R G PARKINS
Meadowside | Shap Road | Kendal | Cumbria | LA9 6NY
Tel:01539 729393
Email: mail@rgparkins.com

Drainage Strategy

Proposed Vehicle Body Workshop
Millom Road, Millom

W Milligan and Sons Ltd

Ref: K39647.DS/001

Version	Date	Prepared By	Checked By	Approved By
Original	28th February 2023	T. Melhuish	O. Sugden	O. Sugden

INDEMNITIES

This report is for the sole use and benefit of W Milligan and Sons Ltd and his professional advisors. RG Parkins & Partners Ltd will not be held responsible for any actions taken, nor decisions made, by any third party resulting from this report.

RG Parkins & Partners Ltd are not qualified to advise on contamination. Any comments contained within this report with regards to contamination are noted as guidance only and the Client should appoint a suitably qualified professional to provide informed advice. The absence of any comments regarding contamination does not represent any form of neglect, carelessness, or failure to undertake our service.

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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

1. INTRODUCTION

1.1 BACKGROUND

This report has been prepared by R. G. Parkins & Partners Ltd (RGP) for W Milligan and Sons Ltd in support of their proposal to construct a new motor body repair works on Millom Road in Millom, Cumbria, in accordance with the National Planning Policy Framework [1][2].

The scheme is currently in for Planning with Copeland District Council (ref: 4/22/2437/0F1), and a detailed drainage strategy has been requested by United Utilities (letter dated 16th January 2023) and CCC LLFA (letter dated 24th January 2023). The following report and associated drainage layout are therefore provided to address these requests.

2. SITE CHARACTERISATION

2.1 SITE LOCATION

The site is located off Millom Road in Millom (see Figure 2.1) at National Grid Reference SD 17772 80279. The total site area covered under the Planning Application is 3847m^2.

