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RESIDENTIAL DEVELOPMENT, HIGHGROVE STREET, READING

DRAINAGE STRATEGY AND FLOOD RISK ASSESSMENT

Client: VJS Projects Ltd.
Prepared By: Richard Jones
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DSA Drawing 20/40863/51 P1 – PRELIMINARY DRAINAGE SECTIONS AND DETAILS

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Existing Runoff. Brownfield & Greenfield

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Proposed Surface Water Drainage Model & Rainfall Simulations

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Foul Water Emergency Storage Calculation

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Extracts of Ground Investigation, Paddock Geo-Engineering, P20-060gi, June 2020

1. INTRODUCTION

1.1 Purpose of Report

- 1.1.1.1 David Smith Associates have been instructed on behalf of VJS Projects Ltd to prepare this Drainage Strategy. This is in support of a planning application for the development of the site to form eight dwellings with associated vehicular access.
- 1.1.1.2 The site is an 'L' shaped parcel of land to the rear of existing properties on Highgrove Street, Reading. The site area is 1330m². Its former use was for vehicle repair and storage. It was laid out as a concrete yard and several building units which were entirely impermeable.
- 1.1.1.3 It is understood that the development has planning permission, and construction works are imminent. Therefore the implementation of the drainage systems shall be through 2021/22.

1.2 Documents Referred To

- 1.2.1.1 The following documents have been referred to.
- Topographical Survey, CH Surveys Ltd, Drg RD04517, 03/05/17.
 - Proposed Site Plans, Highgrove Portfolio Build Ltd, Drg 1079-201, Dec 2020
 - Ground Investigation, Paddock Geo-Engineering, P20-060gi, June 2020

2. EXISTING DRAINAGE AND GROUND CONDITIONS

2.1 Watercourses

- 2.1.1.1 The site is within the catchment of the River Thames which is located approximately 2km to the north. The surrounding area is heavily urbanised and no natural watercourses are known to exist on or near to the site.

2.2 Piped Drainage

- 2.2.1.1 Site investigations have determined that run-off from the existing site drains to a combined drainage system in the north of the site. This discharges to a Thames Water public sewer.

2.3 Ground Conditions

- 2.3.1.1 The ground investigation indicated that the site comprises made ground over deposits of the Boyn Hill Gravel Member in places. This is underlain by the London Clay formation.
- 2.3.1.2 Soil infiltration testing was carried out to BRE Digest 365, with initial infiltration into loose made ground and possibly the Boyn Hill Gravels. Infiltration then halted with insufficient occurring in order to calculate an infiltrate rate and as such are considered a failure.
- 2.3.1.3 Ground water was monitored over May and June 2020, with standing water levels recorded in most wells at less than 1.5m below ground level.
- 2.3.1.4 Based on this geology soakaways as a primary means of surface water disposal for the development are considered unviable.

2.4 Existing Discharge from the Site

- 2.4.1.1 Run off from the existing impermeable areas of the site (1330m²) has been calculated by modelling a basic drainage system in various rainfall events. Based on a 100mm diameter outfall pipe, a flow rate of approximately 10.0 - 13.0 l/s could discharge from the site in storm events between 1 – 100 year probability.
- 2.4.1.2 An assessment of the greenfield runoff has been carried out using the Interim Code of Practice for SuDS method in Microdrainage software. This appears to have centred on particularly permeable soil characteristics and gives a runoff QBAR of 0.1 l/s.

3. PROPOSED DRAINAGE

3.1 Surface Water Drainage Strategy

3.1.1.1 The drainage hierarchy has been considered, as follows:

- Infiltration to ground - considered unviable due to ground conditions.
- Discharge to watercourse – none on or near the site.
- Discharge to combined public sewer – This replicates the existing outfall for the site and is therefore considered a suitable compromise.

3.1.1.2 It is proposed to reduce the discharge flow rate as far as practicable. This is achieved by the final outfall requiring to be pumped, with pumps selected to provide a maximum discharge flow rate of 1.5 l/s.

3.1.1.3 Surface water held back by the outfall flow restriction is proposed to be stored in the engineered sub-base of a permeable paving system to the access road. This shall be comprised of Granular Subbase material Type 3 to the Specification for Highway Works. This has a void ratio of 30% allowing water to move through the material and be stored within it.

3.1.1.4 Rainwater pipes from proposed dwellings shall discharge into the subbase via perforated distribution pipes.

3.1.1.5 The access road through the site is relatively steep, and therefore a system of check dams are proposed within the subbase along the length of the road. Each check dam has a 100mm diameter pipe through to allow water to drain down to the next level. This is further controlled by a 50mm diameter orifice flow control at each check dam. This effectively slows the flow of water from high level areas of the site to the lowest parts of the site.

3.1.1.6 In the event that water builds up behind a check dam, the water level will rise through the permeable paving construction before eventually overflowing to the next storage area downstream.

3.1.1.7 At the lowest area of the site, a wider area of permeable paving is available which allows for the eventual flow of water to this point during and following rainfall events.

- 3.1.1.8 The permeable paving provides a degree of water treatment that is appropriate for the level of contamination risk associated with this relatively small residential development.

3.2 Surface Water Drainage Design

- 3.2.1.1 The main elements of the network have been designed and modelled in Microdrainage software. This results in a relatively complex network with storage structures, orifice plates and overflows have been modelled in an appropriate way to simulate the design proposals.
- 3.2.1.2 The results of the simulations show that there is no flooding from the system in the higher probability modelled storm events. In the 100 year storm event + 40% climate change, minor flooding of less than 2.0m² is indicated. This will be readily accommodated on site by either surface puddles, or runoff to other sections of permeable paving downstream. Other key results are as follows:

Return Period (year)	Critical Storm Duration (minutes)	Discharge Flow Rate (l/s)	Depth of water in 450 deep sub-base at manhole S4 (mm)
1	120	1.5	46
30	180	1.5	190
100 + 40% CC	240	1.5	450

3.3 Foul Water Drainage

- 3.3.1.1 Foul water from proposed dwellings shall discharge to a pumping chamber, with a connection to the existing combined public sewer. The pump and chamber shall be to specialist design and manufacture. A calculation of population equivalent (PE) and emergency storage requirements is included in Appendix **.

4. DRAINAGE MAINTENANCE

4.1 Responsibilities

- 4.1.1.1 Drainage systems within the curtilage of individual dwellings shall be the responsibility of each property owner.
- 4.1.1.2 All drainage on the site outside of the curtilage of individual dwellings shall be the responsibility of VJS Projects Ltd. A Management Company shall be employed by them to carry out inspection and maintenance of the permeable pavement and drainage systems on the site.

4.2 Maintenance Regime

4.2.1 Traditional Drainage Systems

- 4.2.1.1 This generally relates to normal drainage systems including roof gutters, down pipes, underground pipework, manholes/catchpits, slot drains and gullies..
- 4.2.1.2 6 monthly , when extreme rainfall is forecast, and after significant storm events
- Clear leaves litter and debris from all areas of the site and from visible surface features of the drainage system.
 - Inspect all manholes and catchpits, sumps to channel drains and gullies, roof gutters and downpipes, and orifice chambers. Collect and dispose of any silt and debris present. Monitor the speed of silt build up and increase frequency of maintenance if required.
 - Inspect the orifice chambers and ensure the units are clean and operating satisfactorily. Repair or replace if damaged or not operational for any reason.
- 4.2.1.3 At 5-10 year intervals
- CCTV survey of piped drainage system, through to the outfall to the existing combined system. Removal of silt and debris as required. Replace or repair any areas of failure.

