discharge made to the stream and works carried out within 8 m of the top of the river bank requires written consent⁴.

¹ Designbase – Drawing no. DB3192-TOPO – February 2018

² Ridge and Partners LLP – Ground Condition Assessment – October 2017

³ Refer to Table 9.1 of the ground condition assessment report (Ridge 2017)

⁴ In accordance with Flood Risk Activities: Environmental permits [available at <https://www.gov.uk/guidance/flood-risk-activities-environmental-permits>] [accessed 26/02/2018]

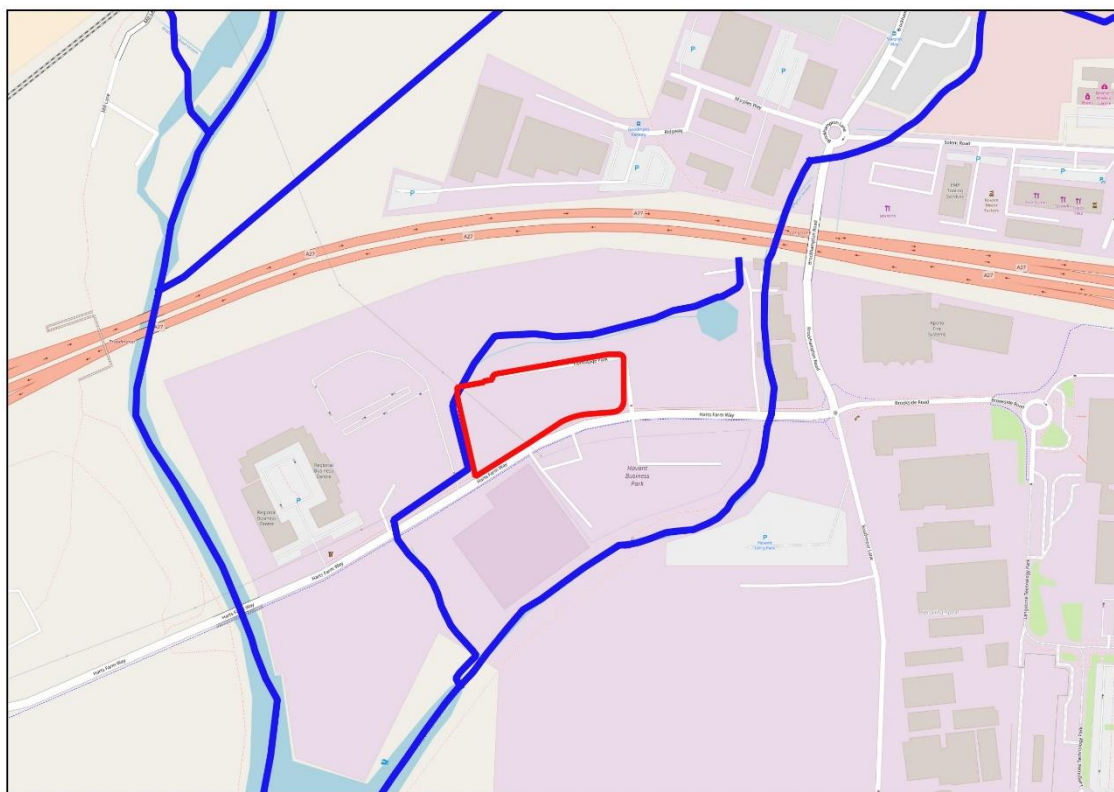


Figure 2 Hydrological Features

2.5 HYDROGEOLOGY

2.5.1. Historic site investigation report⁵ indicate the underlying superficial deposit is classified as a Secondary A aquifer (permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers) and bedrock as a Principal aquifer (layers of rock or drift deposits that have high intergranular and/or fracture permeability providing a high level of water storage).

2.5.2. EA mapping indicate that the site does not lie within a groundwater source protection zone.

2.6 EXSITING SITE

2.6.1. It is understood that the site has been previously developed, however the existing structures were demolished. Currently, the majority of the area is covered by bare ground where the demolished buildings and surfacing had been removed with macadam and concrete associated with access roads and car parking still locally present. The site is overgrown with a number of mature trees to southern and western perimeter (see Figure 3).

⁵ Structural Soils Ltd – Final Report on Site Investigation – August 2013



Figure 3 Existing Site Layout

2.7 PROPOSED DEVELOPMENT

2.7.1. It is proposed to redevelop the site into commercial/industrial uses (50% Class B2 and 50% Class B8) with associated car parking. The proposed site layout is provided in Appendix A, at the rear of this report.

3. SOURCES OF FLOOD RISK

3.1 INTRODUCTION

3.1.1. This section summarises the potential flood risk to the site from various sources. A summary of the flood risk from these sources is given in Table 2.

Table 2 Flood Risk Summary Table

POTENTIAL SOURCE OF FLOODING	OVERALL RISK
Fluvial and Tidal	Low
Surface Water	Low
Groundwater	Medium
Reservoir	Very Low

3.2 FLUVIAL & TIDAL FLOODING

3.2.1. The site is shown by the EA's Flood Zone mapping to be wholly within Flood Zone 1 (Low Risk) (see Figure 4). Flood Zone 1 comprises land assessed as having a 0.1%, or less, Annual Exceedance Probability (AEP) of fluvial or tidal flooding in any given year, equivalent to the 1 in 1,000 year, or more, return period event. As such, the site is concluded to be at a low risk of fluvial & tidal flooding.