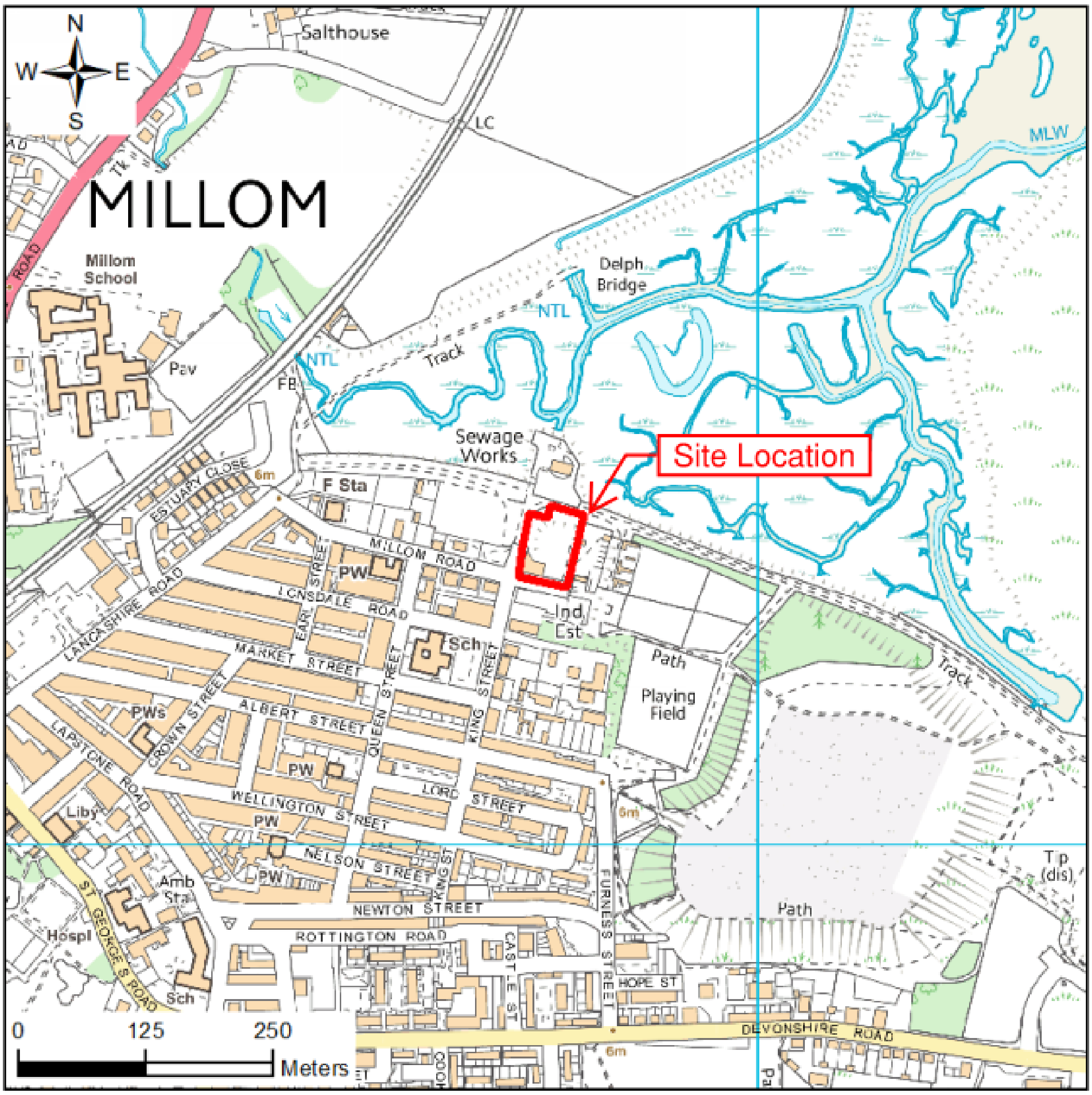

Figure 2.1 Site location

2.2 SITE DESCRIPTION

The site is currently partly occupied by the existing W Milligan and Sons Ltd garage with the remainder of the site being used for vehicle parking. The site is bounded to the north by a raised embankment, to the west by scrubland, to the south by the Millom urban area and to the east by further vehicle parking. The raised embankment is an old railway line that separates this part of Millom from the coastal salt marshes to the north. This embankment which is approximately 2.5-3.0 m higher than the site acts as a de-facto defence against coastal flooding although it is not classified as an Environment Agency asset. It is not clear at this stage who owns or maintains this embankment as a flood defence.

In terms of topography, most of the site is relatively flat with the southern boundary at an elevation of approx. 4.7 mAOD rising to the northern boundary at an elevation of 5.3 mAOD. The embankment beyond rises steeply to an elevation of c. 7.5m AOD.

The site is accessed through Millom via either Millom Road to the south or King Street to the west.

2.3 GEOLOGY & HYDROGEOLOGY

British Geological Survey (BGS)[4] and Land Information Systems (LandIS)[5] mapping indicates the site is underlain by the geological sequences outlined in Table 2.1.

Table 2.1 Site geological summary

Geological Unit	Classification	Description	Aquifer Classification
Soil	Soilscape 21	Loamy and clayey soils of coastal flats with naturally high groundwater	N/A
Drift	Raised Marine Deposits	Sand and gravel	Summary: Secondary Undifferentiated
Solid	Low Furness Basal Formation	Conglomerate And Sandstone, Interbedded	Summary: Secondary A

2.4 HYDROLOGY

The hydrology of the site has three components which are (i) rainfall runoff from the site; (ii) rainfall infiltration into the soils on the site and (iii) the proximity of the sea at high tide when it covers part or all of the Millom saltmarsh. The relative low elevation of the site relative to Millom saltmarsh will also mean that groundwater levels in the site are likely to be close to the surface.

2.5 EXISTING SEWERS

Reference to the United Utilities sewer records (Appendix A) indicates there is a 225 mm dia. surface water sewer located to the east and south of the site, which discharges into a 1200 mm dia. combined sewer located in the King Street to the west. This combined sewer runs to the sewage works located directly to the north of the site, where wastewater flows are pumped c. 1.2km to the east and into the main wastewater treatment works that serve Millom and the surrounding area. There is a combined sewer overflow from the sewage works that discharges into salt marshes.

The manhole at the head of the existing 225 mm dia. surface water sewer in the lane to the east of the site is a shallow brick-built chamber with a depth to invert level of 660mm (CL = 4.511 mAOD; IL = 3.851 mAOD). The next downstream manhole is 1.335m deep (CL = 5.130 mAOD; IL 3.795 mAOD) and the final surface manhole is 1.850m deep (CL = 5.148 mAOD; IL 3.298 mAOD). A small number of highways gullies discharge into this surface water sewer. Based on the existing topography, any site run-off would follow the contours and falls into Millom Road to the south. As such, pre-development run-off would enter the existing surface water sewer via the highways gully network and ultimately into the combined sewer and treatment works.

