

SITE A, WOODSIDE GRANGE

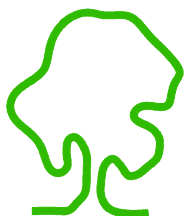


WOODSLANDS ROAD, HASOCKS, BN6 8EX

Flood Risk and Drainage Assessments

December 2023

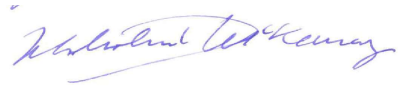
Revised January 2024, February 2024, February 2025, June 2025,
July 2025, August 2025



eas ltd

Environmental Assessment Services Ltd

REPORT DATA SHEET

Requirement	Data
Report Reference	654/Lock/SiteAWoodsideGrangeHassocks/FR&DA
Date	November 2023
Client	Tony Lock
Report type	Flood Risk and Drainage Assessment
Purpose	Submission to Planning
Revisions	January 2024, February 2024 , February 2025, June 2025, July 2025, August 2025
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MR TONY LOCK

SITE A, WOODSIDE GRANGE, WOODSLAND ROAD, HASSOCKS BN6 8EX

Flood Risk and Drainage Assessment

November 2023

(Revised January 2024, February 2024, February 2025, June 2025, July 2025, August 2025)

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MR TONY LOCK

SITE A, WOODSIDE GRANGE, WOODSLAND ROAD, HASSOCKS BN6 8EX

Flood Risk and Drainage Assessment

November 2023

(Revised January 2024, February 2024, February 2025, June 2025, July 2025, August 2025)

1. INTRODUCTION

- 1.1 Environmental Assessment Services Ltd was instructed to prepare a Flood Risk and Drainage Assessment (FR&DA) in relation to a planning application for the proposed development at Woodside Grange, Hassocks.
- 1.2 According to the Environment Agency (EA) Flood Map for Planning, the site lies within Flood Zone 1 (low probability of flooding from rivers and sea).
- 1.3 The site-specific FR&DA is prepared generally in accordance with the Revised National Planning Policy Framework (NPPF) and its accompanying Update Planning Practice Guidance (PPG), published in July 2018.
- 1.4 This report has been prepared for the purposes of addressing the planning authority and Environment Agency's (EA) concerns over the flood risks relating to the proposed development. It should be appreciated that the potential for flooding at a site depends on many factors, some natural, some man-made and some arising from accident, and that these factors may change substantially over the life of the proposed development. The flood risk in this report is assessed on the basis of available information, which is generally catalogued in the report but does not take into account any special risks or requirements of the development unless we have been specifically informed of them.
- 1.5 Any construction levels suggested in this report should therefore be considered as minimum levels and the client and any person appointed to undertake the design of the development should satisfy themselves that any such level is in all respects suitable for their purposes.

2. EXISTING SITE & PROPOSED DEVELOPMENT

- 2.1 The flood risk assessment is prepared for the proposed development of Woodside Grange, Hassocks to provide a five bedroom, single-story, wheelchair accessible dwelling with parking provision and a pool on site A, (the existing tennis court on site A is to remain). Also, two new build two-story houses on site B, (the existing pond is to remain on site B). The existing property and garage are to be retained as a separate neighbouring residence.

- 2.2 The Ordnance Survey reference for the site is TQ 30891 16422 and the site elevation is approximately + 44 m AOD. The total site area is 1.5 acres/0.61 hectares. See Figure 1 in Appendix A.
- 2.3 The site (as existing) is occupied by a house with a parking area, garage and swimming pool, which is to remain as a neighbouring residence. The rest of the property is currently laid to lawn, with extensive mature trees and shrubs. There is an area of hardstanding along the eastern end of site A. The proposed site is bordered to the north and east sides by the Taylor Wimpey housing estate (currently under construction), to the south by site B (two proposed houses) and to the west by a ditch which flows northwards before joining another watercourse that flows under the railway line (the site currently drains surface water into this), the London to Brighton railway line. See Figure 2 for the existing site layout, Figure 3 for the proposed development and Figure 8 for maps of the drainage ditch connection to the watercourse.
- 2.4 According to the British Geological Survey, the site lies on superficial deposits of Lower Greensand over Weald Clay. DEFRA classify the soil at the site as “slowly permeable, seasonally wet but rich based loamy and clay soil.”
- 2.5 The south of the existing site lies on a Principal Aquifer and is in an area of high groundwater vulnerability. This area however is largely situated under the existing property and not the proposed development sites.
- 2.6 The site borders directly onto a small stream/ditch that flows northwards alongside the railway line to the west of the site, this ditch forms the border with the land owner’s site and he has riparian responsibilities for this watercourse. Figure 8 and Figure 3 in Appendix A for maps show the connection of the ditch to the mains watercourse. There is also a drainage ditch towards the northeastern edge of the site. This lies largely under the hedge/vegetation and flows north towards the proposed Taylor Wimpey SuDS Pond. See Figure 8 in Appendix A for map.
- 2.7 The site is to be developed (excluding the existing house and garage and the area at Site B, adjacent to the original house) for use as a five-bedroom dwelling with disabled access and a swimming pool. The tennis courts are to be retained. The proposed development has been revised to ensure that no part of the building or hardstanding are within 5m of an existing watercourse/drainage ditch.
- 2.8 See Figure 3 in Appendix A for the proposed redevelopment plan.

3. FLOOD RISK & DEFENCES

3.1 Outline Flood Risk

- 3.1.1 The EA Flood Map for Planning showed the site as lying within Flood Zone 1, and with a 0.1% (or less) chance of flooding each year from fluvial sources, where defences are not considered. The site is considered at very low risk of flooding from reservoirs or surface water. See Appendix B.

3.1.2 The Level 1 Strategic Flood Risk Assessment (prepared by Gould Baxter – October 2023) provides a review of potential sources of flooding and proposed mitigation.

3.1.3 The EA data provides modelled flood data and maps specific to the vicinity of the site. The modelled data indicates that the site lies within an area of very low risk of flooding from rivers, but partially on an area of low risk from flooding. (Low risk means that this area has a chance of flooding of between 0.1% and 1% each year). Mitigation measures will need to be proposed during the planning process.

3.2 Defences

3.2.1 The SFRA 2015 for the district did not identify any ‘key flood defences’ protecting the site. There are no fluvial defence works affecting the site.

4. ALTERNATIVE SOURCES OF FLOODING

An assessment of alternative sources of flooding potentially affecting the site has also been made from a review of the Level 1 update 2017 and Level 2 West Sussex SFRA 2021 and EA online mapping.

The SFRA addresses alternative sources of flooding including fluvial flooding, surface water flooding, groundwater flooding, flooding from sewers and flooding from reservoirs.

4.1 Surface Water Flooding

4.1.1 The EA’s online surface water flood risk map shows that the site is in an area at very low risk of surface water flooding (1 in 100 $\geq$ 1 in 1000), with a low risk also present at lower levels of the site (1 in 100). See Appendix B.

4.1.2 For consideration of flood mitigation measures, it is necessary to establish a design flood level for a designated Annual Exceedance Probability (AEP). For establishing a design flood level for the site, the probable life of the proposed development needs to be considered to adopt the appropriate climate change allowance, the design AEP and the appropriate flooding scenarios. The factors selected to establish the design flood level are summarised in Table 4.1 below:

TABLE 4.1
FLOOD DESIGN FACTORS

Factor	Value
Probable life of proposed development (residence)	100 years
Annual Exceedance Probability for fluvial flooding	1.0%
Climate change allowance for fluvial flows (2115)	+20% peak river flow

- 4.1.3 The design flood level is usually derived from the Environment Agency Product 4 for the site. It appears unlikely that any flood modelling will have extended this far into Flood Zone 1.

4.2 Sewer Flooding

- 4.2.1 The existing site does not have access to the mains sewer and foul water is disposed of directly into a cesspit on the site.
- 4.2.2 There does not appear to be any record of sewer flooding affecting the site due to the lack of a nearby sewer system.

4.3 Groundwater Flooding

- 4.3.1 Groundwater flooding occurs when the water table rises above the ground surface, normally as a result of persistent rainfall over a prolonged period. Increased levels of development in an area can increase the risk of groundwater flooding.
- 4.3.2 The site is not thought to be at risk from groundwater flooding.

4.4 Reservoirs

- 4.4.1 The EA online flood maps for planning tool shows that the site is not at risk of flooding from reservoirs.

5. HISTORY OF FLOODING

- 5.1 The history of flooding on site is taken from the West Sussex SFRA 2021, flooding has occurred in the village in 2000, 2003, 2008, 2012 and 2014. The West Sussex County Council Strategic Flood Risk Assessment (SFRA) Level 1 was published in 2015. The SFRA considered flood risk within the district and appropriate policy responses. There are no records of historic flooding on site. The closest record of flooding is from the Herring Stream in the centre of the village of Hassocks, at a lower elevation than the site.
- In November 2008 three shops in Keymer Road were flooded due to a blocked drain.
 - In February 2014 a garden was flooded due to a surge of water in the Herring Stream during heavy rainfall, The culvert on Downs View Road was not large enough to handle very heavy rainfall.
- 5.2 No evidence of historic flood events affecting the site, or its immediate surroundings has been found.

6. PLANNING STATUS WITH RESPECT TO FLOODING

- 6.1 The site is shown on the current Environment Agency (EA) flood risk mapping as being within Flood Zone 1 (<0.1% risk of flooding in any year). The site is shown to lie in an area of very low risk of surface water flooding.
- 6.2 Residential dwellings are classified as 'More Vulnerable' under Table 2 of the NPPF PPG. According to Table 3 of the PPG, 'More Vulnerable' development is appropriate within Flood Zone 1.
- 6.3 The site lies within a long developed semi-rural area, which includes residential premises. The Sequential Test would appear to be satisfied and there is no need to apply the Exception Test.

7. RISK TO PROPERTY

- 7.1 The site is in Flood Zone 1 and at very low risk for flooding from surface water. As long as the finished ground floor level is a suitable margin above the design flood level, no flood related risk to property is envisaged.
- 7.2 Recommended finished ground floor level will be 0.6 m above design flood level or 0.3 m above adjacent ground level, whichever is the higher. Ground level at the site varies between +41.50 m OD and +42.66 m OD.

8. RISK TO OCCUPIERS OF THE PROPOSED DEVELOPMENT

- 8.1 As the site lies in Flood Zone 1, is not at risk from coastal, groundwater or reservoir flooding and is at very low risk of surface water flooding, there should be very minimal flood related risk to the proposed occupants of the redevelopment whilst they are on site.

9. LIKELY IMPACT ON FLOOD RISK ELSEWHERE

- 9.1 The site is presently mostly under lawn and tree cover, with a small amount of hard standing. The proposed redevelopment will leave the site partly under hard cover, where the access driveway enters the site and surrounding the house, pool and tennis court area, with the rest laid to lawn. The drainage solutions outlined in the planning will mitigate any increase in surface water flood risk caused by the increase in levels of hard standing at the site. As such it is unlikely that the proposed redevelopment will increase flood risk elsewhere.
- 9.2 Flood risk downstream of the site may be reduced by including sustainable drainage (SuDS) measures in the proposed redevelopment such as permeable paving and rainwater harvesting for irrigation.

10. DRAINAGE

10.1 Surface Water Drainage

The existing surface water appears to drain as greenfield across the land into the ditch that runs along the western boundary of the wider site, alongside the railway embankment. This flows northwards moving to below ground flow further along the embankment, as can be seen through the railway owned surface water drainage chambers. See Figure 7 for photographs of the railway owned surface water drainage chambers and the drainage ditch, Figure 3 for the plan of the site showing the direction of flow of the drainage ditch and watercourse and Figure 8 for a map showing the connection of the ditch to the watercourse in Appendix A.

The preferred surface water drainage option is draining to an attenuation pond on site with controlled flow into the ditch/watercourse, on the western side of the site. The storage volume would be provided by attenuation with freeboard in the pond. See Appendix C for the proposed attenuation drainage design.

An alternative to drainage into the proposed attenuation pond would be to drain directly into the existing pond to the south, (situated on Site B and proposed as the surface drainage solution for that site). See Appendix A Figure 7 for photographs of the pond on Site B. This option has been discounted due to the elevation of the pond on site B being higher than the ground level at Site A.

The attenuation storage with controlled flow discharge will be limited to a maximum of Greenfield Q_{bar} . This is achieved by the use of a vortex valve to control the flow of water to 4 l/s (greenfield Q_{bar}). See Table 10.1 below for a summary of the greenfield flow rates, the full greenfield calculations are shown in Appendix F:

TABLE 10.1
GREENFIELD FLOW RATE

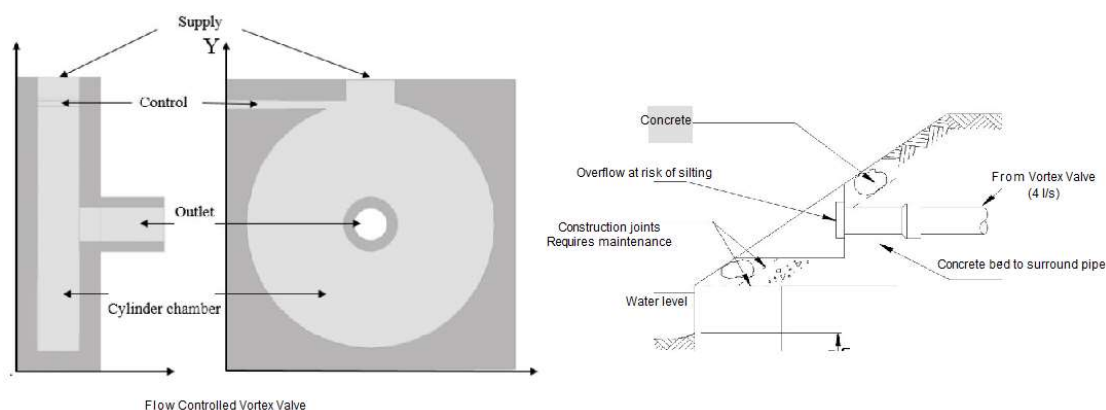
Factor/Return Period	Runoff Rate l/s
Q_{bar}	4.0
1 Year	3.4
30 Year	9.3
100 Year	12.9

See Appendix F for full Greenfields runoff data.