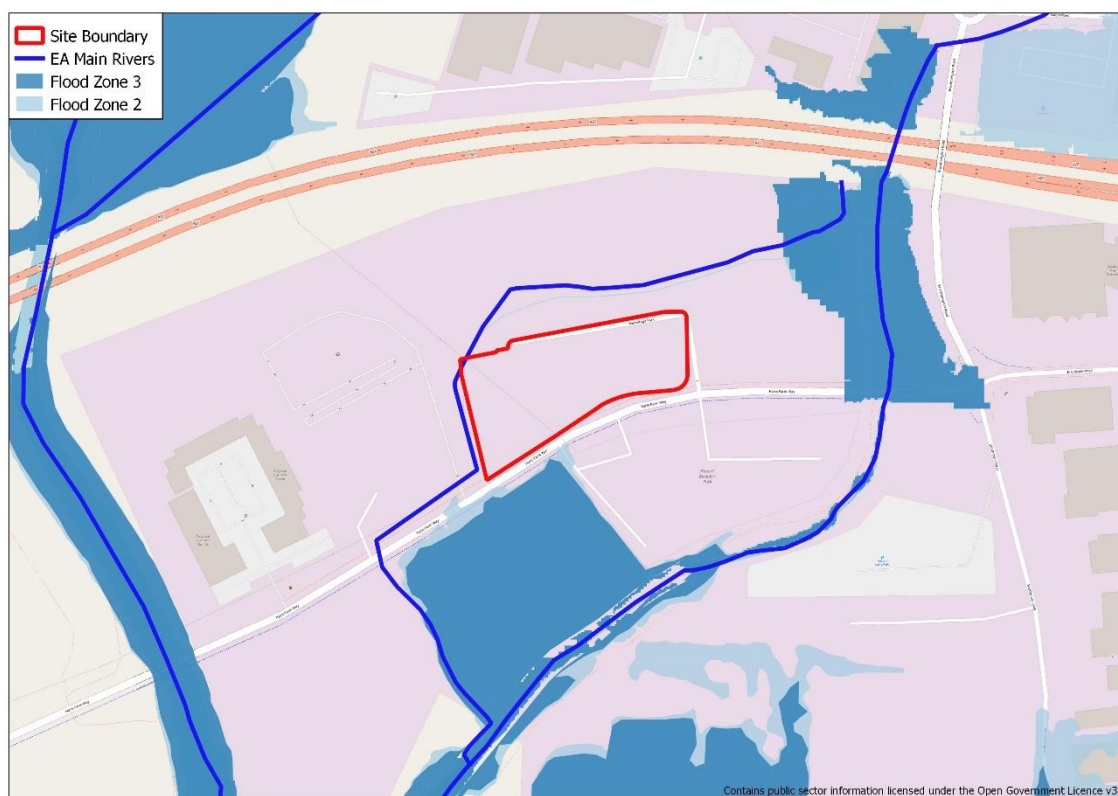


Figure 4 Risk of Flooding from Rivers and Sea

3.3 SURFACE WATER FLOODING

- 3.3.1. Surface water flooding occurs as the result of an inability of intense rainfall, or overland flows, to infiltrate to ground. This often happens when the maximum soil infiltration rate or storage capacity is reached. Such flows either drain into existing land drainage features or follow the general topography which can concentrate flows and lead to localised ponding/flooding.
- 3.3.2. The EA Surface Water Flooding Map (Figure 5) shows that the majority of the site is at a very low risk of flooding from this source (less than 1 in 1,000 (0.1%) annual probability of flooding). There are however, small areas of the site that are shown to be at a low risk (between 1 in 1,000 (0.1%) and 1 in 100 (1%) annual probability), that are assumed to be a result of localised depression areas post demolition.

