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Flood Risk Assessment & Outline Drainage Strategy

Summergrove, Whitehaven

John Swift Homes

Ref: K39183.FRA/001

Version	Date	Prepared By	Checked By	Approved By
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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

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7. INTRODUCTION

7.1 BACKGROUND

This report has been prepared by R. G. Parkins & Partners Ltd (RGP) for John Swift Homes in support of their proposals for a new residential development adjacent to Summergrove, Whitehaven.

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

The following study assesses flood risk to the site and proposed development and demonstrates the proposed development will not adversely affect flood risk elsewhere.

7.2 PLANNING POLICY

The NPPF [1] and its Planning Practice Guidance [2] states "a site-specific flood risk assessment should be provided for all development in Flood Zones 2 and 3. In Flood Zone 1, an assessment should accompany all proposals involving: sites of 1 hectare or more; land which has been identified by the Environment Agency as having critical drainage problems; land identified in a strategic flood risk assessment as being at increased flood risk in the future; or land that may be subject to other sources of flooding, where its development would introduce a more vulnerable use."

7.3 THE DEVELOPMENT IN THE CONTEXT OF PLANNING POLICY

Owing to the size of the development it is classed as major development (over 1 ha) in accordance with The Town and Country Planning Order 2015 [3].

The area covered by the assessment is 8.34 ha (hectares) and by reference to the Environment Agency Flood Map, the site lies in Flood Zone 1.

Table 2 of the NPPF's Planning Practice Guidance [2] classifies each development into a vulnerability class, depending on the type of development, as outlined in Table 7.1. The site is to be developed for a housing development; and is classified as 'More vulnerable'. 'More Vulnerable' development classes are deemed acceptable in terms of flood risk within Flood Zones 1, 2 and 3a but are not generally considered acceptable within Flood Zone 3b.

Due to the site being located in Flood Zone 1 sequential and exception tests are not required.

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Table 7.1 Vulnerability Classification

Vulnerability Classification	Development
Essential Infrastructure	Essential transport infrastructure (including mass evacuation routes) which has to cross the area at risk. Essential utility infrastructure, which has to be located in a flood risk area for operational reasons, including electricity generating power stations and grid and primary substations; and water treatment works that need to remain operational in times of flood. Wind turbines.
Highly Vulnerable	Police and ambulance stations; fire stations and command centres; telecommunications installations required to be operation during flooding. Emergency dispersal points. Basement dwellings. Caravans, mobile homes, and park homes intended for permanent residential use. Installations requiring hazardous substances consent.
More Vulnerable	Hospitals. Residential institutions such as residential care homes, children's homes, prisons and hostels. Buildings used for dwelling houses, student halls of residence, drinking establishments, nightclubs, and hotels. Non-residential uses for health services, nurseries, and education establishments. Landfill and sites used for waste management facilities for hazardous waste. Sites used for holiday or short let caravans and camping, subject to a specific warning and evacuation plan
Less Vulnerable	Police, ambulance, and fire stations which are NOT required to be operational during flooding. Buildings used for shops; financial, professional, and other services; restaurants, cafes and hot food takeaways; offices; general industry, storage and distributions; non-residential institutions not included in the 'more vulnerable' class; and assemble and leisure. Land and buildings used for agriculture and forestry. Waste treatment (except landfill & hazardous waste facilities). Minerals working & processing (except for sand & gravel working). Water treatment works which do not need to remain operational during times of flood. Sewage treatment works, if adequate measures to control pollution and manage sewage during flooding events are in place.
Water-Compatible Development	Flood control infrastructure. Water transmission infrastructure & pumping stations. Sewage transmission infrastructure & pumping stations. Sand & gravel working. Docks, marinas, and wharves. Navigation facilities. Ministry of Defence installations. Ship building, repairing & dismantling, dockside fish processing & refrigeration & compatible activities requiring a waterside location. Water based recreation (excluding sleeping accommodation). Lifeguard and coastguard stations. Amenity open space, nature conservation & biodiversity, outdoor sports and recreation and essential facilities such as changing rooms. Essential ancillary sleeping or residential accommodation for staff required by uses in this category subject to a specific warning & evacuation plan.

8. SITE CHARACTERISATION

8.1 SITE LOCATION

The land is situated in the Galemire area approx. 1 km southeast of Whitehaven, at National Grid Co-ordinates 300008E 515562N. The location is shown in Figure 8.1.

The land is currently accessible from numerous places with gated access from Summergrove Park in the North, via Dalzell Street in the East and an access track in the most westerly corner