The calculated storage volume required would be approximately 88 m³. A 19.3 m diameter pond would be suitable, with the required storage provided by 0.3 m depth of attenuation and 0.15 m of freeboard above the 0.5 m of permanent water depth. An orifice or vortex valve flow restrictor limits the discharge from the pond to the greenfield Q_{bar} (4 l/s). The area of the attenuation pond is calculated to include the attenuation space and freeboard which is considered in the calculation to size the required pond.

The Tennis court is an existing structure on the site; it is to remain as existing and is not included in the development plans. It has not been included in the drainage design as it will continue to drain as with the current situation. It is made of entirely permeable surfacing and as such is not adding to the surface water runoff from the site as existing and will not add to it once developed.

The pond will outflow to the ditch via a 100 mm pipe in a headwall; the outfall to the ditch from the pond is limited to the greenfield flow Q_{bar} by the use of an orifice or vortex valve (located adjacent to the pond) limiting the flow to 4 l/s, (orifice diameter of 66.5 mm). See below for a typical vortex valve and overflow pipe in a headwall. The ditch is located on the western boundary of the wider site (blue line boundary). The ground level is +36.0 m AOD and the invert level of the ditch is +34.0 m AOD.



The data for the drainage layout (chamber diameter and pipe size) are shown in Appendix D and summarised in table 10.2 overleaf.

10.2 Design Storm

The surface water drainage system has been designed for the 1 in 30-year return period event plus a 35% allowance for climate change and tested for surcharge for the 1 in 100-year return period event (1% Annual Exceedance Probability) plus a 45% allowance for climate change with a maximum discharge of 4 l/s. The attenuation pond has been designed for a CV value of 95% and a climate change allowance of 45%. See Appendix C for the proposed drainage design and Appendix D for pipe and chamber layout and sizing of the attenuation pond from the MasterDrain model.

10.3 Hydrological data

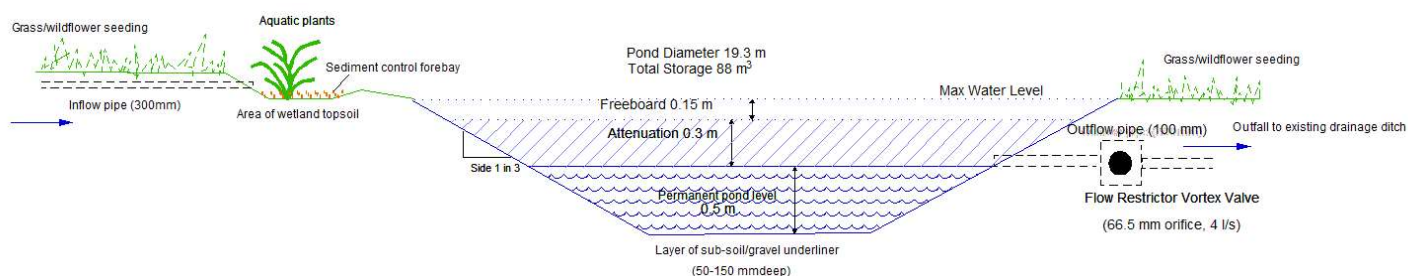
Hydrological data for the site as used in the MasterDrain model is given in Appendix D and summarised in Table 10.2 below:

Table 10.2
Surface Water Chamber and pipe details

Chamber Number	Chamber Diameter (mm)	Pipe size (mm)	Invert Level (m AOD)	Flow Restrictor (l/s)
SW4	600	150	42.16	-
SW3	600	150	41.85	-
SW2	450	225	40.90	-
SW1	450	225	40.40	-
SW5	450	100	41.95	-
SW6	900	300	40.00	-
POND	15600	100	39.10	4
DITCH	-	100	34.0	-

10.4 Drainage Design Model

The attenuation pond has been designed to accommodate the 1 in 100-year return period storm event plus a 45% allowance for climate change. An appropriate attenuation pond size has been calculated as 19.3 m diameter. This allows for a maximum depth of 0.5 m. An attenuation storage of 0.3 m above plus a freeboard of 0.15 m has been added above the attenuation level. The water above the permanent waterline will flow out of the attenuation pond via a vortex valve flow restrictor (limiting the flow to 4 l/s) through a 100 mm pipe to the drainage ditch on the western boundary of the wider site. The outfall into the ditch is 34 m from the northern boundary of the wider site. See Appendix D for the calculations for the attenuation storage and freeboard and Appendix C for a plan of the drainage design and below for a cross section of the attenuation pond (not to scale):



10.5 Extreme Event

An extreme event, which overwhelmed the drainage system, would likely result in water following the existing course and flowing westwards across the surface

of the ground into the existing drainage ditch located parallel to the railway line inside the western red line boundary.

10.6 Additional SuDS Measures

Other SuDS measures that would reduce peak surface water flows include permeable paving for the external areas, rainwater harvesting for garden irrigation and a green roof for the building.

10.7 Foul Sewerage

Foul sewage from the existing house on the site is presently discharged to a cesspit. The proposed dwelling on site A is intended to connect to the public foul sewer (currently under construction) at the edge of the Taylor Wimpey site via Chamber FW.IC.A . This requires an agreement with Taylor Wimpey (already agreed in principle) and Southern Water. See Appendix E for plans for the Taylor Wimpey sewer. It is intended that the foul sewerage on the Taylor Wimpey site be adopted by Southern Water.

There is a swimming pool proposed on Site A which will need to be connected to the foul sewer system via chamber FW3 so that any water from the periodic backwashing the system can be disposed of, as well as water from the occasional draining of the pool (*swimming pools are drained on average every three to five years - pooltroopers.com*) which will need to be via a public sewer system.

The foul sewerage from the proposed development will be connected to the main sewage system at the proposed Foul Chamber (FW.IC.A) situated on the western edge of the Taylor Wimpey site, as outlined in the planning. A pumping station located in chamber FW1 will connect to the main sewer at chamber FW IC A on the Taylor Wimpey site via a 70 mm rising main connection pipe. See the drainage design plan in Appendix C for the connection to the Taylor Wimpey drainage system. The data for the drainage layout (chamber diameter and pipe size) is summarised in table 10.3 below.

Table 10.3
Foul Water Chamber Data

Chamber Number	Chamber Diameter (mm)	Outfall Pipe Size (mm)	Invert Level (m AOD)	Pumping Station
FW3	450	100	42.1	-
FW2	400	100	41.5	-
FW1	600	70	41.1	Yes
FW IC A	900	225	41.8	-

10.8 Drainage Design

A surface water and foul sewerage drainage design has been prepared for the site using the MasterDrain SW and MasterDrain Foul network models. See Appendix C for the drainage plan and Appendix D for the output from the drainage models.

11. SUMMARY

- 11.1 The site lies within Flood Zone 1 and is unlikely to be at risk of flooding from fluvial sources. The site is not at risk from flooding from reservoirs or coastal waters and is at very low risk of flooding from surface water. No other flood risks likely to affect the site have been identified.
- 11.2 A design flood level at the site, based on the 1 in 100-year return period event plus a 45% allowance for climate change, has been derived, based on the Environment Agency's flood model for the location, at +44 m OD. The ground floor level of the residential part of the development and the access to the access road should be 0.6 m above the design flood level or 0.3 m above ground level, whichever is the higher.
- 11.3 There should be no significant flood risk to buildings or occupants and safe access should be available to the northeast onto the access road under all eventualities.
- 11.4 The underlying geology suggests that the drainage of surface water to soakaway will not be practicable and that the surface water at the site may be most suited to being drained to an attenuation pond. An onsite attenuation pond, with a flow-controlled vortex valve, limiting the discharge to 4 l/s (greenfield Qbar) and over-spilling into the ditch on the western boundary of the wider site's blue line boundary (which connects to a watercourse to the north), is the preferred option and would be the simplest and most practicable solution. An appropriate size attenuation pond would be 19.3 m diameter, which allows for a maximum permanent depth of 0.5 m, an attenuation storage of 0.3 m and an additional freeboard of 0.15 m. This will overflow into the drainage ditch on the western blue line boundary via a vortex valve restrictor (4 l/s), through a 100 mm pipe. The approximate location of the outfall is 36 m from the northern boundary of the wider site. See Appendix D for the calculations and sizing for the attenuation pond and the freeboard and Appendix C for a plan of the drainage design.
- 11.5 The alternative to attention drainage would be drainage into the existing pond on site B, this has been discounted due to the higher elevation of the pond.
- 11.6 Additional sustainable drainage (SuDS) measures would be permeable paving for the external parking and access areas, rainwater harvesting and a green roof, this would reduce the impermeable area of the site.
- 11.7 Foul sewage from the redeveloped site will be drained to the public foul sewer at the Southern Water Chamber FW.IC.A (invert level 41.8 m AOD) located in the new Taylor Wimpey estate on the eastern boundary of the site. This requires

the use of a pumping station to pump the foul water from FW1 to the FW IC A via a rising main of 70 mm diameter, the pump will be located in FW1 (the downstream chamber before the southern Water chamber. See Appendix C for a plan of the connection to the Taylor Wimpey chamber and drainage system.

12. RECOMMENDATIONS

- 12.1 It is recommended that the external paved areas should have permeable surfacing laid over a drainage blanket of no-fines stone wrapped in permeable geotextile.
- 12.2 A capacity check should be commissioned from Southern Water to confirm that there is adequate capacity in the existing foul sewer for the increased flow from the redeveloped site.

☆☆☆☆☆☆

APPENDIX A

Figure 1:	Site location
Figure 2:	Existing Site Layout
Figure 3:	Proposed Redevelopment
Figure 4:	Proposed Elevation
Figure 5:	Proposed Roof Plan
Figure 6:	Proposed Floor Plan
Figure 7:	Site Photographs
Figure 8:	Map of Tributary Connection to Watercourse

