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Drainage Strategy

New Petrol Filling Station, Frizington

M & L Richardson & Sons Ltd

Ref: K38912.DS/001

Version	Date	Prepared By	Checked By	Approved By
Original	01/04/2022	J. Brabbins	R. Heron	R. Warwick

Registered Office: RG Parkins & Partners Ltd, Meadowside, Shap Road, Kendal, LA9 6NY
Registered in England & Wales - Company number 04107150

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6. 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

7. INTRODUCTION

7.1 BACKGROUND

This report has been prepared by R. G. Parkins & Partners Ltd (RGP) for M & L Richardson & Sons Ltd in support of their proposals to demolish the vacant Griffin Hotel to allow for the construction of a new petrol filling station (PFS) and an extension to the existing retail store.

RGP has been appointed to undertake a Surface and Foul Water Drainage Strategy in accordance with the National Planning Policy Framework (NPPF) [1][2] to support a planning application that fulfils the requirements of the Local Planning Authority, Environment Agency and the Sewerage Undertaker.

The site is located within Flood Zone 1 and owning to the size of the development (less than 1ha) [3], a Flood Risk Assessment is not required.

8. SITE CHARACTERISATION

8.1 SITE LOCATION

The site is located on the junction of Mill Street and Main Street in Frizington (Figure 8.1). The National Grid Co-Ordinates to the centre of the site are 303360E 517190N.

Figure 8.1 Site Location

8.2 SITE DESCRIPTION

The existing area proposed for redevelopment is approximately 0.229 ha and is classed as Brownfield. The site is currently occupied by The Griffin, with the frontage of the pub facing Mill Street. This public house is vacant/abandoned and is due to be demolished as part of the works. Parking for the former pub wraps around the building to the south west and south east, with a green area to the north west.

The Spar and Post Office building lies adjacent to Main Street, with parking to the rear, and the access road to the immediate south.

The site is bounded by residential dwellings on all sides, with Mill Street and Main Street to the south east and south west.

There are currently 2 no. access points to the site from Main Street and Mill Street.

8.3 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 8.1. The EA Groundwater Vulnerability Map [6] indicates there is a Groundwater Source Protection Zone (Zone III Total Catchment) 5.8 km south west of the site. The development site overlies a major aquifer with 'Medium' vulnerability with soluble rock risk. The development site is not located within a Drinking Water Safeguard Zone for either surface or groundwater.

Table 8.1 Site Geological Summary

Geological Unit	Classification	Description	Aquifer Classification
Soil	Soilscape 18	Slowly permeable seasonally wet slightly acid but base rich loamy and clayey soils	N/A
Drift	Till Devensian	Diamicton- clay, silt, sand and gravel	Secondary (undifferentiated)
Solid	Pennine Middle Coal	Mudstone, siltstone and sandstone	Secondary A

As the site is currently an existing public house with asphalt hardstanding surrounding it is expected that the ground make-up may be variable with a high chance of encountering made ground.

8.4 HYDROLOGY

Reference to the topographic survey and OS mapping indicates there is a watercourse, Lingla Beck crossing under Mill Street, c. 263 m north west of the site. It flows in a southerly direction, passing along the rear of the properties on Lingley Road.

8.5 EXISTING SEWERS

Reference to United Utilities Sewer Records indicates there is a 225 dia. combined sewer running from north to south west along the Spar/Post office's north west boundary. It passes under the access road, connecting into a 450 dia. combined sewer in Main Street.

There is also a 150 dia. foul water sewer in Mill Street, c. 63 m from The Griffin, passing through a dwelling on Mill Street. It discharges into a 225 dia. combined sewer that flows in a westerly direction, towards the rear of the properties on Lingley Road, before turning south.

9. SURFACE WATER DRAINAGE STRATEGY

9.1 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]
- Non Statutory Technical Standards for Sustainable Drainage Systems, Defra, March 2015 [15]

The following assessment and drainage strategy are based on the latest site layout plan by Harry Walters & Livesey Ltd (drawing no. 453-16-P2). Any alterations to the site plan resulting in changes to impermeable areas will require the drainage strategy to be revisited.