Immediately to the west of the existing building there is a short section of private combined drainage that takes both roof run-off and wastewater into the 1200 mm dia. public combined sewer. There is also a short section of private surface water drain to the front of the existing building that takes run-off across Millom Road and into the 225 mm dia. public surface water sewer.

2.6 GROUND INVESTIGATION

Ground investigation was undertaken at the site on 17th November 2022 by GEO Environmental Engineering Ltd[6]. The siteworks comprised 5no. window sample boreholes, in-situ standard penetration testing, gas and groundwater monitoring, laboratory geotechnical and contamination testing.

Geotechnically the ground is very poor, with Made Ground (up to 1.5m thick) over medium dense becoming very loose natural deposits. Shallow groundwater was encountered in all investigation locations, resulting in some of the boreholes being abandoned. In-situ percolation testing was not possible due to the shallow groundwater and as such infiltration-based SuDS will not be effective or feasible at the site. Ongoing monitoring has confirmed that groundwater levels have remained shallow over the winter months.

For further details refer to Geo Environmental Engineering Report No. GEO2022-5585.

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.

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 total site area covered under the Planning Application is 3847m^2.

Table 3.1 Land Cover Areas

Land Cover	Area		Percentage of total site area
	m²	Ha	
Building roof area	378	0.038	10%
Car parking area - rear	280	0.028	7%
Car parking area - front	70	0.007	2%
Concrete ramps	44	0.004	1%
Footpaths	15	0.002	0.5%
Remaining undeveloped areas	3060	0.306	79.5%

To develop the detailed drainage design, only certain surfaces and areas will be positively drained into the surface water network. Positively drained areas include the roof area, concrete ramps, footpaths and the car parking to the front. The rear car parking area is to be finished in a porous, gravel finish and will therefore not be positively drained. All other undeveloped areas will have a porous surface or will have no positive drainage. Table 3.2 summarises this.

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

Land Cover	Area		Percentage of total site area
	m²	Ha	
Total Positively Drained Area	507	0.051	13%
Remaining Undrained Area	3340	0.334	87%

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 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 or additional positively drained areas has been considered. Due to the nature of the development an additional uplift on impermeable area due to urban creep is not considered appropriate as any future redevelopment of the site would be subject to Planning Approval and a new drainage strategy.

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 507m^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	0.35
QBAR	0.41
Q10	0.56
Q30	0.69
Q100	0.85
Q100 + 50% CC	1.27

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.

a. Infiltration

Infiltration testing could not be undertaken at the site by GEO Environmental Engineering during the ground investigation works due to the shallow groundwater encountered. Groundwater levels will be influenced by the tidal range and seasonal variations and will likely be within 1m of any below ground soakaway during the wetter winter months. Groundwater monitoring has confirmed this to be the case. As such, disposal of surface water via infiltration-based SuDS is not considered feasible and is therefore ruled out.

b. Surface Water Body

The Duddon Estuary and associated salt marshes lie directly to the north at a distance of c. 100m. The area is designated as a Special Area of Conservation, SSSI and a protected site under Ramsar. There are no existing surface water drainage outfalls into the estuary within this area. The existing drainage that serves both the dwellings and public highways in Millom comprises various sizes of primarily public combined sewers which discharge into the UU sewage works and pump station located to the north. Due to the size and scale of the proposed vehicle repair workshop and sensitivity of the receiving estuary it is not considered feasible to discharge into this surface water body. Tidal influences would also result in flows being impounded and backing up and flooding into the site and adjacent highway network.

c. Surface Water Body

The nearby 225mm dia. public surface water sewer directly to the south of the site presents the most feasible point of discharge for attenuated surface water generated by the development. It is also clear that existing site (uncontrolled) run-off already enters this system via the highways gully network. UU's formal Planning response (letter dated 16th January 2023) already states that discharge into the nearby UU surface water sewer would be acceptable if infiltration drainage is not feasible. As such, it is proposed that discharge will be into this system via an on-site attenuation system with off-site flows limited to the Greenfield QBAR. Approval for the connection will be subject to a S106 application to United Utilities.