4.2.2 Permeable Paving

12 weekly

For the first 12 months following regular use, additional 2-4mm quartzite/gritstone to BS EN 1097-2:1998 to be brushed into joints at 12 week intervals to accommodate the initial settlement of permeable block paving.

12 monthly , when extreme rainfall is forecast, and after significant storm events

Brush silt and organic matter from all joints with stiff brush and vacuum.

Additional 2-4mm quartzite/gritstone to BS EN 1097-2:1998 to be brushed into joints.

At 5 year intervals

Shallow trial holes to be carried out at a low point of the paving to assess silt and organic matter build up in the joints between pavements and the laying course.

If the silt build up is significant and forms a seal which prevents water flowing into the sub base layers easily, localised areas shall be taken up and the pavements relaid with new laying course and jointing grit.

At 20 year intervals

Deep trial holes to be carried out at a low point of the paving to assess silt and organic matter build up in the joints between pavements, the laying course, the sub base and the surfaces of the geotextiles.

If the silt build up is significant and affects the ability of the system to satisfactorily hold and dispose of surface water, the affected parts shall be taken up and replaced.

4.2.3 Pumps and chambers

The Management Company shall employ the services of specialist pump servicing teams to attend site at regular intervals. The manufacturers and suppliers of the pumping equipment can advise on recommended inspection and servicing intervals, but this is likely to be every 6 months initially.

All manufacturers and suppliers recommendations should be followed for the inspection and maintenance of the pump installations.

High level alarms should be included within the pump chambers, with alerts sent to the Management Company and Land Owner so that immediate action can be taken.

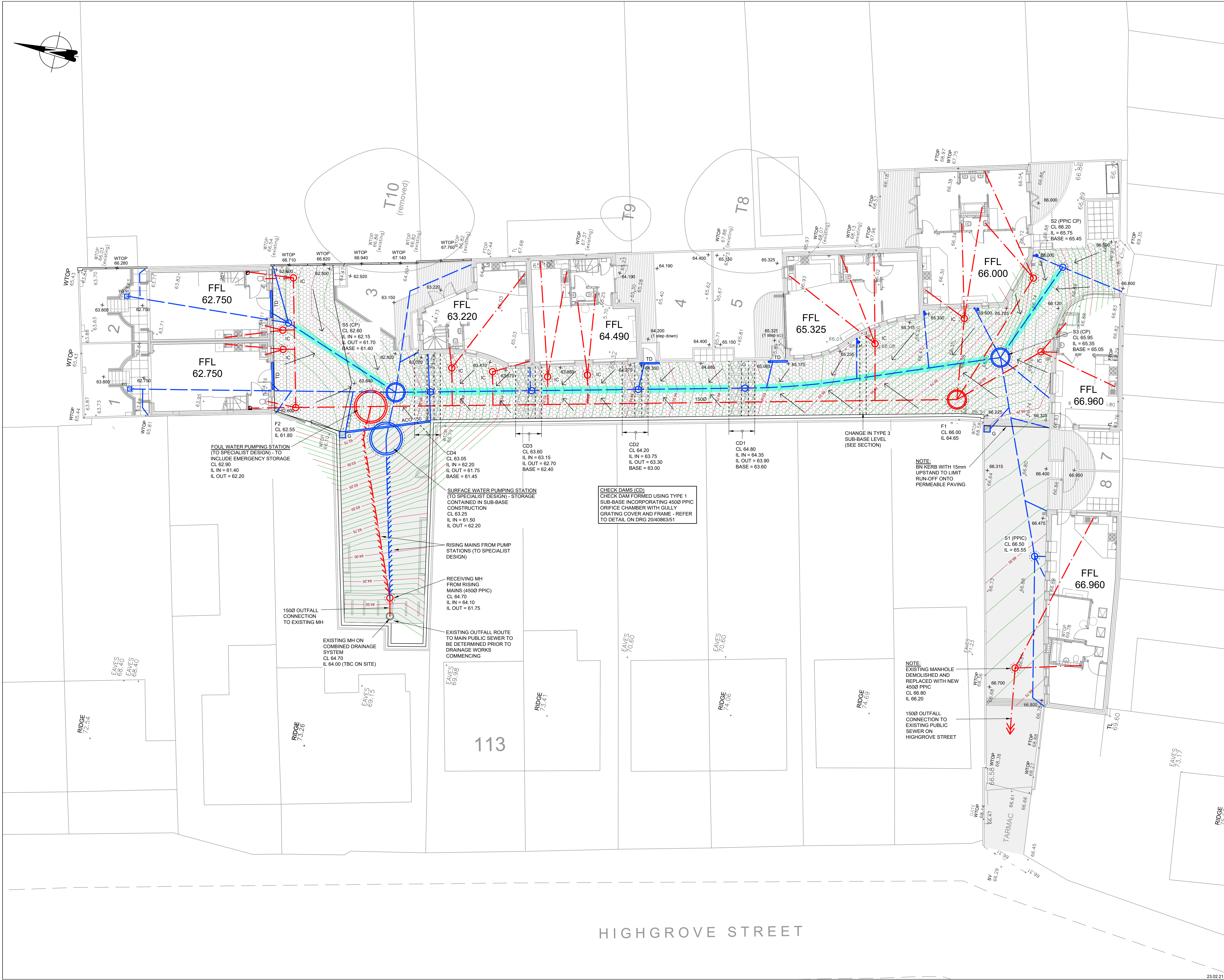
5. CONCLUSION

- 5.1.1.1 The development is situated in Flood Zone 1, where this type of development is appropriate.
- 5.1.1.2 Other sources of flooding have been assessed and none present a significant risk to the development.
- 5.1.1.3 A surface water drainage strategy has been developed, with flow control and storage to manage flows and offer an improvement on the existing situation. This reduces the risk of off-site flooding downstream.
- 5.1.1.4 A strategy for foul water has been developed.
- 5.1.1.5 Maintenance responsibilities, and maintenance regimes for specific features have been established.
- 5.1.1.6 The relevant national and local policies and guidance have been referred to and met, and the development is considered appropriate in relation to drainage and flood risk.