Figure 1: Site Location

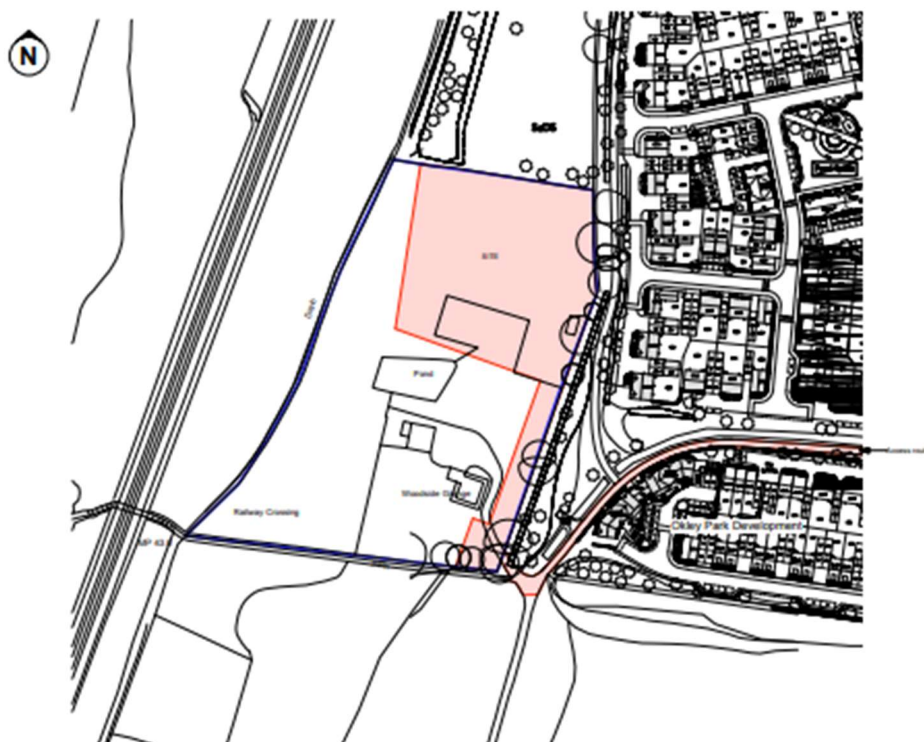


Figure 2: Existing Site Layout

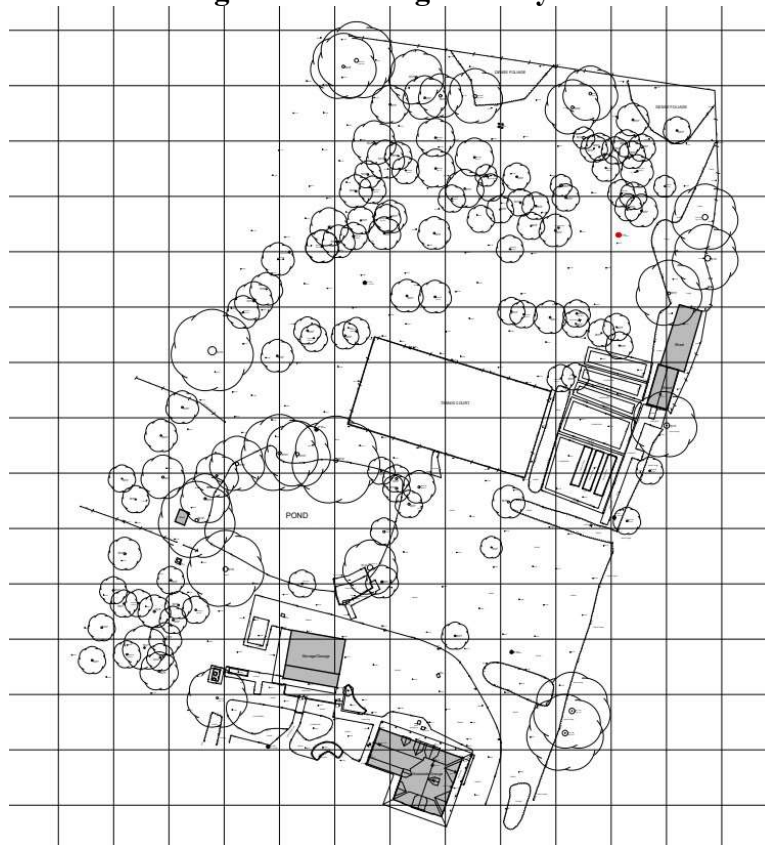
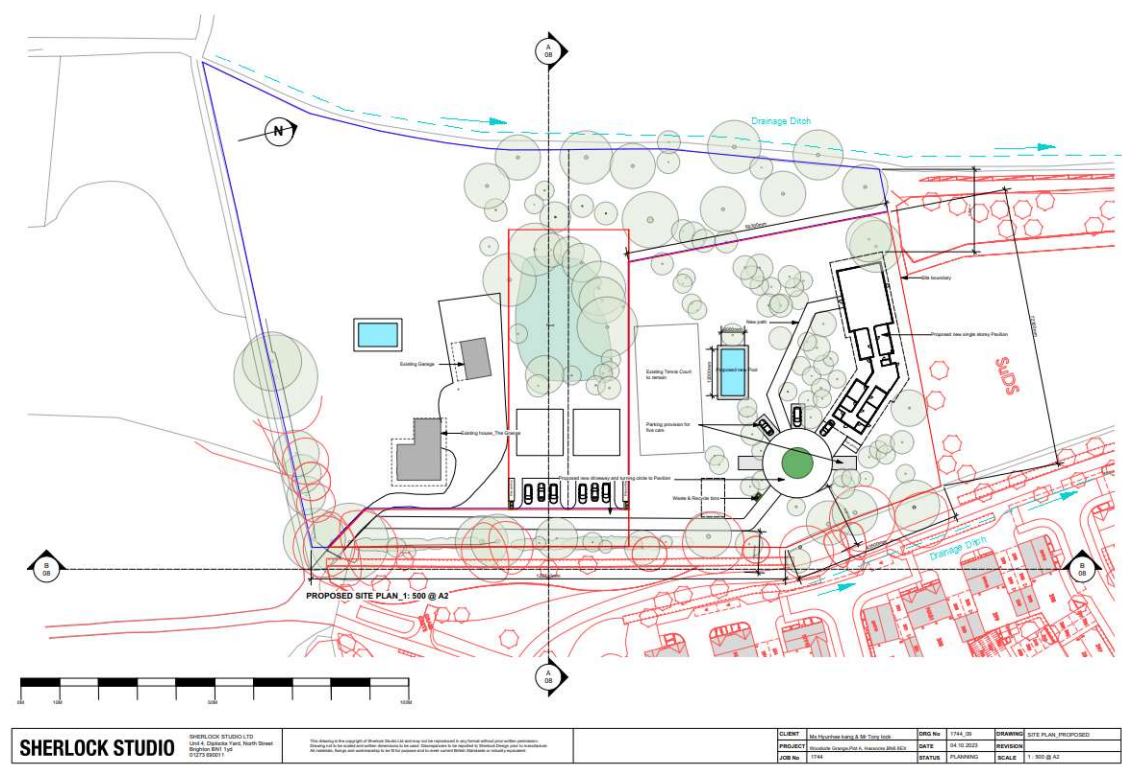
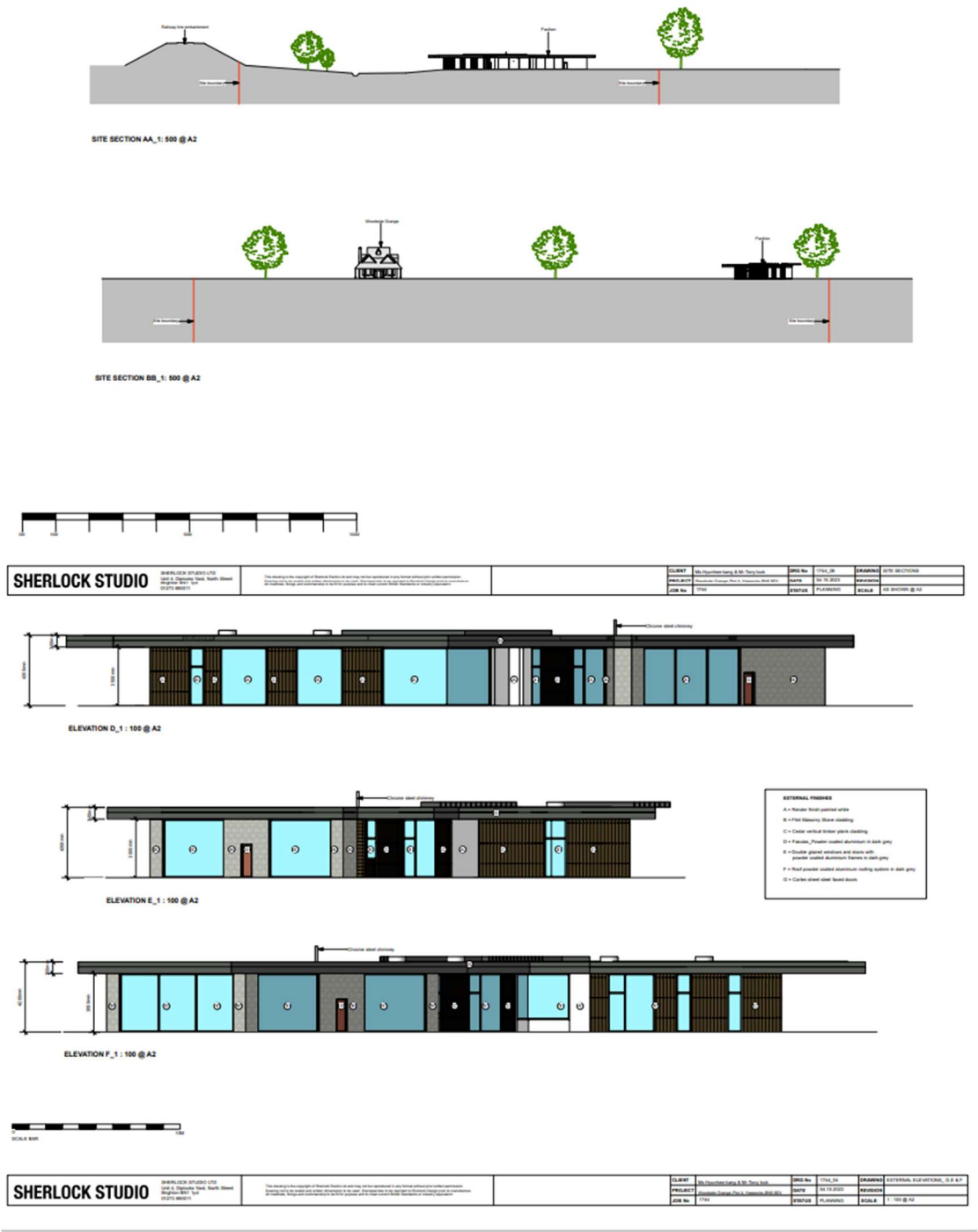


Figure 3: Proposed redevelopment



(Note the distance from the proposed parking area and road and the drainage ditch on the Taylor Wimpey site to the east of the site is greater than 5m)