9.2 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 9.1 shows the measured proposed land cover areas. The highest percentage is hardstanding/ car parking areas at 61% of the total site area. Existing roof areas cover 14%, landscaped areas 14%, forecourt 9%, forecourt cover 6% and proposed roof areas 2%.

Table 9.1 Land Cover Areas

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Proposed Roof Area	52	0.005	2%
Existing Roof Area	322	0.032	14%
Forecourt Area	202	0.020	9%
Forecourt Cover	145	0.015	6%
Total Hardstanding/Car Parking	1395	0.139	61%
Landscaped Areas	317	0.032	14%

Note: The total area in the above table is 106% of the site area as the area beneath the forecourt cover has also been included in the forecourt area.

The site can be subdivided into land cover that could be permeable and that which could be impermeable. Potential impermeable areas are regarded as roof areas, parking, and hardstanding. All other areas (landscaped areas) are regarded as having a permeable surface. Table 9.2 gives the areas of potentially permeable and impermeable land cover. This shows that impermeable areas could cover 86% of the site and permeable areas 14%.

Table 9.2 Area of Potentially Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Total Impermeable Area	1971	0.197	86%
Remaining Permeable Area	317	0.032	14%

The site has previously been developed (brownfield), with a former public house and local convenience store currently on the site. The existing impermeable and permeable land cover can be directly comparable to determine the impact of the redevelopment. Table 9.3 shows that the proposed redevelopment of the site will result in a minor decrease in surface water runoff.

Table 9.3 Area of Existing Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Existing Impermeable Area	2065	0.207	90%
Existing Permeable Area	225	0.023	10%

9.3 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:

9.3.1 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.

Climate change guidance is issued by the Environment Agency and outlines the anticipated changes to extreme rainfall intensity. Table 9.4 shows anticipated changes in extreme rainfall intensity in small and urban catchments. Guidance states that for site-specific flood risk assessments and strategic flood risk assessments, the upper end allowance should be assessed. A climate change allowance of 40% has been selected for the purpose of drainage design based on the 100-year anticipated design life of the proposed development.

Table 9.4 Peak Rainfall Intensity Allowance in Small and Urban Catchments

Applies across all of England	Total potential change anticipated for the '2020s' (2015 to 2039)	Total potential change anticipated for the '2050s' (2040 to 2069)	Total potential change anticipated for the '2080s' (2070 to 2115)
Upper End	10%	20%	40%
Central	5%	10%	20%

9.3.2 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.

9.3.3 VOLUMETRIC RUNOFF COEFFICIENT (CV)

The volumetric runoff coefficient describes the volume of rainfall 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.

9.3.4 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.

9.4 PRE-DEVELOPMENT RUNOFF ASSESSMENT

As the Brownfield 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.

Based on the anticipated design life of the proposed development (100 years), an increase in peak runoff of 40% has been used in the calculations for the post development rate of runoff to account for climate change. Peak runoff rates have been calculated for: (i) the greenfield site with 70% impermeable area contributing to the proposed surface water drainage network and (ii) the current site as 90% brownfield.

Full details of the calculations and the methodology for deriving the Peak Rate of Runoff are in included in Appendix B. A summary of the results is included in Table 9.5.

Table 9.5 Pre-Development Peak Runoff Rates

Rate of Runoff (l/s)		
Event	Greenfield	Brownfield Pre Development
Q1	1.4	20.5
QBAR	1.7	30.2
Q10	2.3	41.1
Q30	2.8	50.3
Q100	3.4	64.9
Q100+ 40% CC	4.8	90.9

Without attenuation or infiltration, the proposed development would decrease the rate of runoff from the developed areas of the site. This decrease is due to the minor increase in landscaped areas.

9.5 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.

Due to the proposed site use as well as predicted ground conditions, it is anticipated that the disposal of surface water via infiltration will be unviable.

The entire impermeable area of the site will require a positive drainage solution. Runoff will be attenuated as far as practical, with discharge to the UU public 225mm dia. combined sewer situated within the site, subject to agreement with UU.

9.6 SURFACE WATER DRAINAGE DESIGN

For clarity, runoff from the forecourt will be identified as foul water due to the potentially high pollutant load. For further information see Section 10.

It is proposed that the existing shop on the site will maintain existing drainage connections.

