DRAINAGE STRATEGY
3901/DS

Cliff Road
Darfield
Barnsley
S73 9HR

Proposed Residential Development

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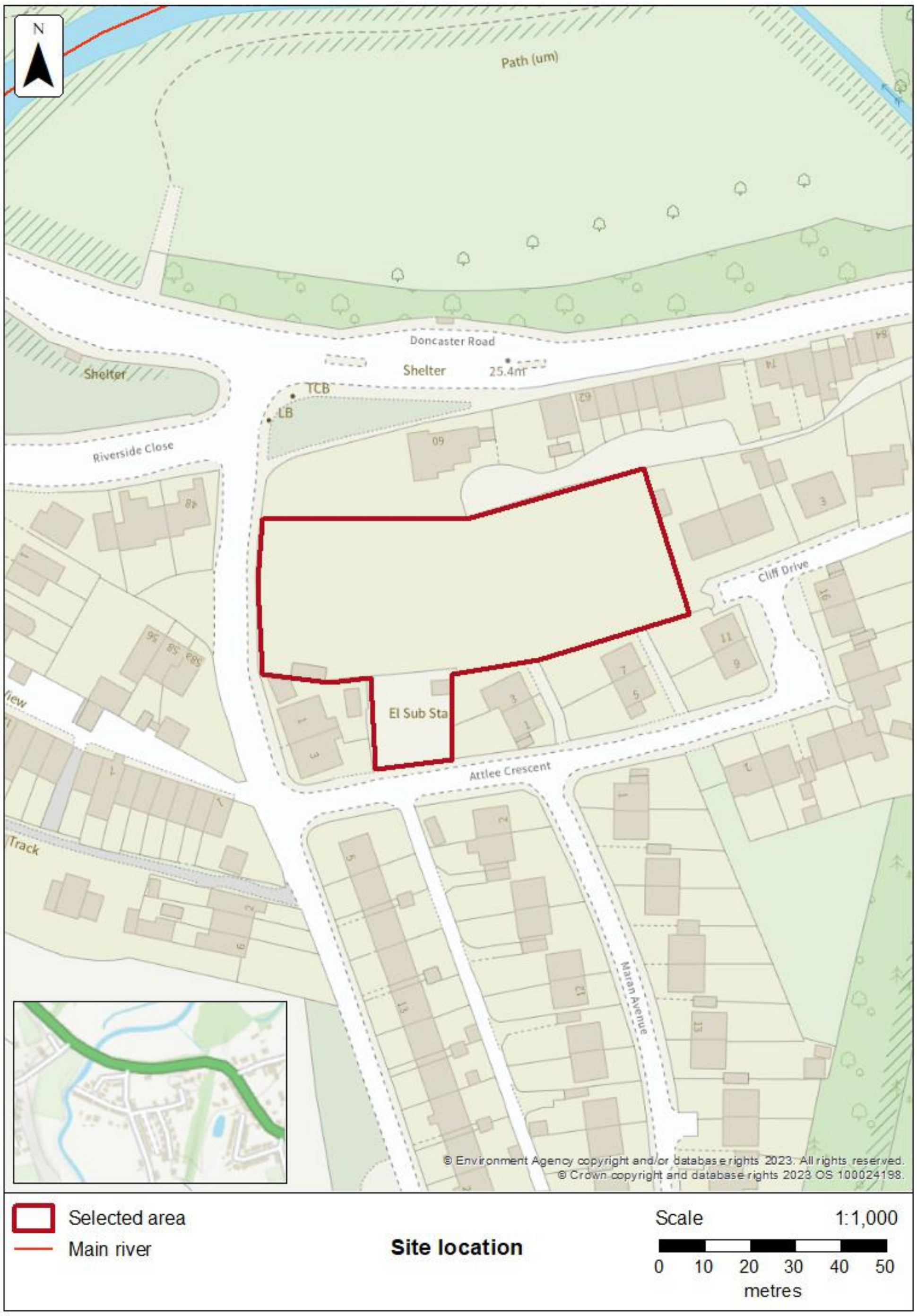
1. Development site

1.1. The development site is a vacant plot to the east of Cliff Road, and south of Doncaster Road, in Darfield, Barnsley. The river Dearne is about 100m west of the site.