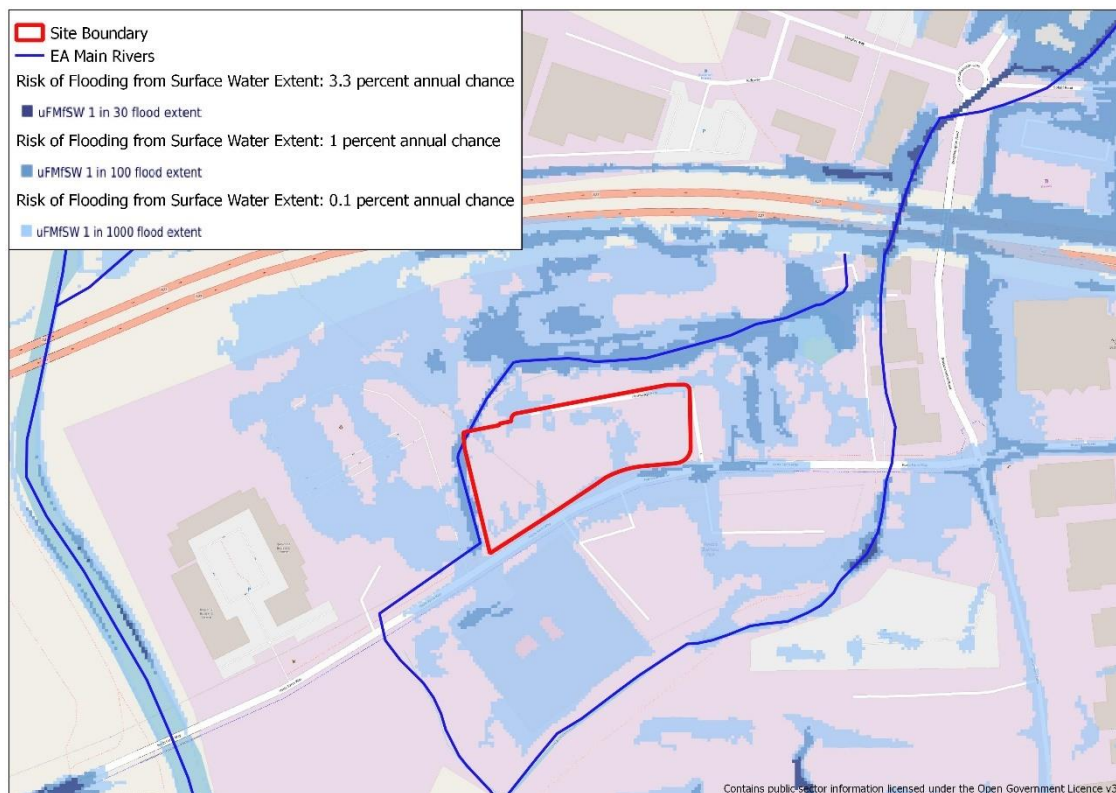


Figure 5 Risk of Flooding from Surface Water

- 3.3.3. Adequate levels and drainage design will eliminate the residual risk of flooding from surface water to the site.

3.4 GROUNDWATER FLOODING

- 3.4.1. PUSH mapping indicates high bedrock permeability underneath the site. The Strategic Flood Risk Assessment⁶ (SFRA) update states that Havant has historically been susceptible to groundwater

⁶ PUSH – Strategic Flood Risk Assessment – Havant Borough Council – 2016 update

flooding. The key risks to new developments in the Havant Borough come from increases in flows in the Lavant Stream as a result of high groundwater levels emerging in East Hampshire District.

3.4.2. The site is located outside the Lavant catchment which was identified to have high risk of groundwater flooding. Considering the historic information and underlying ground conditions, the overall risk to the site is considered to be medium.

3.5 FLOODING FROM RESERVOIRS

3.5.1. Based on EA indicative mapping the site is not at risk of flooding from reservoirs.

4. SURFACE WATER DRAINAGE

4.1 EXISTING SURFACE WATER DRAINAGE

4.1.1. Google earth image from 1999 indicate the presence of existing buildings and associated access road, car park and landscaping within the site boundary. The EA main river mapping shows that the unnamed tributary flows from east to a south-west direction to the north of the site, crossing it on the north-west corner and flowing southwards along the west boundary (refer to Figure 6).



Figure 6 Site Layout in 1999

4.1.2. It is understood that the main river has been diverted along Hermitage Park to the north of the site in 2015⁷. Runoff from this road is collected via gullies and discharged to the unnamed tributary via 3No. outfalls.

4.1.3. The utility survey⁸ indicates that the car park within the western extent of the site is drained via a network of linear drainage channels and gullies. Runoff from the remaining hard paved areas are collected via gullies. There are two existing connection points to the highway drainage in Harts Farm Way. Final discharge point is unknown, however considering the proximity to an open watercourse it is assumed that the highway drain outfalls to the unnamed tributary. For the layout of the existing drainage features refer to Figure 7.

⁷ Upton McGougan – Drawing no. 034.0036-2100 – As built issue 21/07/2015

⁸ DesignBase – Topo Survey of Harts Farm Way, Havant – Drawing no. 03 DB3192-TOPO_USP-520842485 – February 2018



Figure 7 Existing Drainage Features

4.1.4. The site area is approximately 0.9 ha. Based on the historic layout 94% of the site is impermeable⁹. Runoff from the site is assumed to be unrestricted. Existing peak discharge rates were calculated based on the Wallingford Procedure and Modified Rational Method and presented in Table 3. Detailed calculations are provided in Appendix B, at the rear of this report.

Table 3 Existing Runoff Calculations

RETURN PERIOD	1 IN 1	1 IN 2	1 IN 10	1 IN 30	1 IN 100
Peak runoff rate (l/s)	66	86	132	162	209

4.2 DESIGN CALCULATIONS

4.2.1. The proposed site is considered to be 100% impermeable. The design life of industrial/commercial developments is considered to be 60 years. Peak post-development runoff rates were calculated based on the Wallingford Procedure and Modified Rational Method (refer to Table 4). In accordance with the EA climate change allowances guidance¹⁰ the anticipated change in peak rainfall intensity by 2078 is between 20% (central) and 40% (upper end). Considering that the site is located in close proximity to the final discharge point to the sea, the central climate change allowance was applied throughout the calculations. Detailed calculations are provided in Appendix C, at the rear of this report.

⁹ Total site area is 8992 m². Extent of existing landscaping is 845 m². Calculations were performed in QGIS 2.18.15.

¹⁰ Environment Agency – Guidance Flood risk assessments: climate change allowances – February 2016 (last updated February 2017)

Table 4 Post-Development Runoff Calculations

RETURN PERIOD	1 IN 1	1 IN 2	1 IN 10	1 IN 30	1 IN 100
Peak runoff rate including 20% allowance for climate change (l/s)	88	114	177	217	282

4.2.2. The Surface Water Management Plan (SWMP)¹¹ states that surface water arising from a developed site should, as far as practicable, be managed in a sustainable manner in order to replicate greenfield runoff rates arising from the site prior to the proposed development. Measures to manage surface water runoff from proposed developments should also reduce the flood risk to the site itself and downstream receptors, whilst taking into account climate change.

4.3 PROPOSED SURFACE WATER DRAINAGE STRATEGY

4.3.1. The topographic survey indicates that Harts Farm Way is drained via gullies (refer to Figure 7). Runoff from the road is assumed to discharge to Brockhampton Stream, south of the development site, however this is not confirmed by the survey.

4.3.2. The site is understood to drain unrestricted and discharged to the highway drain in Harts Farm Way. It is proposed to provide 50% betterment on the 1 in 1 annual probability peak flow rates, therefore the runoff from the site is to be restricted to 33 l/s. This measure ensures that flood risk is reduced to downstream receptors.