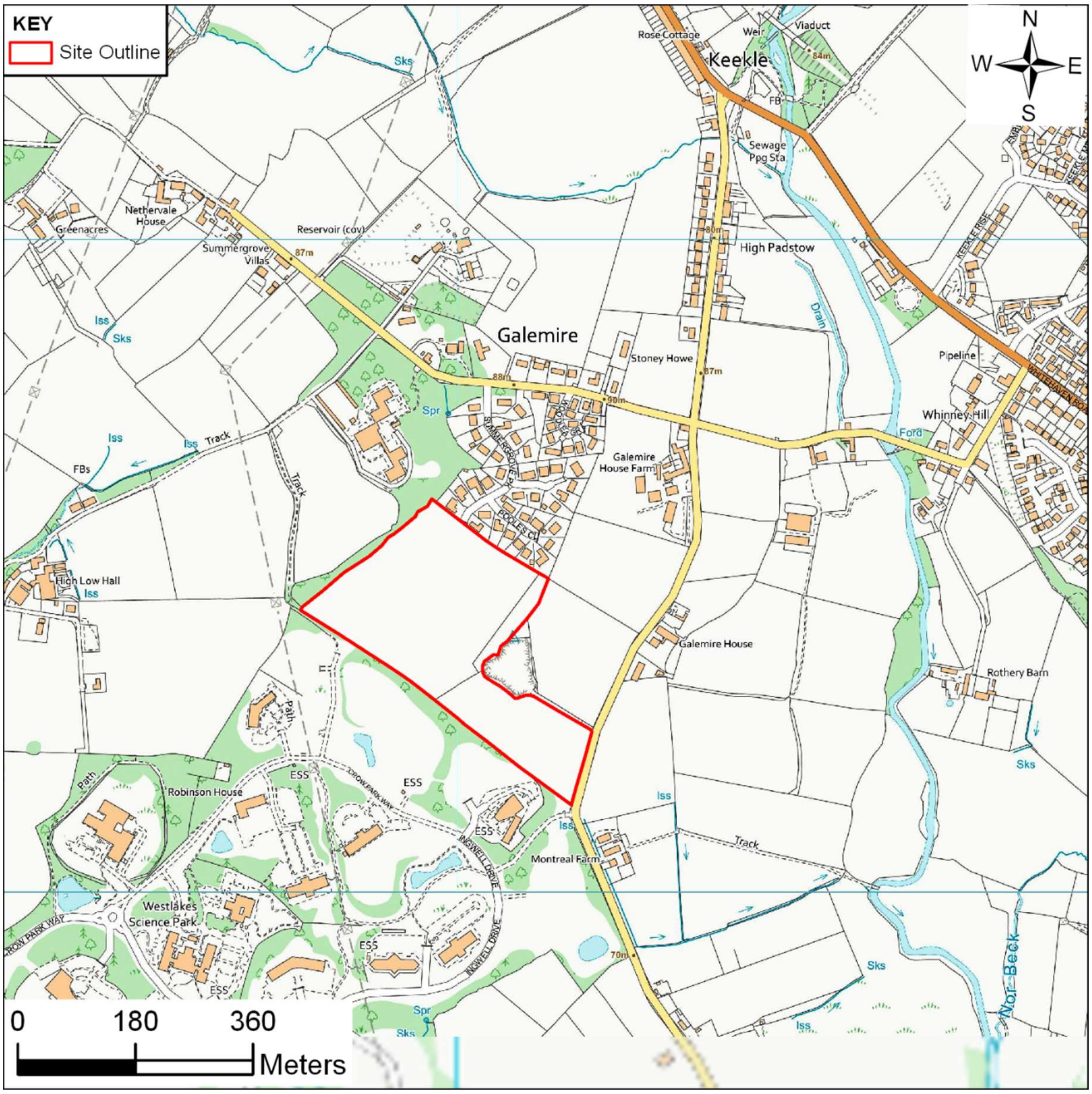

Figure 8.1 Site Location

8.2 SITE DESCRIPTION

The land covered by the assessment covers approximately 8.34 ha. And is currently used as agricultural pasture and can be classed as Greenfield. The land comprises of two approximately rectangular fields which border the now infilled former Galemire Quarry inert landfill site.

The land is bound by Summergrove Park residential estate to the north, agricultural land to the northeast and west, Westlakes Science Park to the south with Dalzell Street forming the smaller fields eastern boundary.

Topographically levels vary significantly across the land, with the survey indicating the site is effectively crested along the walled hedgerow of the two adjoining fields with levels falling both easterly and westerly towards the respective boundaries. A topographic Survey is included in Appendix A for reference.

In the larger more westerly field, the levels typically slope east to west from around 97.50mAOD at the north-eastern field boundary to approximately 86.50mAOD in the west. Along the north eastern boundary of this field there is a strip of land which appears to form a localised mound / bund type feature along the perimeter up to 2.5m high, this feature is thought to be associated with the adjacent infilling of the former Galemire inert landfill quarry site.

In the smaller of the two fields the Levels fall west to east from the walled hedge (crest) from a high of around 95.50mAOD towards the road to a low of around 85.50mAOD.

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 the nearest Groundwater Source Protection Zone is a Zone 3 which is situated approximately 2.5 km south of the site.

The development site overlies a principal aquifer with areas of 'Medium-High' groundwater vulnerability in the eastern areas of the site and 'Medium-Low' vulnerability in the western portion of the site.

Table 8.1 Site Geological Summary

Geological Unit	Classification	Description	Aquifer Classification
Soil	Soilscape 17	Slowly permeable seasonally wet acid loamy and clayey soils	N/A
Drift	None recorded	N/A	N/A
Solid	Bees Sandstone Formation	Sandstone	Principal

8.4 HYDROLOGY

Within the site there appear to be a series of historic land drainage features serving the fields that appear to discharge to informal surface water drainage features located just outside the extents of both the eastern and western boundaries. The localised sloping topography would suggest that these ditches act as the natural drainage route for the existing site.

To the west of the site there is an existing land drainage inspection chamber which outfalls to a drainage ditch that runs parallel to the access track in this location heading in a northerly direction before feeding into the unnamed Ordinary Watercourse marked on the ordnance survey maps which issues on the northern side of the access track to High Low Hall.

On the eastern side there are drainage ditches present on both sides of Dalzell Street which then follow a pathway into a series of drainage channels located in fields to the east of the nearby Montreal Farm before heading in an easterly direction where they to eventually feed into the River Keekle.

The River Keekle is the nearest designated ‘main river’ located approximately 475m east of the development site boundary on the remote side of (Dalzell Street) road.

8.5 EXISTING SEWERS

Existing private land drainage pipework features are evident within the site itself.

Reference to United Utilities Sewer Records indicate the presence of both foul and surface water public sewers located to the north of the development site in the Summergrove Park residential estate site roads.

8.6 GROUND INVESTIGATION

Although no ground investigation has been undertaken as part of this application. The site has been subject to recent potential development enquiries and a phase 2 ground investigation report conducted for a previous development proposal in 2018/2019 has been made available to RGP for information.

With respect to drainage, this ground investigation report included in-situ ground permeability testing in a number of trial pits as part of the intrusive investigations and returned infiltration rate results of between 1.6×10^{-05} m/s and 7.1×10^{-06} m/s.

Although not considered to be completely impermeable the slow rate of infiltration encountered in the site ground conditions is not considered suitable to facilitate a full infiltration drainage solution.

Minor seepage was also encountered in a number of exploratory holes at depths of between 1.20m and 2.10m bgl and groundwater monitoring was also undertaken in a series of boreholes with standing groundwater recorded at depths of between 0.89m and 2.20m bgl. Although not anticipated to be significant, groundwater control measures are recommended to mitigate potential ingress during construction excavations.

For further information refer to the Geo Environmental Engineering Ground investigation report:

- GEO2018-3441– Phase 2 - Ground investigation Report (v2 Issued 2019)

The report is available to view online at Copeland Borough Councils planning website [8] (Planning Reference No. 4-19-2126-0F1).

Based on the above findings the variable levels of permeability across the site combined with fluctuating seasonal groundwater levels could potentially impact the efficiency of any soakaway structures and therefore an alternative surface water disposal solution is recommended for this development. Partial infiltration may be suitable at shallow depth in certain situations.

9. ASSESSMENT OF FLOOD RISK

9.1 BACKGROUND

The following risk assessment has been carried out in accordance with the National Planning Policy Framework [1] and its Planning Practice Guidance [2] on Flood Risk. The broad aim of the guidance is to reduce the number of people and properties within the natural and built environment at risk of flooding. To achieve this aim, planning authorities are required to ensure that flood risk is properly assessed during the initial planning stages.

Responsibility for this assessment lies with the developers and they must demonstrate:

- Whether the proposed development is likely to be affected by flooding.
- Whether the proposed development will increase flood risk in other parts of the hydrological catchment.
- That the measures proposed to deal with any flood risk are sustainable.