The proposed shop extension will discharge to the proposed new surface water network and any foul connections will discharge to the proposed new foul water network.

ACO RoadDrains will be utilised to collect surface water runoff from the forecourt area of the site. The channel drains will be connected to a Full Retention Class 1 separator with alarm to ensure sufficient treatment of surface water runoff from the forecourt area. Eventual discharge shall be to the existing 225mm dia. combined sewer within the site. Combined systems often have overflows directly to watercourses during rainfall events and therefore discharges to them should be treated as direct discharges and passed through a Class 1 separator.

Full retention separators treat the full flow that can be achieved by the drainage system, while a Class 1 is designed to achieve a concentration of less than 5mg/l of oil under standard test conditions. Class 1 separators should be used when the separator is required to remove very small oil droplets.

Typical road gullies at the low-point on the site as well as ACO KerbDrains and RoadDrains (or similar approved) shall be used to collect surface water runoff from the hardstanding areas. These components shall be connected to the new proposed surface water drainage system which will utilise a bypass separator to treat runoff from the remaining areas of the hardstanding not served by the full retention separator, as well as runoff from the forecourt cover. A bypass separator is used when it is considered an acceptable risk not to provide full treatment, they are used where the risk of a large spillage and heavy rainfall occurring at the same time is small.

It is proposed to discharge the surface water to the existing combined sewer running through the proposed site via a new manhole. The surface water will be discharged at a rate to match the greenfield Qbar rate outlined in Table 9.5 (1.7 l/s). Attenuation of surface water will be provided by geocellular crates situated beneath the proposed parking bays to the north of the site. The discharge rate will be restricted by a Hydrobrake flow control device (ref SHE-0064-1700-0811-1700) located in a manhole immediately downstream of the geocellular attenuation structure.

A silt trap will be provided upstream of the proposed bypass separator to avoid siltation of the separator, attenuation, and flow control device.

Although United Utilities will not adopt the surface water system, the surface water drainage network for the positively drained areas shall be constructed to adoptable standards where possible. The level of the receiving combined sewer is relatively shallow, therefore the proposed new surface water system is also at a shallow depth. Where cover levels to the soffit of pipes is less than 1.2m, the pipes should be constructed with concrete surround. The layout of sewers has been considered to minimise excessive gradients, depth and ensure appropriate bends / junctions.

Microdrainage Source Control calculations for the proposal are included in Appendix B. Following planning approval, a more detailed network model shall be completed which may result in slightly reduced storage requirements.

Table 9.6 Breakdown of Drainage Areas

Land Cover	Area		Percentage of total site area
	m^2	ha	
Impermeable Area to SW network	1592	0.159	70%
Impermeable Area to FW network	202	0.020	9%
Impermeable Area to drain as existing	322	0.032	14%
Remaining Permeable Area	317	0.032	14%

For further detail refer to the Drainage Layout Plan (K38912/A1/20) included in Appendix A.

9.7 STORAGE VOLUME

The drainage design has been sized to attenuate runoff during a Q100 event, plus a 40% allowance for future climate change across the design life of the development (100 years). The outline storage estimate has been undertaken using Micro Drainage Source Control, with FEH point descriptors used to model the rainfall and determine the volume of attenuation required.

The storage volume required is 108.8 m^3, and this can be achieved utilising a geocellular crate structure with dimensions of 24m x 6m x 0.8m.

9.8 DESIGNING FOR LOCAL DRAINAGE SYSTEM FAILURE

In accordance with the general principles discussed in CIRIA Report C635 – Designing for Exceedance in Urban Drainage [13] the proposed surface water drainage, where practical, should be designed to ensure there is no increased risk of flooding on the site or elsewhere as a result of extreme rainfall, lack of maintenance, blockages or other causes. These measures are discussed below.

Surface Storage & External Levels –where possible the hardstanding should be 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 should be raised to channel surface water runoff back into the drainage system.

Drainage Contingency – the proposed surface water system will be designed to provide adequate storage volume against flooding for the Q100 event, including a 40% allowance to account for climate change.

Building Layout & Detail – the site will be designed to ensure that the shop is not at risk of flooding from overland flow. The finished floor and threshold level of the proposed shop extension will be in line with existing shop levels, while external footpaths will fall away, ensuring that any flood water runs away from, rather than towards the building.