Figure 4: Proposed Elevation



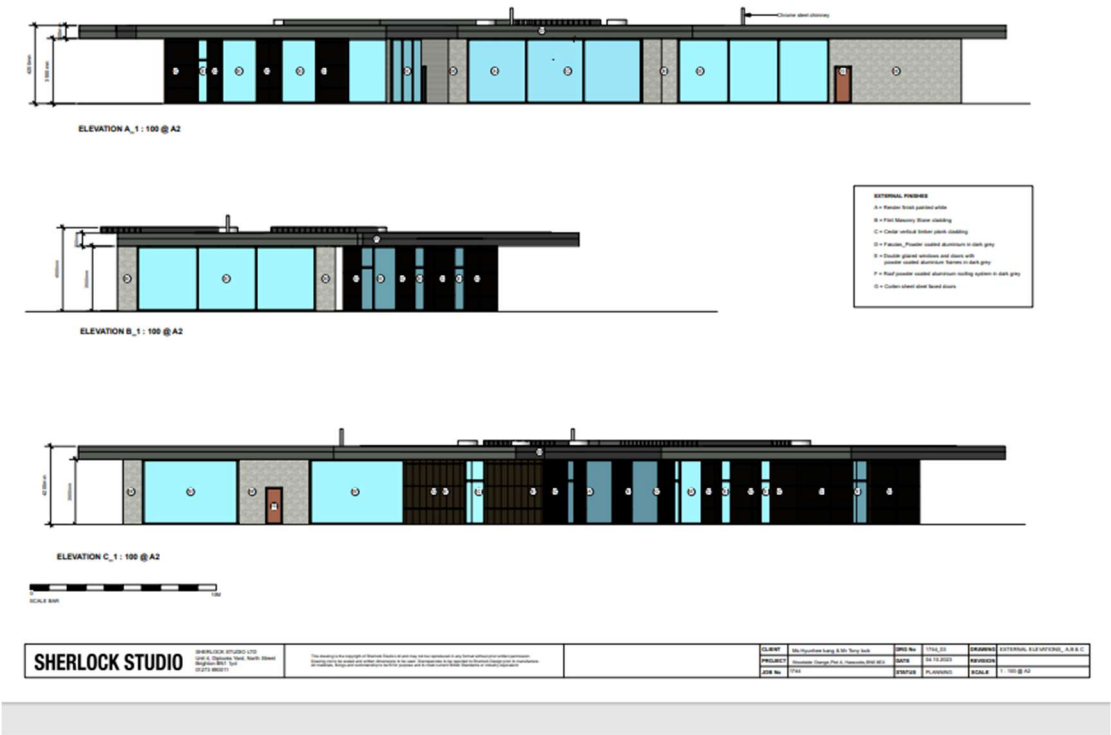


Figure 5: Proposed Roof Plan

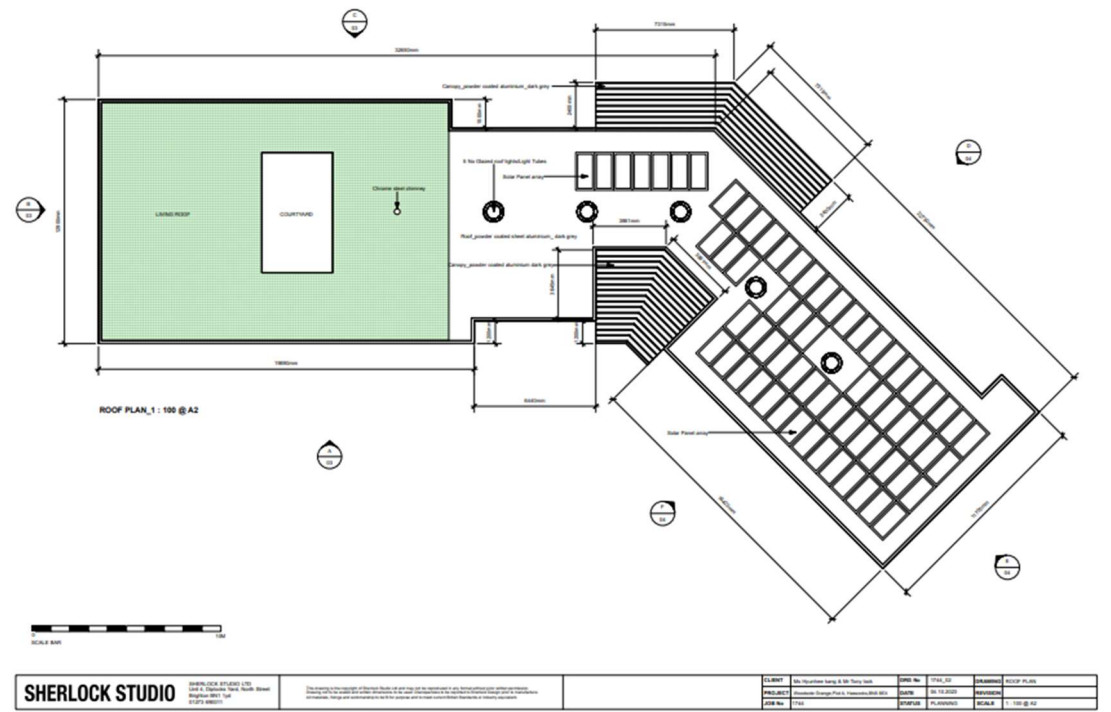


Figure 6: Proposed Floor Plan

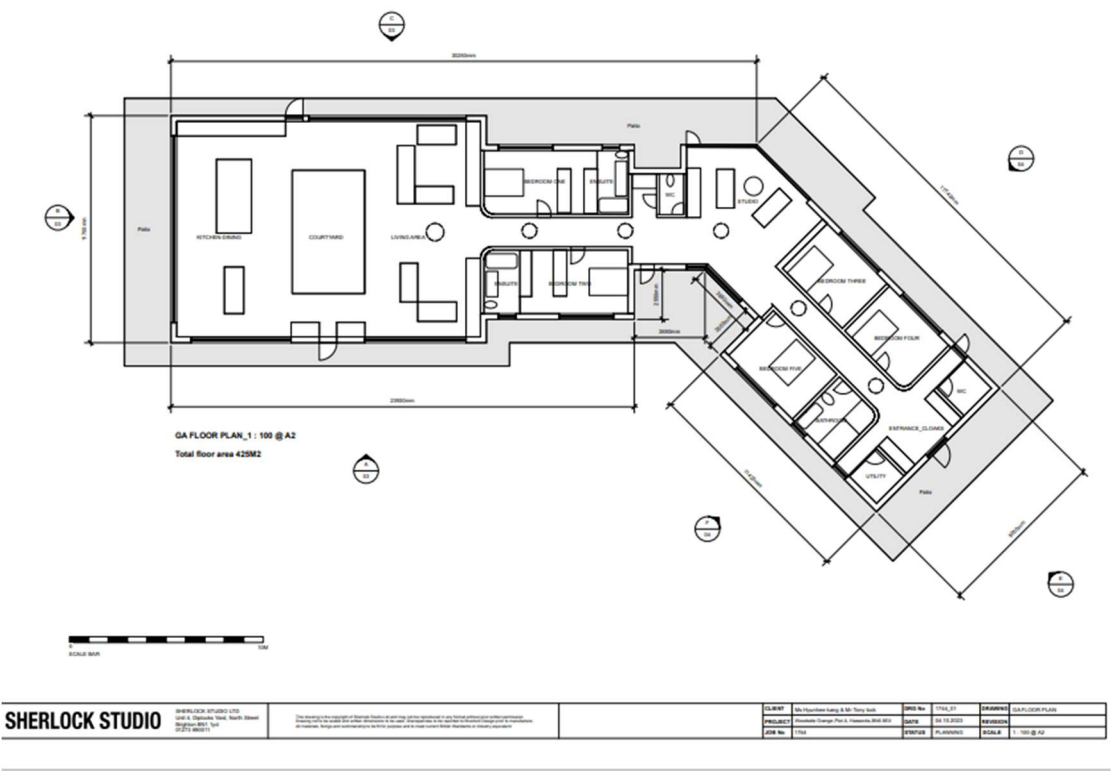


Figure 7: Site Photographs



Existing house



Proposed Site A (west)



Proposed Site A (south, facing tennis court and site B)



Area of hardstanding (south eastern edge of site)



Site of proposed Taylor Wimpey SuDS drainage pond



Taylor Wimpey construction site showing path/location of foul drainage



Drainage construction on Taylor Wimpey site (ditch on the eastern site boundary - greater than 5m from the construction on Site A)



Existing pond located on Site B



Ditch and boundary fence to the west of the site.



Ditch/tributary showing culvert under the footpath southeastern edge of site



Drainage ditch to the east of the site (proposed overflow ditch from the attenuation pond)



The drainage ditch to the east of the site flowing underground northwards



Top of chamber showing the northwards flow of the drainage ditch



Internal view of the 1st chamber



Internal view of the 2nd chamber



Internal view of the 3rd chamber

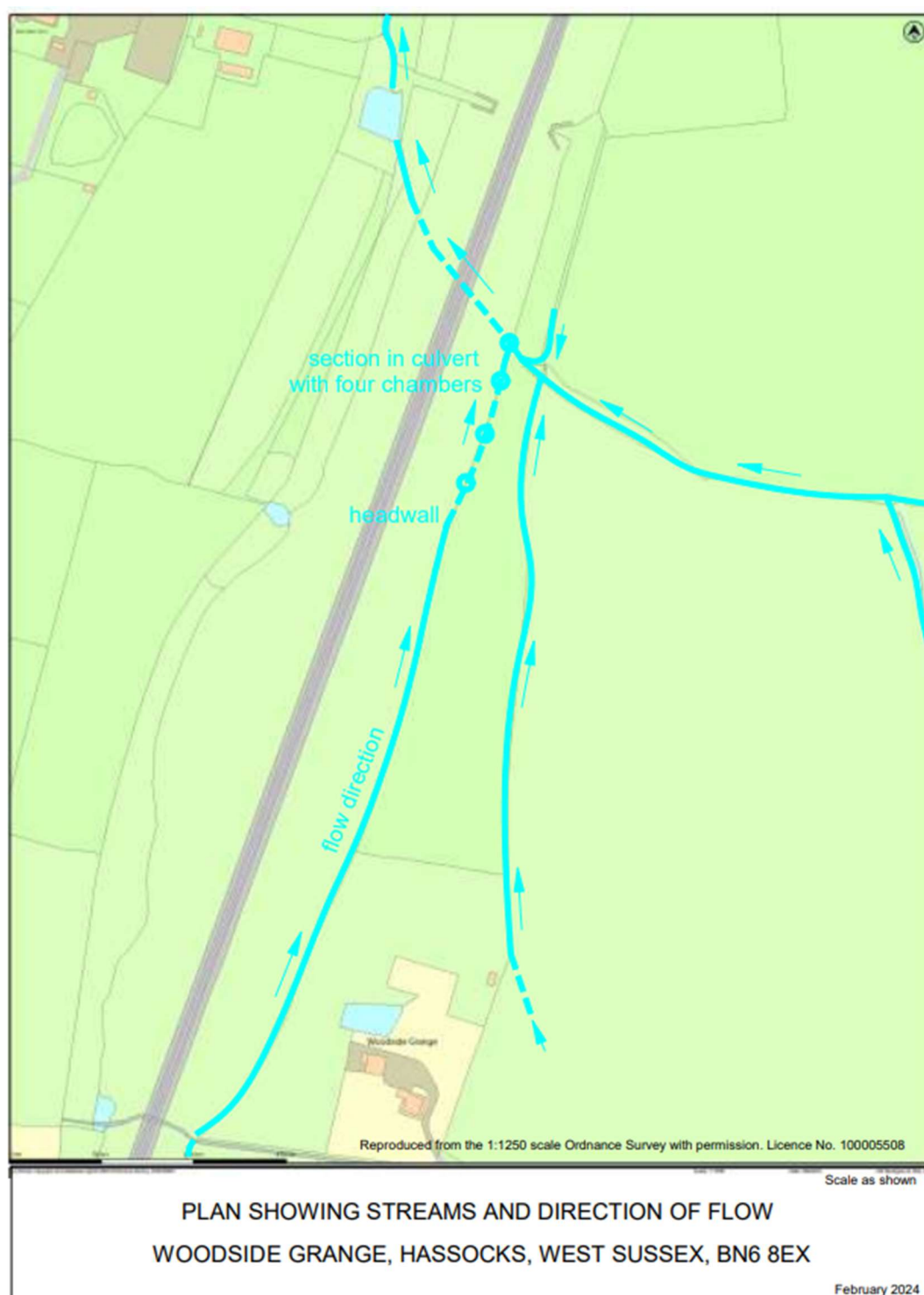


Internal view of the 4th chamber, with drainage ditch from the Taylor Wimpey site (from the east) joining the watercourse (already draining from the south, past the Site A). The watercourse flows northwest under in embankment via this chamber.



Taylor Wimpey ditch flowing from the east.

Figure 8: Map of Drainage Ditch Connection to Watercourse

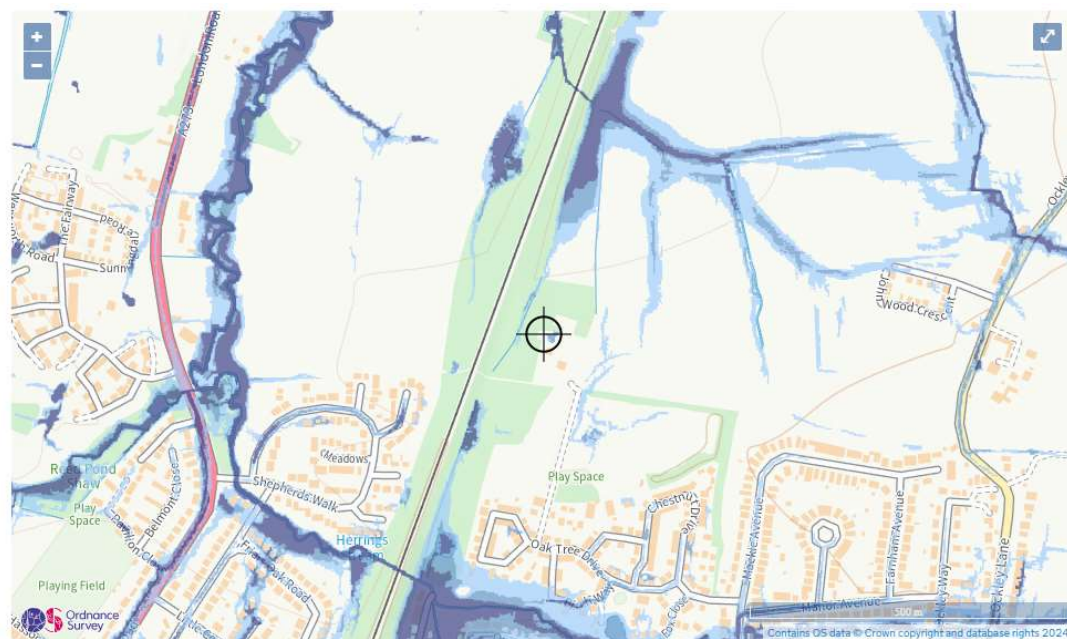


APPENDIX B

Environment Agency Flood Mapping



Site in Flood Zone 1



Extent of flooding from surface water

● High ● Medium ● Low ○ Very low ⊕ Location you selected

APPENDIX C: Proposed Drainage Design

Appendix D: Output From Drainage Design Software



MasterDrain
SW

Environmental Assessment Services Ltd

<http://www.easltd.co.uk>

London Rd, Hickstead
Haywards Heath,
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Tel: 01444 882552
email: info@easltd.co.uk

Job No. 654		
Sheet no. 1		
Date 14/08/25		
By XL	Checked	Reviewed

Project Site A Woodside Grange	
Title Surcharge calcs (Sized at 30 yrs storm) for WOOSIDE GRANGE SITE A.SW CCF = 45%	

Free flowing outlet
Return period = 100 yrs
Climate change factor = 45

PEAK hydrograph values printed
Storm duration = 15 mins
No offline storage

Mean rain intens. 140.00 mm/hr
Storm profile = Summer
Using FSR data

Peak rain intens. 548.80 mm/hr
Sample period = 7.5 secs.

Entry No.	SECT. No.	MANHOLE REF	PIPE CAPACITY l/s	RATE FLOW l/s	PIPE SIZE mm	CHAMBER DIAM/LxW mm	INVERT LEVEL m	WATER LEVEL m	GRND LEVEL m	SURCHARGE		EXCESS FLOW l/s	FLOODED VOL m³	DRAINED AREA (m²) :x FACTOR	STATUS	
1	I	1.01	SW4	51.9	28.2	225	600	42.16	42.28	42.66	0.54	0.12	0.00	0.000	424	OK
2	I	1.02	SW3	104.4	44.5	225	600	41.85	41.95	42.74	0.43	0.10	0.00	0.000	667	OK
3	I	1.03	SW2	64.5	59.2	225	450	40.90	41.09	41.62	0.92	0.19	0.00	0.000	888	OK
4	I	1.04	SW1	71.5	75.0	225	450	40.40	40.79	41.50	1.05	0.39	3.43	0.000	1124	Surch.
5	B	2.01	SW5	15.2	6.8	100	450	41.95	42.00	42.45	0.44	0.05	0.00	0.000	101	OK
6	I	1.05	SW6	106.6	81.7	225	900	40.00	40.16	41.90	0.77	0.16	0.00	0.000	1226	OK
7	I	1.06	POND#	336.8	93.8	300	15600	39.10	39.20	41.53	0.28	0.10	0.00	0.000	1407	OK



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Job No. 654		
Sheet no. 2		
Date 14/08/25		
By XL	Checked	Reviewed

Project	Site A Woodside Grange
Title	Surcharge calcs (Sized at 30 yrs storm) for WOOSIDE GRANGE SITE A.SW CCF = 45%

Notes

Printout headings

- | | | |
|---|---|--|
| 1) Entry no - position in file | 2) Section no - pipe identifier | 3) Manhole ref - Manhole identifier |
| 4) Pipe cap - full bore capacity of that pipe | 5) Rate of flow - calculated flow rate (l/s) ± = flow restrictor. | 6) Pipe diam - outlet pipe diameter (mm) |
| 7) Chamber diam - chamber diam. at base of MH | 8) Invert level - invert level of manhole | 9) Water level - calculated peak water level. |
| 10) Grnd level - ground / cover level | 11) Surch. fract - calc.flow/pipe capacity | 12) Surch. depth - surcharge level above soffit |
| 13) Overflow - surcharged flow rate (l/s) | 14) Flooded vol - volume of water above cover | 15) Upstrm Vol - upstream pipe vol to previous manhole(s) |
| 16) Status - OK - outlet not surcharged | 17) Status - Surcharged - outlet surcharged | 18) Status - Warning - water level within 299mm of cover level |
| 19) Status - Flooded - cover over-topped | 20) § against diameter indicates throttle pipe used. | |

Title box

Hydrograph data

- | | | |
|---|--------------------------------------|--|
| 1) Ret. period - that used to calculate profile | 2) Duration - length of storm (mins) | 3) Profile - either Winter (75%) or Summer (50%) |
|---|--------------------------------------|--|

Flow restrictors

Manhole reference - POND

Restrictor flowrate - 4 l/s

Flood volumes

Check that the upstream storage of the manhole is adequate to take the flood volume - see Upstrm Vol above.



