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Drainage Strategy & Design

Arlecdon Road, Arlecdon, Frizington

Nigel Kay Homes Ltd

Ref: K39479.DS/001

Version	Date	Prepared By	Checked By	Approved By
Original	23rd February 2023	R. Heron	T. Melhuish	T. Melhuish

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Registered in England & Wales - Company number 04107150

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FIGURES

TABLES

GLOSSARY OF TERMS

AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
BGL	Below Ground Level
BGS	British Geological Society
CC	Climate Change
CCC	Cumbria County Council
DSM	Digital Surface Model
DTM	Digital Terrain Model
EA	Environment Agency
FEH	Flood Estimation Handbook
FFL	Finished Floor Level
FRA	Flood Risk Assessment
GIS	Geographical Information System
LiDAR	Light Detection and Ranging
LLFA	Lead Local Flood Authority
NPPF	National Planning Policy Framework
OS	Ordnance Survey
RGP	RG Parkins & Partners Ltd
SFRA	Strategic Flood Risk Assessment
SuDS	Sustainable Drainage System
UU	United Utilities

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1. INTRODUCTION

1.1 BACKGROUND

This report has been prepared by R. G. Parkins & Partners Ltd (RGP) for Nigel Kay Homes Ltd in support of proposals for a residential development on Arlecdon Road, Arlecdon, Frizington, in accordance with the National Planning Policy Framework [1][2].

Copeland District Council issued planning permission for the development in June 2020 (4/20/2086/001), the following report and associated drainage layout is to discharge the following planning conditions.

- **Condition 4**

'No development shall commence until a surface water drainage scheme has been submitted to and approved in wring by the Local Planning Authority'.

- **Condition 5**

'Foul and surface water shall be drained on separate systems'

- **Condition 6**

'Prior to occupation of the development, a sustainable drainage management and maintenance plan for the lifetime of the development shall be submitted to the local planning authority and agreed in writing'. Please note that a separate document, entitled '*Operations & Maintenance Plan'* (K39479.OM/002) has been prepared by RGP detailing the requirements for future maintenance of the drainage system.

2. SITE CHARACTERISATION

2.1 SITE LOCATION

The site is located to the south of Arlecdon Road, Arlecdon, in the north of the village, at National Grid Co-Ordinates 304749E 519284N (Figure 2.1).

Figure 2.1 Site Location

2.2 SITE DESCRIPTION

The greenfield site covers an area of approximately 0.235 ha (c. 2,345 m^2), and at present is utilised as grazing pasture. The site is bounded by Arlecdon Road to the east, residential dwellings and agricultural land to the south, a highway to the west, and a watercourse further west. Further residential dwellings and Arlecdon Road lie to the north.

Topographically, site levels are highest in the south-east of the site, alongside Arlecdon Road, with levels falling from c. 181.200 mAOD to c. 178.00 mAOD in the south-west. Access to the site is from Arlecdon Road.

2.3 GEOLOGY & HYDROGEOLOGY

British Geological Survey (BGS) [3] and Land Information Systems (LandIS) [4] mapping indicates the site is underlain by the geological sequences outlined in Table 2.1. The Defra Magic Maps [5] indicates the site is not located within a 10 km Groundwater Source Protection. The development site overlies a major aquifer with 'Medium-High' vulnerability,

Table 2.1 Site Geological Summary

Geological Unit	Classification	Description	Aquifer Classification
Soil	Soilscape 6	Freely draining slightly acid loamy soils	N/A
Drift	None recorded	N/A	N/A
Solid	Brockram	Breccia	Summary: Principle

2.4 HYDROLOGY

Reference to OS Mapping indicates there are numerous watercourses in the vicinity of the site. The closest is an unnamed watercourse, that lies c. 25 m to the west of the site, approximately 3.5-4.0m lower than the proposed development site. This watercourse is designated as an 'Ordinary Watercourse' and as such is regulated by the LLFA.

There is another unnamed watercourse c.160m south-west of the site that is culverted under the highway and runs to the west.

2.5 EXISTING SEWERS

Reference to the United Utilities sewer records indicates there is a 150 mm dia. foul sewer in Arlecdon Road. It crosses the site frontage, turning in a south westerly direction within the highway, towards the sewage treatment works.

A CCTV drainage survey was undertaken in July 2022 of the foul network and highways network. The survey confirmed the existing foul network is in reasonable condition and a gravity connection from the site will be possible.

The highways drainage network surveyed is located on the adjacent side of the carriageway to the site within Arlecdon Road and takes flows from the gully network along the road. The pipe outfalls into the unnamed watercourse c.22m north-east of the site via a 225mm and 450mm dia. pipework. A trial hole investigation has confirmed that the highways drainage network is c.0.9m deep within the grass verge.

For further information refer to Drain Doctor report no. 2022-07-17837.

2.6 GROUND INVESTIGATION

Ground investigation was undertaken at the site on 13th May 2022 by GEO Environmental Engineering Ltd[6]. GEO were commissioned to carry out soil infiltration tests to determine whether the underlying ground conditions were suitable for infiltration based SuDS.

The work comprised 2 no. trial pits that were excavated to a depth of c. 1.50 mBGL and they encountered topsoil (c. 0.35 m-0.40 m thick) overlying dark reddish brown very silty, sandy fine to medium gravel. It was noted ground conditions become very dense with depth and both trial pits remained dry with no groundwater ingress recorded.

For the infiltration tests, the pits were filled with water and the water level recorded over a maximum duration of 325 minutes (c. 5.4 hours). During this time, the water level dropped slowly, falling between 500-590 mm. Due to time limitations, only one test was possible per trial pit.

In order to calculate a meaningful infiltration rate, it is necessary for the water level to drain completely, or nearly completely. During these tests, the water level did not drop sufficiently for a meaningful infiltration rate to be calculated. As such, the tests were classified as a failure, and the ground is concluded to not be sufficiently permeable to facilitate soakaway drainage.

For further details refer to Geo Environmental Engineering Report No. GEO2022-5231.

3. SURFACE WATER DRAINAGE STRATEGY & DESIGN

3.1 INTRODUCTION

The principal aim of the following drainage strategy is to design the development to avoid, reduce and delay the discharge of rainfall to public sewers and watercourses in order to protect watercourses and reduce the risk of localised flooding, pollution and other environmental damage.

In order to satisfy these criteria this surface water runoff assessment and drainage design has been undertaken in accordance with the following reports and guidance documents:

- SuDS Manual, CIRIA Report C753, 2015 [7]
- Code of Practice for Surface Water Management, BS8582:2013, November 2013[8]
- Rainfall Runoff Management for Developments, Defra/EA, SC030219, October 2013[9]
- Designing for Exceedance in Urban Drainage – Good Practice, CIRIA Report C635, 2006[10]
- Flood Estimation Handbook (FEH)[11]
- Flood Studies Report (FSR), Volume 1, Hydrological Studies, 1993[12]
- Flood Studies Supplementary Report No 14 (FSSR14), Review of Regional Growth Curves, 1983[13]
- Flood Estimation for Small Catchments, Marshall & Bayliss, Institute of Hydrology, Report No. 124 (IoH 124), 1994[14]

The following drainage strategy is based on the latest site layout plan by Alpha Design.

3.2 SITE AREAS

To support the exploration of options for site drainage, the spatial extent of different types of proposed land cover on the site have been measured. Table 3.1 shows the measured proposed land cover areas. The highest percentage is garden and landscaped areas at 32%, roof areas at 29%, total parking and paved areas at 26% and road area at 14%.

Table 3.1 Land Cover Areas

Land Cover	Area		Percentage of total site area
	m²	Ha	
Housing & garage roof areas	685	0.069	29%
Car parking area (16no.)	200	0.020	9%
Access road & footway areas	420	0.042	18%
Gardens, patios & landscaped areas	1040	0.104	44%

To develop the detailed drainage design, only certain surfaces and areas will be positively drained into the surface water network. Positively drained areas include roof areas, car parking, access road and footways. All other areas (principally gardens, landscaping and patios) will either have a permeable surface or will have no positive drainage (i.e. patios will run-off to landscaped or garden areas). Table 3.2 summarises this and shows that positively drained areas will cover 56% of the site and permeable areas 44%.

Table 3.2 Summary of drained and undrained areas into surface water drainage system

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Total Positively Drained Area	1305	0.161	56%
Remaining Undrained Area	1040	0.074	44%

3.3 SURFACE WATER DRAINAGE DESIGN PARAMETERS

The surface water drainage system has been designed on the following basis using the modified rational method and a generated rainfall profile:

3.3.1 CLIMATE CHANGE

Projections of future climate change indicate that more frequent short-duration, high intensity rainfall and more frequent periods of long-duration rainfall are likely to occur over the next few decades in the UK. These future changes will have implications for river flooding and for local flash flooding. These factors will lead to increased and new risks of flooding within the lifetime of planned developments.

The EA have provided a peak rainfall online map showing the anticipated changes in peak rainfall intensity across the UK. Climate change allowances are now provided on a catchment by catchment basis. The site falls within the South West Lakes catchment. Table 3.3 outlines the EA guidance for this catchment, for the anticipated design life of the proposed development.

In line with current guidance and for conservative design, a 50% allowance shall be used within this assessment.

Table 3.3 South West Lakes Management Catchment Peak Rainfall Allowances (1.0 AEP)

South West Lakes (1.0%AEP)	Central Allowance (%)	Upper End Allowance (%)
2050s	30	45
2070s	35	50

3.3.2 URBAN CREEP

BS 8582:2013[8] outlines best practice with regard to Urban Creep. Although not a statutory requirement, future increase in impermeable area due to extensions and introduction of impervious positively drained areas has been considered. An uplift of 10% on impermeable areas associated with plots only has been applied to the contributing area used for surface water drainage design. This equates to an additional area of $66m^2$ (Plot 1 garage not included in the urban creep uplift) which increases the total positively drained area to $1,371m^2$.