The developer must prove to the Local Planning Authority and the Environment Agency that the existing flood risk or the flood risk associated with the proposed development can be satisfactorily managed.

9.2 FLOOD RISK TERMINOLOGY

Flood risk considers both the probability and consequence of flooding.

Flood events are often described in terms of their probability of recurrence or probability of occurring in any one year. The threshold between a medium flood and a large flood is often regarded as the 1 in 100-year event. This is an event which statistical analysis suggests will occur on average once every hundred years. However, this does not mean that such an event will not occur more than once every hundred years. Table 9.1 shows the event return periods expressed in years and annual exceedance probabilities as a fraction and a percentage. For example, a 1 in 100-year event has a 1% probability of occurring in any one year, i.e. a 1 in 100 probability. A 1000-year event has a 0.1% probability of occurring in any one year, i.e. a 1 in 1000 probability.

Table 9.1 Flood Return Periods & Exceedance Probabilities

Return Period (years)	Annual Exceedance Probability (AEP)	
	Fraction	Percentage
2	0.5	50%
10	0.1	10%
25	0.04	4%
50	0.02	2%
100	0.01	1%
200	0.005	0.5%
500	0.002	0.2%
1000	0.001	0.1%

9.3 DATA COLLECTION

The following information was referred to for the Flood Risk Assessment:

- Environment Agency Flood Map for Planning covering the site and adjacent area.
- Environment Agency Surface Water Flood Risk Map
- Environment Agency Reservoir Flood Risk Map
- Environment Agency Historic Flood Map
- United Utilities sewer records
- British Geological Survey Groundwater Flooding Susceptibility Map
- Copeland Borough Council Strategic Flood Risk Assessment
- Development layout plan
- Topographic survey

9.4 STRATEGIC FLOOD RISK ASSESSMENT

Copeland Borough Council commissioned JBA Consulting to produce a Level 1 Strategic Flood Risk Assessment (SFRA) Report [7] finalised in 2021 which refers to the Environment Agency Flood Maps to determine flood risk.

It states there are several historic flooding incidents in Whitehaven, but these are generally attributed to tidal flooding due to the proximity of the town centre to the coastline. Some properties are at risk from the main watercourse, Pow Beck which bisects the town and during extreme events, flooding can be exacerbated in certain areas by insufficient sewer capacities. This site however is located away from the historically affected areas and is not shown to be at risk of flooding.

9.5 ENVIRONMENT AGENCY FLOOD MAP FOR PLANNING

Figure 9.1 is an extract from the EA's Flood Map for Planning [6]. This has been reviewed to assess the level of flood risk to the area. The flood map shows areas that may be at risk of fluvial flooding in a 1% (1 in 100 year, dark blue) or 0.1% (1 in 1000 year, light blue) Annual Exceedance Probability (AEP) event. Alternatively, if the flood risk is tidal the flood map will show areas predicted to be at risk of flooding from the sea in a 0.5% AEP event (1 in 200 year, dark blue) or a 0.1% AEP event (1 in 1000 year, light blue).

The Flood Map shows the current best information on the extent of the extreme flooding from rivers or the sea that would occur without the presence of flood defences. The potential impact of climate change is not considered by the mapping.

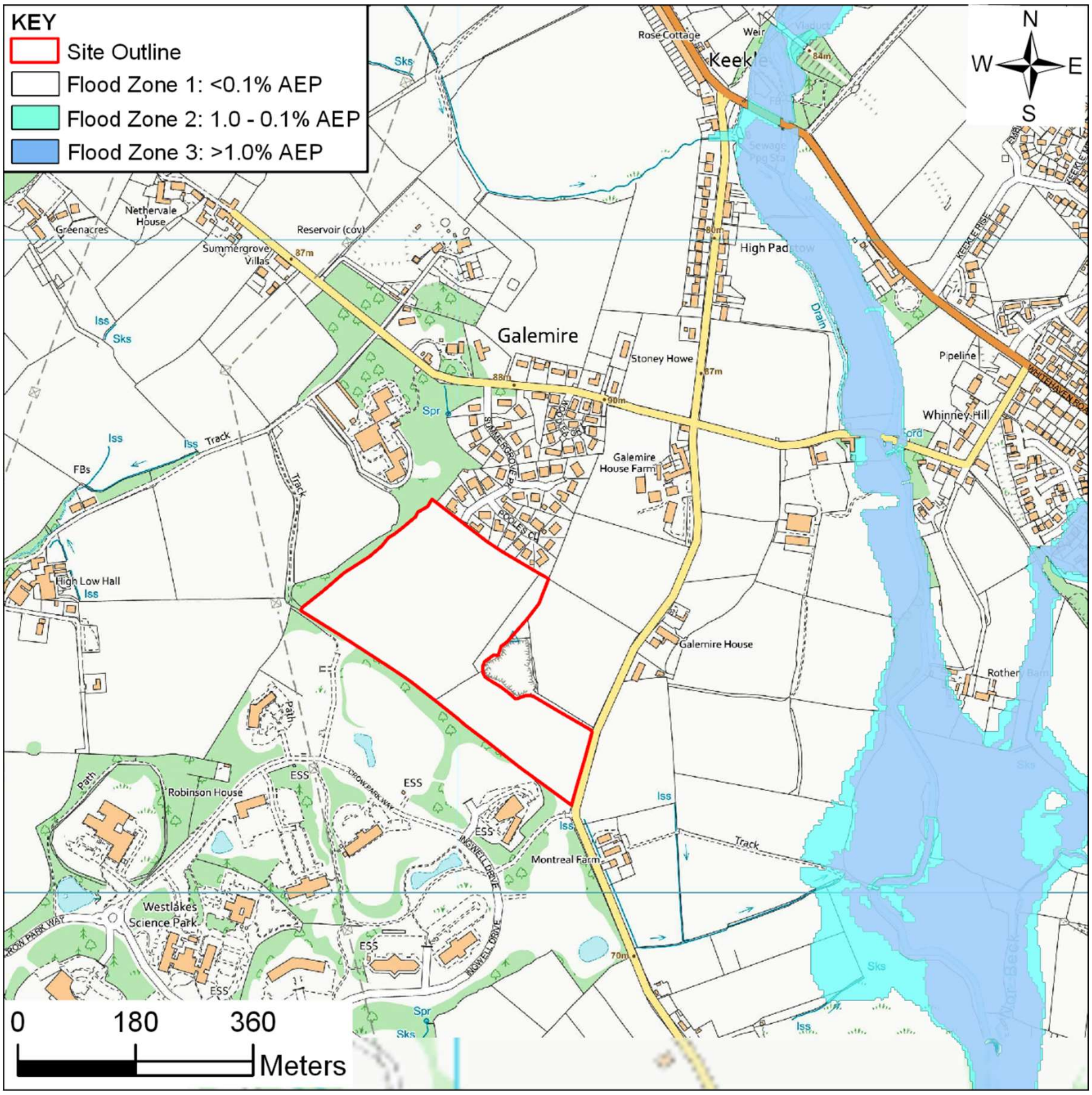

Figure 9.1 Environment Agency Flood Map for Planning

Reference to Figure 9.1 indicates the site lies within Flood Zone 1 “Low Probability”, land assessed as having a less than 0.1% annual probability of flooding (i.e. rivers, lake or sea) in any year by reference to the NPPF and is therefore not considered to be at risk of fluvial flooding.