9.9 SURFACE WATER QUALITY

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.

A number of 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 on site use. For petrol filling stations, the pollutant load is potentially high.

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 in order to treat the runoff. Due to site constraints, it is not possible to provide a SuDS treatment train to sufficiently reduce the pollutant load to acceptable standards for discharge to the combined sewer. For this reason, a bypass separator is required to treat all surface water runoff from the hardstanding areas. Surface water runoff from the roof of the proposed extension will have a low hazard, therefore can be connected to the surface water drainage system downstream of the bypass separator.

9.10 OPERATIONS & MAINTENANCE RESPONSIBILITY

Drainage shall be privately maintained by the site owner. An '*Operations & Maintenance Plan'* will be made available to the site owners upon request detailing the requirements for future maintenance of the drainage system. This can be progressed following planning.

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10. FOUL WATER DRAINAGE STRATEGY

It is proposed that foul water from the development shall be drained via gravity and connected to the existing 225mm dia. combined sewer within the site via an existing manhole.

There are potentially two sources of foul water from the proposed site; runoff from the forecourt, and foul waste from the building.

The forecourt will be surrounded by ACO RoadDrains to collect any runoff from the proposed forecourt. The runoff will then be discharged via a forecourt separator to an existing manhole within the site.

Preliminary foul water discharge calculations should be undertaken in accordance with the British Water Code of Practice Flows and Loads 4. The foul flow from the site will be dependent on the number of waste connections from the site and people using the site.

For further detail refer to the Drainage Layout Plan included in Appendix A.

11. CONCLUSIONS AND RECOMMENDATIONS

In consideration of the Drainage Strategy for the site, the following conclusions and recommendations are made:

- It is predicted that the underlying soils at the site will not be suitable for infiltration type drainage systems. Due to the proposed site use as a petrol filling station, infiltration drainage is advised against due to potential high pollutant load. In line with the SuDS hierarchy, and in the absence of a watercourse / surface water sewer, surface water shall discharge to the 225mm dia. public combined sewer within the site.

- Existing foul and surface water connections from the shop will remain.

- It is proposed that the hardstanding areas will be served by typical highway gullies, ACO KerbDrains, and ACO RoadDrains (or similar approved) with attenuation provided within geocellular crates, and treatment provided by a bypass separator. Roof areas (proposed extension) shall connect to the surface water drainage system downstream of the bypass separator. Discharge shall be restricted via a flow control device, to the pre-development greenfield Qbar rate of 1.7 L/s, thereby providing significant betterment in comparison to the existing drainage situation. Discharge is proposed to the existing combined sewer within the site via a new manhole subject to agreement from United Utilities.

- Any foul flows from the proposed extension shall discharge directly into the existing combined sewer via an existing manhole within the site. Foul flows from the forecourt shall also be discharged to the same manhole within the site, via a forecourt separator subject to agreement from United Utilities. A pre-development enquiry has been submitted to ascertain whether the proposals in principle are acceptable to UU.

- It is proposed that drainage shall be privately maintained by the site owner/s. An '*Operations & Maintenance Plan'* will be made available to the site owners upon requset detailing the requirements for future maintenance of the drainage system.

12. 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, October 2021

[3] Defra/Environment Agency, The Town and Country Planning Order 2015, 2015 No.595, April 2015.

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

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

[6] Defra Magic Maps, 2022. https://magic.defra.gov.uk/MagicMap.aspx.

[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.