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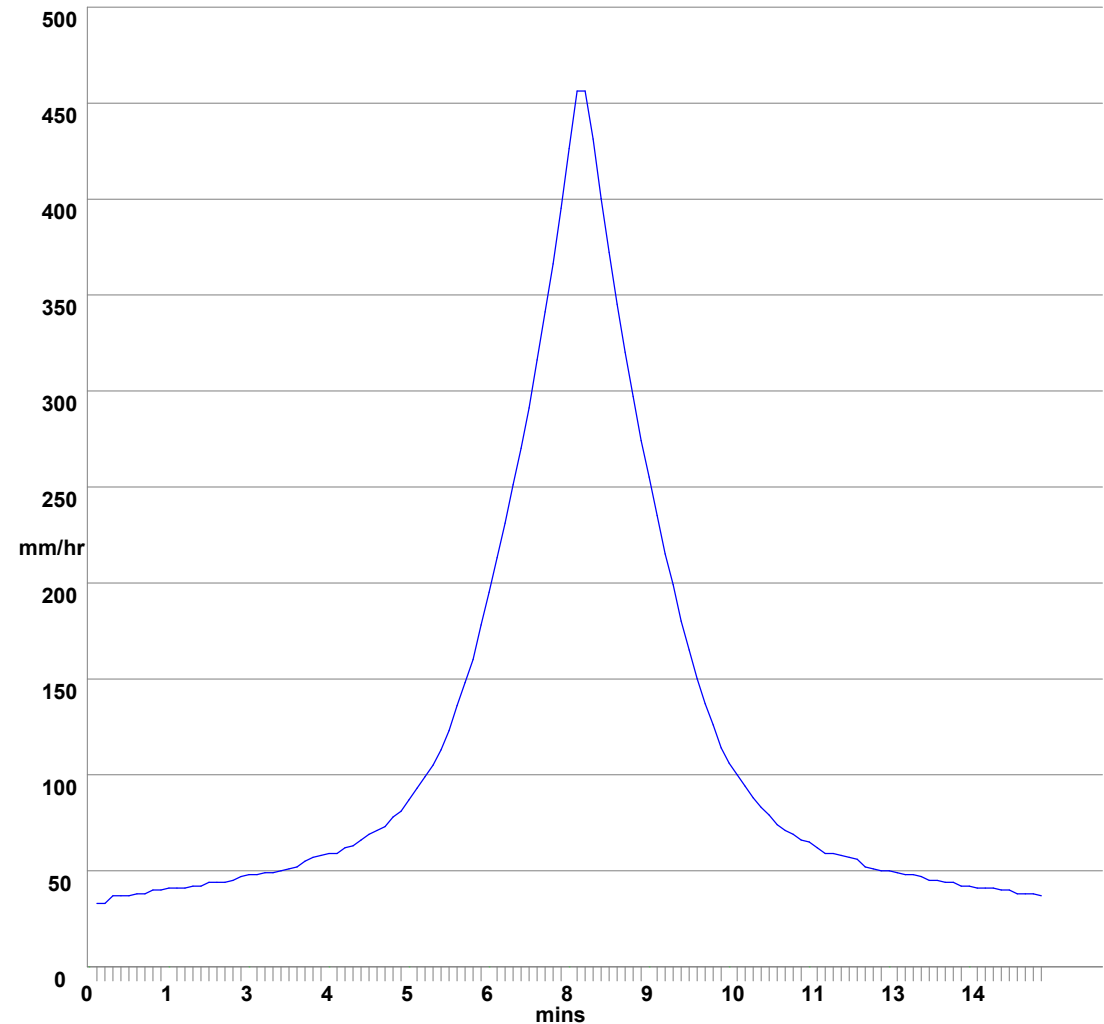
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SW

Project Site A Woodside Grange
Title Surcharge calcs (Sized at 30 yrs storm) for WOOSIDE GRANGE SITE A.SW CCF = 45%

Time mins	Rain mm/hr	Time mins	Rain mm/hr	Time mins	Rain mm/hr
0:08	32.56	5:08	93.04	10:08	100.02
0:15	32.56	5:15	98.85	10:15	94.20
0:22	37.22	5:22	104.67	10:22	88.39
0:30	37.22	5:30	112.81	10:30	82.57
0:38	37.22	5:38	123.28	10:38	79.08
0:45	38.38	5:45	136.07	10:45	74.43
0:52	38.38	5:52	147.70	10:52	70.94
1:00	39.54	6:00	160.49	11:00	68.62
1:08	39.54	6:08	177.94	11:08	66.29
1:15	40.70	6:15	195.38	11:15	65.13
1:22	40.70	6:22	212.83	11:22	61.64
1:30	40.70	6:30	231.43	11:30	59.31
1:38	41.87	6:38	251.20	11:38	59.31
1:45	41.87	6:45	269.81	11:45	58.15
1:52	44.19	6:52	290.75	11:52	56.99
2:00	44.19	7:00	316.33	12:00	55.82
2:08	44.19	7:08	340.75	12:08	52.33
2:15	45.36	7:15	366.34	12:15	51.17
2:22	46.52	7:22	395.41	12:22	50.01
2:30	47.68	7:30	425.65	12:30	50.01
2:38	47.68	7:38	455.89	12:38	48.85
2:45	48.85	7:45	455.89	12:45	47.68
2:52	48.85	7:52	431.47	12:52	47.68
3:00	50.01	8:00	400.07	13:00	46.52
3:08	51.17	8:08	372.16	13:08	45.36
3:15	52.33	8:15	345.41	13:15	45.36
3:22	54.66	8:22	319.82	13:22	44.19
3:30	56.99	8:30	296.56	13:30	44.19
3:38	58.15	8:38	274.46	13:38	41.87
3:45	59.31	8:45	254.69	13:45	41.87
3:52	59.31	8:52	234.92	13:52	40.70
4:00	61.64	9:00	215.15	14:00	40.70
4:08	62.80	9:08	198.87	14:08	40.70
4:15	66.29	9:15	180.26	14:15	39.54
4:22	68.62	9:22	165.14	14:22	39.54
4:30	70.94	9:30	150.03	14:30	38.38
4:38	73.27	9:38	137.23	14:38	38.38
4:45	77.92	9:45	125.60	14:45	38.38
4:52	81.41	9:52	113.97	14:52	37.22
5:00	87.22	10:00	105.83	15:00	37.22

Hydrograph profile derived from data in the Flood Studies Report

Return period= 100 yrs Duration= 15 mins Profile - summer





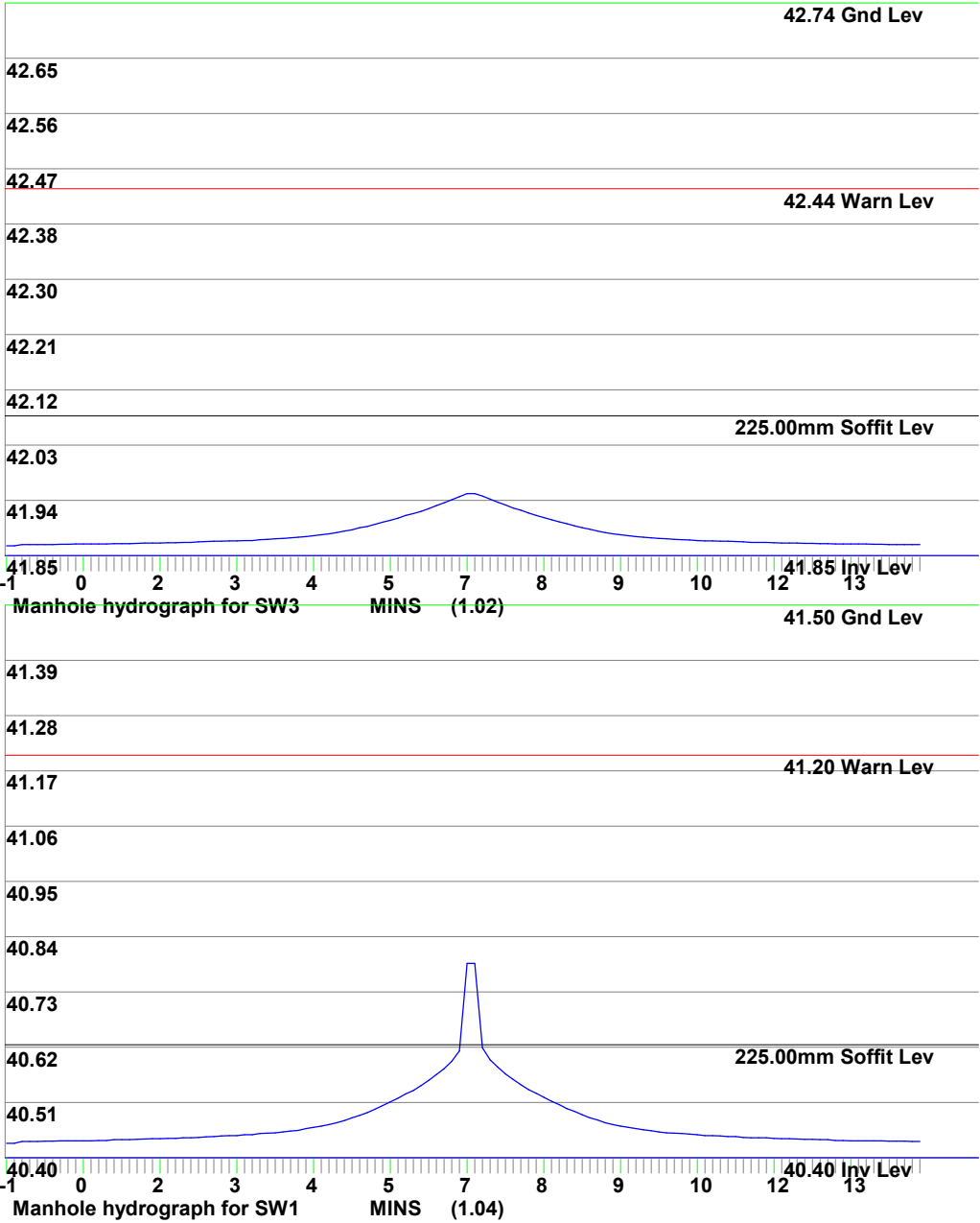
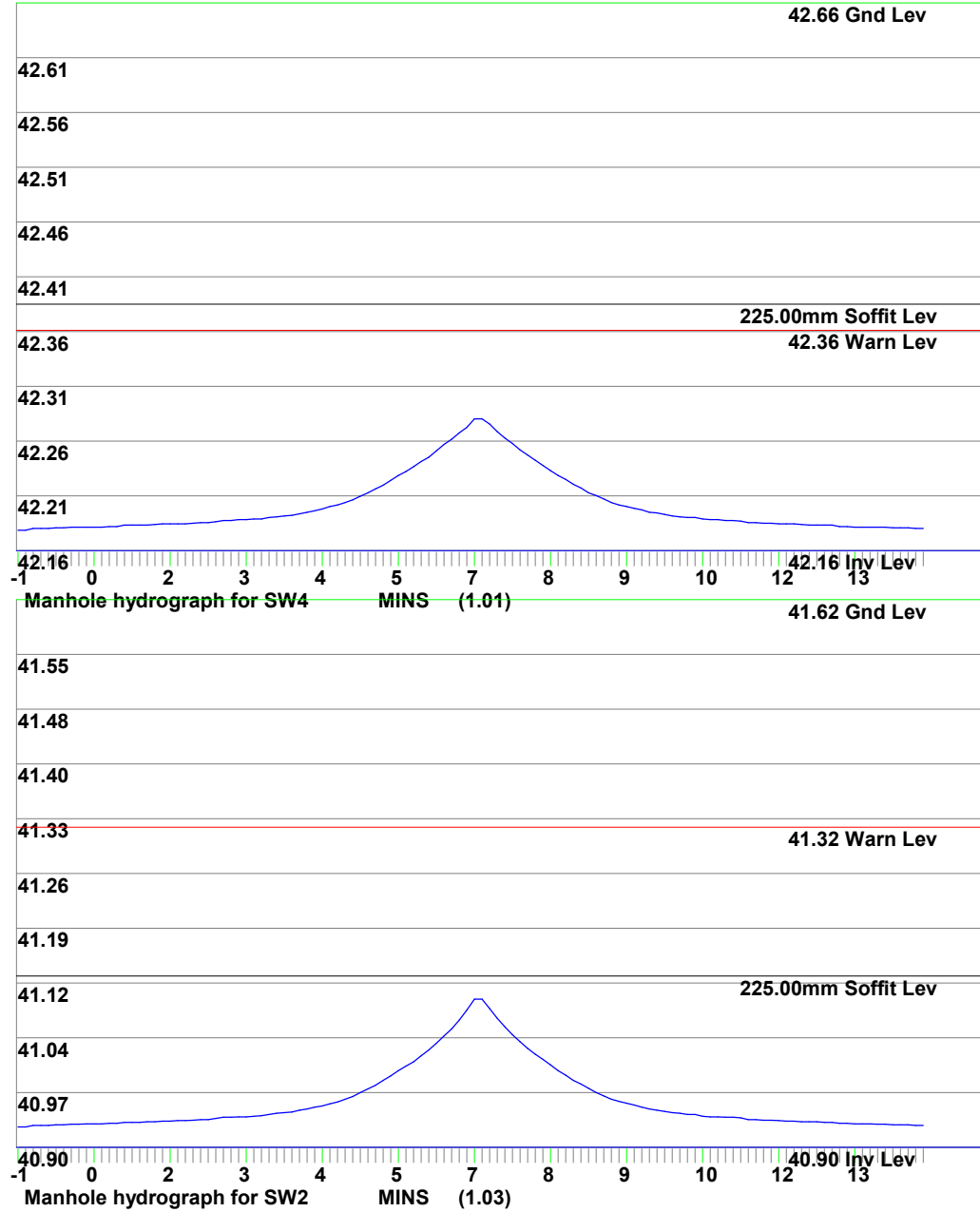
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Job No. 654		
Sheet no. 4		
Date 14/08/25		
By XL	Checked	Reviewed