4.3.3. Due to the lack of infiltration potential on the site, as a result of high groundwater table, runoff generated from the site will need to be stored within the site boundary. The storage volume required providing 50% betterment on the existing flow rates equates to 385 m³ for up to the 1 in 100 year annual probability events including 20% allowance for climate change¹². The storage volume estimates were calculated in MicroDrainage Source Control Modul. The output is provided in Appendix D, at the rear of this report.

4.3.4. The development is planned to be constructed in two phases and subsequently the buildings will be occupied by separate tenants who will be responsible for the maintenance of the drainage network. Due to the phased construction, the required storage volumes are provided for Phase 1 and 2 separately on an area proportion bases.

Phase 1

4.3.5. Phase 1 covers approximately 63% of the total site area. Runoff from all hard standings are to be collected via private drains.

¹¹ Hampshire County Council – Surface Water Management Plan Strategic Assessment and Background Information – August 2012

¹² Estimated volume only to allow for a discharge rate of 33 l/s for all storm events – final volume to be established at detailed design stage.

4.3.6. It is proposed to provide below ground attenuation for up to the 1 in 100 year annual probability events including 20% allowance for climate change underneath the car parking. The required storage volume equates to 243 m³. No above ground storage to be provided within Phase 1.

4.3.7. Discharge is to be made via gravity to the highway drain via the existing connection.

Phase 2

4.3.8. Phase 2 covers approximately 37% of the total site area. Runoff from all hard standings are to be collected via private drains.

4.3.9. It is proposed to provide below ground attenuation for up to the 1 in 30 year annual probability events including 20% allowance for climate change underneath the car parking area located in the north-west of the site. The required storage volume equates to 99 m³.

4.3.10. Rainfall events between the 1 in 30 and 1 in 100 annual probability events to be stored above ground within the south-west extent of the site. The required storage volume equates to 43 m³. The average maximum depth of surface water ponding across the designated area is to be no greater than 50 mm.

4.3.11. Discharge is to be made via gravity to the highway drain via the existing connection.

4.3.12. For the proposed layout of the alternative surface water drainage design refer to Figure 8.

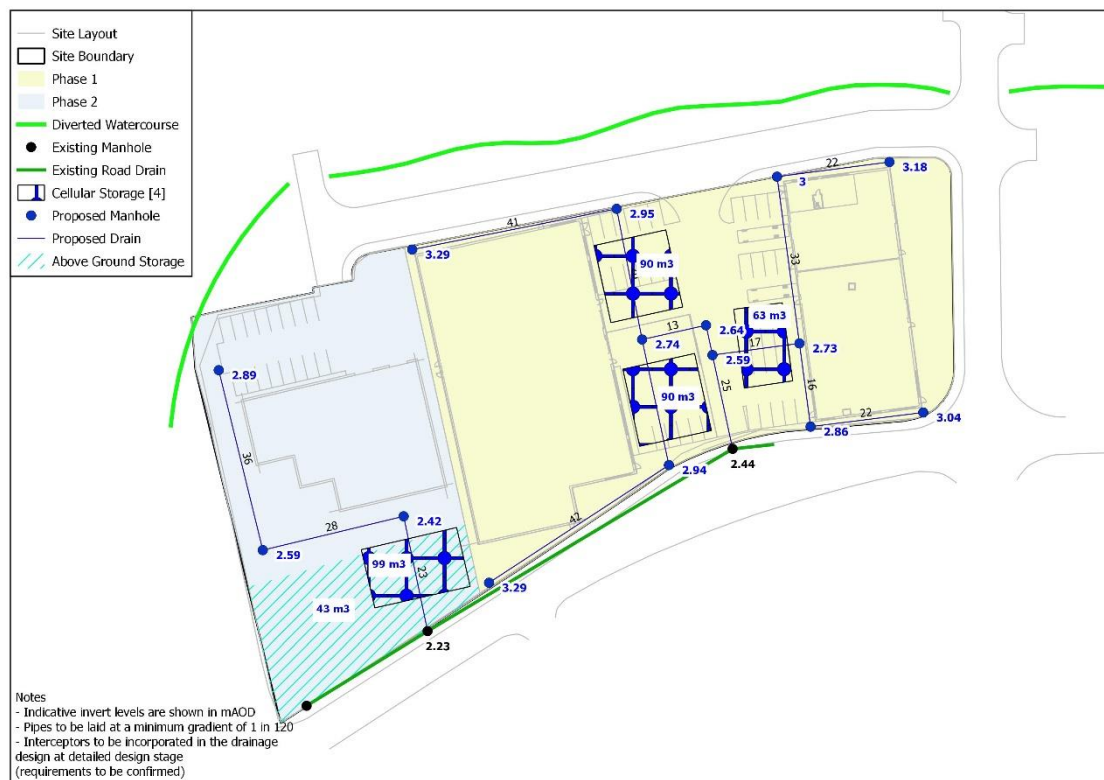


Figure 8 Proposed Surface Water Drainage Design

4.3.13. The combined surface water and foul drainage strategy layout is provided in Appendix E, at the rear of this report.

4.4 ALTERNATIVE SURFACE WATER DRAINAGE DESIGN OPTION

4.4.1. Should connection to the existing highway drain be rejected, the below alternative design can be taken forward at detailed design stage for the management of surface water on site.

Phase 1

4.4.2. Phase 1 covers approximately 63% of the total site area. Discharge from all hard standings are to be collected via private drains.