1.2. The local area is predominantly residential in nature.

1.3. A location plan is shown below.



2. Development proposals

2.1. The development proposal is construction of 10No. new detached dwellings.

2.2. The first five dwellings are to be accessed from a private drive off Cliff Road, and the other five from a private drive off Attlee Crescent to the south.

3. Drainage Strategy

3.1. For this study, the aspects to be considered include:

- Existing site levels and topography
- Site soil characteristics
- Existing nearby infrastructure and sewers
- Discharge rates
- Planning policy regarding runoff controls and pollution control.
- Management and maintenance of the system
- Flood risk

3.2. This documentation has been prepared in a "desktop" fashion using available information. It should be understood there are limitations on its accuracy and accurate surveys of the site should be performed before works commence to ensure compatibility.

4. Design Constraints

4.1. The site is sloped. The drainage system should ideally not necessitate significant level changes and earthworks to minimise costs.

4.2. The site has no existing surface drainage presently.

4.3. The proposed entrance from Cliff Road is at the low point of the site. Runoff from the private drives should not be allowed to flow onto the public highway.

4.4. It is not clear what the permeability of the soils at the site is likely to be. It is understood that the site is probably underlain by Mexbrough Rock sandstone but may have superficial alluvial deposits of clay, silt, sand or gravel.

4.5. The drainage proposals should be steered by the NPPF Hierarchy of drainage options.

4.6. Yorkshire Water mapping shows the site is surrounded by combined surface water & foul sewers.

4.7. The design standard is to be 1 in 30 year + 30% climate change allowance, with sufficient storage on site for 1 in 100 year + 40% climate change allowance without flood water exiting the site.

4.8. Any outflow from the site should be limited to the estimated greenfield rate, subject to maintenance and reliability concerns.

4.9. The proposal should not cause an increase in off-site flood risk.

5. Surface Water Design

5.1. Following NPPF drainage hierarchy, surface water will be preferentially discharged via:

i) Infiltration
ii) Surface water bodies
iii) Surface water sewers
iv) Combined sewers

5.2. Due to the slope of the site, soakaways near the east end of the site are unlikely to be feasible. Infiltrated water is likely to reissue at lower parts of the site.

5.3. The soil characteristics at the site are not known. The preliminary drainage design shall incorporate the possibility of infiltration being feasible, subject to on site percolation testing before the drainage design is finalised.

5.4. No watercourses flow either on the site or adjacent to it. Direct discharge to watercourse is not feasible.

5.5. A Yorkshire Water surface water sewer runs to the north west of the site within the main road. Connection to this system is proposed.

5.6. Surface water discharge should be limited to the greenfield runoff rate. The 1 in 1 year greenfield runoff rate is estimated at 0.47L/s. See appendix B at the end of this document.

5.7. Restrictions of less than 2.5L/s are considered impractical due to risk of blockage. Therefore, it is proposed that the discharge rate be limited to 2.5L/s. This is subject to agreement of Yorkshire Water and any other relevant parties.

5.8. The catchment area is 1420m2, based on drawings provided by the client. This corresponds to the highlighted magenta area below. The entire catchment is presumed 100% impermeable paved.

5.9. The proposed impermeable area will result in a runoff that far exceeds the 2.5L/s limit during design storms. An attenuation system is required to balance the difference in inflow and outflow from the site.

5.10. A vortex flow control device is proposed. These have additional reliability benefits over simple orifices and get closer to the ideal design point of 2.5L/s over a wide range of heads, however they are still imperfect. Less than the designed 2.5L/s will flow at low heads. Appendix C includes design data for a relevant flow control device, showing the average flow over the head range is around 1.97L/s. A more conservative value of 1.88L/s has been used in the calculations, a 25% reduction.