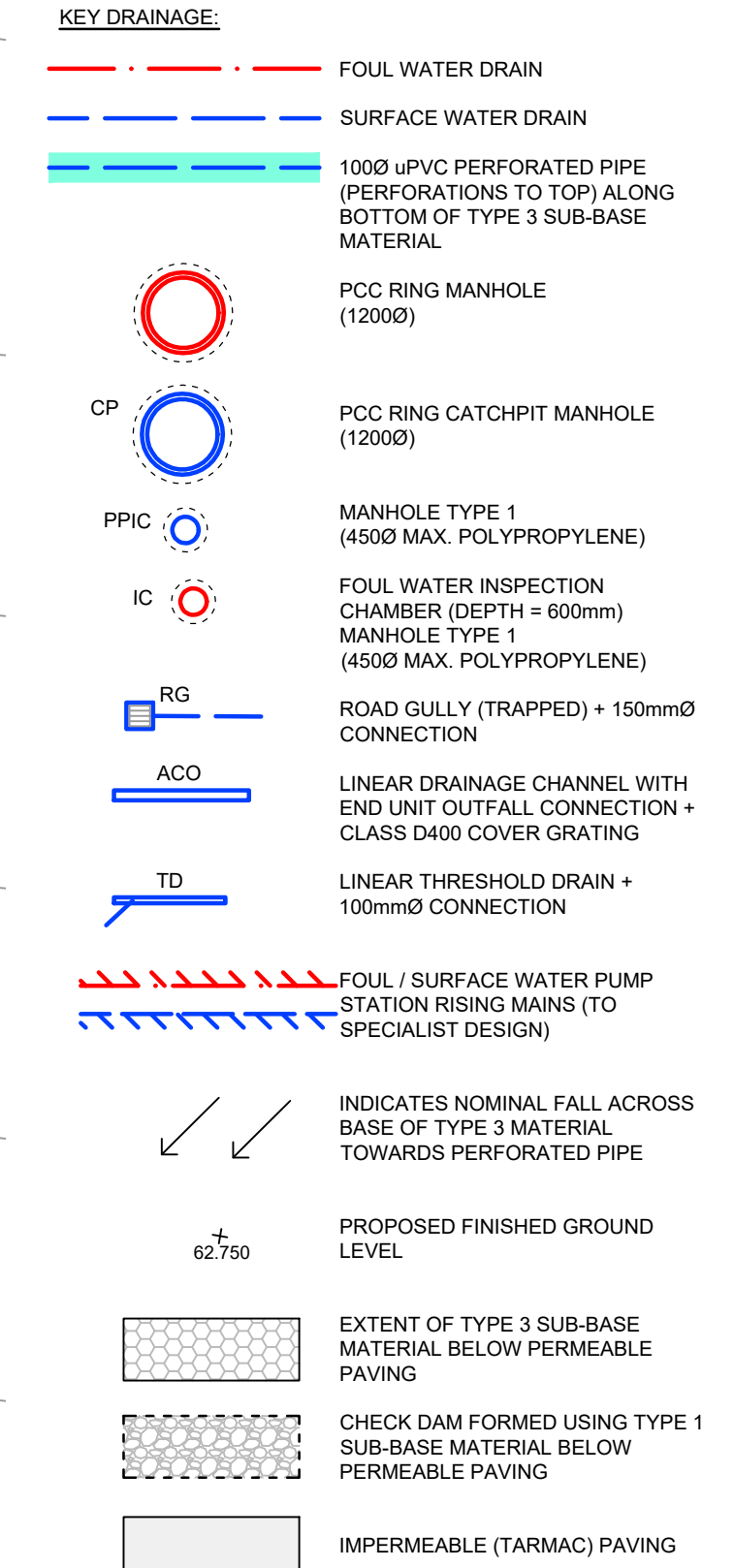
APPENDIX A

DSA Drawing 20/40863/50 P1 – PRELIMINARY DRAINAGE LAYOUT

DSA Drawing 20/40863/51 P1 – PRELIMINARY DRAINAGE SECTIONS AND DETAILS



- Notes**
1. IF IN DOUBT - ASK !!! DO NOT SCALE
 2. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL ARCHITECTS AND ENGINEERS DRAWINGS.
 3. ALL WORK TO BE CARRIED OUT IN ACCORDANCE WITH THE RELEVANT BRITISH STANDARDS, CODES OF PRACTICE AND BUILDING PRACTICE.
 4. ALL DIMENSIONS TO BE CHECKED PRIOR TO STARTING THE WORKS ON SITE. ANY DISCREPANCIES TO BE REPORTED TO THE ENGINEER IMMEDIATELY.
 5. CONTRACTOR TO ASCERTAIN THE LOCATION OF SERVICES ON SITE PRIOR TO STARTING THE WORK.
 6. ALL DIMENSIONS FOR CONSTRUCTION ARE TO BE OBTAINED FROM SITE MEASUREMENTS OR ARCHITECTS SETTING OUT DRAWINGS PRIOR TO MANUFACTURE/BUILDING.
- CDM 2015 DESIGNER NOTES**
- IN ADDITION TO THE HAZARDS, AND RISKS NORMALLY ASSOCIATED WITH THE TYPE OF WORK DETAILED ON THIS DRAWING, NOTE THE FOLLOWING SIGNIFICANT RISKS AND INFORMATION.
- CONSTRUCTION:**
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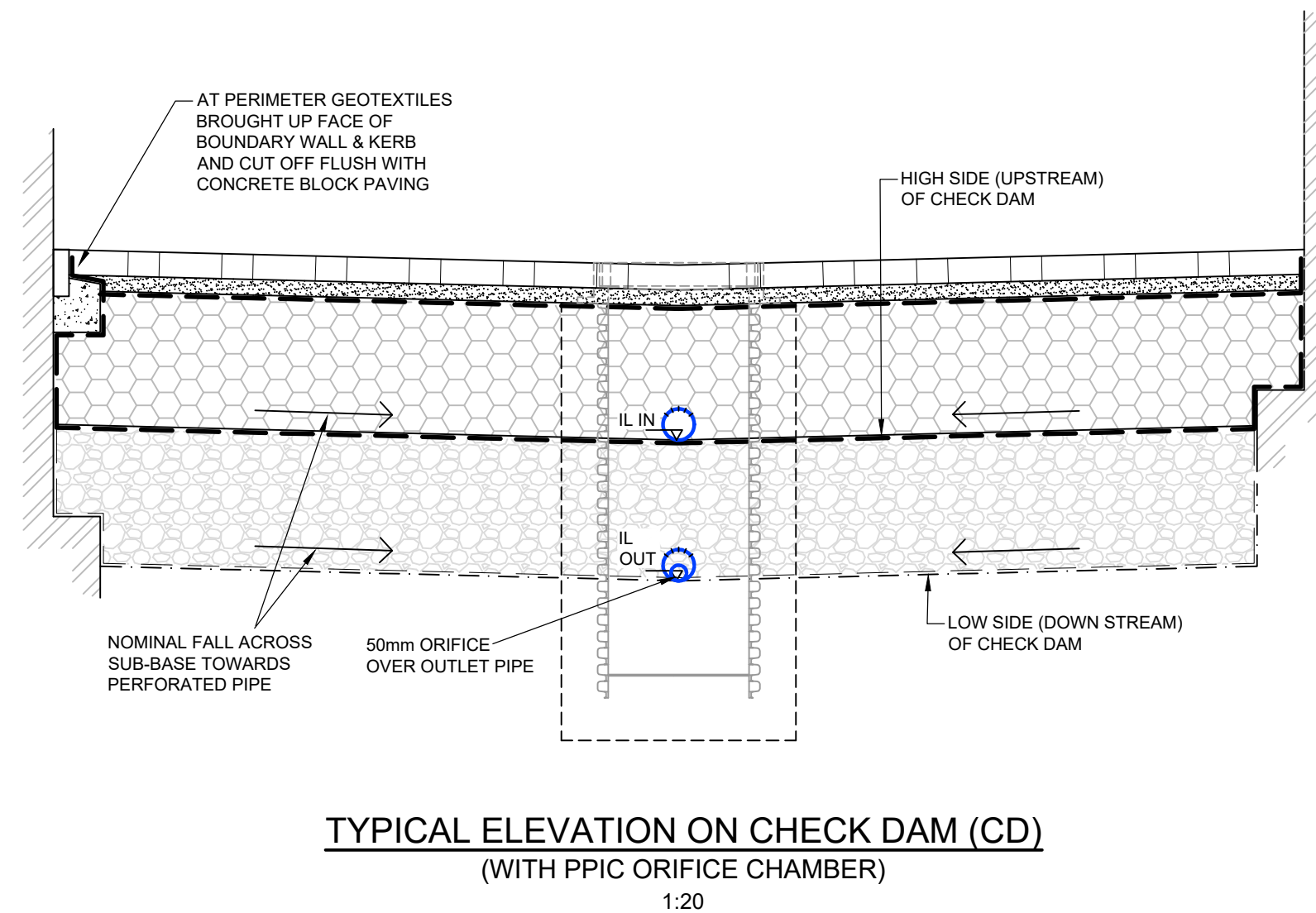
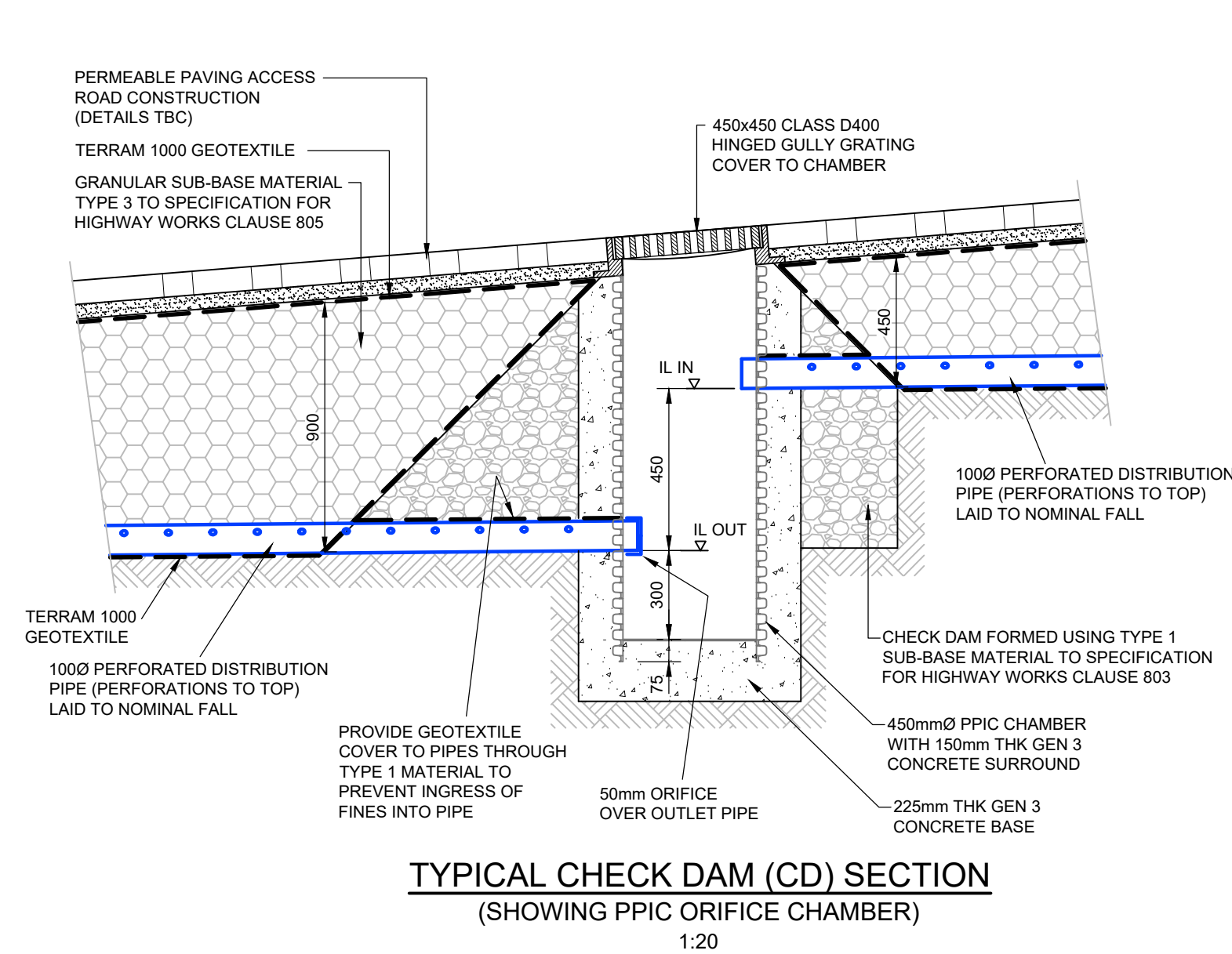
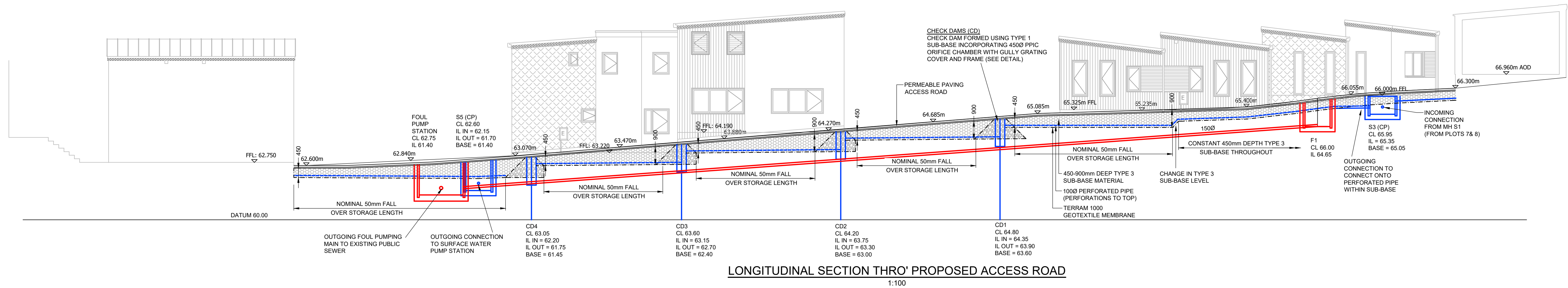


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TITLE	PRELIMINARY DRAINAGE LAYOUT		
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APPENDIX B

Existing Runoff. Brownfield & Greenfield

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8 Duncan Close Moulton Park Northampton NN3 6WL	HIGGROVE ST, READING 20/40863	
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Micro Drainage	Network 2019.1	

STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Storm