3.6 SURFACE WATER DRAINAGE DESIGN

It is proposed that surface water runoff from all positively drained areas will be attenuated within a single geocellular tank located under the car parking area to the rear of the development. A silt trap inspection chamber 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 wrapped in an impermeable geomembrane to prevent groundwater ingress or seepage. The detailed design and installation of the tank will need to take due consideration of groundwater inflow and potential floatation.

The tank will be accessible via a row of inspections cells to facilitate future access and maintenance. A mini-flow control chamber incorporating a 16mm orifice will be located downstream of the attenuation tank restricting discharge to the greenfield runoff rate (QBAR) of 0.4 l/s.

The car parking area to the front will be constructed using conventional asphalt surfacing and will drain into a channel drain located at the back of the highway kerb. The channel drain will discharge into the mini-flow control chamber, ensuring that there is no uncontrolled run-off onto the highway.

The rear car parking area will be re-constructed using an open-graded stone/gravel surface finish, to mimic and improve the current situation. Whilst it is recognised that shallow groundwater will impact infiltration during high tides or wet winter months, it is concluded that some shallow permeable surfaces should be utilised to reduce the impact on the downstream public sewers.

Full details of the drainage proposals are included on RGP drawing K39647-100 included in Appendix A.

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)
0.4	0.4	6.0 x 15.0	34.2

The critical storm event has been calculated to be the 480 min winter storm and under this event the water level within the tank will be 0.399m. 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 southern and eastern boundaries where the levels are lowest. Existing site runoff enters the highway and flows into the gully network.

In the unlikely event of blockage in the geocellular attenuation tank post-construction, exceedance flows will be flood from the silt trap inspection chamber (SW03) towards the new site access and into the lane to the east. Any exceedance flows from the mini-flow control chamber (SW01) to the front would fall directly into the existing gully located in Millom Road – refer to RGP drainage plan for details.

3.8.2 BUILDING LAYOUT & DETAIL

The finished floor level to the new building has been set at 5.0 mAOD to ensure that thresholds are not at risk of flooding from overland flow. Threshold levels have been set 150mm above external areas, and external footpath fall away from the thresholds, ensuring that any flood water runs away from, rather than towards the building.

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. Due to the scale of the development and the fact that the existing surface water sewer discharges into a combined sewer and ultimately into the wastewater treatment works that serves Millom, it is considered that proprietary treatment systems are not necessary for this scheme.

The drainage proposal primarily serves the roof area of the new building and as such the pollutant load from suspended solids, metals and hydrocarbons is very low. An upstream silt trap will be provided to remove silt prior to flows entering the geocellular tank. The rear car parking area will have a porous finish and any pollutants would infiltrate into the ground and not enter the drainage system. The area of car parking to the front is very small and does not constitute an additional level of treatment, other than the channel drain and sump solution proposed.

3.10 OPERATIONS & MAINTENANCE RESPONSIBILITY

All on site drainage will remain private and will be maintained by the site owner. An '*Operations & Maintenance Plan'* (K39647.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 building shall be drained via a private gravity foul water drain to the rear of the building and car parking area with discharge into the existing 1200mm dia. combined sewer. A new saddle connection will be required, and the connection will be subject to a S106 application to United Utilities.

5. CONCLUSIONS AND RECOMMENDATIONS

The proposed Drainage Strategy can be summarised as follows:

- Ground investigation undertaken on 17th November 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 6m x 15m x 0.4m deep, thereby providing 34.2m^3 of storage within the site.

- A mini-flow control chamber incorporating a 16mm orifice will be located downstream of the geocellular tank restricting discharge to the greenfield development QBAR runoff rate of 0.4 lit/sec. The tank will be wrapped in an impermeable geomembrane to prevent groundwater ingress or seepage. The detailed design and installation of the tank will need to take due consideration of groundwater inflow and potential floatation.

- The rear car parking area will be constructed in an open graded stone/gravel surfacing and will not be positively drained into the system. The small front car parking area will be constructed in conventional impermeable surfacing and will drain into the system via a channel drain.

- Attenuated discharge from the site shall be into the existing 225mm dia. public surface water sewer located to the south of the site via a lateral connection. Approval for the connection will be subject to a S106 application to United Utilities.

- Exceedance flows in the event of blockage would either be towards the lane to the east or Millom Road to the south.

- All on site drainage will remain private and will be maintained by the site owner. An '*Operations & Maintenance Plan'* has been prepared detailing the requirements for future maintenance of the drainage system.