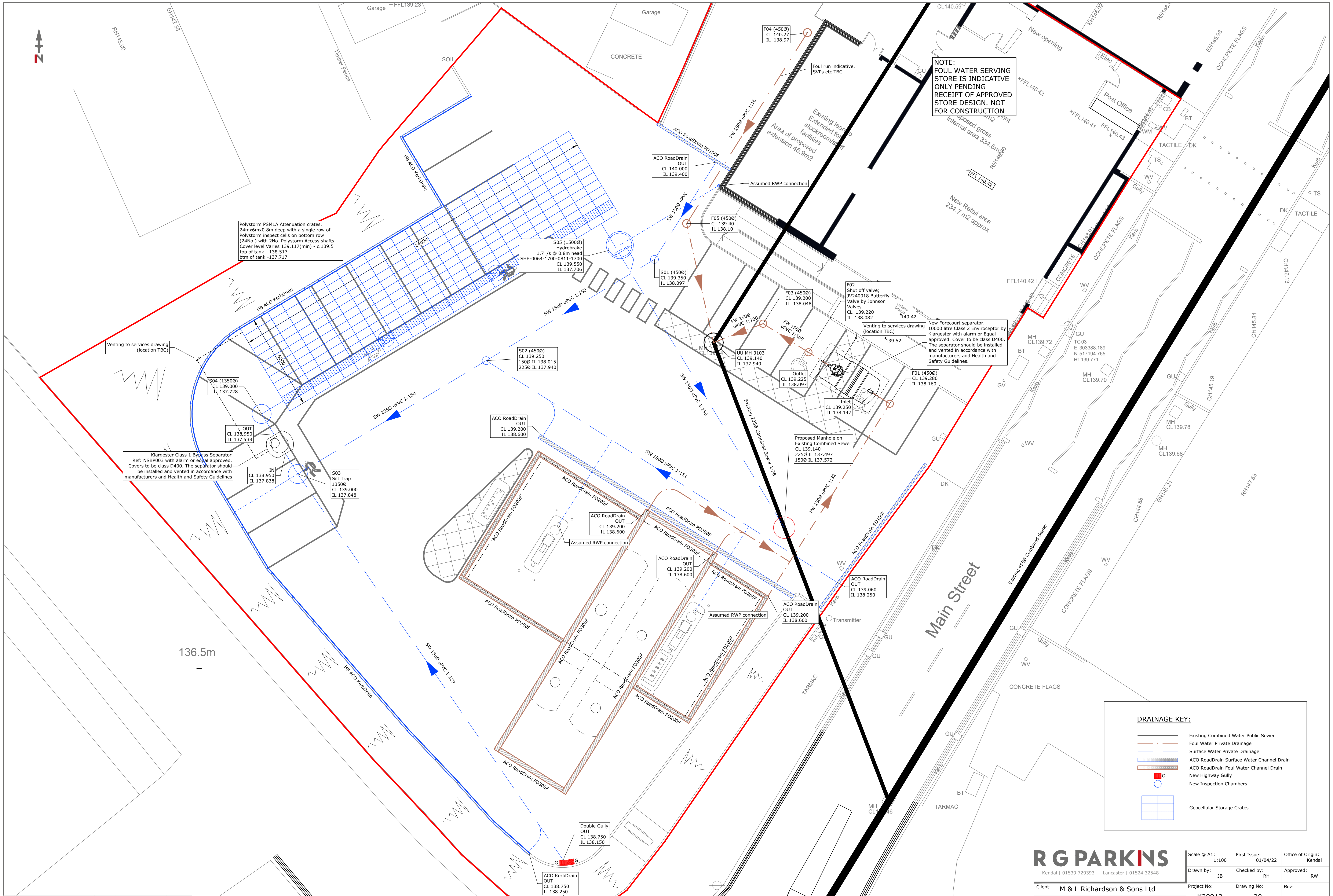
[14] Marshall & Bayliss, 1994. Flood Estimation for Small Catchments, Report No. 124 (IoH 124), Institute of Hydrology.

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

[16] 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

DRAINAGE LAYOUT



Rev	Description	Date	Revised by	Checked by	Approved

Issue Purpose: Planning

Do not scale from this drawing

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Kendal | 01539 729393 Lancaster | 01524 32548

Client: M & L Richardson & Sons Ltd	Scale @ A1: 1:100	First Issue: 01/04/22	Office of Origin: Kendal
Project: New Petrol Filling Station, Frizington	Drawn by: JB	Checked by: RH	Approved: RW
Drawing Title: Drainage Layout Plan	Project No: K38912	Drawing No: 20	Rev: -
	BIM No: -		

APPENDIX B

PRE-DEVELOPMENT RUNOFF CALCULATIONS

DRAINAGE CALCULATIONS

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:

Brownfield Site to Brownfield Site (Reduced Impermeable Area)