The inclusion of 10% is highly conservative due to the provision of adequate parking on the site and the density of the properties.

3.3.3 PERCENTAGE IMPERMEABILITY (PIMP)

The percentage impermeability (PIMP) for all impermeable areas is modelled as 100%. The entirety of the impermeable areas is to be positively drained.

3.3.4 VOLUMETRIC RUNOFF COEFFICIENT (CV)

The volumetric runoff coefficient describes the volume of surface water which runs off an impermeable surface following losses due to infiltration, depression storage, initial wetting and evaporation. The coefficient is dimensionless. Default industry standard volumetric runoff coefficients are 0.75 for summer and 0.84 for winter and are used for design.

3.3.5 RAINFALL MODEL

The calculations use the REFH2 unit hydrograph methodology in line with best practice as outlined in the SuDS Manual[7]. The calculations use the most up to date available catchment descriptors (2013) provided by the Centre for Ecology and Hydrology Flood Estimation Handbook web service.

3.4 PRE-DEVELOPMENT GREENFIELD RUNOFF ASSESSMENT

As the site covers an area of less than 200 ha the Greenfield calculations have been undertaken in accordance with methodology described in IoH 124[14]. For catchments of less than 50 ha the Greenfield runoff rate is scaled according to the size of the catchment in relation to a 50-hectare site. The calculation has been based on the total proposed positively drained area of 1,305m^2, as summarised in Table 3.2.

Full details of the calculations and the methodology for deriving the Peak Rate of Runoff are in included in Appendix B, and a summary included in Table 3.4.

Table 3.4 Pre-Development Greenfield Runoff Rates

Rate of Runoff (l/s)	
Event	**Greenfield**
Q1	1.2
QBAR	1.4
Q10	1.9
Q30	2.4
Q100	2.9
Q200	3.3

3.5 SURFACE WATER DISPOSAL

Surface water disposal has been considered in line with the hierarchy outlined in the SuDS Manual[7]. The approach considers infiltration drainage in preference to disposal to watercourse, in preference to discharge to sewer.

Infiltration testing undertaken at the site by GEO Environmental Engineering confirmed that the ground is not suitably permeable to facilitate soakaway drainage. For further information refer to Section 2.6.

There are no surface water sewers in Arlecdon, reference to the sewer records show the village is mainly served by public foul sewers. As such discharge to public sewers is not possible.

Following an onsite meeting with the LLFA in October 2022, it was agreed that disposal of attenuated and treated surface water would be to the public highways network located directly to the north of the development site. During site investigations, a highways gully (G2 in Drain Doctor report) was surveyed, and dye tested confirming this 225 mm dia. pipe connected into a 450mm dia. highways drain that discharged into the watercourse. A trial hole was excavated within the highways verge and confirmed the depth of the 225mm diameter as c.0.9m below existing ground level.

3.6 SURFACE WATER DRAINAGE DESIGN

It is proposed that surface water runoff from positively drained area will be attenuated within a single geocellular tank located under the car parking area to the rear of Plots 4 – 8. A silt trap manhole will be located upstream of the tank, which will provide surface water treatment and access for maintenance. Silt traps isolate silt and other particles by encouraging settlement into removal silt buckets, preventing ingress into the tank. The tank will be founded at a suitable level providing a minimum depth of cover of 600mm over the top.

Geocellular attenuation tanks provide high void ratios (95%), resulting in a high storage volume capacity. They are lightweight, easy to install, robust and are also capable of managing high-flow events.

The tank will be fully accessible via access turrets and a row of inspections cells to facilitate future access and maintenance. A flow control chamber incorporating a Hydrobrake will be located downstream of the attenuation tank restricting discharge to the greenfield runoff rate (QBAR) of 1.4 l/s. A Hydrodynamic Separator will be installed downstream of the flow control chamber to provide an enhanced level of treatment of flows prior to offsite discharge into the downstream highways drainage network and watercourse.

The access road and car parking areas will be constructed using conventional surfacing in the form of asphalt and block paving respectively. The access road will fall towards the parking bays, where a series of channel drains and a double gully in the low spot of the road will collect and convey flows towards the geocellular tank.

Full details of the drainage proposals are included on RGP drawings K39479-10, 11 and 12, included in Appendix A.

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3.7 STORAGE VOLUME

The drainage design has been sized to attenuate runoff during a Q100 event plus a 50% allowance for future climate change to ensure adequate drainage over the design life of the development (100 years).

The drainage system has been modelled using MicroDrainage Source Control. FEH catchment descriptors have been used to model the rainfall and determine the volume of attenuation required. In order to contain the 100-year design storm, including a 50% climate change design storm flows, the storage volume outlined in Tables 3.5 is required.

Table 3.5 Storage Requirement (Geocellular Storage)

Tank Depth (m)	Discharge Rate (l/s)	Tank Dimensions (m)	Storage Requirement (m^3)
0.8	1.4	26 x 5.5	109

The critical storm event has been calculated to be the 720 min winter storm and under this event the water level within the tank will be 0.772m, meaning that there is some spare capacity available. A copy of the calculations is included in Appendix B.

3.8 DESIGNING FOR LOCAL DRAINAGE SYSTEM FAILURE

In accordance with the general principles discussed in CIRIA Report C635 – Designing for Exceedance in Urban Drainage [10] the proposed surface water drainage, where practical, should be designed to ensure there is no increased risk of flooding to the proposed dwellings on the site or elsewhere as a result of extreme rainfall, lack of maintenance, blockages or other causes. These measures are discussed below.

3.8.1 BLOCKAGE & EXCEEDANCE

The sustainable drainage system has been designed to attenuate a 100-year design storm including a 50% allowance for climate change. The drainage system will also provide capacity for lower probability (greater design storm events) which are not critical duration.

Based on the existing topography, overland flows follow the contours towards the gated access located in the south-western corner of the site where the levels are lowest. Existing site runoff enters the highway and flows in a southerly direction towards the existing grass verge and drainage channel.

In the unlikely event of blockage in the geocellular attenuation tank post-construction, exceedance flows will be flood from the silt trap manhole (SW05) towards the new double gully arrangement, located at the lowest levels within the access road. Any further exceedance flows would then be channelled via a new 750mm wide by 150mm deep grass swale which will route along the southern boundary around the gable of Plot 8 and towards the highway. As such, any future exceedance flows from the development site will match the existing Greenfield situation, albeit with less likelihood of occurring.

3.8.2 SURFACE STORAGE & EXTERNAL LEVELS

The site levels have been designed to offer additional surface water storage volume and conveyance of flood water should the SuDS and drainage system fail, flood or exceed capacity. Where appropriate, the kerb lines have been raised to channel surface water runoff back into the drainage system or onto the existing highway.

3.8.3 BUILDING LAYOUT & DETAIL

The finished floor levels to the new dwellings have been designed and situated to ensure that they are not at risk of flooding from overland flow. Threshold levels have been set 150mm above external paved areas, and external footpaths typically fall away from the thresholds, ensuring that any flood water runs away from, rather than towards the dwellings.

3.9 SURFACE WATER TREATMENT

The treatment of surface water is not a statutory requirement. Water quality remains a material consideration but there are no prescriptive standards to be imposed in terms of treatment train management. In the absence of a design standard, the SuDS manual has been used which outlines best practice.

Pollutants such as suspended solids, heavy metals and organic pollutants may be present in surface water runoff, the quantity and composition of the runoff is highly dependent upon site use. For housing developments, the pollutant load is very low. The SuDS Manual[7] outlines best practice with regards to treatment of surface water by SuDS components prior to discharge to the environment. SuDS components can be effective in reducing the amount of pollutants within the surface water discharged and therefore environmental impact of the development. SuDS components may be installed in series to form a treatment train to treat the runoff.

Due to the density of dwellings, it is not feasible to implement any surface SuDS components, however, it is proposed to install a StormCleanser™ hydrodynamic separator by FP McCann downstream of the Hydrobrake chamber to ensure surface water is treated prior to discharge to the highways drainage system, and thereby the downstream open watercourse. Table 3.6 – Table 3.8 summarise the pollution hazard and mitigation indices for each type of runoff.

Table 3.6 Pollution Hazard & Mitigation Indices - Roof Areas

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.2	0.2	0.05
Pollution Mitigation	0.5	0.4	0.8
Treatment Suitability	**Adequate**	**Adequate**	**Adequate**

Table 3.7 Pollution Hazard & Mitigation Indices - Parking Areas

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.4	0.8
Treatment Suitability	**Adequate**	**Adequate**	**Adequate**

Table 3.8 Pollution Hazard & Mitigation Indices - Road Areas

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.4	0.8
Treatment Suitability	**Adequate**	**Adequate**	**Adequate**

3.10 OPERATIONS & MAINTENANCE RESPONSIBILITY

All on site drainage will remain private and will be maintained by the site owner in the first instance, and then a third-party management company. An '*Operations & Maintenance Plan'* (K39479.OM/002) has been prepared by RGP detailing the requirements for future maintenance of the drainage system.

4. FOUL WATER DRAINAGE STRATEGY

It is proposed that foul water from the development shall be drained via private gravity foul water plot drainage within the site before connecting into existing 150mm dia. public foul water sewers. Plots 1 – 3 will connect via a new junction into the sewer located under the footway in Arlecdon Road. Plots 4 – 8 will connect via a new vertical backdrop into the existing UU foul water sewer manhole located in the highway to the south-west of the site. A CCTV drainage survey has confirmed that the sewers are sufficiently deep to enable convention gravity connections.

The new connections will be subject to formal application to UU under S106 agreements. Foul water discharge calculations have been undertaken for the 8 no. dwellings, in accordance with the Design and Construction Guidance for Foul and Surface Water Sewers [17], as shown in Table 4.1.