4.4.3. It is proposed to provide below ground attenuation for up to the 1 in 100 year annual probability events including 20% allowance for climate change underneath the car parking. The required storage volume equates to 243 m³. No above ground storage to be provided within Phase 1.

Phase 2

4.4.4. Phase 2 covers approximately 37% of the total site area. Discharge from all hard standings are to be collected via private drains.

4.4.5. It is proposed to provide below ground attenuation for up to the 1 in 30 year annual probability events including 20% allowance for climate change underneath the car parking area located in the north-west of the site. The required storage volume equates to 99 m³.

4.4.6. Rainfall events between the 1 in 30 and 1 in 100 annual probability events to be stored above ground within the south-west extent of the site. The required storage volume equates to 43 m³. The average maximum depth of surface water ponding across the designated area is to be no greater than 50 mm.

4.4.7. Final discharge from the site is proposed to be made to the adjacent watercourse via 2No. new outfalls. Due to the shallow depth of this unnamed tributary surface water runoff from the site is required to be pumped.

4.4.8. For the proposed surface water drainage layout refer to Figure 9.

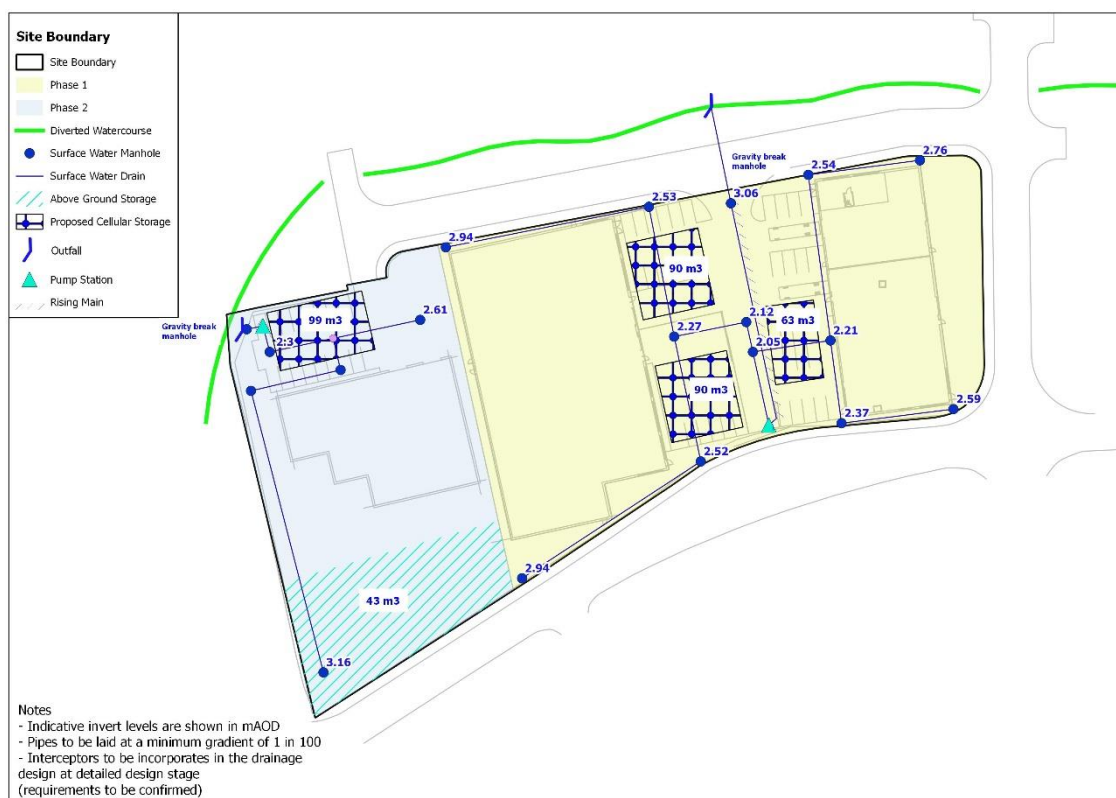


Figure 9 Alternative Surface Water Drainage

4.4.9. The combined surface water and foul drainage strategy layout is provided in Appendix E, at the rear of this report.

4.5 MAINTENANCE

4.5.1. Once the site becomes operational regular inspection and maintenance of trapped gullies, channel drain sumps, catchpits, attenuation tanks, and hydrobrake flow control chambers, will be required. The attenuation tanks and hydrobrake flow control chambers will be supplier design, with suitable access for inspection and future maintenance.

4.5.2. Maintenance is proposed to be in accordance with Table 21.3 of the SuDS manual CIRIA 753 as shown below. A full Operation and Maintenance Manual will be required for the site to include all details of the drainage scheme.

TABLE 21.3 Operation and maintenance requirements for attenuation storage tanks		
Maintenance schedule	Required action	Typical frequency
Regular maintenance	Inspect and identify any areas that are not operating correctly. If required, take remedial action	Monthly for 3 months, then annually
	Remove debris from the catchment surface (where it may cause risks to performance)	Monthly
	For systems where rainfall infiltrates into the tank from above, check surface of filter for blockage by sediment, algae or other matter; remove and replace surface infiltration medium as necessary.	Annually
	Remove sediment from pre-treatment structures and/or internal forebays	Annually, or as required
Remedial actions	Repair/rehabilitate inlets, outlet, overflows and vents	As required
Monitoring	Inspect/check all inlets, outlets, vents and overflows to ensure that they are in good condition and operating as designed	Annually
	Survey inside of tank for sediment build-up and remove if necessary	Every 5 years or as required

Figure 10 General Maintenance Requirement Schedule

4.5.3. Maintenance of the overall drainage network will be the responsibility of the tenants.

4.5.4. Regular inspection of the pump station is required by the tenants. The detailed maintenance requirements of the surface water pump station to be provided by the manufacturer.