9.6 SURFACE WATER FLOOD RISK

Surface water flooding is that which results from extreme rainfall rather than overflowing rivers. This type of flooding typically occurs when extreme rainfall causes water to run down slopes and collect in depressions in the landscape or where runoff is focussed into an area where drainage is insufficient. It can also cause erosion resulting in the partial or complete blockage of drains or culverts.

Figure 9.2 shows an extract from the EA Surface Water Flood Risk Map [6]. This has four risk classifications from very low probability (<0.1% AEP) to high probability (>3.3% AEP).

The EA surface water flood map indicates that the entire site area proposed for development is at 'Very Low' risk of surface water flooding with the risk of flooding being less than 0.1% AEP (1 in 1000 year).

Immediately outside the site boundary there is an area of 'low to medium Probability' flooding shown in the area of the former quarry. This appears to be attributed to a localised depression in this area. Levels fall in the opposite direction to the development site which would not be affected.

Surface water run-off from the site is currently directed towards the drainage ditches on each respective side of the field boundaries. Any development resulting in an increase in impermeable areas could cause additional run-off if not properly managed.

It is therefore proposed to incorporate sufficient SuDS measures and attenuation storage to mitigate this as part of the overall Drainage Strategy. This is discussed in further detail in Section 10.0.

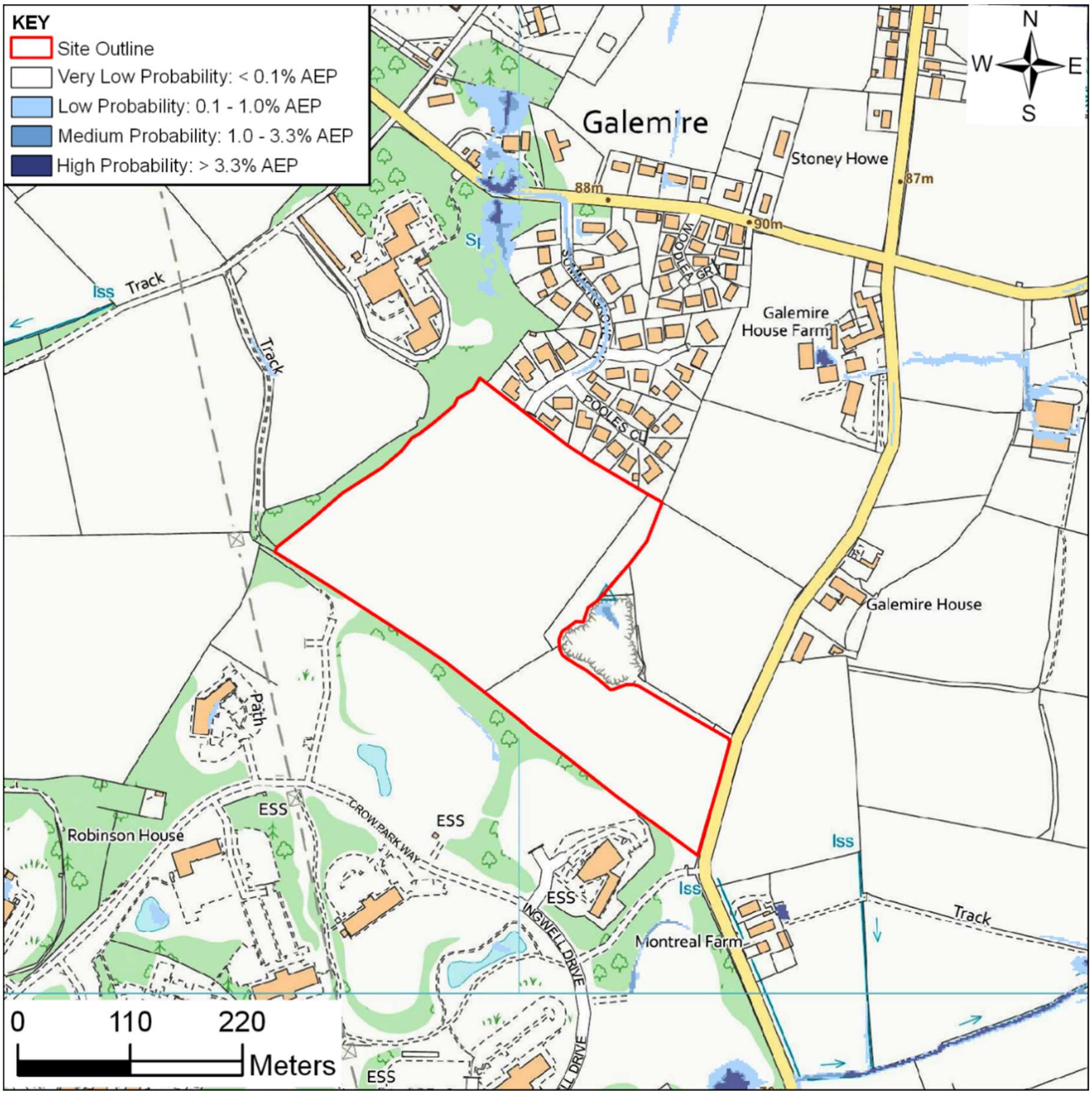

Figure 9.2 Environment Agency Surface Water Flood Map

9.7 GROUNDWATER FLOOD RISK

Groundwater flooding occurs when water levels in the ground rise above the ground surface. It is most likely to occur in low lying areas underlain by permeable drift and rocks.

Some minor groundwater ingress was encountered in some of the trial pit and boreholes conducted as part of the ground investigations (Section 8.6) but was not considered to be significant.

It is likely that groundwater levels will fluctuate throughout the year but given the sloping topography and elevation this site is unlikely to be significantly affected by groundwater flooding.

9.8 FLOODING FROM RESERVOIRS, CANALS OR OTHER ARTIFICIAL SOURCES

The likelihood of reservoir flooding is considered to be much lower than other forms of flooding. Current reservoir regulation, which has been further enhanced by the Flood and Water Management Act, aims to make sure that all reservoirs are properly maintained and monitored to detect and repair any problem.