Table 4.1 Peak Foul Flow Rates

Sewerage Sector Design & Construction Guidance Clause B3.1	
Peak Load based on Number of Dwellings, 8 no. units @ 4000 l/day	32,000
Peak Flow Rate from Site (l/s)	0.37

The estimated peak foul flow rate for the development is 0.37 lit/sec. For further details, refer to the Drainage Layout Plan included in Appendix A.

5. CONCLUSIONS AND RECOMMENDATIONS

The proposed Drainage Strategy can be summarised as follows:

- Ground investigation undertaken on 13th May 2022 by GEO Environmental Engineering Ltd concluded that the site is not suitable for infiltration-based SuDS drainage and an off-site surface water drainage solution is required.

- It is proposed that surface water runoff from positively drained areas will be attenuated within a geocellular attenuation tank, measuring 26m x 5.5m x 0.8m deep, thereby providing 109m^3 of storage within the site.

- A flow control chamber incorporating a Hydrobrake will be located downstream of the geocellular tank restricting discharge to the greenfield development QBAR runoff rate of 1.4 lit/sec.

- The access road and driveways will be constructed using conventional surfacing in the form of asphalt and block paving respectively. The access road will fall towards the parking bays, where a series of channel drains and a double gully in the low spot of the road will collect and covey flows towards the geocellular tank.

- Attenuated discharge from the site shall be to the existing 225mm dia. highways drainage system located to the north-west of the site. Discussions with CCC LLFA & Highways have confirmed that the proposed discharge route and connection from the development is acceptable.

- Treatment of surface water is proposed by a proprietary Hydrodynamic Separator, located downstream of the flow control chamber. An upstream silt trap will also help to remove any potential pollutants and silt from entering the tank. The tank will be fully accessible via access turrets and inspections cells to facilitate future access and maintenance.

- An exceedance channel is proposed to route any future flood flows around the gable of Plot 8 and towards the existing highway, as per the existing flow path for the site.

- It is proposed foul water drainage shall discharge via gravity connections into the existing public foul water sewers, via a direct connection for Plots 1 – 3 and a new backdrop connection onto an existing manhole for Plots 4 – 8. A CCTV drainage survey confirmed the sewer is in good condition and there is sufficient depth to enable a gravity connection.

6. REFERENCES

[1] Ministry of Housing, Communities and Local Government, National Planning Policy Framework, July 2018.

[2] Ministry of Housing, Communities and Local Government, Planning Practice Guidance to the National Planning Policy Framework, December 2022

[3] British Geological Survey, 2022. Geoindex. http://mapapps2.bgs.ac.uk/geoindex/home.html

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[7] CIRIA, The SuDS Manual, Report C753, 2015.

[8] BS8582:2013, Code of Practice for Surface Water Management, November 2013.

[9] DEFRA/EA, Rainfall Runoff Management for Developments, SC030219, October 2013.

[10] CIRIA, Designing for Exceedance in Urban Drainage – Good Practice, Report C635, London, 2006.

[11] Centre for Ecology and Hydrology, Flood Estimation Handbook, Vols. 1 – 5 & FEH CD-ROM 3, 2009.

[12] Institute of Hydrology, Flood Studies Report, Volume 1, Hydrological Studies, 1993.

[13] Institute of Hydrology, Flood Studies Supplementary Report No 14 – Review of Regional Growth Curves, August 1983.

[14] Marshall & Bayliss, 1994. Flood Estimation for Small Catchments, Report No. 124 (IoH 124), Institute of Hydrology.

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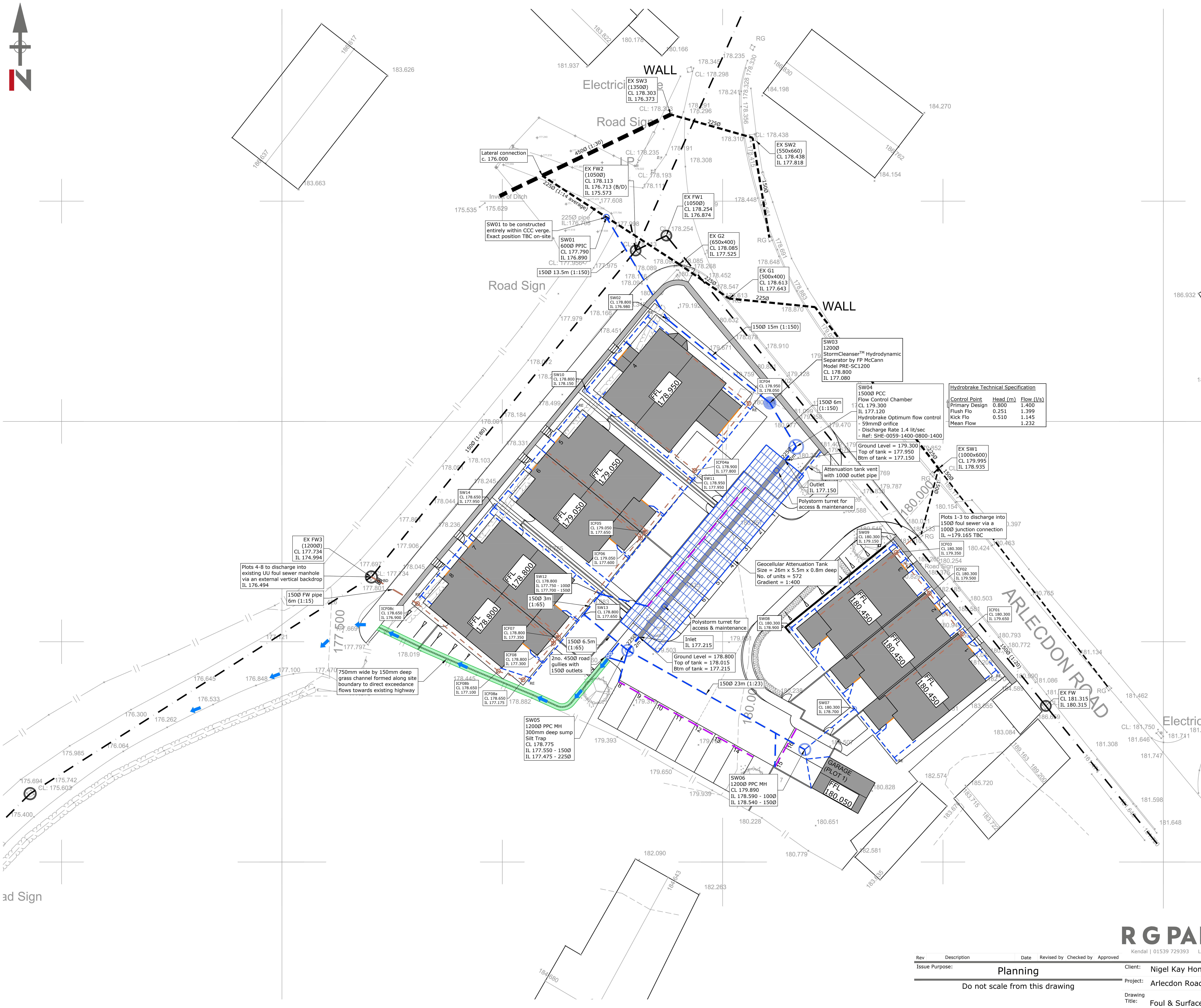
[17] Water UK, Design and Construction Guidance for Foul & Surface Water Sewers Offered for Adoption Under the Code for Adoption Agreements for Water and Sewage Companies Operating Wholly or Mainly in England, Approved Version 10, October 2019

APPENDIX A

DRAWINGS



DRAINAGE KEY

- Existing 150Ø UU Foul Water Public Sewer
- Existing 150/225/450Ø CCC Highways drainage
- Proposed 100/150Ø Surface Water Plot Drainage
- Proposed 100Ø Foul Water Plot Drainage
- Proposed 150Ø Channel Drain with sump unit
- Proposed threshold drain to front/rear access
- 460Ø PPIC Manhole Inspection Chamber
- Route of exceedance flows from attenuation tank

Hydrobrake Technical Specification

Control Point	Head (m)	Flow (l/s)
Primary Design	0.800	1.400
Flush Flo	0.251	1.399
Kick Flo	0.510	1.145
Mean Flow		1.232