5. FOUL WATER DRAINAGE

5.1.1. There are no existing buildings located within the boundary, therefore it is understood that currently there is no foul flow from the site.

5.1.2. Post-development foul flows are predicted to increase to 0.99 l/s, as a result of the development. The design flow calculations are presented in Table 5 below.

Table 5 Foul Design Flow Calculations¹³

DESIGN FLOW	FLOW ELEMENT (L/S/HA)	AREA (HA)	FLOW RATE (L/S)
Domestic Flow Element	0.6	0.9	0.54
Trade Effluent Flow (normal industry only)	0.5	0.9	0.45
TOTAL			0.99

5.1.3. Foul water from the buildings to be collected via private drains. It is proposed to discharge foul flows from the new buildings to Southern Water public sewers. Connections from the new buildings to be made via gravity at manhole no. 6802 and a new connection between manhole no. 7902 and 7701. The proposed foul drainage layout is provided in Figure 11.

¹³ Based on Sewers for Adoption 7th Edition

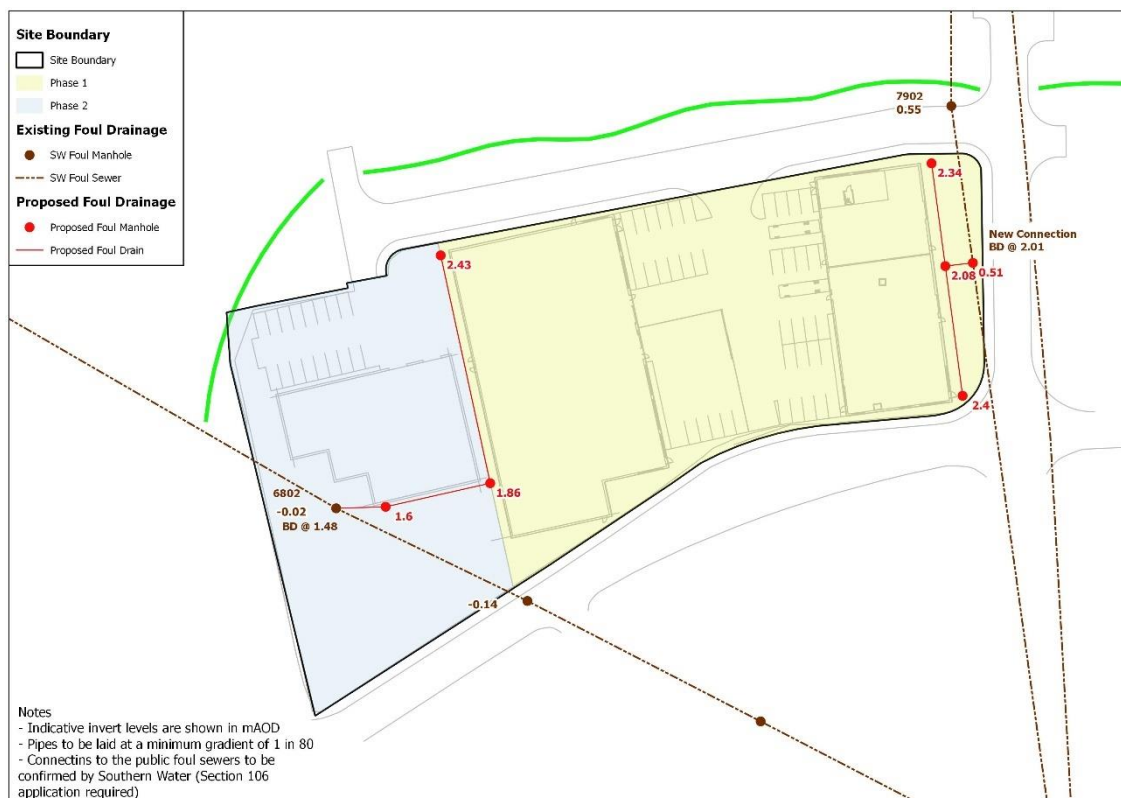


Figure 11 Proposed Foul Water Drainage

5.1.4. A Level 1 capacity check was submitted to confirm that the proposed additional flows can be accommodated within the Southern Water public network. Official response yet to be obtained.

5.1.5. The combined surface water and foul drainage strategy layout is provided in Appendix E, at the rear of this report.

6. CONCLUSIONS

- 6.1.1. The site is currently brownfield and has been previously developed. Currently, the majority of the area is covered by bare ground where the demolished buildings and surfacing had been removed. For the drainage design purposes, it is understood that 94% of the site is impermeable.
- 6.1.2. It is proposed to redevelop the site into commercial/industrial uses (50% Class B2 and 50% Class B8) with associated car parking.
- 6.1.3. The groundwater level underlying the site is between 1.35m and 2.07 mbgl. Given the shallow depth of the groundwater, it is considered that the site is generally unsuitable for the use of soakaways.
- 6.1.4. An existing EA main river flows from east to a south-west direction to the north of the site, crossing it on the north-west corner and flowing southwards along the west boundary. This watercourse has been diverted along Hermitage Park road to the north of the site in 2015.
- 6.1.5. Existing drainage features on site were recorded as part of the topographic survey. There are two connection points from the site to the highway drain in Harts Farm Way.
- 6.1.6. It is proposed to provide 50% betterment on the existing surface water runoff rates from the site. Storage volume is to be provided via a combination of below ground attenuation tanks and above ground storage. Discharge is proposed to the adjacent unnamed tributary or alternatively to the existing highway drain in Harts Farm Way. Consent for discharging into a main river is required from the EA.
- 6.1.7. There are no existing buildings located within the boundary, therefore it is considered that currently there is no foul flow from the site. Post-development foul flows are predicted to increase to 0.99 l/s. It is proposed to collect foul water via private drains and discharge via gravity to Southern Water public sewers. A Level 1 capacity check was submitted to confirm that the proposed additional flows can be accommodated within the Southern Water public network. Official response yet to be obtained.