5.11. Calculations of the required volume to balance inflow and outflow are included in Appendix D. For the 1 in 30-year storm + 30% climate change, 51m3 of storage volume is required. For the 1 in 100 storm + 40% climate change, 79m3 of storage is required.

5.12. The access to the main road of the site is nearly at the lowest point of the site. No scope for earthworks has been allowed. If there is insufficient underground volume, the drains will overflow at the lowest point and allow water out of the site and onto the main road. This is unlikely to be acceptable to highway authorities. Therefore, it has been decided that the underground volume must be sized to store the 1 in 100-year storm + 40% climate change allowance.

5.13. The storage volume is to be provided by a geocellular crate tank under the private drive. A tank of dimensions 9.5m x 6m x 1.5m with 95% voids would provide a volume of 81m3. Adjoining manholes and pipework will also provide additional volume but are not necessary to meet the 79m3 requirement.

5.14. Drain depths from Yorkshire Water records indicate that drainage via gravity only should be possible.

5.15. Silt traps/catch pits should be included before the attenuation tank to reduce siltation and therefore volume reduction of the tank over time.

5.16. Due to the slope of the site, there should be no issues achieving sufficient gradients for self-cleansing flow in the sewers.

5.17. The crate tank is to be designed by specialist for the specific conditions and may require a concrete cover slab. Any drains at shallow depth should also have concrete cover in accordance with the building regulations or other guidance.

5.18. If the crate tank is constructed without an impermeable membrane surround, it will act as a soakaway. Infiltration testing should be undertaken to confirm suitability of the underlying soils.

6. <u>Foul Drainage Design</u>

6.1. Yorkshire Water combined sewers are present at west edge of the site.

6.2. A simple foul sewer run at the front of plots 1a-5a and rear of 1-2 should be sufficient for the foul drainage needs of the development.

6.3. Joining to the 225dia combined sewer at the west of the site should be possible.

7. <u>Maintenance</u>

7.1. The drains are to be private and therefore maintenance is the responsibility of the owner.

7.2. Maintenance actions required include:

Action	Frequency
Inspect system for debris and siltation	At handover
Inspect system for debris and siltation	3 Monthly
Inspect paving above tank for deformation	6 Monthly
Clean tank and silt trap of debris and silt. Remove oil from bypass separator.	6 Monthly
Repairs to system	As required

8. Flood Risk

8.1. A separate flood risk assessment has been prepared for the site. The west edge of the site is in flood zone 3. Higher ground areas of the site are outside any flood zone.

8.2. The proposed development will not increase the risk of flooding elsewhere. All surface water for a 1 in 100 storm + climate change allowance is to be contained on site.

9. Conclusion

9.1. The proposed development is for 10No. dwellings which involves impermeable paving of an area of about 1420m2.

9.2. The NPPF hierarchy of drainage options has been applied. Infiltration and discharge to watercourse are not feasible.

9.3. Attenuated discharge to a surface water sewer is proposed.

9.4. The proposal can be built without increase of flood risk either on or off site.

P Wade Eur.Ing, B.Tech (Hons), C.Eng, RICS DipAdj, MICE, MIStructE.

30.01.2024

APPENDIX A – Greenfield Runoff Rate Estimation

hrwallingford

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Richard Sixsmith
Site name:	Cliff Road
Site location:	Darfield

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	53.53528° N
Longitude:	1.36491° W
Reference:	2006620482
Date:	Jan 18 2024 13:46

Runoff estimation approach: IH124

Site characteristics

Total site area (ha): .35

Methodology

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

Soil characteristics

	Default	Edited
SOIL type:	2	2
HOST class:	N/A	N/A
SPR/SPRHOST:	0.3	0.3

Hydrological characteristics

	Default	Edited
SAAR (mm):	609	609
Hydrological region:	3	3
Growth curve factor 1 year:	0.86	0.86
Growth curve factor 30 years:	1.75	1.75
Growth curve factor 100 years:	2.08	2.08
Growth curve factor 200 years:	2.37	2.37