Rev	Description	Date	Revised by	Checked by	Approved

Issue Purpose: Planning

Do not scale from this drawing

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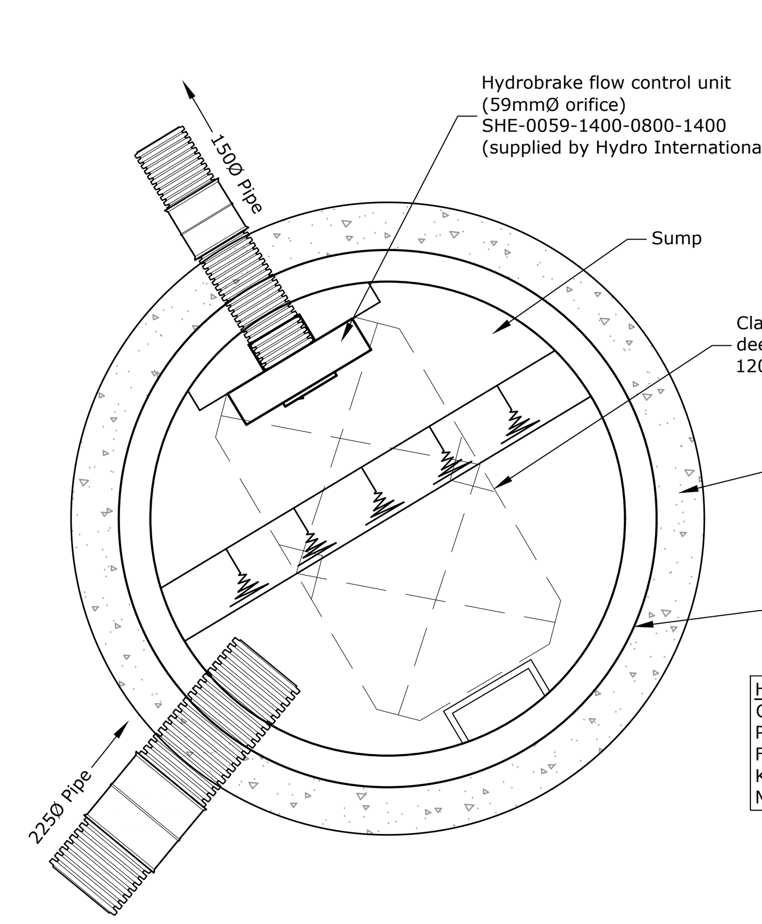
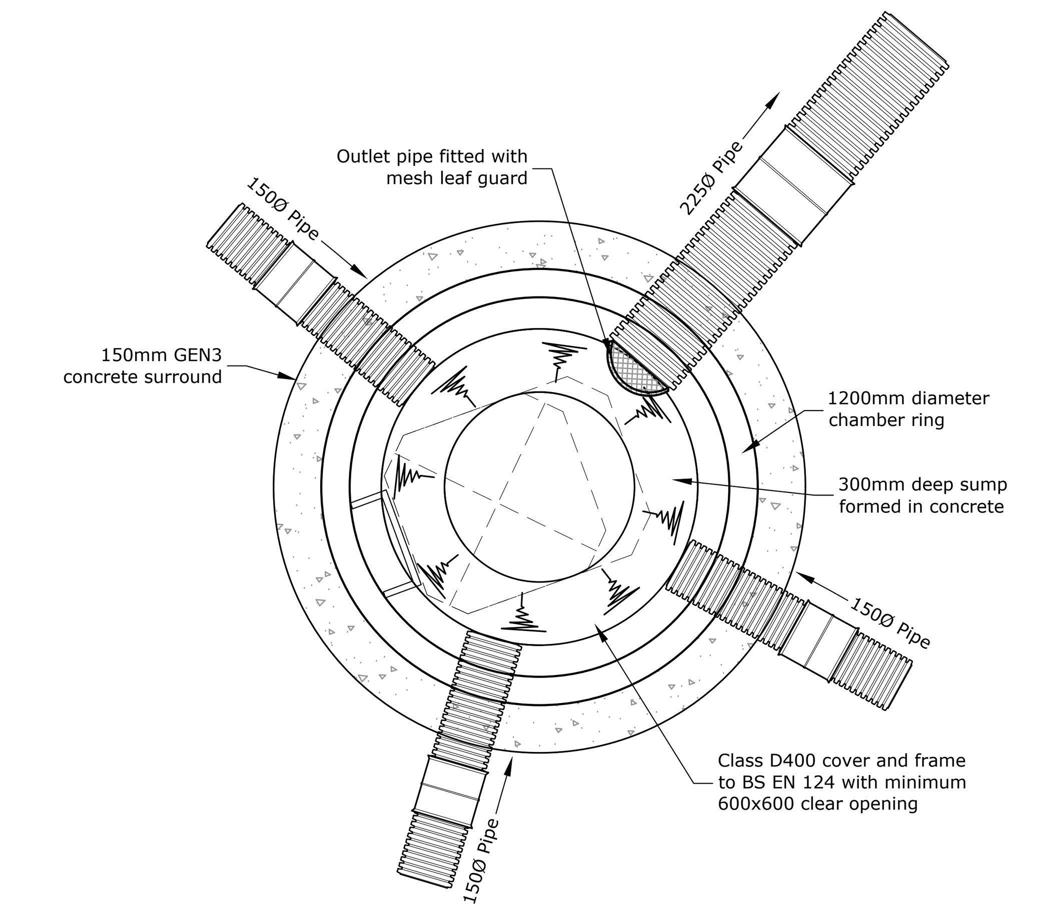
Client: Nigel Kay Homes Ltd
Project: Arlecdon Road, Arlecdon
Drawing Title: Foul & Surface Water Drainage Plan

Scale @ A1: 1:200	First Issue: 23/02/23	Office of Origin: Kendal
Drawn by: RH	Checked by: TM	Approved: TM
Project No: K39479	Drawing No: 10	Rev:
BIM No:		

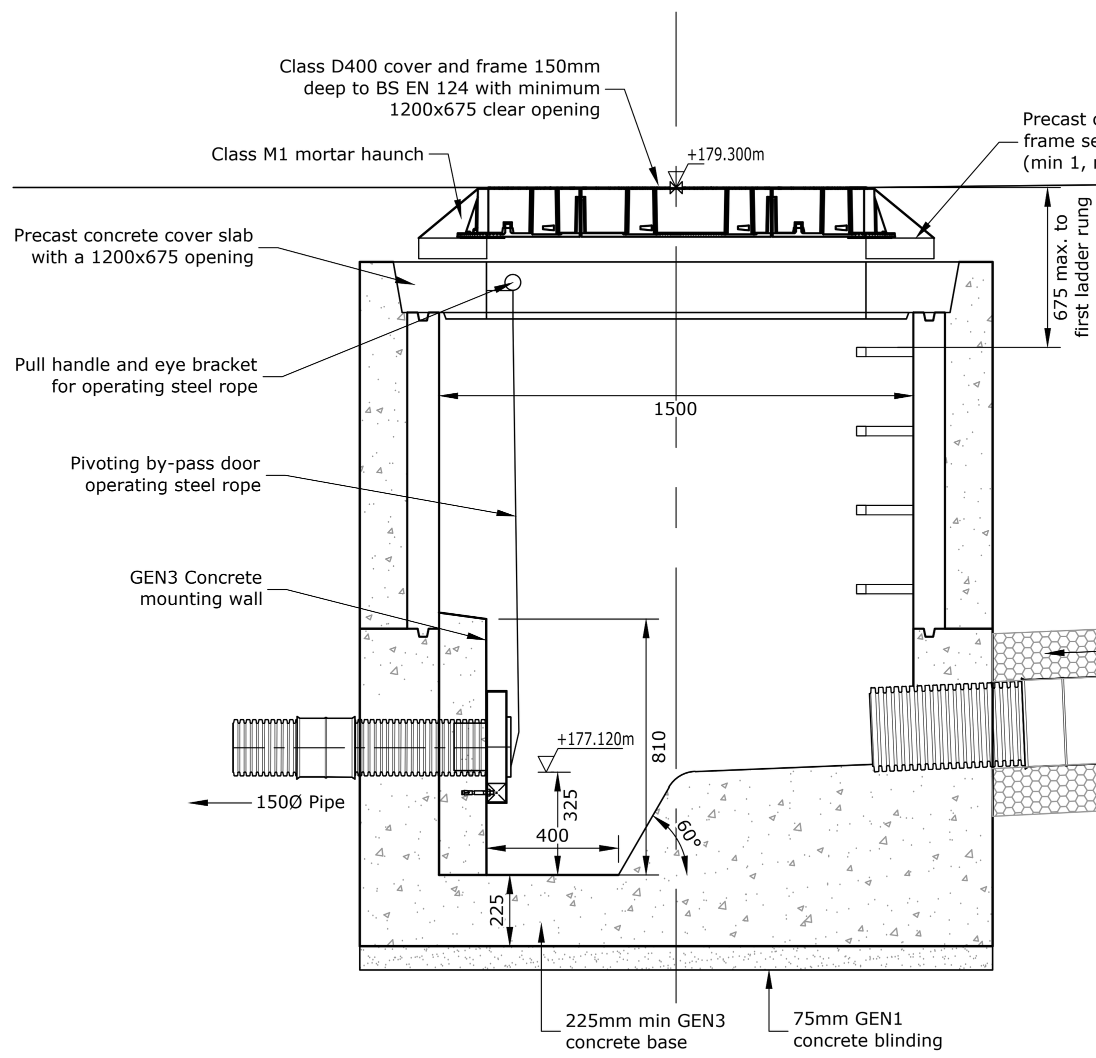


Hydro-brake optimum technical specification		
Control point	Head (m)	Flow (l/s)
Primary Design	0.800	1.400
Flush-Flo	0.251	1.399
Kick-Flo	0.510	1.145
Mean Flow		1.232