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales

Return Period (years)	1	PIMP (%)	100
M5-60 (mm)	18.900	Add Flow / Climate Change (%)	0
Ratio R	0.400	Minimum Backdrop Height (m)	0.000
Maximum Rainfall (mm/hr)	0	Maximum Backdrop Height (m)	0.000
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	1.200
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	500

Designed with Level Soffits

Time Area Diagram for Storm

Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	0.095	4-8	0.038

Total Area Contributing (ha) = 0.133

Total Pipe Volume (m³) = 0.157

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
S1.000	10.000	0.100	100.0	0.133	5.00	0.0	0.600	o	100	Pipe/Conduit	
S1.001	10.000	0.100	100.0	0.000	0.00	0.0	0.600	o	100	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
S1.000	0.00	5.22	65.500	0.133	0.0	0.0	0.0	0.77	6.0	0.0
S1.001	0.00	5.43	65.400	0.133	0.0	0.0	0.0	0.77	6.0	0.0

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PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S1.000	o	100	SS1	66.500	65.500	0.900	Open Manhole	450
S1.001	o	100	SS3	66.500	65.400	1.000	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S1.000	10.000	100.0	SS3	66.500	65.400	1.000	Open Manhole	1200
S1.001	10.000	100.0	S	66.500	65.300	1.100	Open Manhole	0

Free Flowing Outfall Details for Storm

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
S1.001	S	66.500	65.300	0.000	0	0

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Micro Drainage Source Control 2019.1		

ICP SUDS Mean Annual Flood

Input

Return Period (years)	1	Soil	0.150
Area (ha)	0.133	Urban	0.000
SAAR (mm)	700	Region Number	Region 6