Project Site A Woodside Grange	By XL	Checked	Reviewed
Title Surcharge calcs (Sized at 30 yrs storm) for WOOSIDE GRANGE SITE A.SW CCF = 45%			

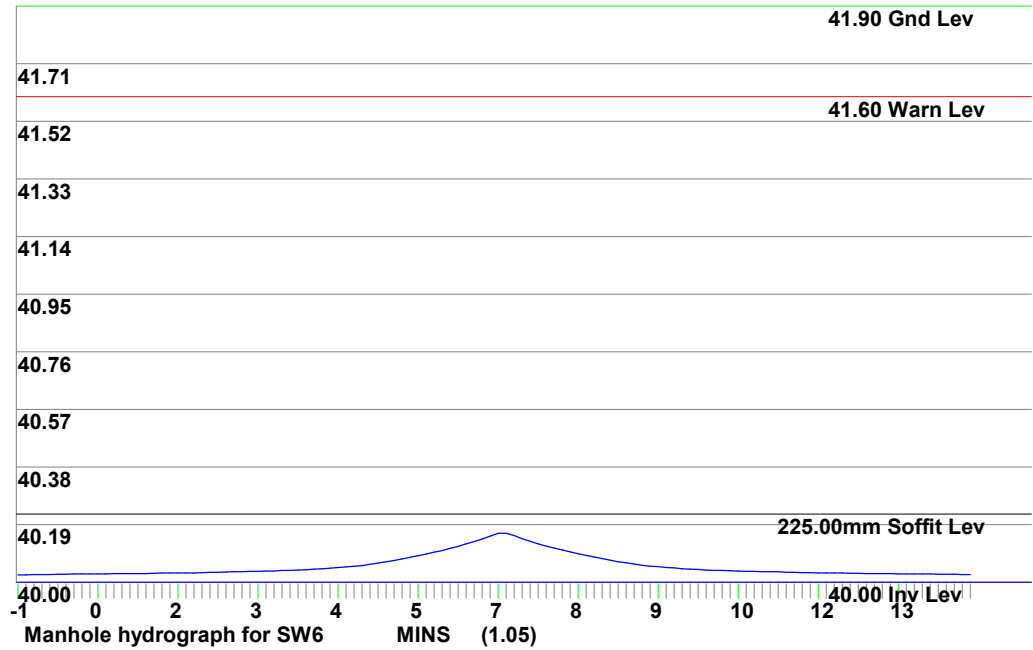




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Project	Site A Woodside Grange
Title	Surcharge calcs (Sized at 30 yrs storm) for WOOSIDE GRANGE SITE A.SW CCF = 45%





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Project Site A Woodside Grange				By XL	Checked	Reviewed
Title Surcharge calcs (Sized at 30 yrs storm) for WOOSIDE GRANGE SITE A.SW CCF = 45%						



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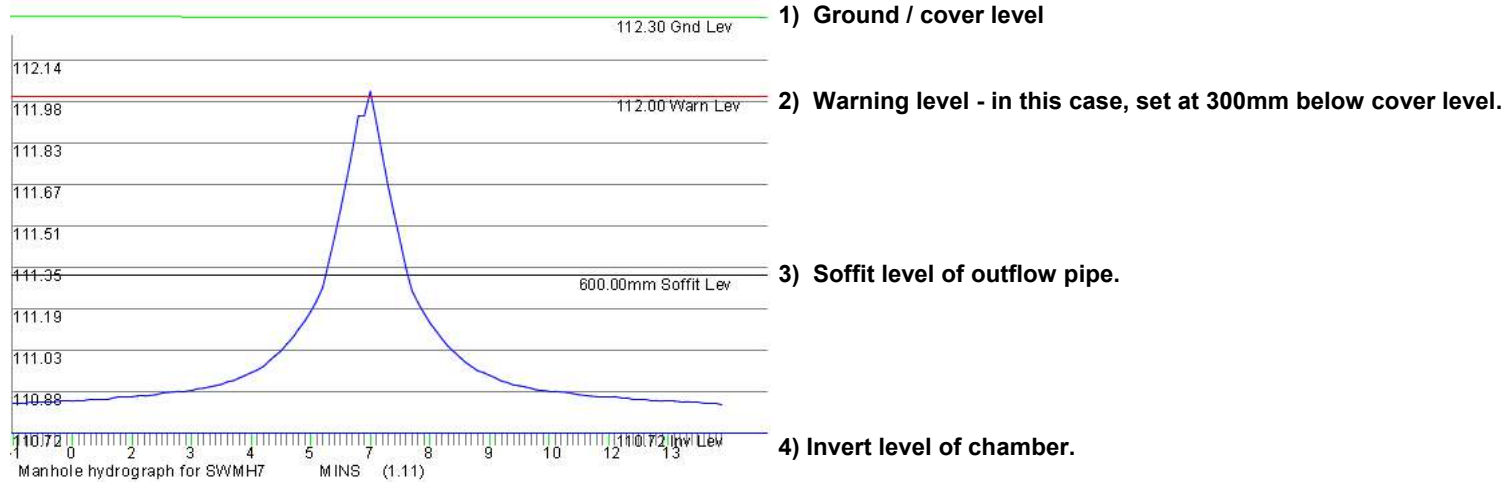
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Sheet no. 7		
Date 14/08/25		
By XL	Checked	Reviewed

Project
Site A Woodside Grange

Title
Surcharge calcs (Sized at 30 yrs storm) for WOOSIDE GRANGE SITE A.SW CCF = 45%



Notes

- Lower section of the graph shows the water depth filling the channel. Channel is assumed to be a full pipe diameter in depth.
- Upper section of the graph shows the water depth filling the chamber. Chamber has a greater width/diameter than the channel, so increases in depth are proportionally less.
- The top of the graph clips the warning level and would be marked thus on the printout.
- In many cases the invert of the offline storage is required to enter at the channel soffit level, meaning that the pipe will still surcharge but flooding risk reduced.
- The diagram above is a general one and is not part of the current calculation.

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		Date 14/08/25 12:00		
Project Site A Woodside Grange		By XL	Checked	Reviewed
Title Manhole printout for WOOSIDE GRANGE SITE A.SW				

Manhole ref.	X ref	Y ref	Form	Cham diam. or length	Rect. width	Chamb. height	Shaft height	MH Volume	Exit Diam	Exit Crown	Exit Invert	Chamb const	Slab level	Cover Level	Dwnstr MH
SW4	192.24	8.56	circ	600	N/A	0.50		0.14	225	42.410	42.160	N/A	42.110	42.660	SW3
SW3	223.28	7.76	circ	600	N/A	0.89		0.25	225	42.100	41.850	N/A	41.800	42.740	SW2
SW2	235.02	28.06	circ	450	N/A	0.72		0.11	225	41.150	40.900	N/A	40.850	41.620	SW1
SW1	229.05	59.9	circ	450	N/A	1.10		0.17	225	40.650	40.400	N/A	40.350	41.500	SW6
SW5	198.41	33.03	circ	450	N/A	0.50		0.08	100	42.075	41.950	N/A	41.900	42.450	SW6
SW6	208.16	62.49	circ	900	N/A	1.90		1.21	225	40.250	40.000	N/A	39.950	41.900	POND
POND	188.46	54.53	circ	15600	N/A	2.43		464.46	300	39.425	39.100	N/A	39.050	41.530	DITCH

All manholes to be as per 'Sewers for Adoption'

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Date
14/08/25 12:00

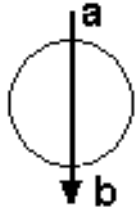
Project
Site A Woodside Grange

Title
Manhole printout for WOOSIDE GRANGE SITE A.SW

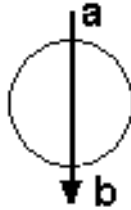
By
XL

Checked

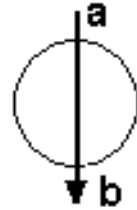
Reviewed



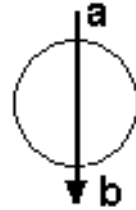
MH = SW4
a=head
b = 225



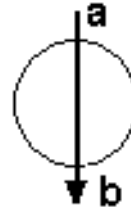
MH = SW3
a= 225
b = 225



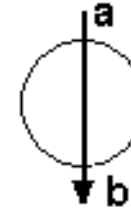
MH = SW2
a= 225
b = 225



MH = SW1
a= 225
b = 225



MH = SW5
a=head
b = 100



MH = SW6
a= 225
b = 225
c = 100

MH = POND
a= 225
b = 300

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Project Site A Woodside Grange		By XL	Checked	Reviewed	
Title Manhole printout for WOOSIDE GRANGE SITE A.SW					

These explanatory notes should be read in conjunction with the Manhole printout

- 1) Manhole ref - the reference for the manhole in question
- 2) Form - either circular or rectangular
- 3) Chamber diam or length - diameter of chamber if circular, or length if rectangular - in metres
- 4) Rectangular width - if form is rectangular, this gives the width of the chamber
- 5) Chamber height - height of chamber from invert
- 6) Shaft height - height of access shaft if applicable, from top of chamber to ground level
- 7) Manhole volume - cubic capacity of manhole chamber
- 8) Exit diameter - diameter of pipe leaving this manhole
- 9) Exit crown - crown level of pipe leaving this manhole
- 10) Exit invert - invert level of pipe leaving this manhole
- 11) Chamb const - manhole construction :-
 - N/A - not specified
 - Conc - concrete rings
 - Poly - polypropylene rings surrounded by selected fill
 - Poly/conc - polypropylene rings surrounded by concrete
 - Brick - brick built with waterproof render
 - In situ - concrete cast in situ
- 12) Slab level - surface level of base slab (if no sump, this is placed at 50mm below exit invert to allow levels manipulation).
- 13) Manhole diameters of 9999 have exceeded the sizes available in SfA
- 14) Manhole diameters may have to be modified due to the number of branches



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Project Site A Woodside Grange
Title Hydrograph storage analysis (Winter profile) for HASSOCKS

Data:-

Location = HASSOCKS Grid reference = TQ3015
M5-60 (mm) = 20.4 r = 0.35
Soil index = 0.45 SAAR (mm/yr) = 910
Return period = 100 WRAP = 4
UCWI = 0.0 Climate change = +45%

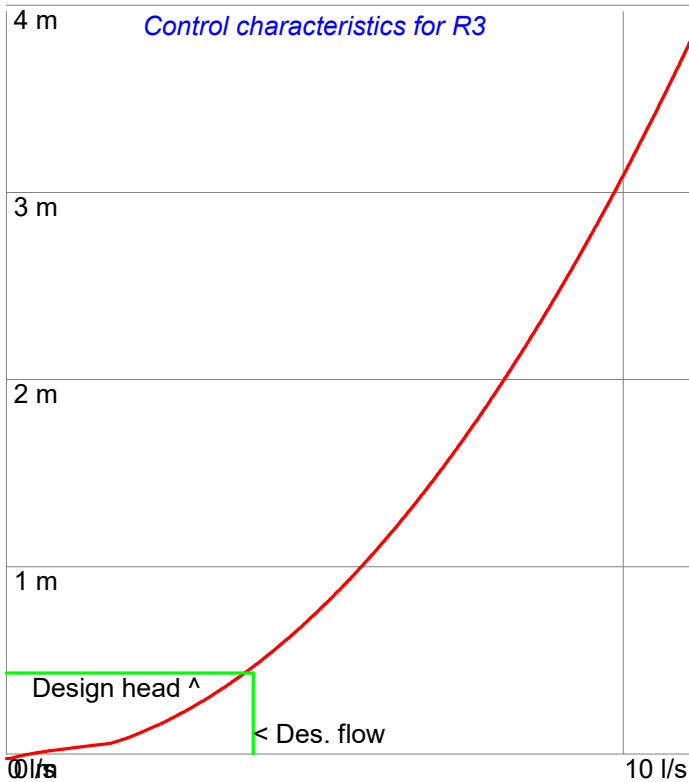
Clayey, or loamy over clayey soils with an impermeable layer at shallow depth.