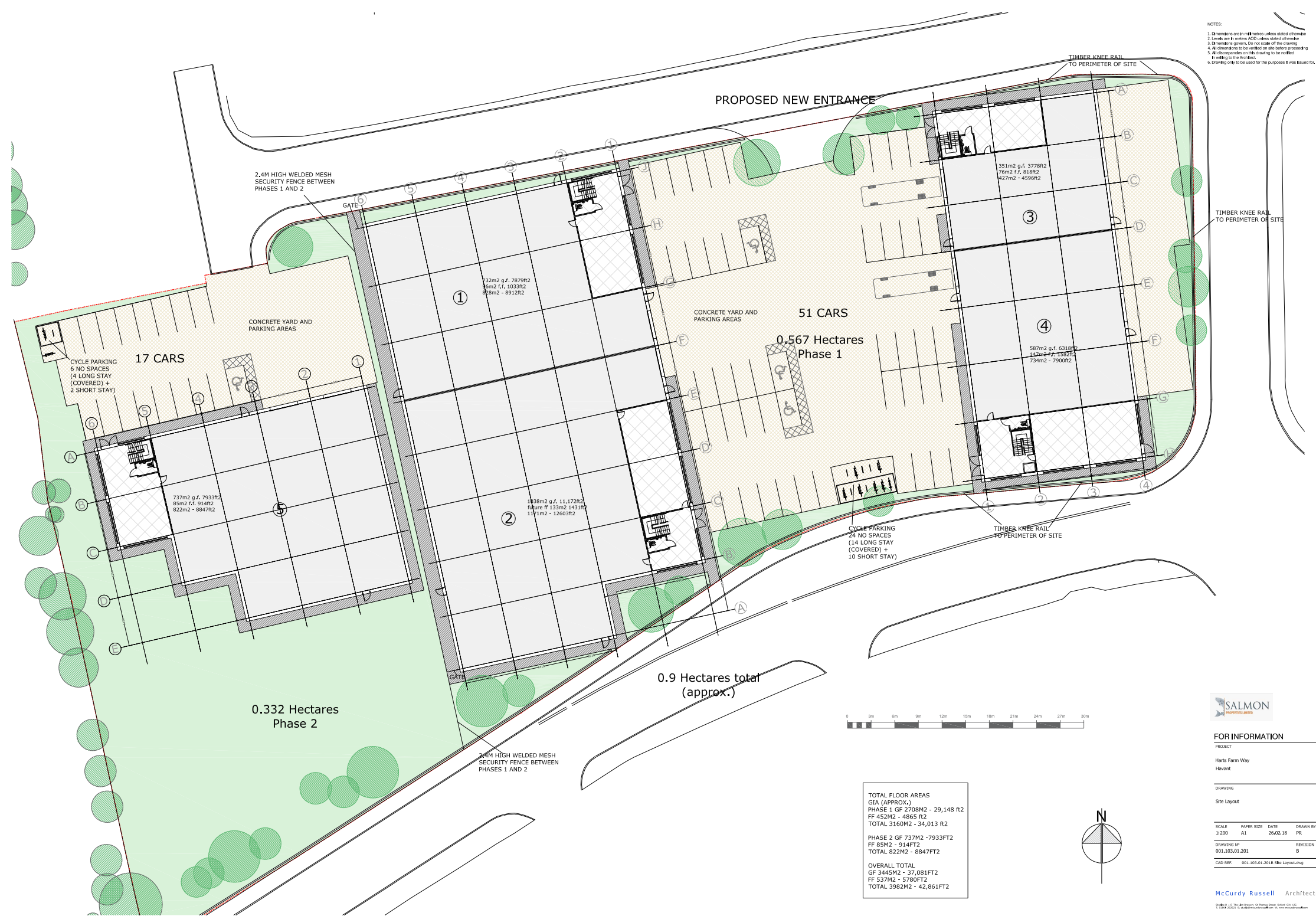
APPENDICES



APPENDIX A

PROPOSED SITE LAYOUT

- NOTES:
1. Dimensions are in millimetres unless stated otherwise
 2. Levels are in meters ACD unless stated otherwise
 3. Dimensions given. Do not scale off the drawing
 4. All dimensions to be verified on site before proceeding
 5. All dimensions on this drawing to be verified in writing to the Architect
 6. Drawing only to be used for the purposes it was issued for.



FOR INFORMATION

PROJECT
Harts Farm Way
Havant

DRAWINGS
Site Layout

SCALE	PAPER SIZE	DATE	DRAWN BY
1:200	A1	26.02.18	PR
DRAWING NO	REVISION		
001.103.01.201	B		
CAD REF.	001.103.01.201B Site Layout.dwg		

McCurdy Russell Architects

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APPENDIX B

EXISTING RUNOFF RATE CALCULATIONS

 Ridge and Partners LLP Partnership House Moorside Road, Winchester SO23 7RX	Project Harts Farm Way				Job no. 5004378	
	Calcs for Salmon Property				Start page no./Revision 1	
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DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 1 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min = 11.9 mm
Factor Z2 (Wallingford procedure)	Z2 = 0.61
Rainfall for 15min storm with 1 year return period	M1_15min = Z2 × M5_15min _i = 7.3 mm
Design rainfall intensity	I _{max} = M1_15min / D = 29.2 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8147 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 66.1 l/s