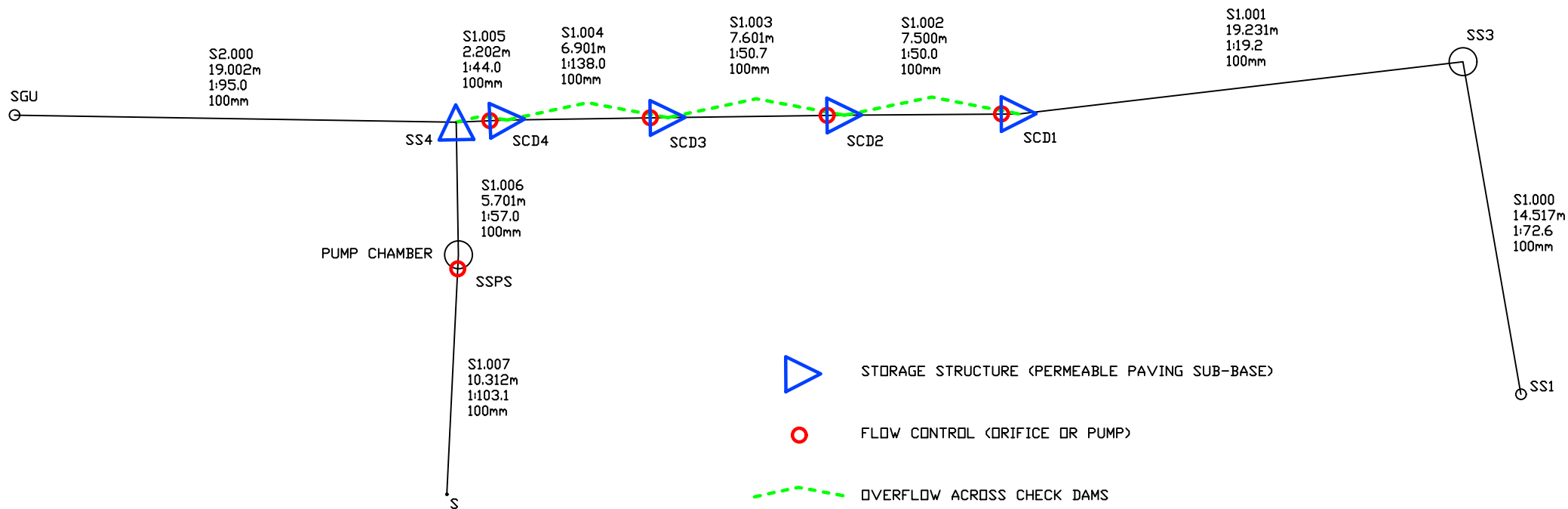
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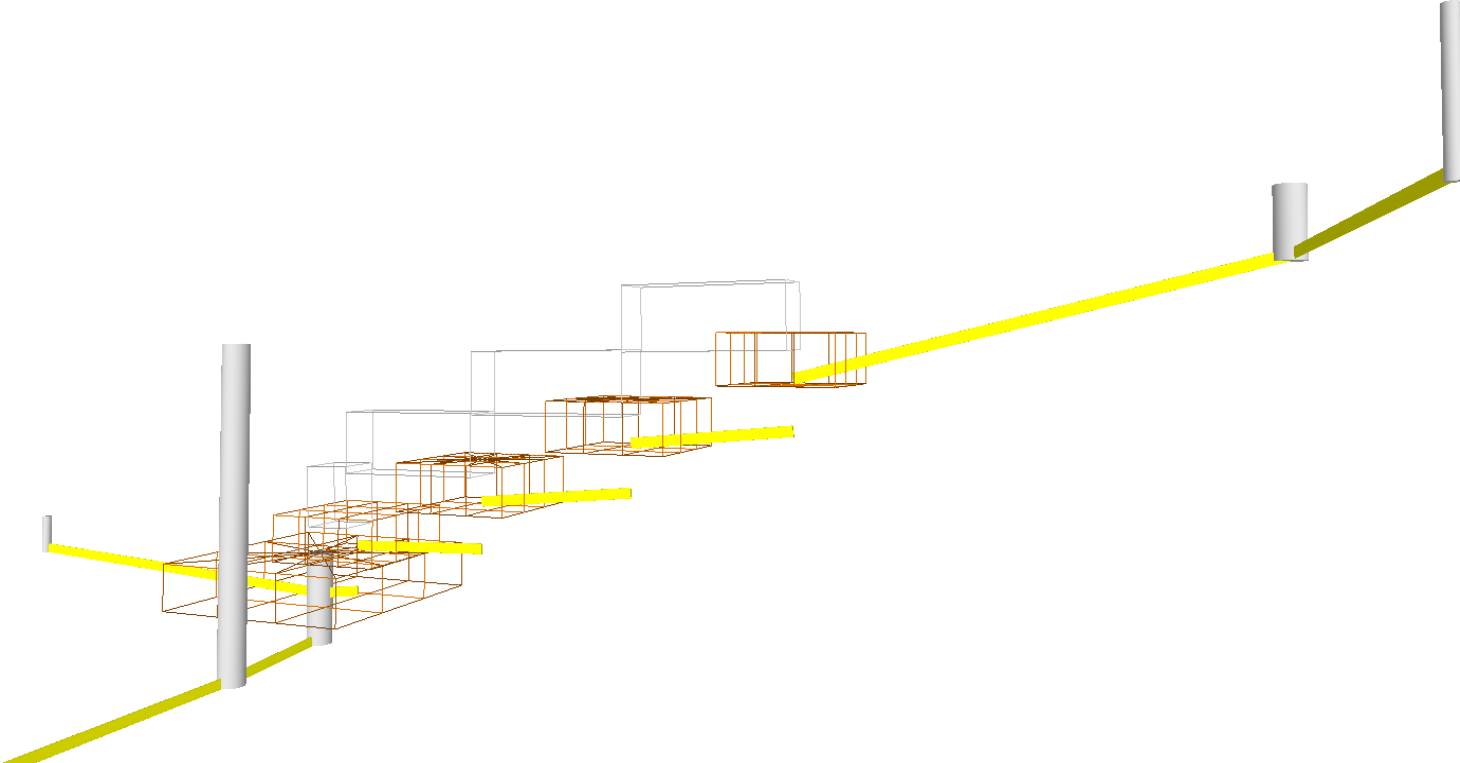
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QBAR Urban	0.1
Q1 year	0.0
Q1 year	0.0
Q30 years	0.1
Q100 years	0.2

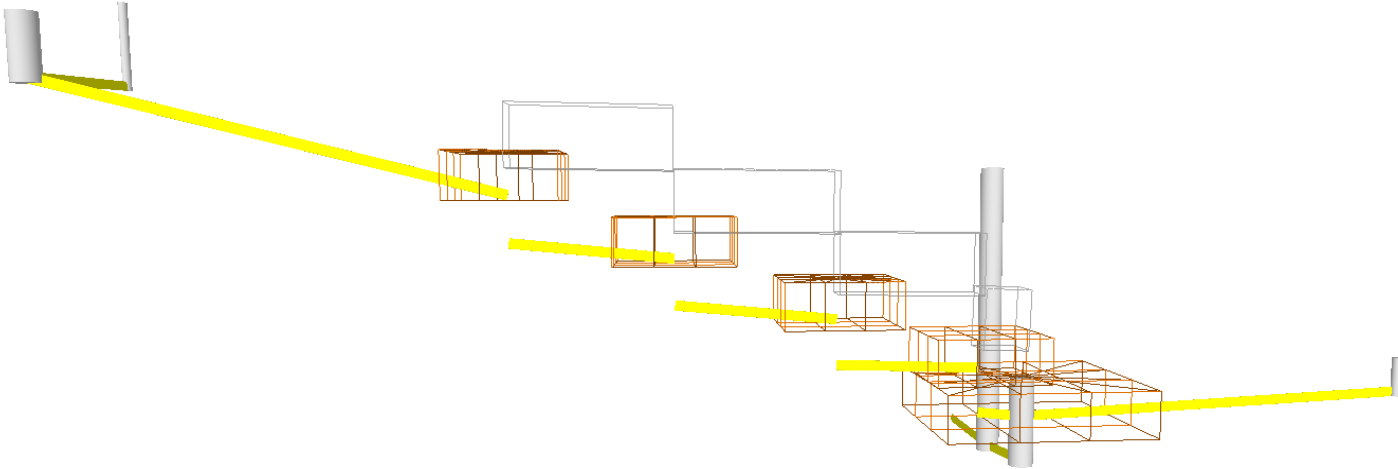
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APPENDIX C