- Foul water from the building shall be drained via a private gravity foul water drain to the rear of the building and car parking area with discharge into the existing 1200mm dia. combined sewer. A new saddle connection will be required, and the connection will be subject to a S106 application to United Utilities.

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

[4] Land Information System (LANDIS)- Soilscapes viewer, Accessed December 2022 http://www.landis.org.uk/soilscapes

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[6] GEO Environmental Engineering Ltd, March 2023. GEO2022-5585: Millom Road, Millom- Ground Investigation Report.

[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.

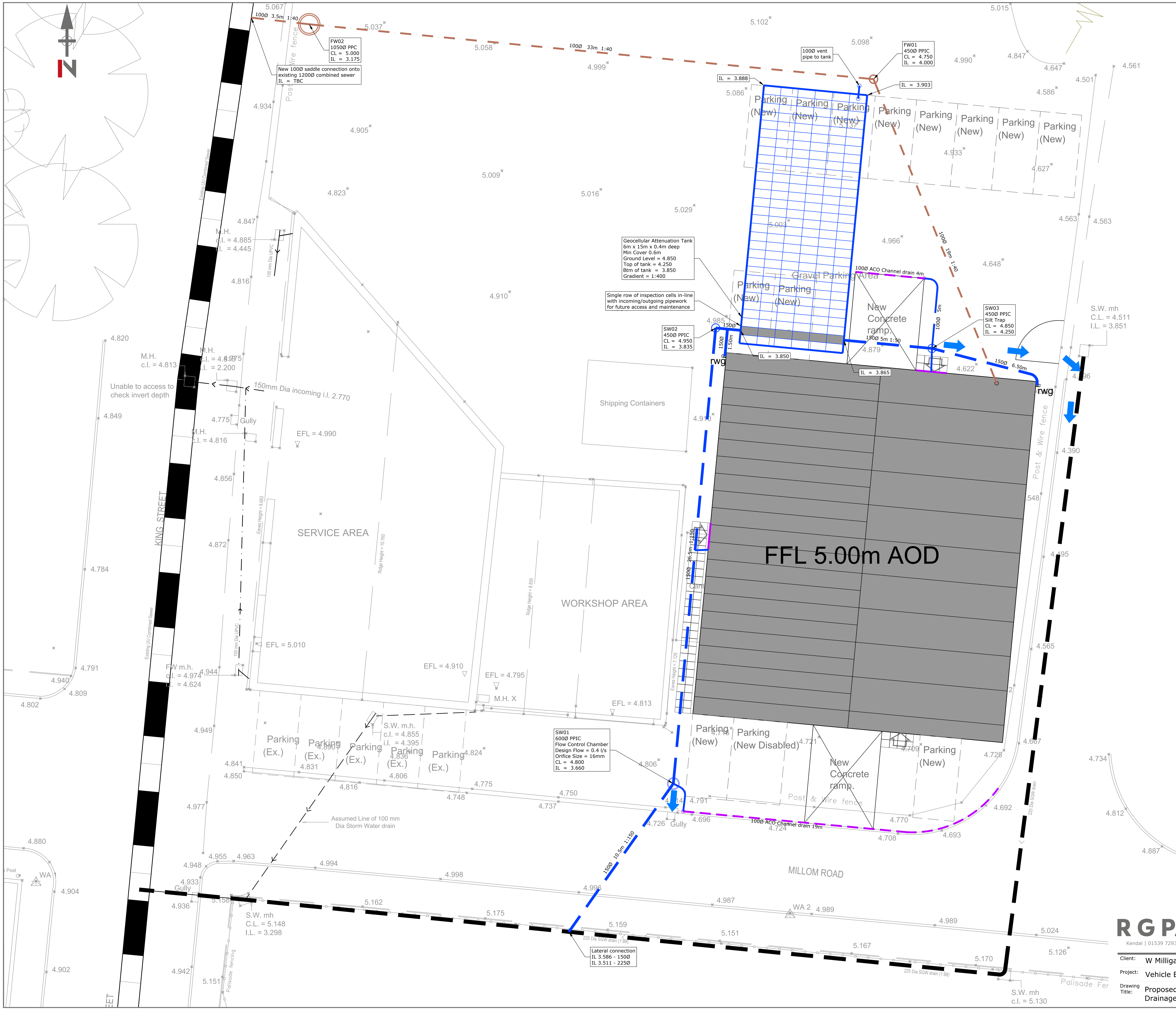
[15] Department for Environment, Food and Rural Affairs, Non-Statutory Technical Standards for Sustainable Drainage Systems, March 2015