SW04 - Flow Control Chamber
Plan
Scale 1:20



SW05 - Silt Trap
Plan
Scale 1:20



SW04 - Flow Control Chamber
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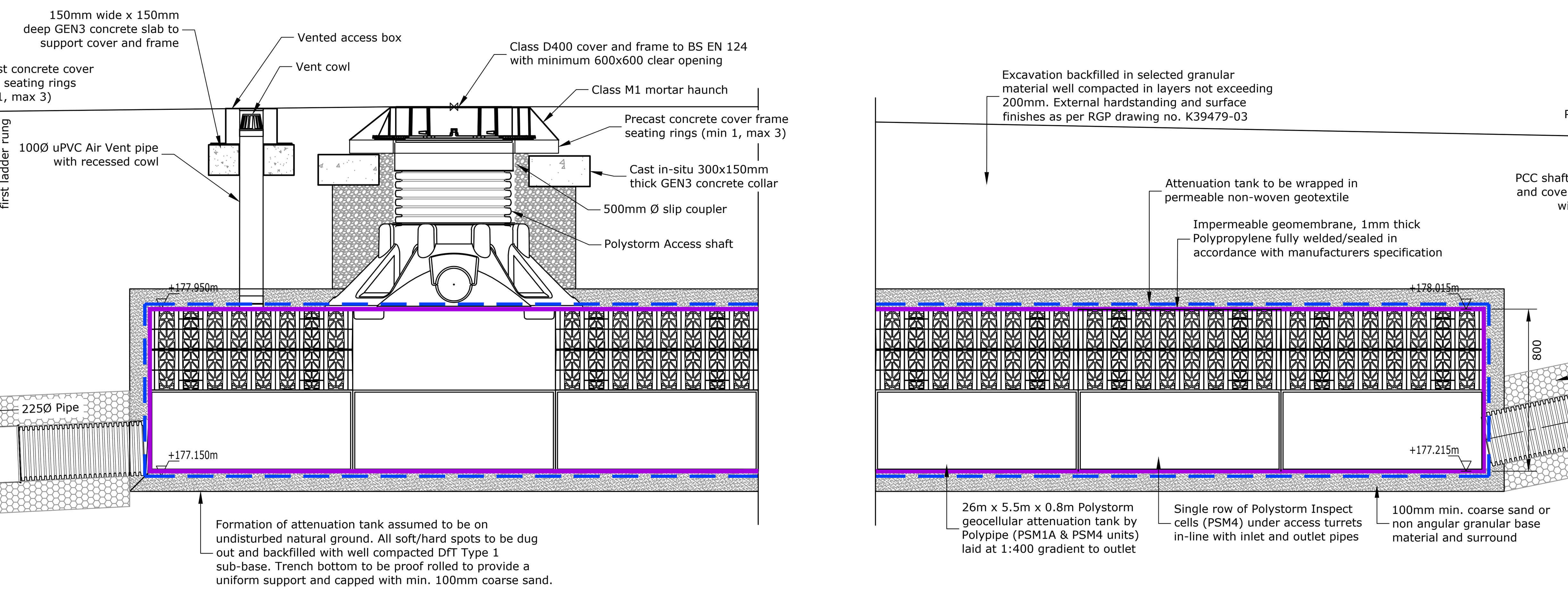
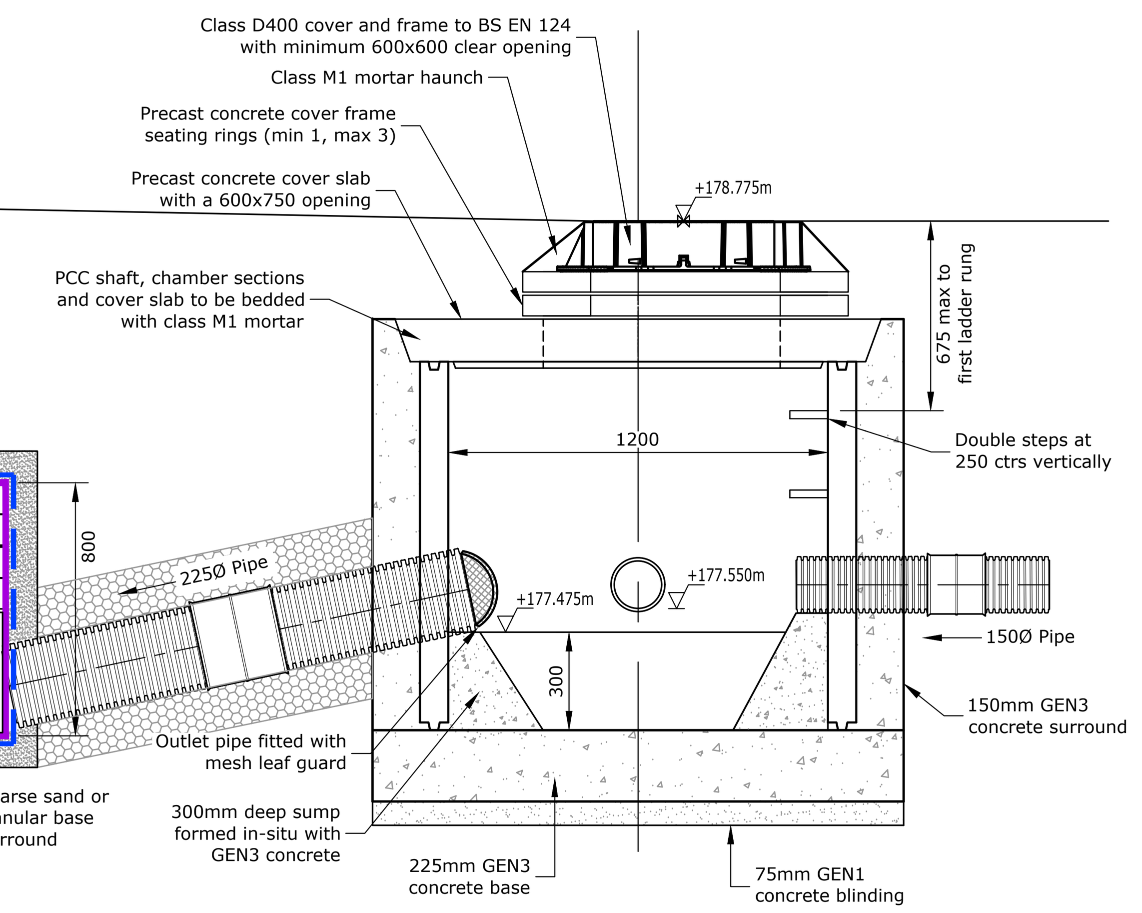


Section Through Geocellular
Attenuation Tank
Scale 1:20



SW05 - Silt Trap
Section
Scale 1:20

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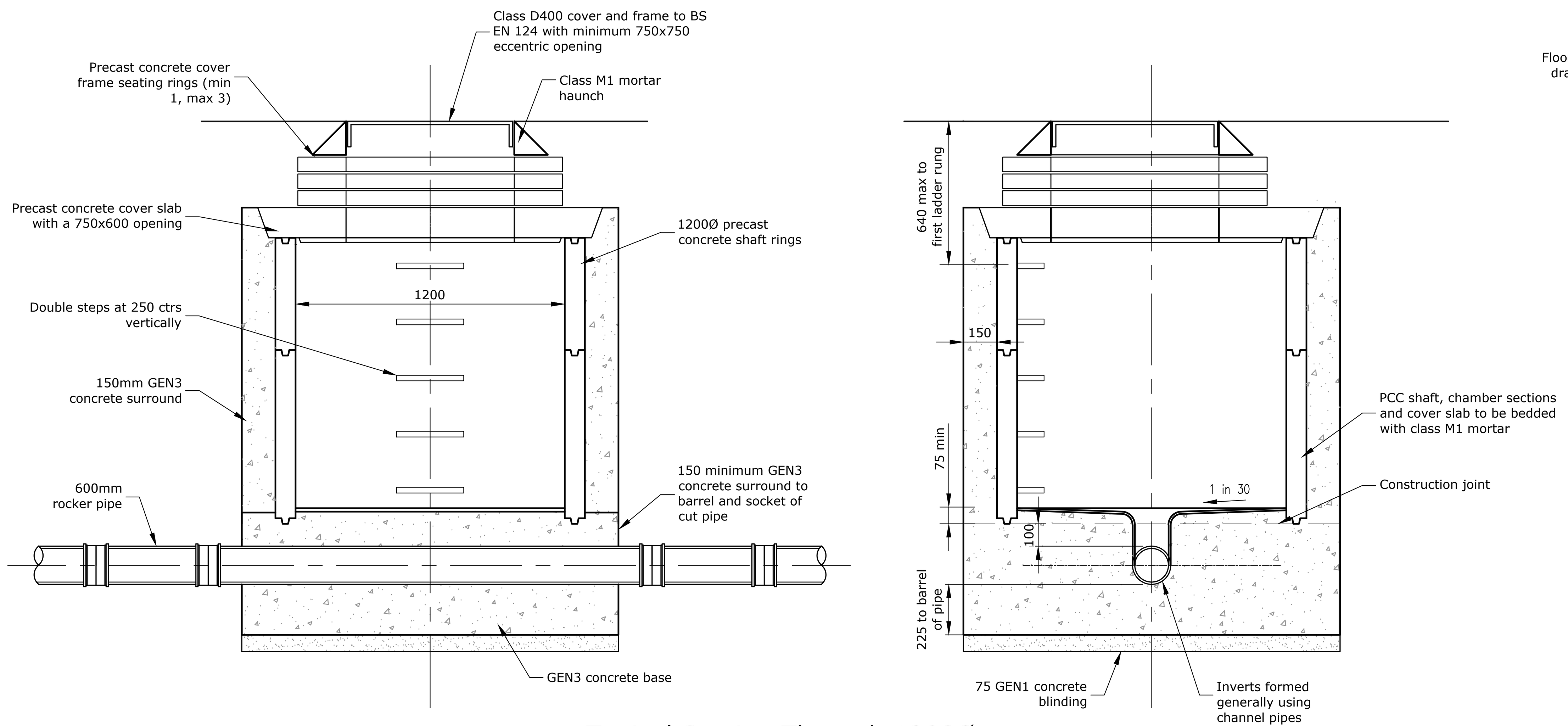
Scale @ A1: As shown	First Issue: 22/02/23	Office of Origin: Kendal
Drawn by: RH	Checked by: RW	Approved: TM
Project No: K39479	Drawing No: 11	Rev:
BIM No:		

Client: Nigel Kay Homes Ltd
Project: Arlecdon Road, Arlecdon
Drawing Title: Geocellular Attenuation Tank Construction Details