Notes

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

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

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

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

Greenfield runoff rates

	Default	Edited
Q_{BAR} (l/s):	0.54	0.54
1 in 1 year (l/s):	0.47	0.47
1 in 30 years (l/s):	0.95	0.95
1 in 100 year (l/s):	1.13	1.13
1 in 200 years (l/s):	1.28	1.28

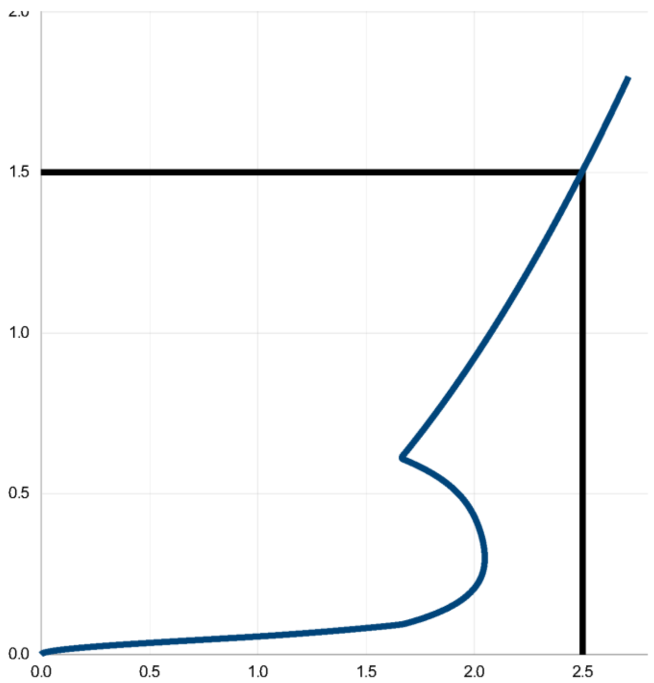
APPENDIX B – Vortex Flow Control Device Data

Technical Specification		
Control Point	Head (m)	Flow (l/s)
Primary Design	1.500	2.500
Flush-Flo™	0.300	2.049
Kick-Flo®	0.609	1.660
Mean Flow		1.973

BBA APPROVAL INSPECTION TESTING CERTIFICATION CERTIFICATE No 08/4596

wrc approved PT/503/0422

hydro-int.com/patents



Head (m)	Flow (l/s)
0.000	0.000
0.052	0.904
0.103	1.722
0.155	1.909
0.207	2.002
0.259	2.042
0.310	2.049
0.362	2.037
0.414	2.011
0.466	1.969
0.517	1.901
0.569	1.791
0.621	1.674
0.672	1.735
0.724	1.794
0.776	1.850
0.828	1.905
0.879	1.958
0.931	2.009
0.983	2.059
1.034	2.108
1.086	2.155
1.138	2.201
1.190	2.246
1.241	2.290
1.293	2.333
1.345	2.375
1.397	2.417
1.448	2.457
1.500	2.497

DESIGN ADVICE !	The head/flow characteristics of this SHE-0068-2500-1500-2500 Hydro-Brake® Optimum Flow Control are unique. Dynamic hydraulic modelling evaluates the full head/flow characteristic curve. **The use of any other flow control will invalidate any design based on this data and could constitute a flood risk.**
DATE	30/01/2024 13:26
Site	Cliff Road
DESIGNER	Richard Sixsmith
Ref	3901

Hydro International A CRH COMPANY

SHE-0068-2500-1500-2500

Hydro-Brake® Optimum

APPENDIX C – Attenuation Calculations

1 IN 30 YEAR + 30% STORMS

Impermeable Area	=	1420	m2
M5_60min	=	18.9	mm
$P_{climate}$	=	30	%

Duration (min)	Growth Factor Z1	M5 falls (mm) + $P_{climate}$	Growth Factor Z2	30 Year Rainfall (mm)	Volume (m^3)	Flow (L/s)
5	0.36	8.8	1.48	13.1	19	62.0
10	0.51	12.5	1.51	18.9	27	44.8
15	0.62	15.2	1.53	23.3	33	36.8
30	0.79	19.4	1.54	29.9	42	23.6
60	1.00	24.6	1.53	37.6	53	14.8
120	1.22	30.0	1.51	45.3	64	8.9
240	1.48	36.4	1.48	53.8	76	5.3
360	1.67	41.0	1.46	59.9	85	3.9
600	1.90	46.7	1.44	67.2	95	2.7
1440	2.42	59.5	1.39	82.6	117	1.4