The Ordnance Survey map indicates that there are no reservoirs, canals or artificial structures in the close proximity of the proposed development site.

9.9 FLOODING FROM SEWERS

United Utilities (UU) do not provide information on flood risk from their assets and there have been no reports within the SFRA in this locality.

The existing foul and surface water sewers located in Summergrove Park Road are at elevations typically higher than the majority of the development site with levels falling away and any surcharging of the sewer would result in flood flows following the gradient of the existing highway downhill and away from the site. It is therefore concluded that the site is not at risk of flooding from these sources.

10. SURFACE WATER DRAINAGE STRATEGY

10.1 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[9]
- Code of Practice for Surface Water Management, BS8582:2013, November 2013[10]
- Rainfall Runoff Management for Developments, Defra/EA, SC030219, October 2013[11]
- Designing for Exceedance in Urban Drainage – Good Practice, CIRIA Report C635, 2006[12]
- Flood Estimation Handbook (FEH)[13]
- Flood Studies Report (FSR), Volume 1, Hydrological Studies, 1993[14]
- Flood Studies Supplementary Report No 14 (FSSR14), Review of Regional Growth Curves, 1983[15]
- Flood Estimation for Small Catchments, Marshall & Bayliss, Institute of Hydrology, Report No. 124 (IoH 124), 1994[16]
- Non-Statutory technical Standards for Sustainable Drainage Systems, Defra, March 2015[17]
- Water UK, Design and Construction Guidance for Foul & Surface Water Sewers March 2020[18]
- Defra, Recommendations to Update Non-Statutory Technical Standards for Sustainable Drainage Systems (SuDS), February 2021[19]
- HR Wallingford: Tools for the design and evaluation of Sustainable Drainage Systems (SuDS) [20]

The following assessment and drainage strategy are based on the indicative landscape layout plan provided for the initial phase of development. As the development site plan is progressed the level of impermeable areas will become more evident allowing the drainage strategy to be revisited and assessed more accurately in line with the finalised development proposals.

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10.2 SITE AREAS

Based on the sloping topography of the existing Greenfield site it can be concluded that the entirety of the site area drains downslope towards the respective boundaries from the natural crest point along the existing adjoining central field boundary and to the respective existing drainage ditches in these localities. As such the pre-development Greenfield runoff rates will be calculated based on the total site area.

Table 10.1 therefore gives the total area of land cover proposed to be positively drained.

Table 10.1 Area of Positively Drained Land Cover

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Total Positively Drained Area	83,400	8.34	100%

The above figures can then be subdivided into drainage areas that are reflective of the existing crested site topography, whereby existing overland flows would follow the contours with different areas of the site draining to the Western and Eastern boundaries respectively. These Areas have been calculated and split into Areas A and B respectively as shown in Table 10.2.

Table 10.2 Area of Positively Drained Land Cover

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Area A	60,880	6.088	73%
Area B	22,520	2.252	27%

10.3 PRE-DEVELOPMENT GREENFIELD RUNOFF ASSESSMENT

As the total site covers an area of less than 200 ha, (8.34 ha) the Greenfield calculations have been undertaken in accordance with methodology described in IoH 124 [15]. 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.

With reference to Table 10.2 the pre-development greenfield runoff assessment has been calculated based on the total site area of 83,400 m^2.

However, as the site will potentially be a phased development the equivalent runoff rates based on the initial phase of development have also been explored for the Phase 1 site area totalling 35,500 m^2 shown in Table 10.3. A legal plan indicating the phasing of development is included in Appendix A for reference.

Table 10.3 Area of Positively Drained Land Cover - Phase 1

Phase 1 Development Land Cover	Area	
	m²	Ha
Area A	28,000	2.80
Area B	5,500	0.55

Full details of the calculations and the methodology for deriving the Greenfield Runoff Rate are in included in Appendix B. A summary of the results is included in Table 10.4.

Table 10.4 Pre-Development Peak Runoff Rates

Greenfield Rate of Runoff (l/s)		
Event	Phase 1 Development Area	Full Developed Site Area
Q1	25.9	64.5
QBAR	29.8	74.2
Q10	41.1	102.4
Q30	50.7	126.1
Q100	62.0	154.3
Q100 + 40% CC	86.8	216.0

The above figures can then be subdivided into drainage areas reflective of Areas A and B respectively and the discharge rates factored in line with the respective contributing site areas relating to the phase of development to mimic existing site conditions as shown in Table 10.5.

Table 10.5 Pre-Development Peak Runoff Rates

Greenfield Rate of Runoff (l/s)				
Event	Phase 1 Development Area		Full Developed Site Area	
	Area A	Area B	Area A	Area B
Q1	4.3	21.7	47.1	17.4
QBAR	4.9	24.9	54.1	20.0
Q10	6.8	34.4	74.7	27.6
Q30	8.3	42.3	92.0	34.0
Q100	10.2	51.8	112.6	41.7
Q100 + 40% CC	14.2	72.5	157.7	58.3

Without attenuation, the proposed development would significantly increase the rate of Runoff from the developed areas of the site.

To mitigate against the potential increase in runoff, it is proposed to contain and attenuate runoff within the development site before being released at a controlled rate to the existing drainage ditches to match the pre-development Greenfield QBAR rate for the various stages of development.

A summary of the relative catchments and discharge rates is outlined on Drawing No. K39183-100 included in Appendix A.

10.4 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:

10.4.1 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 10.6 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 10.6 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%

10.4.2 URBAN CREEP

BS 8582:2013 [10] 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 (excluding roads) is typically applied to the contributing area.

The inclusion of 10% is highly conservative due to the anticipated provision of adequate parking on the site and considering the limited extension potential of the dwellings due to site topography.

In this situation as the number of properties proposed for the development are currently unknown a conservative assumed impermeable percentage of the site area has been used in calculations to account for the anticipated density of housing at this site and allows for urban creep.

Once the development layout has been finalised the impermeable areas associated with the number of dwellings can be recalculated and urban creep can be applied.

10.4.3 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.

10.4.4 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 typically 0.75 for summer and 0.84 for winter for drainage design.

In line with the latest design guidance a Cv Value of 1.0 has been used for all impermeable areas and a factored contribution (0.3) of the sites pervious areas has been included for conservative design.

In February 2021 DEFRA published a paper listing recommendations to update the non-statutory technical standards for SuDS [19] and in cases such as this where it is proposed that 100% of the development site including green/garden areas will be positively drained, it is recommended that it would be more appropriate to assume 100% runoff from impermeable areas and 30% runoff from pervious surfaces.