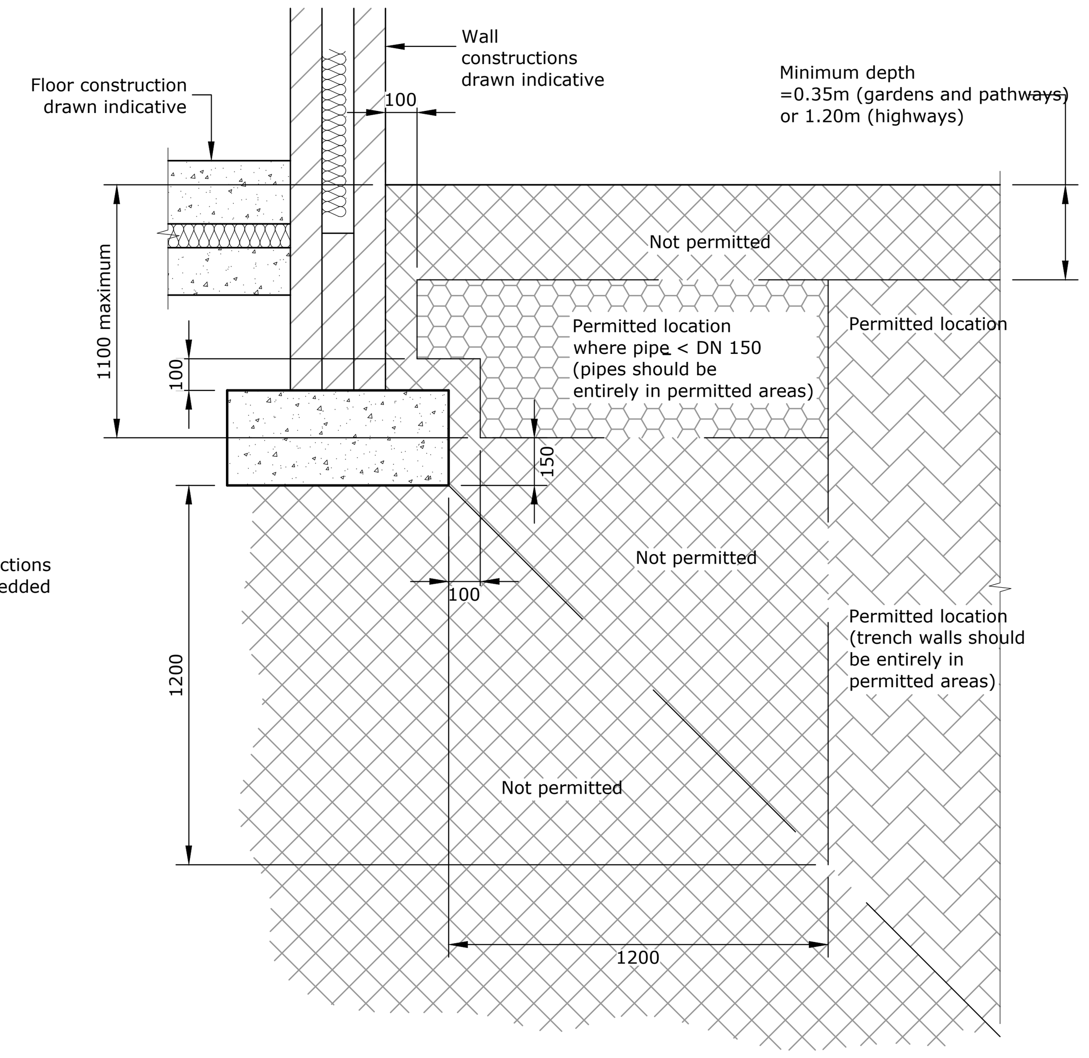
Rev	Description	Date	Revised by	Checked by	Approved

Issue Purpose: Planning

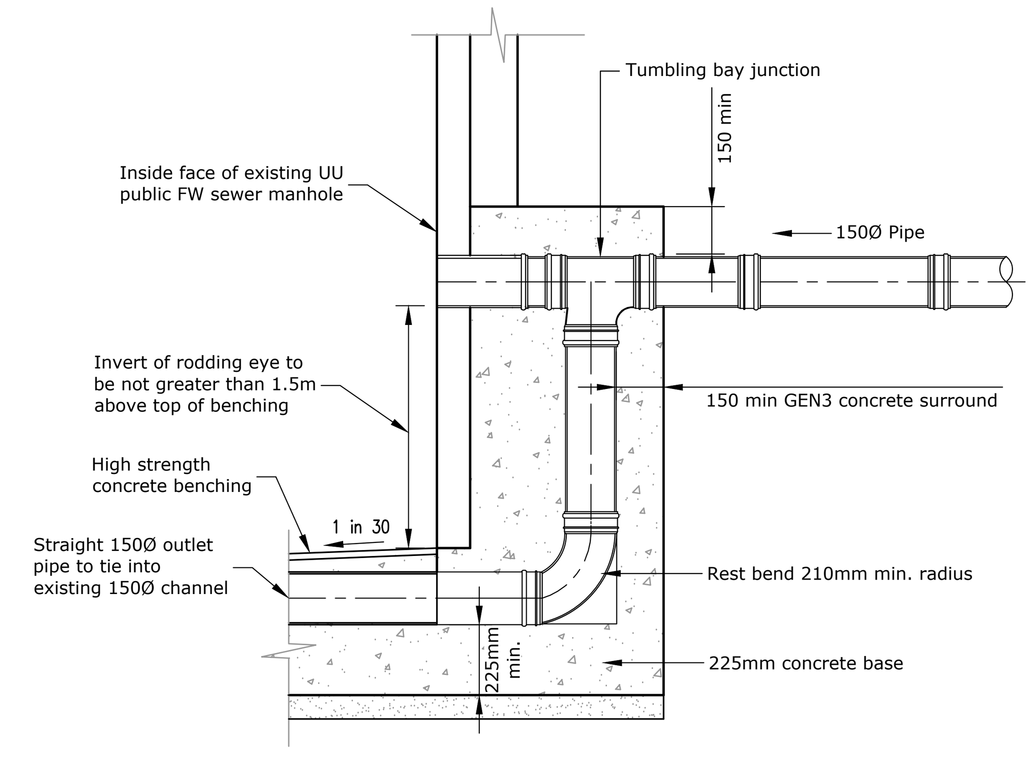
Do not scale from this drawing



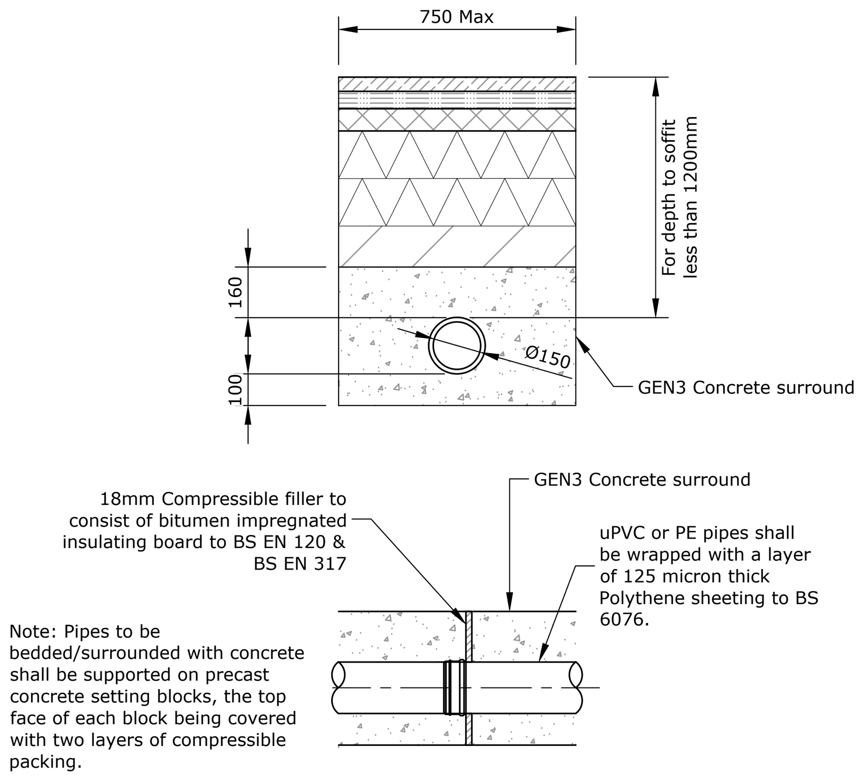
Typical Section Through 1200Ø Manholes < 1.5m to Soffit
SCALE 1:20



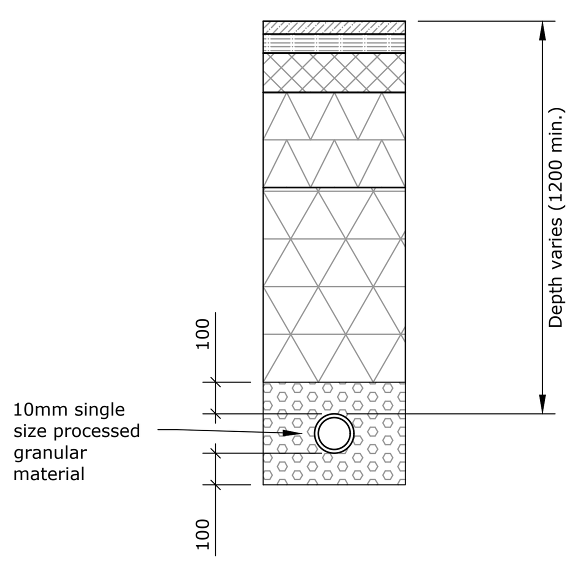
Permitted Location Of Sewers And Lateral Drains In Proximity To Buildings
SCALE 1:20



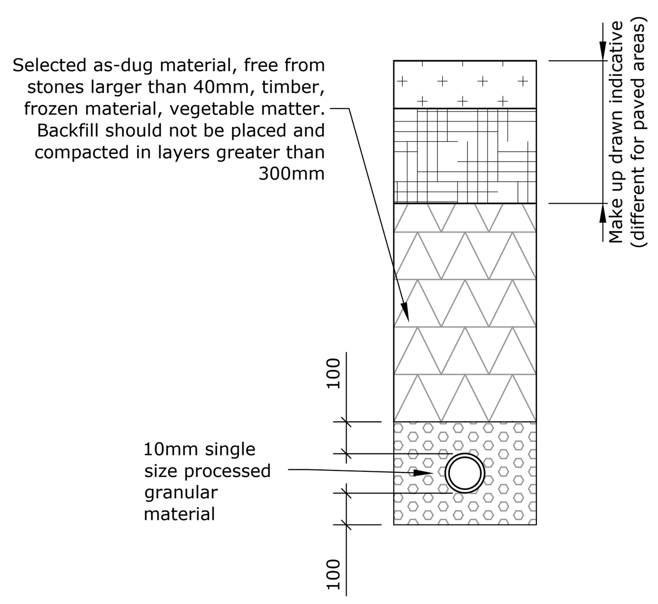
Typical Vertical Backdrop Detail
Plot 4-8 FW Connection onto UU manhole
SCALE 1:20