Results Summary

Rate of Run-Off (l/s)			
Event	**Greenfield**	**Brownfield**	**Post-Development**
Q1	1.4	20.5	15.8
QBAR	1.7	30.2	23.3
Q10	2.3	41.1	31.7
Q30	2.8	50.3	38.8
Q100	3.4	64.9	50.1
Q100 + 40% CC	4.8	90.9	70.1

SITE AREAS (LAND COVER AREAS)

Existing Impermeable & Permeable Land Cover

Total Site Area: **0.229** ha **2290** m²

Existing Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total Impermeable Area	2065	0.207	90%
Remaining Permeable Area	225	0.023	10%

Proposed Land Cover Areas

Land Cover	Area		Percentage of total site area
	m²	ha	
Total Proposed Roof Area	52	0.005	2%
Existing Roof Area	322	0.032	14%
Forecourt	202	0.020	9%
Forecourt Cover	145	0.015	6%
Total Hardstanding/Car Parking	1395	0.139	61%
Landscaped Areas	317	0.032	14%

Proposed Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Impermeable Area to SW network	1592	0.159	70%
Impermeable Area to FW network	202	0.020	9%
Impermeable Area to drain as existing	322	0.032	14%
Remaining Permeable Area	317	0.032	14%

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	=	**1349**	mm	From UKSuds website (point data)

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

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. This seems reasonable based on ground investigation.

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

$QBAR_{rural}$	=	0.520	m^3/s
	=	520.1	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	=	**1592**	m^2	Excluding significant open space which would remain disconnected from the positive drainage system during flood events.
	=	0.002	km^2	
	=	0.159	ha	
$QBAR_{rural\ site}$	=	0.00166	m^3/s	
	=	**1.66**	l/s	

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	1.44
2	0.93	1.54
5	1.19	1.97
10	1.38	2.29
25	1.64	2.72
30	1.7	2.82
50	1.85	3.06
100	2.08	3.45
200	2.32	3.84
500	2.73	4.52
1000	3.04	5.04

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

ESTIMATE OF BROWNFIELD RUNOFF

Total site impermeable area, A = **2065** m²

M5-60 rainfall depth **20** mm *[Flood Studies Report (NERC, 1975)]*

Ratio M5-60/M5-2Day, r **0.30** *[The Wallingford Proceedure - V4 Modified Rational Method, Fig A.2 (Hydraulics Research, 1983)]*

Storm Duration 15 mins — Anticipated critical duration for the site - usually 15 minutes

Duration factor, Z1 0.59 — *[The Wallingford Proceedure - V4 Modified Rational Method, Fig A.3b (Hydraulics Research, 1983)]*

M5-15 rainfall depth = 11.8 mm

Return period ratio, Z2

M1-15	0.61
M10-15	1.23
M30-15	1.50
M100-15	1.94

[The Wallingford Proceedure - V4 Modified Rational Method, Table A1 (Hydraulics Research, 1983)]

Rainfall

	Depth (mm)	Intensity, i (mm/hr)
M1-15	7.2	29
M10-15	14.5	58
M30-15	17.8	71
M100-15	22.9	92

Peak discharge, Qp = Cv Cr i A

Where:

Cv = Volumetric Runoff Coefficient
Cr = Routing Coefficient
i = Rainfall intensity (mm/hour)

Cv = 0.95
Cr = 1.3

Peak Runoff

	l/s
Q1	20.5
Q10	41.1
Q30	50.3
Q100	64.9

ESTIMATION OF QBAR (BROWNFIELD RUNOFF RATE)

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

Return Period	Ordinate
1	0.87
2	0.93
5	1.19
10	1.38
25	1.64
30	1.70
50	1.85
100	2.08
200	2.32
500	2.73
1000	3.04

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

Ordinate used	Qbar l/s
10 year	29.8
30 year	29.6
100 year	31.2

Proposed Brownfield Runoff, Qbar = 30.19 l/s

Using the average Qbar derived from three ordinates.