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DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 2 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min = 11.9 mm
Factor Z2 (Wallingford procedure)	Z2 = 0.79
Rainfall for 15min storm with 2 year return period	M2_15min = Z2 × M5_15min _i = 9.4 mm
Design rainfall intensity	I _{max} = M2_15min / D = 37.8 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8147 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 85.5 l/s

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	RH	12/02/2018	KS	15/02/2018		

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 10 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min = 11.9 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.23
Rainfall for 15min storm with 10 year return period	M10_15min = Z2 × M5_15min _i = 14.6 mm
Design rainfall intensity	I _{max} = M10_15min / D = 58.4 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8147 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 132.3 l/s

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DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 30 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min = 11.9 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.50
Rainfall for 15min storm with 30 year return period	M30_15min = Z2 × M5_15min _i = 17.9 mm
Design rainfall intensity	I _{max} = M30_15min / D = 71.6 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8147 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 162.0 l/s

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DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 100 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 0 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min = 11.9 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.94
Rainfall for 15min storm with 100 year return period	M100_15min = Z2 × M5_15min _i = 23.1 mm
Design rainfall intensity	I _{max} = M100_15min / D = 92.4 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8147 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 209.1 l/s

APPENDIX C

POST-DEVELOPMENT RUNOFF CALCULATIONS

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DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 1 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 20 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min × (1 + p _{climate}) = 14.3 mm
Factor Z2 (Wallingford procedure)	Z2 = 0.62
Rainfall for 15min storm with 1 year return period	M1_15min = Z2 × M5_15min _i = 8.8 mm
Design rainfall intensity	I _{max} = M1_15min / D = 35.3 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8992 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 88.3 l/s

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DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 2 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 20 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min × (1 + p _{climate}) = 14.3 mm
Factor Z2 (Wallingford procedure)	Z2 = 0.80
Rainfall for 15min storm with 2 year return period	M2_15min = Z2 × M5_15min _i = 11.4 mm
Design rainfall intensity	I _{max} = M2_15min / D = 45.6 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8992 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 113.9 l/s

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	RH	12/02/2018	KS	15/02/2018		

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 10 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 20 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min × (1 + p _{climate}) = 14.3 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.24
Rainfall for 15min storm with 10 year return period	M10_15min = Z2 × M5_15min _i = 17.7 mm
Design rainfall intensity	I _{max} = M10_15min / D = 70.7 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8992 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 176.5 l/s

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	RH	12/02/2018	KS	15/02/2018		

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 30 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 20 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min × (1 + p _{climate}) = 14.3 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.52
Rainfall for 15min storm with 30 year return period	M30_15min = Z2 × M5_15min _i = 21.7 mm
Design rainfall intensity	I _{max} = M30_15min / D = 86.9 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8992 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 217.1 l/s

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	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	RH	12/02/2018	KS	15/02/2018		

DESIGN RAINFALL

In accordance with the Wallingford Procedure

Tedds calculation version 2.0.00

Design rainfall intensity

Location of catchment area	Other
Storm duration	D = 15 min
Return period	Period = 100 yr
Ratio 60 min to 2 day rainfall of 5 yr return period	r = 0.350
5-year return period rainfall of 60 minutes duration	M5_60min = 19.3 mm
Increase of rainfall intensity due to global warming	p _{climate} = 20 %
Factor Z1 (Wallingford procedure)	Z1 = 0.62
Rainfall for 15min storm with 5 year return period	M5_15min _i = Z1 × M5_60min × (1 + p _{climate}) = 14.3 mm
Factor Z2 (Wallingford procedure)	Z2 = 1.98
Rainfall for 15min storm with 100 year return period	M100_15min = Z2 × M5_15min _i = 28.3 mm
Design rainfall intensity	I _{max} = M100_15min / D = 113.0 mm/hr

Maximum surface water runoff

Catchment area	A _{catch} = 8992 m ²
Percentage of area that is impermeable	p = 100 %
Maximum surface water runoff	Q _{max} = A _{catch} × p × I _{max} = 282.3 l/s

APPENDIX D

STORAGE VOLUME CALCULATION



Variables

Results

Design

Overview 2D

Overview 3D

Vt

Variables

FSR Rainfall

Return Period (years)

Region

Map

M5-60 (mm)

Ratio R

Cv (Summer)

Cv (Winter)

Impermeable Area (ha)

Maximum Allowable Discharge (l/s)

Infiltration Coefficient (m/hr)

Safety Factor

Climate Change (%)

100

England and Wales

19.300

0.350

0.750

0.840

0.899

33.0

0.00000

2.0

20



Variables

Results

Design

Overview 2D

Overview 3D

Vt

Results

Global Variables require approximate storage of between 236 m³ and 385 m³.

These values are estimates only and should not be used for design purposes.



Variables

Results

Design

Overview 2D

Overview 3D

Vt

Variables

FSR Rainfall

Return Period (years)

Region

Map

M5-60 (mm)

Ratio R

Cv (Summer)

Cv (Winter)

Impermeable Area (ha)

Maximum Allowable Discharge (l/s)

Infiltration Coefficient (m/hr)

Safety Factor

Climate Change (%)

30

England and Wales

19.300

0.350

0.750

0.840

0.899

33.0

0.00000

2.0

20



Variables

Results

Design

Overview 2D

Overview 3D

Vt

Results

Global Variables require approximate storage of between 158 m³ and 269 m³.

These values are estimates only and should not be used for design purposes.



APPENDIX E

DRAINAGE STRATEGY