The advocated approach cannot be applied using current industry software which is based on the modified rational method and uses a global Cv value. Contributing area from garden and landscaped areas of the site can instead be factored by a value of 0.3, therefore, for conservative design a Cv value of 1.0 has been used and an additional contribution for the pervious areas of the catchment has been included in the model, factored by a value of 0.3. This is the same methodology adopted by HR Wallingford for storage assessment as detailed on their website [20].

It was also observed in comments made as part of the previous planning application[8] by CCC in their capacity as the Lead Local Flood Authority that a Cv value of 1.0 was requested for any drainage modelling calculations for this particular site.

Therefore, for this design a more conservative Cv value of 1.0 has been used for all impermeable areas, as described in Section 10.6.

10.4.5 RAINFALL MODEL

The calculations use the REFH2 unit hydrograph methodology in line with best practice as outlined in the SuDS Manual [8]. 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.

10.5 SURFACE WATER DISPOSAL

Surface water disposal has been considered in line with the hierarchy outlined in the SuDS Manual [9]. The approach considers infiltration drainage in preference to disposal to watercourse, in preference to discharge to sewer.

Cumbria County Council as Lead Local Flood Authority prefer design in accordance with the Cumbria Design Guide which identifies the following hierarchy of techniques to be used:

- **Prevention**: Prevention of runoff by good site design and the reduction of impermeable areas.
- **Source Control**: Dealing with water where and when it falls
- **Site Control**: Management of water in the local area
- **Regional Control**: Management of runoff from sites

10.5.1 INFILTRATION

In-situ permeability testing was undertaken as part of the ground investigation for the previous planning application at this site (See Section 8.6) and although not classed as entirely impermeable the slow infiltration rates encountered concluded that disposal of surface water via. soakaways would not form an effective drainage solution for this site.

On this basis it is therefore considered that disposal of surface water using a full infiltration-based SuDS is not viable for this proposed development and an attenuation-based strategy should be progressed.

10.5.2 POSITIVE DRAINAGE – WATERCOURSE

It is therefore proposed that the development site will require a positive drainage solution. Runoff will be stored and attenuated to match the pre-development Greenfield QBAR rates, with controlled discharge directed from suitably sized attenuation structures for the defined site catchment areas to the respective existing drainage ditches located in close proximity to the site boundaries at the western and eastern extremities of the site.

10.5.3 CONSIDERATION OF SUDS COMPONENTS

A full range of SuDS components and techniques have been considered for the development of the site and their applicability to the site is discussed below.

- **Green roofs –** Discounted due to cost and limits of water volume retention.

- **Soakaways –** insufficient infiltration rates to support effective drainage via. soakaways rule out infiltration as a viable full site drainage strategy.

- **Water butts –** These are suitable for the site, but their effectiveness would depend on them being empty prior to a period of significant rainfall. This could occur during the summer when occupiers are likely to use the water but unlikely during the autumn and winter Irrelevant for drainage design due to their inability to provide reliable stormwater storage.

- **Permeable paving –** Unsuitable road and driveway gradients and inconsistent soil permeability across the site for permeable block paving solutions to work efficiently.

- **Swales –** Would require large areas within the site ideally adjacent to road areas, could potentially be accommodated within the site layout depending upon the available space and scale of development proposed.

- **Filter drains, Infiltration trenches, and basins** – Variable levels of soil permeability across the site rule out a full infiltration as a viable drainage strategy. Inclusion of perforated land drainage pipes is considered suitable as a means to drain gardens and landscaped areas.

- **Detention basins** – Detention basins are the preferred option for this site as there is sufficient space available to accommodate the large areas required for this type of solution. As well as providing attenuation and treatment for site surface water run-off the basins would provide attractive permanent features within the context of an urban development. The required size of the proposed storage structures has been calculated and is detailed in Section 10.7.

- **Ponds / wetland –** As explained above.

- **Rain gardens –** discounted due to high capital and maintenance costs. Maintenance cannot practically be enforced.

- **Geocellular attenuation tank systems –** could be used in conjunction with the detention basins to balance the overall size of any proposed above ground storage structures and could be explored as an alternative / additional SuDS component if required at detailed design stage. These tanks would be wrapped and sealed with an impermeable geomembrane to provide a water-tight structure with flows controlled via an HydroBrake or Orifice flow control device.

- **Oversized Pipes –** could be used in conjunction with or as an alternative to geocellular attenuation tanks to provide additional online storage capabilities.

10.6 SURFACE WATER DRAINAGE DESIGN

The proposed surface water network serving the entire developable area of the site has been modelled using Micro Drainage Source Control (results are included in Appendix B).

The site layout has been divided into the 2 No. separate sub-catchments as detailed in Section 10.3 with Area A covering the western catchment and Area B covering the eastern catchment. Within each area the assumed extent of permeability associated with roads, driveways and all other impermeable areas have been conservatively calculated to equate to approximately 50% of the overall site area.
The drainage design has been sized to store a future 1% AEP event of critical duration without any flooding. Future climate change (40%) and urban creep (10% to housing roof areas only) is accounted for within the calculations.

The new recommended Cv value / contributing areas approach therefore results in the modified contributing catchment areas detailed in Table 10.7 being utilised to model the surface water attenuation requirements for this site.

Based on the landscape plan provided we have calculated the impermeable area for phase 1 to be approximately 50% of the site area. This percentage was then used to form the assumed overall impermeable area for future development of the remainder of the site.

Table 10.7 Modified Contributing Catchment Areas

Site Area	Impermeable Area (50%) m^2	Factored Pervious Area Contribution (30%) m^2	Total Contributing Area Modelled	
			m^2	Ha
Area A	30,440	9,132	39,572	0.396
Area B	11,260	3,378	14,638	0.146

It is proposed that all impermeable site areas i.e. roof, driveway and road areas will drain via. gravity through a network of pipes and chambers into shared private detention basins located in the natural respective low points of the site. The new detention basins will be formed as permanent features in areas designated as open space and will be designed to incorporate shallow, grassed slopes to provide important amenity and biodiversity benefits to the development.

Roof water, driveway and road runoff will connect directly into the surface water pipe network upstream of the detention basins, with inspection and manhole chambers utilised to route the new pipework to suit the proposed development layout and allow for future inspection and maintenance. Proposed ground levels will need to fall consistently depending on which area they are located, in order to enable gravity connections to the drainage system.

In addition, all gardens and landscaped areas will benefit from infiltration trenches and perforated pipes to allow these areas to drain effectively towards the nearest attenuation systems. This will mitigate the risk of uncontrolled overland flows from garden areas.

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Silt traps will be located upstream of each detention basins and any other proposed attenuation systems, which will provide surface water treatment and access for maintenance. Silt traps isolate silt and other particles by encouraging settlement into sumps, preventing ingress into SuDS components.