ESTIMATE OF BROWNFIELD RUNOFF

Total site impermeable area, A = **1592** m²

M5-60 rainfall depth **20** mm — *[Flood Studies Report (NERC, 1975)]*

Ratio M5-60/M5-2Day, r **0.30** — *[The Wallingford Proceedure - V4 Modified Rational Method, Fig A.2 (Hydraulics Research, 1983)]*

Storm Duration 15 mins — Anticipated critical duration for the site - usually 15 minutes

Duration factor, Z1 0.59 — *[The Wallingford Proceedure - V4 Modified Rational Method, Fig A.3b (Hydraulics Research, 1983)]*

M5-15 rainfall depth = 11.8 mm

Return period ratio, Z2

M1-15	0.61
M10-15	1.23
M30-15	1.50
M100-15	1.94

[The Wallingford Proceedure - V4 Modified Rational Method, Table A1 (Hydraulics Research, 1983)]

Rainfall

	Depth (mm)	Intensity, i (mm/hr)
M1-15	7.2	29
M10-15	14.5	58
M30-15	17.8	71
M100-15	22.9	92

Peak discharge, Qp = Cv Cr i A

Where:

Cv = Volumetric Runoff Coefficient
Cr = Routing Coefficient
i = Rainfall intensity (mm/hour)

Cv = 0.95
Cr = 1.3

Peak Runoff

	l/s
Q1	15.8
Q10	31.7
Q30	38.8
Q100	50.1

ESTIMATION OF QBAR (BROWNFIELD RUNOFF RATE)

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

Return Period	Ordinate
1	0.87
2	0.93
5	1.19
10	1.38
25	1.64
30	1.70
50	1.85
100	2.08
200	2.32
500	2.73
1000	3.04

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

Ordinate used	Qbar l/s
10 year	22.9
30 year	22.8
100 year	24.1

Proposed Brownfield Runoff, Qbar = **23.28** l/s

Using the average Qbar derived from three ordinates.

Summary of Results for 100 year Return Period (+40%)

Half Drain Time : 556 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	137.960	0.243	0.0	1.7	1.7	33.2	O K
30 min Summer	138.060	0.343	0.0	1.7	1.7	46.9	O K
60 min Summer	138.176	0.459	0.0	1.7	1.7	62.7	O K
120 min Summer	138.266	0.549	0.0	1.7	1.7	75.2	O K
180 min Summer	138.319	0.602	0.0	1.7	1.7	82.4	O K
240 min Summer	138.353	0.636	0.0	1.7	1.7	87.0	O K
360 min Summer	138.388	0.671	0.0	1.7	1.7	91.8	O K
480 min Summer	138.400	0.683	0.0	1.7	1.7	93.4	O K
600 min Summer	138.403	0.686	0.0	1.7	1.7	93.8	O K
720 min Summer	138.401	0.684	0.0	1.7	1.7	93.6	O K
960 min Summer	138.391	0.674	0.0	1.7	1.7	92.2	O K
1440 min Summer	138.360	0.643	0.0	1.7	1.7	88.0	O K
2160 min Summer	138.298	0.581	0.0	1.7	1.7	79.5	O K
2880 min Summer	138.234	0.517	0.0	1.7	1.7	70.7	O K
4320 min Summer	138.111	0.394	0.0	1.7	1.7	53.9	O K
5760 min Summer	138.026	0.309	0.0	1.7	1.7	42.3	O K
7200 min Summer	137.968	0.251	0.0	1.7	1.7	34.3	O K
8640 min Summer	137.927	0.210	0.0	1.7	1.7	28.7	O K
10080 min Summer	137.897	0.180	0.0	1.7	1.7	24.6	O K
15 min Winter	137.990	0.273	0.0	1.7	1.7	37.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	115.813	0.0	34.7	22
30 min Summer	82.547	0.0	49.5	37
60 min Summer	56.333	0.0	67.5	66
120 min Summer	35.155	0.0	84.3	124
180 min Summer	26.748	0.0	96.2	184
240 min Summer	22.029	0.0	105.7	242
360 min Summer	16.711	0.0	120.3	362
480 min Summer	13.693	0.0	131.4	458
600 min Summer	11.705	0.0	140.4	512
720 min Summer	10.282	0.0	148.0	576
960 min Summer	8.349	0.0	160.2	706
1440 min Summer	6.205	0.0	178.6	984
2160 min Summer	4.562	0.0	197.0	1404
2880 min Summer	3.674	0.0	211.5	1820
4320 min Summer	2.750	0.0	237.6	2552
5760 min Summer	2.269	0.0	261.3	3280
7200 min Summer	1.979	0.0	285.0	3968
8640 min Summer	1.785	0.0	308.5	4672
10080 min Summer	1.647	0.0	332.1	5352
15 min Winter	115.813	0.0	38.8	22