[16] Innovyze, 2022, Micro Drainage Source Control

[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 225Ø UU Surface Water Public Sewer
- Existing 1200Ø UU Combined Water Public Sewer
- Proposed 100/150Ø Surface Water Plot Drainage
- Proposed 100Ø Foul Water Plot Drainage
- Proposed 100Ø Aco Channel Drain with sump unit
- 460Ø PPIC Manhole Inspection Chamber
- Route of exceedance flows from attenuation tank

R G PARKINS
Kendal | 01539 729393 Lancaster | 01524 32548

Client:	W Milligan & Sons Ltd	Scale @ A1: 1:100	First Issue: 28/02/23	Office of Origin: Kendal
Project:	Vehicle Body Workshop, Millom	Drawn by: CA	Checked by: TM	Approved: TM
Drawing Title:	Proposed Foul & Surface Water Drainage Plan	Project No: K39647	Drawing No: 100	Rev:
		BIM No:		

APPENDIX B

CALCULATIONS

R G PARKINS
97 King Street | Lancaster | LA1 1RH
Tel:01524 32548
Email: office@rgparkinslancaster.co.uk

Wallingford Runoff Estimation	Job Number K39647	Page Number 1 of 4	
	Calc by CA	Check by TM	
Vehicle Body Workshop Millom	Date 28/02/2023	Revised	

DESIGN BASIS MEMORANDUM - PEAK RATE OF RUN-OFF CALCULATION

Design Brief
The following peak rate of run-off calculations have been undertaken to determine changes in peak flow resulting from the development of a greenfield or brownfield site. These calculations are for the **Peak Rate of Run-Off** requirements only.

Background Information & References
The site area **is less than** 200ha and the Greenfield (pre-development) calculation has been undertaken in accordance with methodology described by Marshall & Bayliss, Institute of Hydrology, Report No. 124, Flood Estimation for Small Catchments, 1994 (IoH 124).

In addition, the following references have been used in the preparation of these calculations:

- Interim Code of Practice for Sustainable Drainage Systems (SUDS), CIRIA, 2004
- CIRIA, The SUDS Manual, Report C753, 2015
- Designing for Exceedance in Urban Drainage - good practice, CIRIA Report C635, 2006
- Flood Estimation Handbook (FEH)
- Flood Studies Report (FSR), Volume 1, Hydrological Studies, 1993
- Flood Studies Supplementary Report No 2 (FSSR2), The Estimation of Low Return Period Floods
- Flood Studies Supplementary Report No 14 (FSSR14), Review of Regional Growth Curves, 1983
- Planning Practice guidance of the National Planning Policy Framework, Recommended national precautionary sensitivity ranges for peak rainfall intensities, peak river flows, offshore wind speeds and wave heights.

Proposed Land Use Changes
Changes to the existing site are as follows:

Greenfield Site to Redevelopment Site

Results Summary

Rate of Run-Off (l/s)	
Event	**Greenfield**
Q1	0.35
QBAR	0.41
Q10	0.56
Q30	0.69
Q100	0.85
Q100 + 50% CC	1.27

97 King Street | Lancaster | LA1 1RH
Tel:01524 32548
Email: office@rgparkinslancaster.co.uk

SITE AREAS (LAND COVER AREAS)

Existing Impermeable & Permeable Land Cover

Total Site Area: **0.3847** ha **3847** m²

Proposed Land Cover Areas

Land Cover	Area		Percentage of total site area
	m²	ha	
Building roof area	378.0	0.038	10%
Car parking area - rear	280.0	0.028	7%
Car parking area - front	70.0	0.007	2%
Concrete ramps	44.0	0.004	1%
Footpaths	15.0	0.002	0.5%
Remaining undeveloped areas	3060.0	0.306	79.5%

Proposed Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total positively drained area	507.0	0.051	13%
Remaining undrained area	3340.0	0.334	87%

R G PARKINS
97 King Street | Lancaster | LA1 1RH
Tel:01524 32548
Email: office@rgparkinslancaster.co.uk

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Vehicle Body Workshop Millom	Date 28/02/2023		Revised

ESTIMATION OF QBAR (RURAL) (GREENFIELD RUNOFF RATE)