Percentage runoff = 100.0% (manual setting)

Imperv. area = 1290 m² Pervious area = 76 m²
Total area = 1366 m² Equiv area = 1366 m² (Tot. area x % runoff).
Total runoff = 126.9 m³ Discharge rate = 4.000 l/s

Design Head = 0.5m Peak flow = 4.08 l/s
Control device = R3 Orifice diam = 66.5 mm
Max. calc. depth = 0.52 m Available depth = 0.0 m³
Mean discharge = 2.03 l/s

Available system storage = 0.00 m³ under a system plane at the design head level.
Offline storage = 0.0 m³
Total storage = 88.0 m³ Peak input flow = 17.63 l/s



Head (m)	Flow (l/s)	Head (m)	Flow (l/s)
0.01	0.06	2.01	8.02
0.05	0.70	2.05	8.10
0.10	1.79	2.10	8.20
0.15	2.19	2.15	8.29
0.20	2.53	2.20	8.39
0.25	2.83	2.25	8.49
0.30	3.10	2.30	8.58
0.35	3.35	2.35	8.67
0.40	3.58	2.40	8.76
0.45	3.79	2.45	8.85
0.50	4.00	2.50	8.94
0.55	4.20	2.55	9.03
0.60	4.38	2.60	9.12
0.65	4.56	2.65	9.21
0.70	4.73	2.70	9.30
0.75	4.90	2.75	9.38
0.80	5.06	2.80	9.47
0.85	5.22	2.85	9.55
0.90	5.37	2.90	9.63
0.95	5.51	2.95	9.72
1.00	5.66	3.00	9.80
1.05	5.80	3.05	9.88
1.10	5.93	3.10	9.96
1.15	6.07	3.15	10.04
1.20	6.20	3.20	10.12
1.25	6.32	3.25	10.20
1.30	6.45	3.30	10.28
1.35	6.57	3.35	10.35
1.40	6.69	3.40	10.43
1.45	6.81	3.45	10.51
1.50	6.93	3.50	10.58
1.55	7.04	3.55	10.66
1.60	7.16	3.60	10.73
1.65	7.27	3.65	10.81
1.70	7.38	3.70	10.88
1.75	7.48	3.75	10.95
1.80	7.59	3.80	11.03
1.85	7.69	3.85	11.10
1.90	7.80	3.90	11.17
1.95	7.90	3.95	11.24
2.00	8.00	4.00	11.31

Calculation data provided by Crown Water Ltd, SL5 7NT



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By	XL	Checked	Reviewed
Title			

Project **Site A Woodside Grange**

Title **Hydrograph storage analysis (Winter profile) for HASSOCKS**

Storage curves for a 5 hours storm.

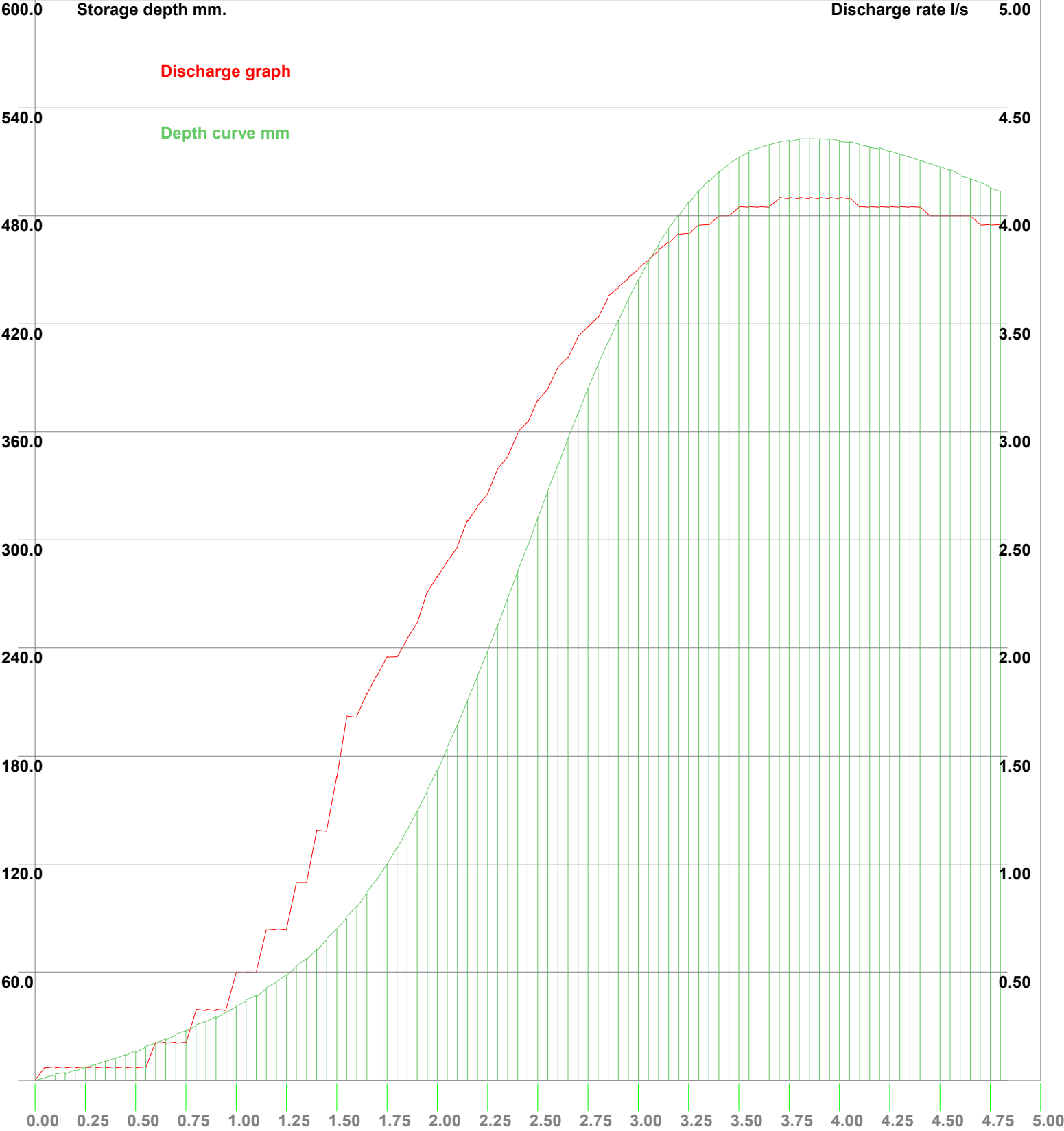
Storage depth mm.

Discharge rate l/s

5.00

Discharge graph

Depth curve mm





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				Sheet no. 3		
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Project Site A Woodside Grange				By XL	Checked	Reviewed
Title Hydrograph storage analysis (Winter profile) for HASSOCKS						

Incremental rainfall figures.

Storm Mins	Storage Depth mm	Control Flow l/s	Storm Mins	Storage Depth mm	Control Flow l/s
3.0	1.4	0.06	153.0	327.1	3.20
6.0	2.8	0.06	156.0	341.8	3.30
9.0	4.3	0.06	159.0	356.3	3.35
12.0	5.7	0.06	162.0	370.5	3.44
15.0	7.2	0.06	165.0	384.2	3.49
18.0	8.8	0.06	168.0	397.4	3.53
21.0	10.5	0.06	171.0	410.1	3.62
24.0	12.3	0.06	174.0	422.2	3.67
27.0	14.2	0.06	177.0	433.8	3.71
30.0	16.2	0.06	180.0	444.7	3.75
33.0	18.4	0.06	183.0	454.8	3.79
36.0	20.6	0.18	186.0	464.1	3.84
39.0	22.8	0.18	189.0	472.7	3.88
42.0	25.1	0.18	192.0	480.6	3.92
45.0	27.5	0.18	195.0	487.7	3.92
48.0	30.1	0.32	198.0	494.0	3.96
51.0	32.6	0.32	201.0	499.7	3.96
54.0	35.1	0.32	204.0	504.6	4.00
57.0	37.8	0.32	207.0	508.7	4.00
60.0	40.7	0.50	210.0	512.2	4.04
63.0	43.8	0.50	213.0	515.2	4.04
66.0	47.1	0.50	216.0	517.6	4.04
69.0	50.7	0.70	219.0	519.5	4.04
72.0	54.5	0.70	222.0	520.9	4.08
75.0	58.5	0.70	225.0	521.9	4.08
78.0	62.9	0.91	228.0	522.5	4.08
81.0	67.5	0.91	231.0	522.8	4.08
84.0	72.7	1.15	234.0	522.7	4.08
87.0	78.0	1.15	237.0	522.4	4.08
90.0	83.9	1.41	240.0	521.7	4.08
93.0	90.1	1.68	243.0	520.8	4.08
96.0	96.6	1.68	246.0	519.7	4.04
99.0	103.9	1.79	249.0	518.4	4.04
102.0	111.8	1.88	252.0	517.1	4.04
105.0	120.2	1.96	255.0	515.7	4.04
108.0	129.3	1.96	258.0	514.2	4.04
111.0	139.1	2.04	261.0	512.5	4.04
114.0	149.5	2.12	264.0	510.7	4.04
117.0	160.5	2.26	267.0	508.9	4.00
120.0	172.2	2.33	270.0	507.1	4.00
123.0	184.5	2.40	273.0	505.1	4.00
126.0	197.4	2.47	276.0	502.9	4.00
129.0	210.6	2.59	279.0	500.7	4.00
132.0	224.3	2.65	282.0	498.3	3.96
135.0	238.4	2.71	285.0	496.0	3.96
138.0	252.8	2.83	288.0	493.5	3.96
141.0	267.5	2.88	291.0	491.0	3.96
144.0	282.5	2.99	294.0	488.4	3.92
147.0	297.4	3.05	297.0	485.9	3.92
150.0	312.3	3.15	300.0	483.3	3.92

Using the Get Max button causes the program to step through a series of storm durations until a maximum volume is obtained.

Each duration is sampled 600 times and the results recorded. The storm durations (hrs) are:-

0.25, 0.5, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 12, 15, 18, 20, 24, 30, 36, 42, 48, 54, 60, 66, 72, 84, 96, 120, 150, 175, 200, 250, 300, 375, 500, 750, 1000, 1250, 1500, 1570, 2000, 2500, 3000, 3500, 4000

It should be noted that the six hour storm frequently requested rarely demonstrates the worst case for storage.



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Job No.	654		
Sheet no.	4		
Date	14/08/25		
By	XL	Checked	Reviewed