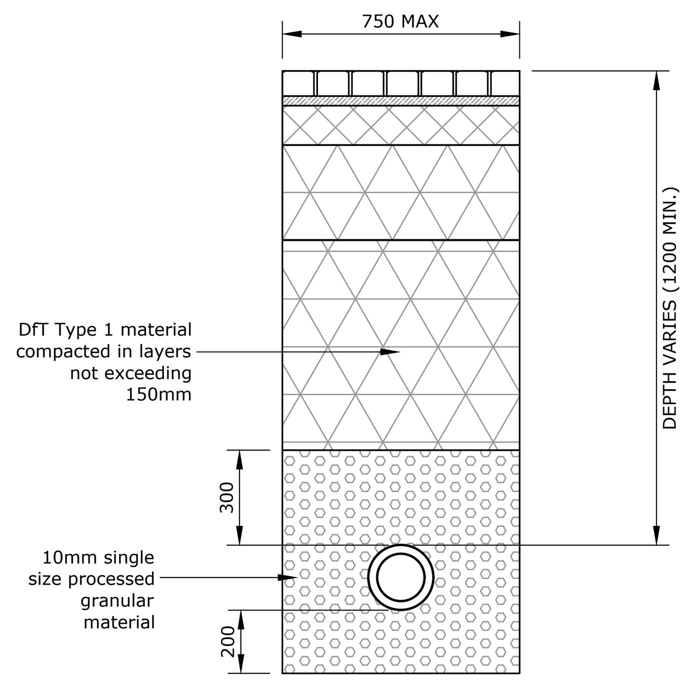
Concrete Surround For 150Ø Upvc Pipe
SCALE 1:20



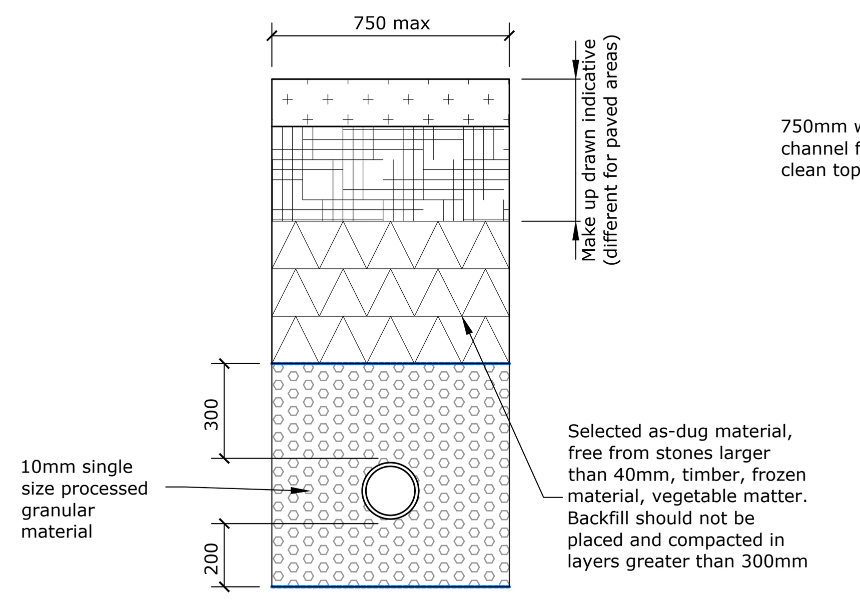
Pipe Bedding For 100Ø Pipes in Hard Landscaped Areas
SCALE 1:20



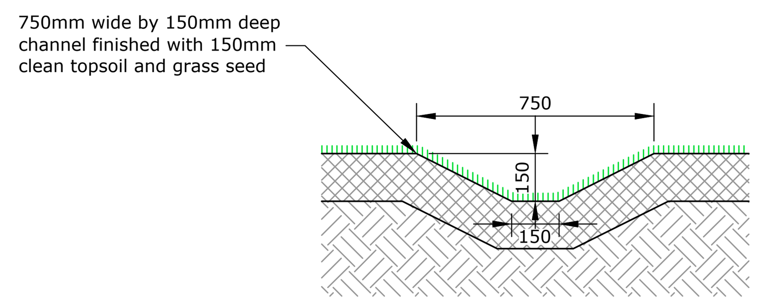
Pipe Bedding For 100Ø Pipes In Soft Landscaped Areas
SCALE 1:20



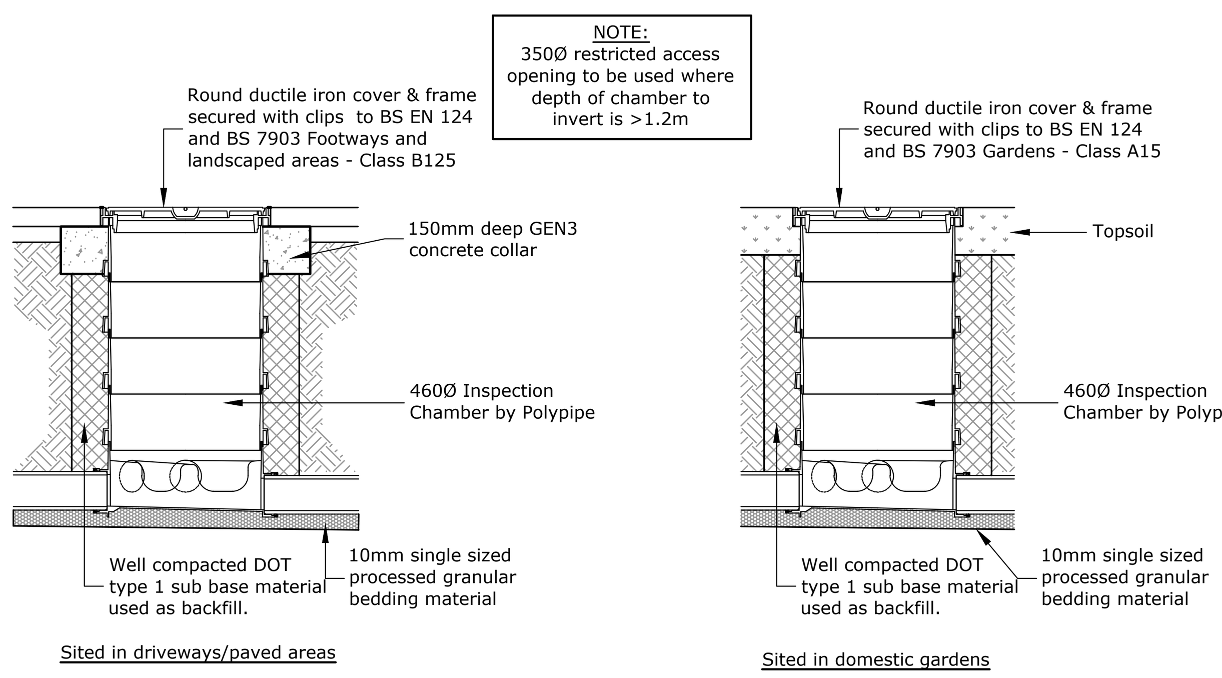
Type 7
Embedment Class S
For 150Ø Pipes In Hard Landscaped Areas
SCALE 1:20



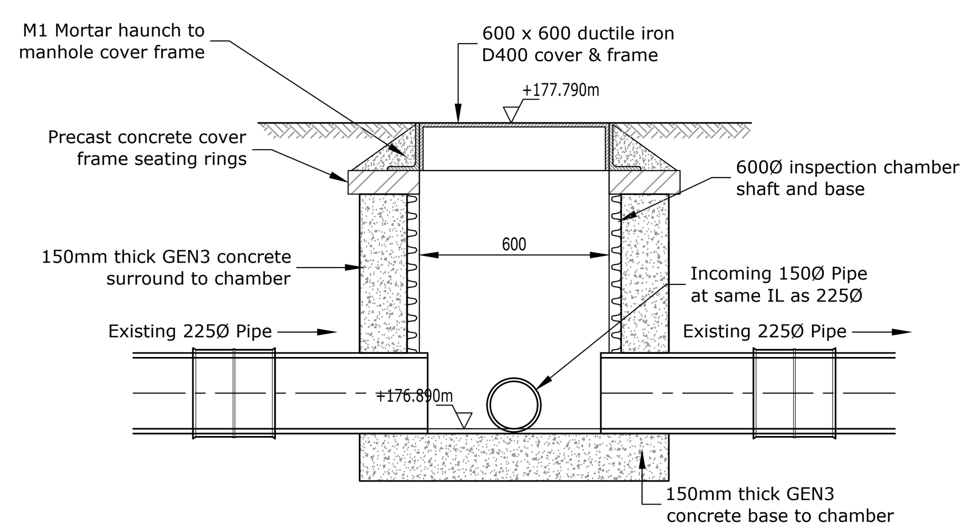
Type 7
Embedment Class S
For 150Ø Pipes In Soft Landscaped Areas
SCALE 1:20