Hydrobrake flow control chambers will restrict discharge from each shared detention basin system. With the cumulative discharge from the initial phase of development culminating in a total discharge rate of 29.8 l/s with the overall future development of the site equalling discharge at 74.2 l/s in line with the calculated pre-development greenfield runoff QBAR rates.

10.7 VOLUMETRIC STORAGE

A storage assessment has been undertaken for the Q100+40% CC storm event and the detention basins have been designed with sufficient capacity to contain flows without causing flooding, the results are provided in Table 10.8.

Despite the potential for phased development, it is anticipated that the basins will be constructed to cater for the full scale of the future development to minimise disruption and therefore the volumetric requirements of the basins (A & B) were sized based on the total overall contributing areas and respective discharge rates for each area, using a standard depth of 1.5m with 1:3 gradient side slopes.

To allow for future changes to the development it is proposed that the runoff from Phase 1 would be stored in the respective (oversized) basins with discharge from the site controlled to match the equivalent QBAR discharge rates for the current level of development for that Phase.

Therefore, it is proposed that for the first phase of development Hydrobrakes would be installed controlling the discharge to 24.9 l/s for Area A and 4.9 l/s for Area B respectively.

Upon commencement of any additional phases of the development these hydrobrakes would be replaced in line with the increased QBAR discharge rate for the remainder of the development.

Table 10.8 Attenuation Storage Volumes

Site Area	Development Phase	Impermeable Catchment Area (m^2)	Volume to TWL (m^3)	Development Controlled Discharge Rate QBAR (l/s)	
				Phase 1	Full Site
Area A	Phase 1	18,200	1,344.0	24.9	
	Full Site	39,572	2856.1		54.2
Area B	Phase 1	3,575	265.6	4.9	
	Full Site	14,638	1028.2		20.0
		Total cumulative discharge		29.8	74.2

Note: TWL - Top Water Level for the Q100 + 40%CC event

For indicative information refer to the Outline Drainage Layout Plan included in Appendix A.

10.8 OTHER BENEFITS OF DEVELOPMENT

The development site in its current agricultural form as sparse grazing pasture on sloping land, underlain by relatively impermeable soil, provides little in the way of natural flood defence or attenuation to overland flows and stormwater runoff. The land in its current form also lacks any meaningful biodiversity or amenity value and provides limited benefits to the surrounding community.

It is envisaged that the proposed development site will tie into the existing topography via the careful design of engineered slopes and retaining walls. Slopes, gardens and open space areas will be carefully landscaped using a variety of plants, shrubs and trees with clean imported granular topsoil, providing a net gain in biodiversity and enhanced storage/protection against overland flows. Any retaining walls will be positively drained using heel drains with discharge into the main surface water system.

As such the existing hydraulic regime of the site will be modified whereby overland and subsurface flows will be intercepted, attenuated, and re-directed by below ground structures, positive drainage, infiltration trenches and service trenches.

Hydraulic gradients and velocities will be reduced, and the risk of downstream flooding would not be increased. Any unlikely surface emergence of any groundwater on-site will be intercepted by land drainage systems and directed into the various SuDS attenuation systems.

In addition, the proposed detention basin volumetric storage and controlled discharge to match QBAR for the development will greatly improve on the existing situation by reducing the rate and volume of water for more extreme storm events i.e. Greenfield Q100 flows for the overall site would be restricted to 74.2 l/s in comparison to the existing uncontrolled run-off rate of 216 l/s.

10.9 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 driveway/car parking areas will 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 will be raised to channel surface water runoff back into the drainage system or onto the existing highway.

Drainage Contingency – the sustainable drainage systems have been conservatively designed to attenuate a 100-year design storm including a 40% allowance for climate change, using a Cv value of 1.0 and a factored percentage of all pervious areas. The drainage system will also provide capacity for lower probability (greater design storm events) which are not critical duration.

Building Layout & Detail – the dwellings will be designed and situated to ensure that they are not at risk of flooding from overland flow. The finished floor and threshold levels of the proposed new dwellings will be set above the external levels, and external footpaths will fall away from the dwellings, ensuring that any flood water runs away from, rather than towards the properties.

Blockage and exceedance – In the unlikely case of exceedance or blockage of the detention basins, associated silt traps and/or flow control chambers, spills would occur from the lowest access cover around the properties. Exceedance flows shall be retained on site within the drainage system as far as practical and in the case of extreme events site levels will be set to divert any exceedance flows to fall towards and disperse into the more permeable green space areas where they would be contained and intercepted by positive land drainage which will discharge exceedance flows back into the detention basins, once the water level has subsided. A high-level overflow on the flow control chambers should prevent any spills from these manhole covers during exceedance events, however in the unlikely event that this or the basin were to overtop, exceedance flows would be directed towards the existing drainage ditches via. overflow spillways and channels where they would be contained and routed away from the site towards the outfall points.

10.10 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.

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[9] 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.

The simple index approach as outlined in the SuDS manual has been used to assess the pollution hazard indices and proposed treatment components, the calculations are included in Appendix B.

For the three categories of runoff areas served by the drainage system, Roof areas, residential parking and residential roads, treatment is proposed by directing all surface water runoff via. detention basins before discharge off site.

Tables 10.9 – 10.11 summarise the pollution hazard and mitigation indices for this type of runoff and show that adequate treatment of surface water runoff is provided by the use of detention basins.

Table 10.9 Pollution Hazard & Mitigation Indices - Roof Areas

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.2	0.2	0.05
Pollution Mitigation	0.5	0.5	0.6
Treatment Suitability	ADEQUATE	ADEQUATE	ADEQUATE

Table 10.10 Pollution Hazard & Mitigation Indices - Residential Parking

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.5	0.6
Treatment Suitability	ADEQUATE	ADEQUATE	ADEQUATE

Table 10.11 Pollution Hazard & Mitigation Indices - Residential Roads

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.5	0.6
Treatment Suitability	ADEQUATE	ADEQUATE	ADEQUATE

10.11 OPERATIONS & MAINTENANCE RESPONSIBILITY

It is envisaged that all access roads and below ground drainage is to remain private and will be maintained by a third-party management company established by John Swift Homes.

A SuDS 'Operations & Maintenance Plan' will be made available at detailed design stage to suit the finalised development parameters which will specify the requirements for future maintenance of the drainage system.

11. FOUL WATER DRAINAGE STRATEGY

It is proposed that foul water from the site shall be connected to the existing 150mm diameter foul sewer located within Summergrove Park to the north of the site.

Conventional gravity sewers will be utilised within the site to the maximum extent possible where site levels allow. However, due to the existing crested topography of the site it is anticipated that some plots will be located in naturally lower areas that will require foul flows to be pumped to higher ground within the site to allow disposal via a final gravity connection to the existing 150mm foul public sewer. In these isolated cases it is anticipated that the affected dwellings would be serviced by individual private pumping systems and therefore operation and maintenance of these systems would fall under the responsibility of the plot owner.