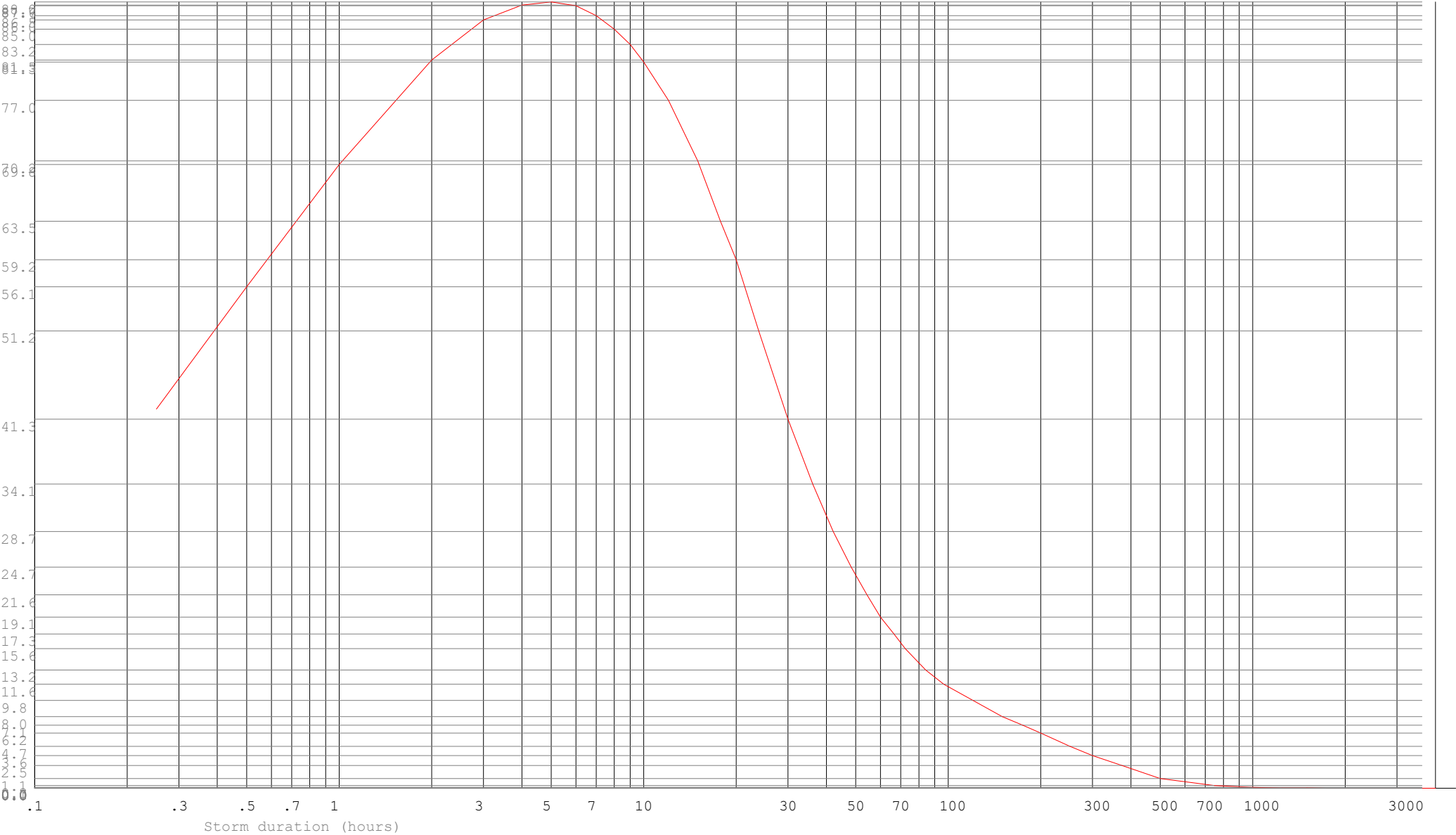
MasterDrain
SW

Project Site A Woodside Grange

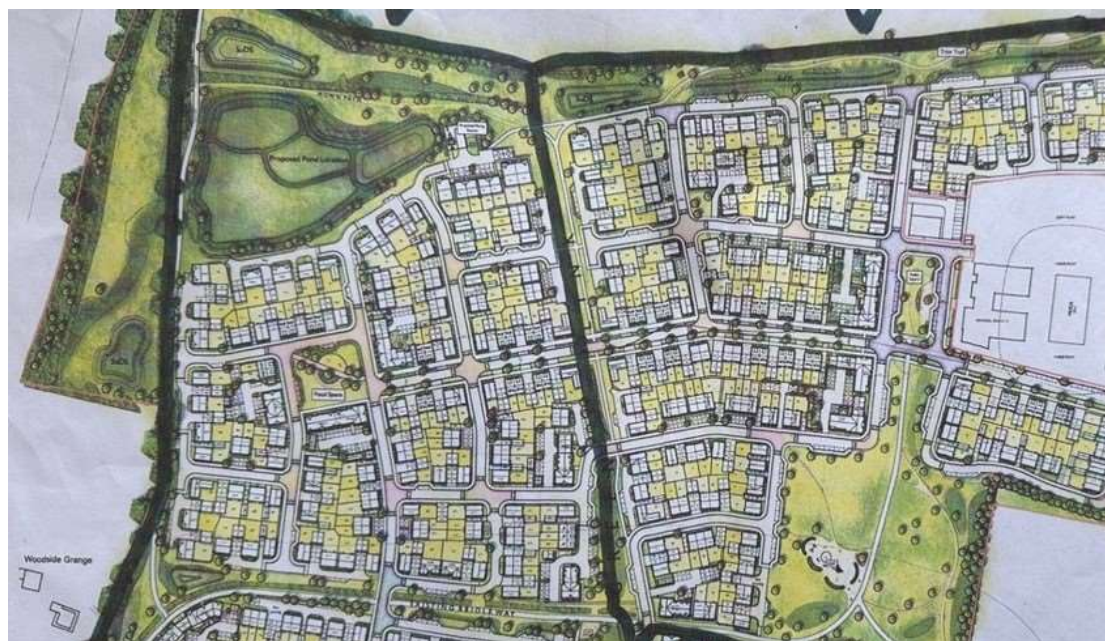
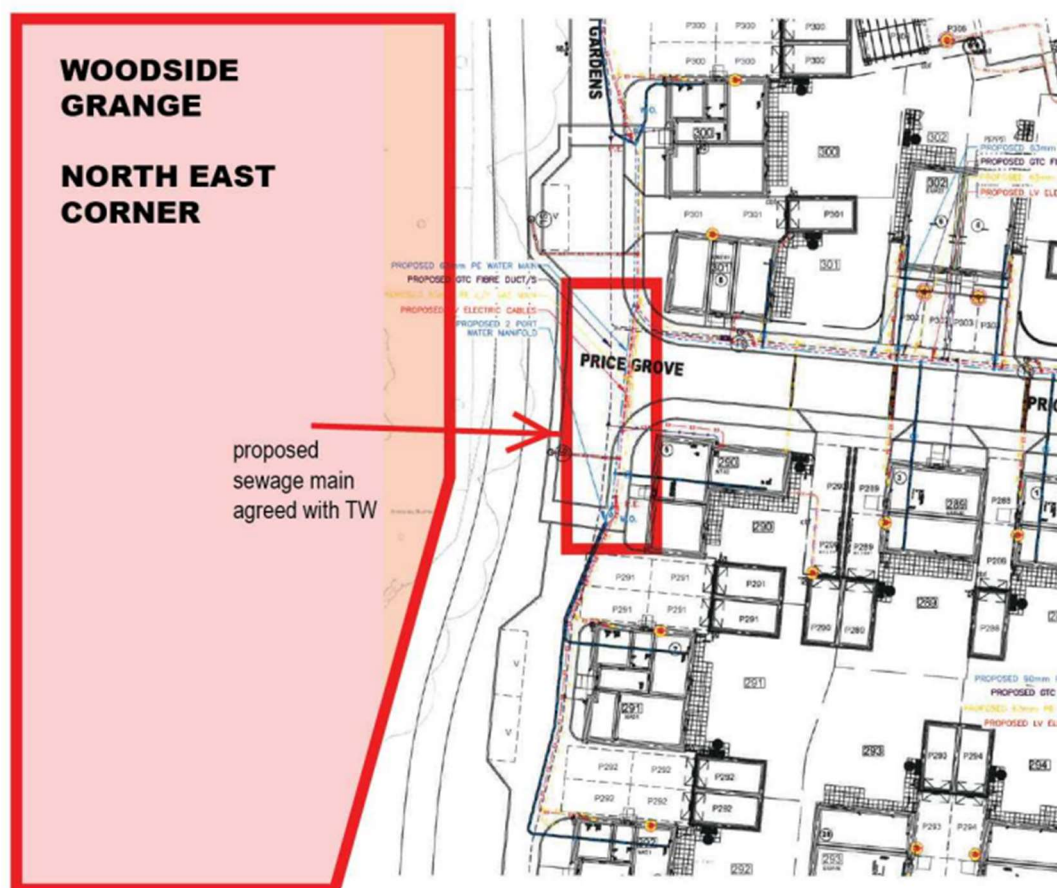
Title Hydrograph storage analysis (Winter profile) for HASSOCKS

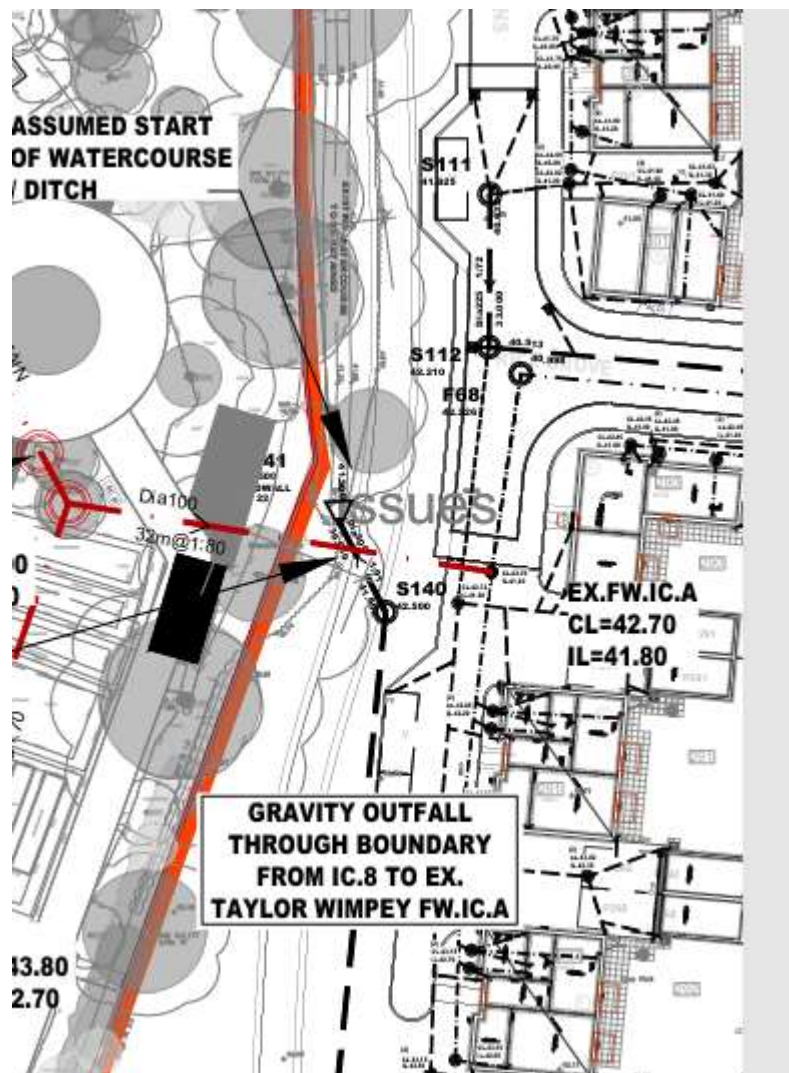
Sequential storage volume at specific storm durations.

m³



APPENDIX E: Taylor Wimpey Proposed Sewer and Drainage Plans





APPENDIX F: Greenfield Runoff Rates for Site

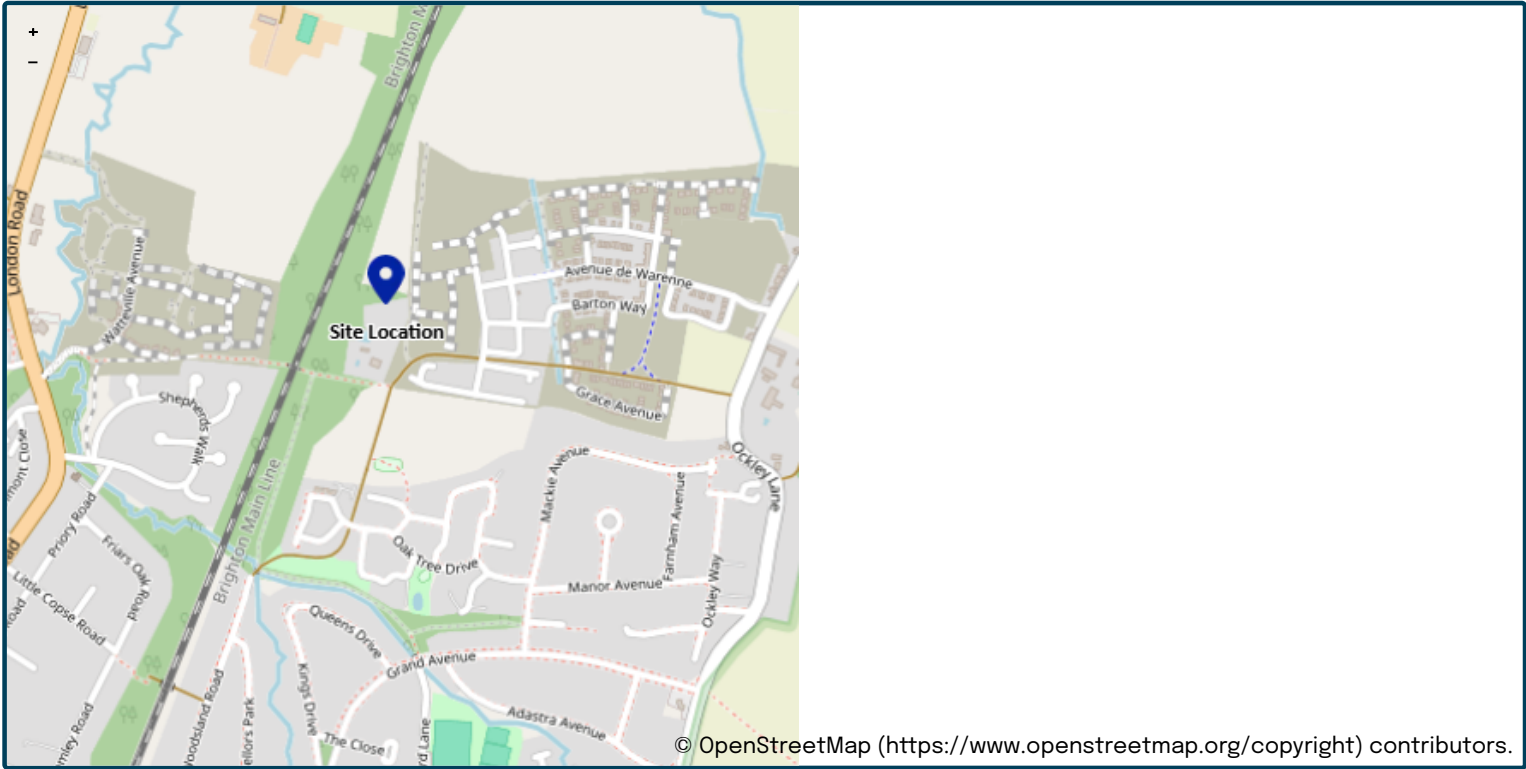
This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance “Rainfall runoff management for developments”, SC030219 (2013), the SuDS Manual C753 (CIRIA, 2015) and the non-statutory standards for SuDS (Defra, 2015). 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	11/06/2025
Calculated by	Xanthe Lyford
Reference	
Model version	2.0.1

Location

Site name	Site A Woodside Grange
Site location	



Site easting	530895
Site northing	116481

Site details

Total site area (ha)	0.64	ha
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Greenfield runoff

Method

Method	IH124	
	<u>My value</u>	<u>Map value</u>
SAAR (mm)	880mm	880
How should SPR be derived?	WRAP soil type	
WRAP soil type	4	4
SPR	0.47	
QBar (IH124) (l/s)	4l/s	

Growth curve factors

	<u>My value</u>	<u>Map value</u>
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	

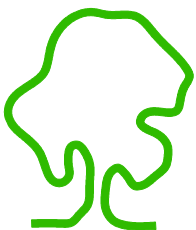
Results

Method	IH124	
Flow rate 1 year (l/s)	3.4	l/s
Flow rate 2 year (l/s)	3.6	l/s
Flow rate 10 years (l/s)	6.5	l/s
Flow rate 30 years (l/s)	9.3	l/s
Flow rate 100 years (l/s)	12.9	l/s
Flow rate 200 years (l/s)	15.1	l/s

Disclaimer

This report was produced using the Greenfield runoff rate estimation tool (2.0.1) developed by HR Wallingford and available at [uksuds.com](https://www.uksuds.com/) (<https://www.uksuds.com/>).

The use of this tool is subject to the UK SuDS terms and conditions and licence agreement, which can both be found at [uksuds.com/terms-conditions](https://www.uksuds.com/terms-conditions) (<https://www.uksuds.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.



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