Proposed Surface Water Drainage Model & Rainfall Simulations



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STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for Storm

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - England and Wales

Return Period (years)	1	PIMP (%)	100
M5-60 (mm)	18.900	Add Flow / Climate Change (%)	0
Ratio R	0.400	Minimum Backdrop Height (m)	0.000
Maximum Rainfall (mm/hr)	0	Maximum Backdrop Height (m)	0.000
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	1.200
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	500

Designed with Level Soffits

Time Area Diagram for Storm

Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	0.081	4-8	0.024

Total Area Contributing (ha) = 0.105

Total Pipe Volume (m³) = 0.730

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
S1.000	14.517	0.200	72.6	0.016	5.00	0.0	0.600	o	100	Pipe/Conduit	
S1.001	19.231	1.000	19.2	0.032	0.00	0.0	0.600	o	100	Pipe/Conduit	
S1.002	7.500	0.150	50.0	0.003	0.00	0.0	0.600	o	100	Pipe/Conduit	
S1.003	7.601	0.150	50.7	0.009	0.00	0.0	0.600	o	100	Pipe/Conduit	
S1.004	6.901	0.050	138.0	0.009	0.00	0.0	0.600	o	100	Pipe/Conduit	
S1.005	2.202	0.050	44.0	0.025	0.00	0.0	0.600	o	100	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
S1.000	0.00	5.27	65.550	0.016	0.0	0.0	0.0	0.90	7.1	0.0
S1.001	0.00	5.45	65.350	0.048	0.0	0.0	0.0	1.77	13.9	0.0
S1.002	0.00	5.56	63.900	0.051	0.0	0.0	0.0	1.09	8.6	0.0
S1.003	0.00	5.68	63.300	0.060	0.0	0.0	0.0	1.09	8.5	0.0
S1.004	0.00	5.86	62.700	0.069	0.0	0.0	0.0	0.65	5.1	0.0
S1.005	0.00	5.89	62.200	0.094	0.0	0.0	0.0	1.16	9.1	0.0

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Micro Drainage	Network 2019.1	

Network Design Table for Storm

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
S2.000	19.002	0.200	95.0	0.011	5.00	0.0	0.600	o	100	Pipe/Conduit	
S1.006	5.701	0.100	57.0	0.000	0.00	0.0	0.600	o	100	Pipe/Conduit	
S1.007	10.312	0.100	103.1	0.000	0.00	0.0	0.600	o	100	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
S2.000	0.00	5.40	62.350	0.011	0.0	0.0	0.0	0.79	6.2	0.0
S1.006	0.00	5.98	61.700	0.105	0.0	0.0	0.0	1.02	8.0	0.0
S1.007	0.00	6.21	61.600	0.105	0.0	0.0	0.0	0.76	5.9	0.0

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PIPELINE SCHEDULES for Storm

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S1.000	o	100	SS1	66.500	65.550	0.850	Open Manhole	450
S1.001	o	100	SS3	65.950	65.350	0.500	Open Manhole	1200
S1.002	o	100	SCD1	64.800	63.900	0.800	Junction	
S1.003	o	100	SCD2	64.200	63.300	0.800	Junction	
S1.004	o	100	SCD3	63.600	62.700	0.800	Junction	
S1.005	o	100	SCD4	63.050	62.200	0.750	Junction	
S2.000	o	100	SGU	62.750	62.350	0.300	Open Manhole	450
S1.006	o	100	SS4	62.600	61.700	0.800	Open Manhole	1200
S1.007	o	100	SSPS	64.700	61.600	3.000	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S1.000	14.517	72.6	SS3	65.950	65.350	0.500	Open Manhole	1200
S1.001	19.231	19.2	SCD1	64.800	64.350	0.350	Junction	
S1.002	7.500	50.0	SCD2	64.200	63.750	0.350	Junction	
S1.003	7.601	50.7	SCD3	63.600	63.150	0.350	Junction	
S1.004	6.901	138.0	SCD4	63.050	62.650	0.300	Junction	
S1.005	2.202	44.0	SS4	62.600	62.150	0.350	Open Manhole	1200
S2.000	19.002	95.0	SS4	62.600	62.150	0.350	Open Manhole	1200
S1.006	5.701	57.0	SSPS	64.700	61.600	3.000	Open Manhole	1200
S1.007	10.312	103.1	S	64.700	61.500	3.100	Open Manhole	0

Free Flowing Outfall Details for Storm

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
S1.007	S	64.700	61.500	0.000	0	0

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Online Controls for Storm

Orifice Manhole: SCD1, DS/PN: S1.002, Volume (m³): 0.1

Diameter (m) 0.050 Discharge Coefficient 0.600 Invert Level (m) 63.900

Orifice Manhole: SCD2, DS/PN: S1.003, Volume (m³): 0.1

Diameter (m) 0.050 Discharge Coefficient 0.600 Invert Level (m) 63.300

Orifice Manhole: SCD3, DS/PN: S1.004, Volume (m³): 0.1

Diameter (m) 0.050 Discharge Coefficient 0.600 Invert Level (m) 62.700

Orifice Manhole: SCD4, DS/PN: S1.005, Volume (m³): 0.1

Diameter (m) 0.050 Discharge Coefficient 0.600 Invert Level (m) 62.200

Depth/Flow Relationship Manhole: SSPS, DS/PN: S1.007, Volume (m³): 3.5

Invert Level (m) 61.600

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.5000	4.000	1.5000

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Offline Controls for Storm

Pipe Manhole: SCD1, DS/PN: S1.002, Loop to PN: S1.003

Diameter (m)	2.000	Roughness k (mm)	0.600
Section Type	600 Culvert	Entry Loss Coefficient	0.500
Slope (1:X)	100.0	Coefficient of Contraction	0.600
Length (m)	2.000	Upstream Invert Level (m)	64.650

Pipe Manhole: SCD2, DS/PN: S1.003, Loop to PN: S1.004

Diameter (m)	2.000	Roughness k (mm)	0.600
Section Type	600 Culvert	Entry Loss Coefficient	0.500
Slope (1:X)	100.0	Coefficient of Contraction	0.600
Length (m)	1.500	Upstream Invert Level (m)	64.050

Pipe Manhole: SCD3, DS/PN: S1.004, Loop to PN: S1.005

Diameter (m)	2.000	Roughness k (mm)	0.600
Section Type	600 Culvert	Entry Loss Coefficient	0.500
Slope (1:X)	100.0	Coefficient of Contraction	0.600
Length (m)	1.500	Upstream Invert Level (m)	63.450

Pipe Manhole: SCD4, DS/PN: S1.005, Loop to PN: S1.006

Diameter (m)	2.000	Roughness k (mm)	0.600
Section Type	600 Culvert	Entry Loss Coefficient	0.500
Slope (1:X)	100.0	Coefficient of Contraction	0.600
Length (m)	1.500	Upstream Invert Level (m)	62.900

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Storage Structures for Storm