Fig. 1

1 IN 100 YEAR + 40% STORMS

Impermeable Area	=	1420	m2
M5_60min	=	18.9	mm
$P_{climate}$	=	40	%

Duration (min)	Growth Factor Z1	M5 falls (mm) + $P_{climate}$	Growth Factor Z2	100 Year Rainfall (mm)	Volume (m^3)	Flow (L/s)
5	0.36	9.5	1.90	18.1	26	85.7
10	0.51	13.5	1.97	26.6	38	62.9
15	0.62	16.4	2.00	32.8	47	51.8
30	0.79	20.9	2.03	42.4	60	33.5
60	1.00	26.5	2.00	52.9	75	20.9
120	1.22	32.3	1.95	62.9	89	12.4
240	1.48	39.2	1.90	74.4	106	7.3
360	1.67	44.2	1.86	82.2	117	5.4
600	1.90	50.3	1.81	91.0	129	3.6
1440	2.42	64.0	1.71	109.5	155	1.8

Fig. 2

1 IN 100 YEAR EXCEEDENCE DESIGN

Due to the head-discharge relationship of the flow control device, less than the desired flow limit will occur at low heads. The assumed discharge rate limit is reduced by a factor P_{hydro}, increasing the storage volume required of the tank.

Reduction in flow due to head-discharge relationship P_{hydro} = 25.0 %

Discharge Rate Limit Q_{lim} = 2.50 L/s

$Q_{reduced}$ = 1.88 L/s

Table Calculations

Inflow **I** = 100 yr Rainfall Volume — See Fig. 2

Outflow (Restricted Outlet) **O** = $Q_{reduced}$* Duration

Volume Required V_{req} = I - O

Duration (min)	Inflow (m3)	Outflow (Restricted Outlet) (m3)	Storage Required (m3)
5	25.7	0.6	25.1
10	37.7	1.1	36.6
15	46.6	1.7	44.9
30	60.3	3.4	56.9
60	75.1	6.8	68.4
120	89.4	13.5	75.9
240	105.7	27.0	78.7
360	116.7	40.5	76.2
600	129.2	67.5	61.7
1440	155.5	162.0	0.0

Fig 4

Max storage volume required V_{req_max} = 78.7 m3

1 IN 30 YEAR ATTENUATION DESIGN

Due to the head-discharge relationship of the flow control device, less than the desired flow limit will occur at low heads. The assumed discharge rate limit is reduced by a factor P_{hydro}, increasing the storage volume required of the tank.