Summary of Results for 100 year Return Period (+40%)

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	138.103	0.386	0.0	1.7	1.7	52.8	O K
60 min Winter	138.236	0.519	0.0	1.7	1.7	70.9	O K
120 min Winter	138.339	0.622	0.0	1.7	1.7	85.1	O K
180 min Winter	138.401	0.684	0.0	1.7	1.7	93.6	O K
240 min Winter	138.443	0.726	0.0	1.7	1.7	99.3	O K
360 min Winter	138.489	0.772	0.0	1.7	1.7	105.6	O K
480 min Winter	138.508	0.791	0.0	1.7	1.7	108.2	O K
600 min Winter	138.512	0.795	0.0	1.7	1.7	108.8	O K
720 min Winter	138.507	0.790	0.0	1.7	1.7	108.1	O K
960 min Winter	138.494	0.777	0.0	1.7	1.7	106.3	O K
1440 min Winter	138.448	0.731	0.0	1.7	1.7	100.0	O K
2160 min Winter	138.353	0.636	0.0	1.7	1.7	87.0	O K
2880 min Winter	138.254	0.537	0.0	1.7	1.7	73.5	O K
4320 min Winter	138.056	0.339	0.0	1.7	1.7	46.4	O K
5760 min Winter	137.938	0.221	0.0	1.7	1.7	30.2	O K
7200 min Winter	137.870	0.153	0.0	1.6	1.6	20.9	O K
8640 min Winter	137.831	0.114	0.0	1.6	1.6	15.6	O K
10080 min Winter	137.808	0.091	0.0	1.5	1.5	12.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
30 min Winter	82.547	0.0	55.4	36
60 min Winter	56.333	0.0	75.6	66
120 min Winter	35.155	0.0	94.4	122
180 min Winter	26.748	0.0	107.8	180
240 min Winter	22.029	0.0	118.4	238
360 min Winter	16.711	0.0	134.7	352
480 min Winter	13.693	0.0	147.2	462
600 min Winter	11.705	0.0	157.3	566
720 min Winter	10.282	0.0	165.8	654
960 min Winter	8.349	0.0	179.5	746
1440 min Winter	6.205	0.0	200.1	1058
2160 min Winter	4.562	0.0	220.6	1516
2880 min Winter	3.674	0.0	236.9	1968
4320 min Winter	2.750	0.0	266.1	2684
5760 min Winter	2.269	0.0	292.6	3352
7200 min Winter	1.979	0.0	319.2	4032
8640 min Winter	1.785	0.0	345.5	4672
10080 min Winter	1.647	0.0	371.9	5344

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 303357 517197 NY 03357 17197
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 %	+40

Time Area Diagram

Total Area (ha) 0.160

Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)
0	4	0.080	4	8	0.080

Model Details

Storage is Online Cover Level (m) 139.117

Cellular Storage Structure

Invert Level (m) 137.717 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	144.0	0.0	0.801	0.0	0.0
0.800	144.0	0.0			

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0064-1700-0811-1700
Design Head (m)	0.811
Design Flow (l/s)	1.7
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	64
Invert Level (m)	137.706
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.811	1.7
Flush-Flo™	0.250	1.7
Kick-Flo®	0.515	1.4
Mean Flow over Head Range	-	1.5

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	1.5	1.200	2.0	3.000	3.1	7.000	4.6
0.200	1.7	1.400	2.2	3.500	3.3	7.500	4.7
0.300	1.7	1.600	2.3	4.000	3.5	8.000	4.9
0.400	1.6	1.800	2.4	4.500	3.7	8.500	5.0
0.500	1.4	2.000	2.6	5.000	3.9	9.000	5.2
0.600	1.5	2.200	2.7	5.500	4.1	9.500	5.3
0.800	1.7	2.400	2.8	6.000	4.3		
1.000	1.9	2.600	2.9	6.500	4.4		