Cellular Storage Manhole: SCD1, DS/PN: S1.002

Invert Level (m) 64.350 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.30
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	25.0	0.0	0.451	0.0	0.0
0.450	25.0	0.0			

Cellular Storage Manhole: SCD2, DS/PN: S1.003

Invert Level (m) 63.750 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.30
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	30.0	0.0	0.451	0.0	0.0
0.450	30.0	0.0			

Cellular Storage Manhole: SCD3, DS/PN: S1.004

Invert Level (m) 63.150 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.30
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	30.0	0.0	0.451	0.0	0.0
0.450	30.0	0.0			

Cellular Storage Manhole: SCD4, DS/PN: S1.005

Invert Level (m) 62.600 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.30
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	30.0	0.0	0.451	0.0	0.0
0.450	30.0	0.0			

Cellular Storage Manhole: SS4, DS/PN: S1.006

Invert Level (m) 62.150 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

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Cellular Storage Manhole: SS4, DS/PN: S1.006

Depth (m)	Area (m ²)	Inf. Area (m ²)	Depth (m)	Area (m ²)	Inf. Area (m ²)
0.000	95.0	0.0	0.451	0.0	0.0
0.450	95.0	0.0			

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1 year Return Period Summary of Critical Results by Maximum Level (Rank 1)
for Storm

PN	US/MH Name	Overflow Act.	Water		Surcharged		Flooded		Pipe		Status
			Level (m)	Depth (m)	Volume (m³)	Flow / Cap.	Overflow (l/s)	Flow (l/s)			
S1.004	SCD3	14	63.172	0.372	0.000	0.68	0.0	3.5	SURCHARGED*		
S1.005	SCD4	21	62.695	0.395	0.000	0.57	0.0	3.6	SURCHARGED*		
S2.000	SGU		62.383	-0.067	0.000	0.24		1.4		OK	
S1.006	SS4		62.196	0.396	0.000	0.37		2.7	SURCHARGED		
S1.007	SSPS		62.193	0.493	0.000	0.27		1.5	SURCHARGED		

		US/MH	Level
PN	Name	Exceeded	
S1.000	SS1		
S1.001	SS3	4	
S1.002	SCD1		
S1.003	SCD2		
S1.004	SCD3		
S1.005	SCD4		
S2.000	SGU		
S1.006	SS4	1	
S1.007	SSPS		

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30 year Return Period Summary of Critical Results by Maximum Level (Rank 1)
for Storm

PN	US/MH Name	Overflow Act.	Water		Surcharged		Flooded		Pipe		Status
			Level (m)	Depth (m)	Volume (m³)	Flow / Cap.	Overflow (l/s)	Flow (l/s)			
S1.004	SCD3	14	63.403	0.603	0.000	0.72	0.0	3.7	FLOOD RISK*		
S1.005	SCD4	21	62.901	0.601	0.000	0.69	0.7	4.3	FLOOD RISK*		
S2.000	SGU		62.406	-0.044	0.000	0.58		3.5		OK	
S1.006	SS4		62.340	0.540	0.000	0.31		2.2	FLOOD RISK		
S1.007	SSPS		62.331	0.631	0.000	0.27		1.5	SURCHARGED		

PN	US/MH Name	Level Exceeded	
S1.000	SS1		
S1.001	SS3	4	
S1.002	SCD1		
S1.003	SCD2		
S1.004	SCD3		
S1.005	SCD4		
S2.000	SGU		
S1.006	SS4	1	
S1.007	SSPS		

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100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

Simulation Criteria

Areal Reduction Factor 1.000 Additional Flow - % of Total Flow 0.000
 Hot Start (mins) 0 MADD Factor * 10m³/ha Storage 2.000
 Hot Start Level (mm) 0 Inlet Coefficient 0.800
 Manhole Headloss Coeff (Global) 0.500 Flow per Person per Day (l/per/day) 0.000
 Foul Sewage per hectare (l/s) 0.000

Number of Input Hydrographs 0 Number of Storage Structures 5
 Number of Online Controls 5 Number of Time/Area Diagrams 0
 Number of Offline Controls 4 Number of Real Time Controls 0

Synthetic Rainfall Details

Rainfall Model FSR Ratio R 0.400
 Region England and Wales Cv (Summer) 0.750
 M5-60 (mm) 18.800 Cv (Winter) 0.840

Margin for Flood Risk Warning (mm) 300.0 DVD Status OFF
 Analysis Timestep Fine Inertia Status OFF
 DTS Status ON

Profile(s) Summer and Winter
 Duration(s) (mins) 15, 30, 60, 120, 180, 240, 360, 480, 600,
 720, 960, 1440, 2160, 2880, 4320, 5760,
 7200, 8640, 10080
 Return Period(s) (years) 1, 30, 100
 Climate Change (%) 0, 0, 40

PN	US/MH Name	Storm	Return Period	Climate Change	First (X) Surcharge	First (Y) Flood	First (Z) Overflow
S1.000	SS1	15 Winter	100	+40%	30/15 Winter		
S1.001	SS3	15 Winter	100	+40%	30/15 Summer	100/15 Summer	
S1.002	SCD1	15 Winter	100	+40%	1/15 Summer		30/15 Summer
S1.003	SCD2	30 Winter	100	+40%	1/15 Summer		100/15 Summer
S1.004	SCD3	30 Winter	100	+40%	1/15 Summer		100/15 Summer
S1.005	SCD4	30 Winter	100	+40%	1/15 Summer		30/60 Winter
S2.000	SGU	240 Winter	100	+40%	100/15 Winter		
S1.006	SS4	240 Winter	100	+40%	1/15 Summer	100/240 Winter	
S1.007	SSPS	240 Winter	100	+40%	1/15 Summer		

PN	US/MH Name	Water Overflow Act.	Surcharged Level (m)	Flooded Depth (m)	Volume (m³)	Flow / Cap.	Overflow (l/s)	Pipe Flow (l/s)	Status
S1.000	SS1		66.292	0.642	0.000	1.28		8.7	FLOOD RISK
S1.001	SS3		65.952	0.502	1.718	1.13		15.2	FLOOD
S1.002	SCD1	18	64.664	0.664	0.000	0.52	12.5	4.4	FLOOD RISK*
S1.003	SCD2	12	64.067	0.667	0.000	0.51	16.0	4.4	FLOOD RISK*

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100 year Return Period Summary of Critical Results by Maximum Level (Rank 1) for Storm

			Water	Surcharged	Flooded				Pipe	
	US/MH	Overflow	Level	Depth	Volume	Flow /	Overflow	Flow		
PN	Name	Act.	(m)	(m)	(m³)	Cap.	(l/s)	(l/s)	Status	
S1.004	SCD3	14	63.469	0.669	0.000	0.74	18.5	3.8	FLOOD RISK*	
S1.005	SCD4	21	62.923	0.623	0.000	0.69	25.3	4.3	FLOOD RISK*	
S2.000	SGU		62.600	0.150	0.000	0.21		1.2	FLOOD RISK	
S1.006	SS4		62.600	0.800	0.042	0.25		1.8	FLOOD	
S1.007	SSPS		62.591	0.891	0.000	0.27		1.5	SURCHARGED	

PN	US/MH Level		Exceeded
	Name		
S1.000	SS1		
S1.001	SS3		4
S1.002	SCD1		
S1.003	SCD2		
S1.004	SCD3		
S1.005	SCD4		
S2.000	SGU		
S1.006	SS4		1
S1.007	SSPS		

APPENDIX D

Foul Water Emergency Storage Calculation



◆ David Smith Associates ◆

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Project No: 20/40863

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Made By: *pm*

Revision:

Date: 17/06/21

Checked By:

Project: HIGHGROVE STREET, PEARING - WASTEWATER EMERGENCY STORAGE

BRITISH WATER CODE OF PRACTICE

Flows + Loads 4

(POPULATION EQUIVALENT)

$$6 \text{ No. } 2 \text{ Bedroom} = 6 \times 4 \text{ PE} = 24 \text{ PE}$$

$$2 \text{ No. } 1 \text{ Bedroom} = 2 \times 3 \text{ PE} = 6 \text{ PE}$$

$$= \underline{\underline{30 \text{ PE}}}$$

$$30 \text{ PE} \times 150 \text{ l/person/day} = 4500 \text{ l/day}$$

$$= 4.5 \text{ m}^3 \text{ STORAGE REQUIRED.}$$

ASSUMING 2100 ϕ WET WELL

$$\pi \times 1.05^2 = 3.46$$

$$4.5 / 3.46 = 1.30 \text{ DEPTH REQUIRED BELOW INCOMING GRAVITY PIPE.}$$

APPENDIX E

Extracts of Ground Investigation, Paddock Geo-Engineering, P20-060gi, June 2020

125 Highgrove Street, Reading, RG1 5EJ

GROUND INVESTIGATION



Oak and Lime Limited

June 2020

P20-060gi

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ROOTS

Roots were encountered within borehole WS3 only as rootlets within the Made Ground to a maximum of 0.70m depth.

VISUAL CONTAMINATION

The borehole arisings were assessed for visual contaminants and summarised below.

Visual Contamination

Borehole	Visual / Olfactory Evidence of Contamination and Strata
WS1	Brick and flint within Made Ground between 0.11m and 0.85m. No hydrocarbon or agrichemical odours, staining or VOCs.
WS2	Brick, ash and flint within Made Ground between 0.10m and 0.75m. No hydrocarbon or agrichemical odours, staining or VOCs.
WS3	Brick and flint within Made Ground between 0.10m and 0.70m. No hydrocarbon or agrichemical odours, staining or VOCs.
WS4	Brick, flint and occasional ash/coal within Made Ground between 0.12m and 0.70m. No hydrocarbon or agrichemical odours, staining or VOCs.
WS5	Brick, flint and concrete within Made Ground between 0.14m and 0.50. No hydrocarbon or agrichemical odours, staining or VOCs.
WS6	Brick and flint within Made Ground between 0.10m and 0.90m. Odours noted from material between 0.10m and 0.90m depth.

4.2 Groundwater Conditions

No significant groundwater strikes were encountered within any of the exploratory points undertaken. Lenses of sand were observed within boreholes WS6 that contained perched water within the sand lenses. The sand layer at the base of the Boyn Hill Gravel Member 1.90-2.10m in WS5 was also noted to be damp but did not lead to a steady groundwater level within the borehole at the time of drilling.

Subsequent monitoring of the groundwater level was carried out within the wells installed in WS1, WS3, WS4 and WS6. Monitoring has been carried out on three occasions in May and June 2020 today and further monitoring is planned. The groundwater levels to date are summarised below.

Groundwater Levels

Well	Depth (m begl)			
	19/05/20	26/05/20	02/06/20	09/06/20
	Struck Depth	Steady Level		
WS1	DRY	DRY	DRY	DRY
WS3	DRY	1.60	1.55	1.40
WS4	DRY	1.61	1.40	1.32
WS6	DRY	1.30	Inaccessible	1.31

8.5 Bearing Capacity

Should a conventional spread foundation option be considered, then at a minimum depth of 0.60m bgl or 0.2m into the Boyn Hill Gravel Member deposits on the southern end of site, an allowable bearing capacity would be of the order of **130kPa**.

At the northern end of site, for a conventional spread foundation at a minimum depth of 1.25m bgl into the fine grained London Clay Formation deposits, an allowable bearing capacity would be of the order of **160kPa**.

These estimates further include a factor of safety of 3 against general shear failure and should keep settlements within tolerable limits.

Any excavations for the footings should be inspected by a suitably qualified person to assess the variability of the soils and groundwater conditions. If, following inspection, the soil conditions differ from those identified within this geotechnical appraisal the recommendations may require reassessment. Any roots, organic matter, and in particular any 'soft/loose' or otherwise unsuitable material encountered at the founding depth should be removed prior to pouring of any concrete.

8.6 Piled Foundations

Should assessed minimum foundation depths be too deep for conventional foundations or the bearing capacities derived above not be sufficient for the proposed development then consideration of alternative foundation solutions such as piles.

Piled foundations are likely to provide a satisfactory foundation option for the proposed development with piles extending into the deeper London Clay Formation deposits at a greater depth than investigated during this ground investigation.

Where trees are removed, depending upon pile types and design, piles should be sleeved within the zone of potential desiccation to prevent uplift forces whilst the ground recovers soil water deficits to stable moisture contents.

A cable percussion borehole with attendant sampling, testing and supplementary ground investigation report will be required to provide a further range of geotechnical parameters and to assist structural engineer and piling contractor with the piled foundation design. The advice of a specialist piling contractor should be sought who can provide an assessment of the suitability of their piles.

8.7 Sub-Surface Concrete

The worst case for the site is Design Sulphate Class DS-1. The worst case Aggressive Chemical Environment for Concrete (ACEC) site classification is AC-1s.

8.8 Surface Water Soakaways and Soil Permeability

Infiltration testing was carried out on the site within the boreholes WS2 and WS5 within the Boyn Hill Gravel Member strata employing the BRE365 methodology. These tests indicated initial percolation into the disturbed near surface Made Ground and possibly the near surface Boyn Hill Gravel Member

but then infiltration halted with insufficient occurring in order to be able to calculate a infiltration rate and as such are considered a failure.

Infiltration Testing Results

Borehole	Cycle	Depth of Test (m begl)	Infiltration Factors (ms⁻¹)
WS2	1	0.00 – 3.00m – N/A	Insufficient infiltration and collapse within boreholes – Test failed
WS5	1	0.00 – 3.00m – N/A	Insufficient infiltration and collapse within boreholes – Test failed

Given the infiltration values calculated, it is considered that conventional soakaways are likely to be ineffective on the site within the underlying natural Boyn Hill Gravel Member deposits. As such another method for disposal of surface water further down the hierarchy of preference should be sought.

However, it is recommended that more detailed testing within trial pits to the BRE365 methodology be carried out to confirm the potential for conventional soakaways following site clearance, especially in the southern area of the site underlain by granular soils.

8.9 Access Roadways and Parking Areas

In situ CBR determinations were not carried out at the time of the siteworks due to the hardstanding coverage of the site.

We understand that the site is to be levelled with some 1.5m removed at the northern end of the site. As such pavement foundations will likely be placed onto coarse grained Boyn Hill Gravel Member deposits on the southern end of the site and onto fine grained London Clay Formation deposits on the northern part of the site.

The TRL Laboratory Report 1132 – The Structure of Bituminous Road, Appendix C Table C1 allows an estimate or CBR value based on soil equilibrium suction index. Where a pavement foundation will bear onto the encountered fine grained London Clay Formation soils a CBR value of 2.5-4.0% can be employed.

The TRL Laboratory Report 1132 – The Structure of Bituminous Road, Appendix C Table C1 allows an estimate or CBR value based on soil equilibrium suction index. Where a pavement foundation will bear onto the encountered coarse grained Boyn Hill Gravel Member soils a CBR value of up to 20% can be employed.

Therefore, given the above results a conservative design CBR value of 2.5% is considered appropriate across the site for initial design and it is recommended that further in-situ CBR testing be performed employing plate loading once the site has been cleared to confirm these values.