IoH 124 based on research on small catchments < 25 km2

Method is based on regression analysis of response times
using catchments from 0.9 to 22.9 km^2

$QBAR_{rural}$ is mean annual flood on rural catchment
$QBAR_{rural}$ depends on SOIL, SAAR and AREA most significantly

$QBAR_{rural}$ = $\boxed{0.00108 \times AREA^{0.89} \times SAAR^{1.17} \times SOIL^{2.17}}$

For SOIL refer to FSR Vol 1, Section 4.2.3 and 4.2.6 and IoH 124

Contributing watershed area

Area, A	=	500000	m^2	insert 50 ha for EA
	=	0.500	km^2	small catchment method
	=	50.000	ha	
SAAR	=	**1082**	mm	From FEH Web Service (point data)

Soil index based on soil type, SOIL = $\frac{(0.1S1+0.3S2+0.37S3+0.47S4+0.53S5)}{(S1+S2+S3+S4+S5)}$

Where: S1	=		%	
S2	=		%	
S3	=		%	
S4	=	**100**	%	
S5	=		%	
		100	%	
So, SOIL	=	**0.47**		

UK Suds website provides a value of 4 based on the equivalent Host value.

Note: for very small catchments it is far better to rely on local site investigation information.

$QBAR_{rural}$	=	0.402	m^3/s
	=	401.8	l/s

Small rural catchments less than 50 ha
The Environment Agency recommends that this method should be used for development sizes from
0 to 50 ha and should linearly interpolate the formula to 50 ha.

So, catchment size	=	**507**	m^2	Excluding significant open space which would remain disconnected from the positive drainage system during flood events.
	=	0.001	km^2	
	=	0.051	ha	
$QBAR_{rural\ site}$	=	0.00041	m^3/s	
	=	**0.41**	l/s	

R G PARKINS
97 King Street | Lancaster | LA1 1RH
Tel:01524 32548
Email: office@rgparkinslancaster.co.uk

GREENFIELD RETURN PERIOD ORDINATES

QBAR can be factored by the UK FSR regional growth curves for return periods <2 years and for all other return periods to obtain peak flow estimates for required return periods.

These regional growth curves are constant throughout a region, whatever the catchment type and size.

See Table 2.39 for region curve ordinates
Use FSSR2 Growth Curves to estimate Qbar

Reference- Pg 173-FSR V.1, ch 2.6.2

Region = **10**

Use Figure A1.1 to determine region

GREENFIELD RETURN PERIOD FLOW RATES

Return Period	Ordinate	Q (l/s)
1	0.87	0.35
2	0.93	0.38
5	1.19	0.48
10	1.38	0.56
25	1.64	0.67
30	1.7	0.69
50	1.85	0.75
100	2.08	0.85
200	2.32	0.95
500	2.73	1.11
1000	3.04	1.24

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

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

Half Drain Time : 781 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	3.978	0.128	0.0	0.3	0.3	11.0	O K
30 min Summer	4.030	0.180	0.0	0.3	0.3	15.4	O K
60 min Summer	4.090	0.240	0.0	0.4	0.4	20.5	O K
120 min Summer	4.136	0.286	0.0	0.4	0.4	24.4	O K
180 min Summer	4.162	0.312	0.0	0.4	0.4	26.6	O K
240 min Summer	4.177	0.327	0.0	0.4	0.4	28.0	O K
360 min Summer	4.192	0.342	0.0	0.4	0.4	29.3	O K
480 min Summer	4.195	0.345	0.0	0.4	0.4	29.5	O K
600 min Summer	4.191	0.341	0.0	0.4	0.4	29.1	O K
720 min Summer	4.185	0.335	0.0	0.4	0.4	28.6	O K
960 min Summer	4.171	0.321	0.0	0.4	0.4	27.5	O K
1440 min Summer	4.145	0.295	0.0	0.4	0.4	25.2	O K
2160 min Summer	4.120	0.270	0.0	0.4	0.4	23.1	O K
2880 min Summer	4.101	0.251	0.0	0.4	0.4	21.5	O K
4320 min Summer	4.070	0.220	0.0	0.3	0.3	18.8	O K
5760 min Summer	4.045	0.195	0.0	0.3	0.3	16.7	O K
7200 min Summer	4.026	0.176	0.0	0.3	0.3	15.0	O K
8640 min Summer	4.009	0.159	0.0	0.3	0.3	13.6	O K
10080 min Summer	3.996	0.146	0.0	0.3	0.3	12.5	O K
15 min Winter	3.994	0.144	0.0	0.3	0.3	12.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	118.525	0.0	11.3	22
30 min Summer	83.819	0.0	16.0	37
60 min Summer	56.764	0.0	21.7	66
120 min Summer	34.926	0.0	26.7	126
180 min Summer	26.220	0.0	30.0	184
240 min Summer	21.311	0.0	32.6	244
360 min Summer	15.761	0.0	36.1	362
480 min Summer	12.637	0.0	38.6	480
600 min Summer	10.607	0.0	40.5	574
720 min Summer	9.173	0.0	42.1	620
960 min Summer	7.265	0.0	44.4	744
1440 min Summer	5.203	0.0	47.7	1010
2160 min Summer	3.772	0.0	51.9	1424
2880 min Summer	3.029	0.0	55.6	1824
4320 min Summer	2.247	0.0	61.9	2640
5760 min Summer	1.840	0.0	67.5	3408
7200 min Summer	1.591	0.0	72.9	4184
8640 min Summer	1.423	0.0	78.3	4936
10080 min Summer	1.303	0.0	83.7	5744
15 min Winter	118.525	0.0	12.6	22