Reduction in flow due to head-discharge relationship P_{hydro} = 25.0 %

Discharge Rate Limit Q_{lim} = 2.50 L/s

$Q_{reduced}$ = 1.88 L/s

Table Calculations

Inflow **I** = 30 yr Rainfall Volume — See Fig. 1

Outflow (Restricted Outlet) **O** = $Q_{reduced}$* Duration

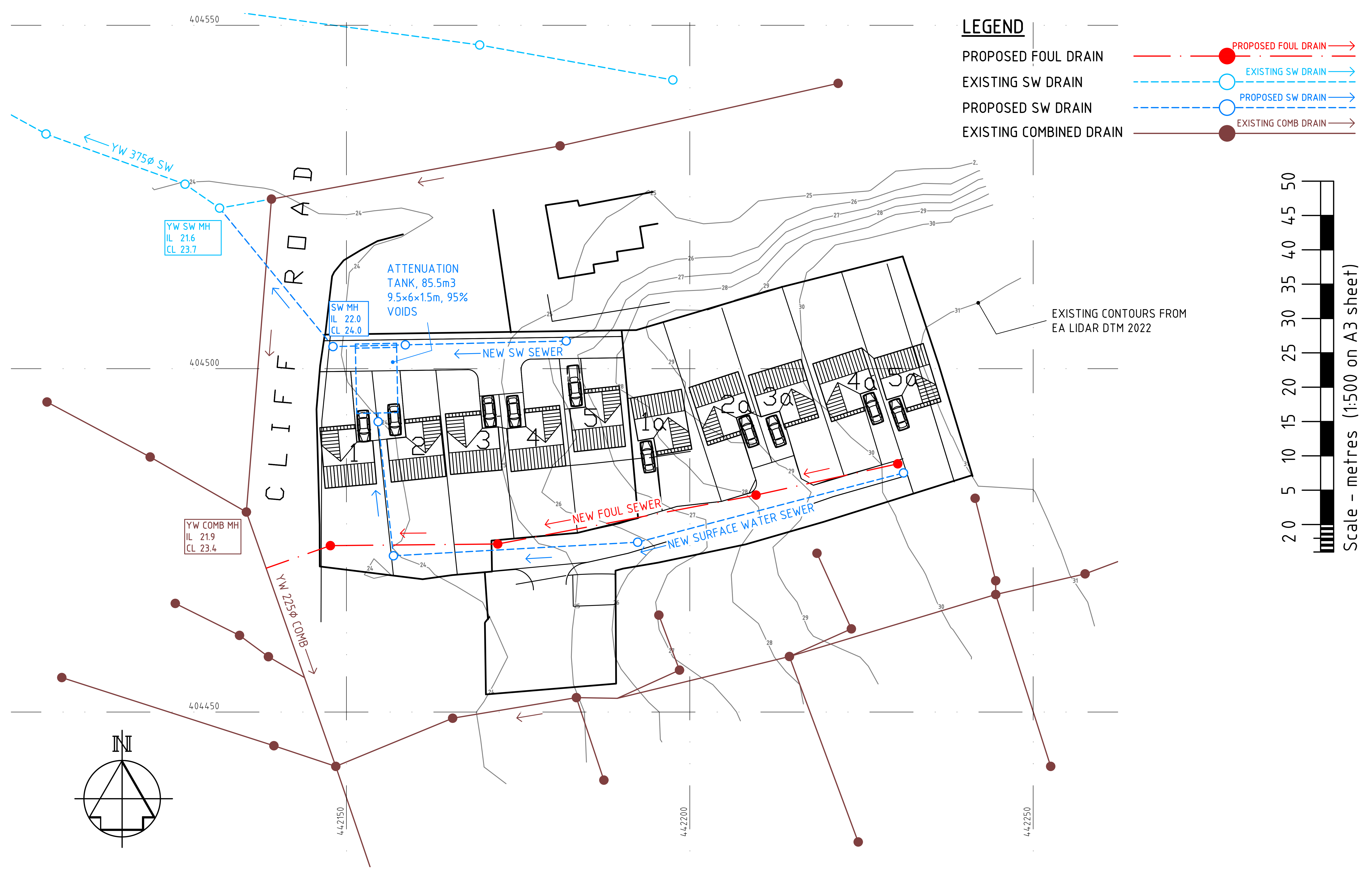
Volume Required V_{req} = I - O

Duration (min)	Inflow (m3)	Outflow (Restricted Outlet) (m3)	Storage Required (m3)
5	18.6	0.6	18.0
10	26.9	1.1	25.7
15	33.1	1.7	31.4
30	42.4	3.4	39.1
60	53.4	6.8	46.6
120	64.3	13.5	50.8
240	76.4	27.0	49.4
360	85.1	40.5	44.6
600	95.5	67.5	28.0
1440	117.4	162.0	0.0

Fig. 3

Max storage volume required V_{req_max} = 50.8 m3

APPENDIX D – Drainage Drawing



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PROJECT: CLIFF ROAD, DARFIELD

TITLE: CONCEPTUAL DRAINAGE PLAN

DATE:	JAN 24	SCALE:	1:500 ON A3 SHEET
DRAWN:	RS	DRG. No:	3901/01 REV: –