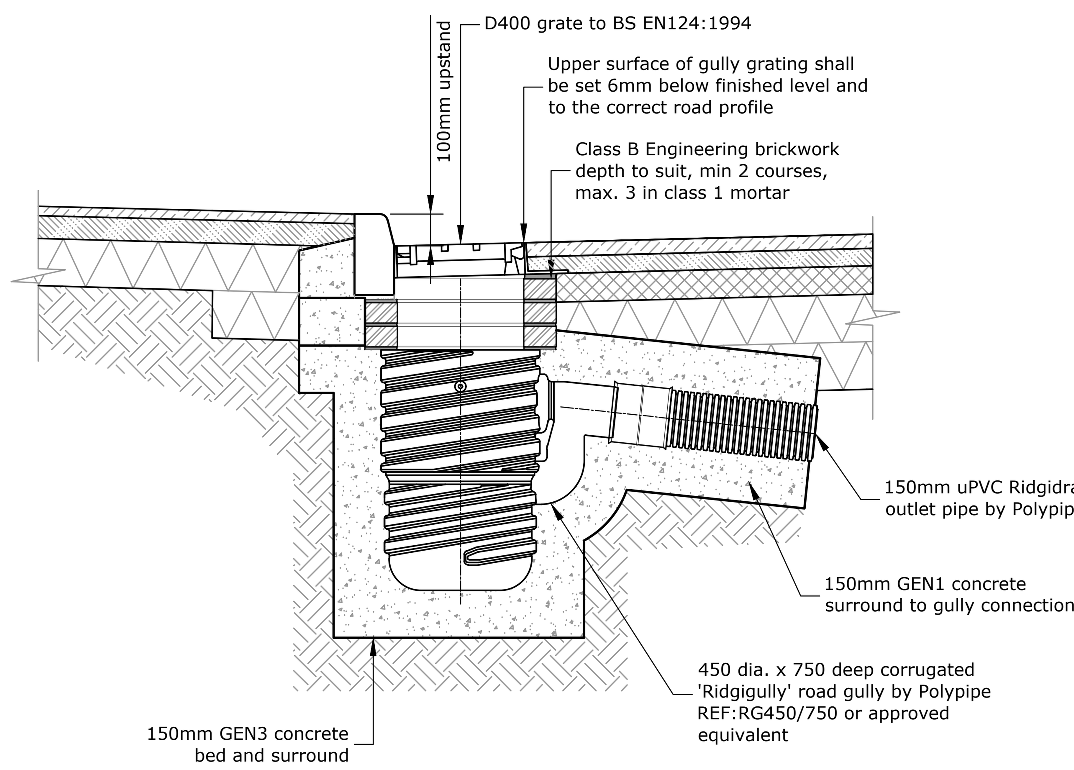
Typical Section Through
Exceedance Channel Past Plot 8
SCALE 1:20



Typical Inspection Chamber Detail
Scale 1:20



SW01 600Ø Chamber Detail
SCALE 1:20



Typical Road Gully Detail
SCALE 1:20

Rev	Description	Date	Revised by	Checked by	Approved

Issue Purpose: Planning

Do not scale from this drawing

R G PARKINS
Kendal | 01539 729393 Lancaster | 01524 32548

Scale @ A1: 1:20	First Issue: 22/02/23	Office of Origin: Kendal	
Drawn by: RH	Checked by: RW	Approved: TM	
Client: Nigel Kay Homes Ltd	Project No: K39479	Drawing No: 12	Rev:
Project: Arlecdon Road, Arlecdon			
Drawing Title: General Drainage Construction Details	BIM No:		

APPENDIX B

CALCULATIONS

Print Close Report

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Troy Melhuish
Site name:	Alrecdon Road
Site location:	Arlecdon

Site Details

Latitude:	54.55972° N
Longitude:	3.47445° W
Reference:	2926223088
Date:	Feb 23 2023 11:32

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.

Runoff estimation approach IH124

Site characteristics

Total site area (ha): 0.1305

Methodology

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

Soil characteristics	Default	Edited
SOIL type:	4	4
HOST class:	N/A	N/A
SPR/SPRHOST:	0.47	0.47

Hydrological characteristics	Default	Edited
SAAR (mm):	1379	1379
Hydrological region:	10	10
Growth curve factor 1 year:	0.87	0.87
Growth curve factor 30 years:	1.7	1.7
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):	1.39	1.39
1 in 1 year (l/s):	1.21	1.21
1 in 30 years (l/s):	2.37	2.37
1 in 100 year (l/s):	2.9	2.9
1 in 200 years (l/s):	3.3	3.3

Summary of Results for 100 year Return Period (+50%)

Half Drain Time : 643 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	177.371	0.221	0.0	1.4	1.4	30.0	O K
30 min Summer	177.463	0.313	0.0	1.4	1.4	42.6	O K
60 min Summer	177.572	0.422	0.0	1.4	1.4	57.4	O K
120 min Summer	177.660	0.510	0.0	1.4	1.4	69.3	O K
180 min Summer	177.715	0.565	0.0	1.4	1.4	76.7	O K
240 min Summer	177.751	0.601	0.0	1.4	1.4	81.6	O K
360 min Summer	177.792	0.642	0.0	1.4	1.4	87.3	O K
480 min Summer	177.810	0.660	0.0	1.4	1.4	89.7	O K
600 min Summer	177.816	0.666	0.0	1.4	1.4	90.5	O K
720 min Summer	177.818	0.668	0.0	1.4	1.4	90.7	O K
960 min Summer	177.813	0.663	0.0	1.4	1.4	90.0	O K
1440 min Summer	177.790	0.640	0.0	1.4	1.4	86.9	O K
2160 min Summer	177.740	0.590	0.0	1.4	1.4	80.1	O K
2880 min Summer	177.689	0.539	0.0	1.4	1.4	73.2	O K
4320 min Summer	177.585	0.435	0.0	1.4	1.4	59.1	O K
15 min Winter	177.398	0.248	0.0	1.4	1.4	33.7	O K
30 min Winter	177.503	0.353	0.0	1.4	1.4	47.9	O K
60 min Winter	177.626	0.476	0.0	1.4	1.4	64.7	O K
120 min Winter	177.728	0.578	0.0	1.4	1.4	78.5	O K
180 min Winter	177.790	0.640	0.0	1.4	1.4	86.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	121.194	0.0	30.5	22
30 min Summer	86.821	0.0	43.9	37
60 min Summer	59.586	0.0	60.9	66
120 min Summer	37.281	0.0	76.3	126
180 min Summer	28.419	0.0	87.2	184
240 min Summer	23.437	0.0	95.9	244
360 min Summer	17.805	0.0	109.3	362
480 min Summer	14.599	0.0	119.5	480
600 min Summer	12.487	0.0	127.7	552
720 min Summer	10.973	0.0	134.7	612
960 min Summer	8.919	0.0	145.9	742
1440 min Summer	6.631	0.0	162.3	1010
2160 min Summer	4.882	0.0	180.4	1428
2880 min Summer	3.937	0.0	193.9	1848
4320 min Summer	2.949	0.0	217.7	2600
15 min Winter	121.194	0.0	34.2	22
30 min Winter	86.821	0.0	49.2	36
60 min Winter	59.586	0.0	68.2	66
120 min Winter	37.281	0.0	85.4	124
180 min Winter	28.419	0.0	97.7	182

Summary of Results for 100 year Return Period (+50%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
240 min Winter	177.833	0.683	0.0	1.4	1.4	92.8	O K
360 min Winter	177.885	0.735	0.0	1.4	1.4	99.8	O K
480 min Winter	177.910	0.760	0.0	1.4	1.4	103.3	O K
600 min Winter	177.921	0.771	0.0	1.4	1.4	104.7	O K
720 min Winter	177.922	0.772	0.0	1.4	1.4	104.9	O K
960 min Winter	177.913	0.763	0.0	1.4	1.4	103.6	O K
1440 min Winter	177.881	0.731	0.0	1.4	1.4	99.2	O K
2160 min Winter	177.805	0.655	0.0	1.4	1.4	89.0	O K
2880 min Winter	177.727	0.577	0.0	1.4	1.4	78.4	O K
4320 min Winter	177.558	0.408	0.0	1.4	1.4	55.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	23.437	0.0	107.4	238
360 min Winter	17.805	0.0	122.4	354
480 min Winter	14.599	0.0	133.8	466
600 min Winter	12.487	0.0	143.0	574
720 min Winter	10.973	0.0	150.8	676
960 min Winter	8.919	0.0	163.3	772
1440 min Winter	6.631	0.0	181.4	1082
2160 min Winter	4.882	0.0	202.0	1544
2880 min Winter	3.937	0.0	217.2	1996
4320 min Winter	2.949	0.0	243.9	2772

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 304747 519278 NY 04747 19278
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	4320
Climate Change %	+50

Time Area Diagram

Total Area (ha) 0.137

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.068	4	8	0.069

Model Details

Storage is Online Cover Level (m) 178.800

Cellular Storage Structure

Invert Level (m) 177.150 Safety Factor 1.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	143.0	143.0	0.900	0.0	193.4
0.800	143.0	193.4			

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0059-1400-0800-1400
Design Head (m)	0.800
Design Flow (l/s)	1.4
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	59
Invert Level (m)	177.150
Minimum Outlet Pipe Diameter (mm)	75
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.800	1.4
Flush-Flo™	0.251	1.4
Kick-Flo®	0.510	1.1
Mean Flow over Head Range	-	1.2

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake® Optimum as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	1.2	1.200	1.7	3.000	2.6	7.000	3.8
0.200	1.4	1.400	1.8	3.500	2.7	7.500	3.9
0.300	1.4	1.600	1.9	4.000	2.9	8.000	4.0
0.400	1.3	1.800	2.0	4.500	3.1	8.500	4.2
0.500	1.2	2.000	2.1	5.000	3.2	9.000	4.3
0.600	1.2	2.200	2.2	5.500	3.4	9.500	4.4
0.800	1.4	2.400	2.3	6.000	3.5		
1.000	1.5	2.600	2.4	6.500	3.7		