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
30 min Winter	4.053	0.203	0.0	0.3	0.3	17.3	O K
60 min Winter	4.121	0.271	0.0	0.4	0.4	23.1	O K
120 min Winter	4.173	0.323	0.0	0.4	0.4	27.6	O K
180 min Winter	4.204	0.354	0.0	0.4	0.4	30.2	O K
240 min Winter	4.223	0.373	0.0	0.4	0.4	31.9	O K
360 min Winter	4.243	0.393	0.0	0.4	0.4	33.6	O K
480 min Winter	4.249	0.399	0.0	0.4	0.4	34.1	O K
600 min Winter	4.247	0.397	0.0	0.4	0.4	34.0	O K
720 min Winter	4.242	0.392	0.0	0.4	0.4	33.5	O K
960 min Winter	4.226	0.376	0.0	0.4	0.4	32.1	O K
1440 min Winter	4.195	0.345	0.0	0.4	0.4	29.5	O K
2160 min Winter	4.157	0.307	0.0	0.4	0.4	26.3	O K
2880 min Winter	4.128	0.278	0.0	0.4	0.4	23.8	O K
4320 min Winter	4.079	0.229	0.0	0.3	0.3	19.6	O K
5760 min Winter	4.040	0.190	0.0	0.3	0.3	16.3	O K
7200 min Winter	4.010	0.160	0.0	0.3	0.3	13.7	O K
8640 min Winter	3.985	0.135	0.0	0.3	0.3	11.5	O K
10080 min Winter	3.965	0.115	0.0	0.3	0.3	9.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
30 min Winter	83.819	0.0	17.9	37
60 min Winter	56.764	0.0	24.3	66
120 min Winter	34.926	0.0	29.9	124
180 min Winter	26.220	0.0	33.6	182
240 min Winter	21.311	0.0	36.5	240
360 min Winter	15.761	0.0	40.5	354
480 min Winter	12.637	0.0	43.3	466
600 min Winter	10.607	0.0	45.4	576
720 min Winter	9.173	0.0	47.1	682
960 min Winter	7.265	0.0	49.8	782
1440 min Winter	5.203	0.0	53.4	1082
2160 min Winter	3.772	0.0	58.1	1540
2880 min Winter	3.029	0.0	62.3	1988
4320 min Winter	2.247	0.0	69.3	2816
5760 min Winter	1.840	0.0	75.6	3640
7200 min Winter	1.591	0.0	81.8	4464
8640 min Winter	1.423	0.0	87.7	5192
10080 min Winter	1.303	0.0	93.8	5952

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 317742 480299 SD 17742 80299
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+50

Time Area Diagram

Total Area (ha) 0.051

Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)
0	4	0.026	4	8	0.025

Model Details

Storage is Online Cover Level (m) 4.950

Cellular Storage Structure

Invert Level (m) 3.850 Safety Factor 2.0
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	90.0	90.0	0.500	0.0	106.8
0.400	90.0	106.8			

Orifice Outflow Control

Diameter (m) 0.016 Discharge Coefficient 0.600 Invert Level (m) 3.650