Under Section 106 of The Water Industry Act 1991, 'the owner / occupier of any premises shall be entitled to have his drain or sewer communicate with the public sewer of any sewerage undertaker and thereby to discharge foul water and surface water from those premises or that private sewer'. Unless 'the making of the communication would be prejudicial to the undertaker's sewerage system'.

A pre-development enquiry has been submitted to United Utilities to confirm acceptability of connection for foul water drainage from the development for the proposed initial phase of development. Their response provides approval in principle to discharge foul flows via a new connection to their asset. Correspondence is included in Appendix C for reference.

Preliminary foul water discharge calculations have been undertaken for the new dwellings in accordance with the Design and Construction Guidance for Foul and Surface Water Sewers [18].

Based on the landscape plan provided it is assumed that a maximum of 35 properties could be constructed as part of the initial phase of development. For conservative design this number of properties has been used to calculate the anticipated foul flows for this stage of development as shown in Table 11.1 below.

Table 11.1 Peak Foul Flow Rates – Phase 1 Development

Sewerage Sector Design and Construction Guidance Clause B3.1	
Peak Load Based on Number of Dwellings – 35 No. @ 4000 L/day	140,000
Peak Foul Flow Rate from Site (l/s)	1.62

This estimates predicted peak foul flow rates from the initial phase of development to be 1.62 l/s.

Using the same logic these figures have been scaled up in anticipation of any potential future development of the remainder of the site to provide an indicative conservative estimate of peak foul flows for the overall site as shown in Table 11.2.

Table 11.2 Peak Foul Flow Rates – Potential Future Development

Sewerage Sector Design and Construction Guidance Clause B3.1	
Peak Load Based on Number of Dwellings – 90 No. @ 4000 L/day	360,000
Peak Foul Flow Rate from Site (l/s)	4.16

The estimated predicted peak foul flow rate from the overall development of the site is 4.16 l/s.

12. CONCLUSIONS AND RECOMMENDATIONS

In consideration of the Flood Risk Assessment and Drainage Strategy for the new development the following conclusions and recommendations are made:

- The site is located in Flood Zone 1 with a predicted annual probability of flooding from rivers or the sea of less than 0.1% AEP (1 in 1000).
- The site is not considered to be at significant risk of flooding from surface water, groundwater, reservoirs, canals, or any artificial structures.
- Previous ground investigations have shown that the underlying ground conditions across the site have variable levels of permeability and high groundwater and are not deemed suitable for an infiltration-based SuDS solution for a development of this scale.
- The existing sloping topography is more suited to an interception and attenuation-based surface water drainage strategy.
- It is proposed that surface water drainage shall be positively drainage and attenuated, using 2 no. detention basins, with individual flow control devices restricting cumulative discharge to the match pre-development Greenfield QBAR rates.
- The respective detention basins have been sized on the premise of the whole site area being developed in the future thus requiring attenuation. The basins were sized based on the individual contributing areas and respective QBAR discharge rates to contain flows based on a Q100 + 40% storm event.
- Due to the potential for phased development, it is proposed that the basins are constructed to contain surface water generated for the full future developed area of the site with Hydrobake chambers controlling runoff to match the initial phase of development which could then be upsized to account for future development.
- Controlled cumulative runoff for the first phase of the development would therefore be restricted to match the equivalent Greenfield QBAR rate of 29.8 l/s prior to discharge to nearby existing drainage ditches.
- It is envisaged that the Hydrobrakes would then be replaced in line with future development to match the overall site Greenfield QBAR rate of 72.4 l/s to replicate existing conditions.
- Adequate treatment of surface water runoff generated by the development will be provided by the detention basins.
- Foul flows from the site will ultimately discharge via. a new connection to the existing 150mm diameter public foul sewer located in Summergrove Park residential estate site road. A pre-development enquiry has provided an approval in principle for connection from UU.

- Should planning permission be granted, the developer will accept pre-commencement planning conditions controlling detailed design of the drainage systems. At this point it is recommended that the drainage strategy is revisited and re-assessed as part of the detailed design process.

- In addition to these measures, a SuDS Operations and Maintenance Plan will be made available detailing future maintenance requirements of all sustainable drainage systems at detailed design stage to suit the finalised development scale and layout.

13. REFERENCES

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

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

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APPENDIX A

DRAWINGS

TOPOGRAPHIC SURVEY

INDICATIVE DRAINAGE LAYOUT

Revision Notes:
RevA:07/09/2017–Road to East added

N

Survey Note:
Positions are relative to OSGB36 National Grid, using Transformation OSTN02. Heights are relative to Ordnance Datum Newlyn, using Geoid Model OSGM02.
Contours @ 0.5m intervals.

Survey Key:

Wall			Cover
Kerb			Man Hole
Fence			Fire Hydrant
Vegetation Line		⊗WS	Water Supply
Tree		LP	Lamp Post
Bush		EP	Electricity Pole
Gate		TP	Telegraph Pole
Drop Kerb			Anchor Point
O/H Telecomms			Bench Mark
O/H Power			Bore Hole
		AV	Air Valve
		FP	Fire Point
		⊕SV	Stop Valve
		CATV	Cable TV

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URL: www.SpatialDataLtd.com

Client: Swift Homes

Project: Summergrove, Whitehaven

Site Survey: August 2016
Scale:1:1000 @ A1
Crew:DH Chk: NR Ref: KRH

Controlled discharge to outfall to existing drainage ditch via. existing pipework.
Approximate location of existing land drainage outfall chamber from site
HYDROBRAKE FLOW CONTROL CHAMBER:
Phase 1: Greenfield QBAR Design Flow = 24.9 l/s
Full Developed Site: Greenfield QBAR Design Flow = 54.2 l/s
Incoming surface water site drainage
Indicative Size of Detention Basin required for full developed eastern site area SW attenuation based on assumed total impermeable area of 65% (Area A).
Storage Volume required = 2856m^3
Surface Area required = 2465m^2 x 1.5m deep
Max side slope gradient 1:3
Equivalent Greenfield QBAR Design Flow = 54.2 l/s
Foul sewage to discharge via. new connection to existing public foul sewer located in Summergrove Park residential estate highway.
Foul sewage to discharge to final chamber located within site boundary to allow connection to existing UU manhole via. short section of gravity pipework. Foul drainage routing to be confirmed to suit final layout.
Summergrove (Conference Centre)
POOLES CLOSE
Track
93.9m
89.3m
83.5m
Area B
Ash Lee
Galemire House
DRAINAGE KEY
Proposed surface water drainage
Proposed foul drainage
Existing public foul sewer
SITE CATCHMENT AREAS
Area A - Phase 1 [2.800 Ha]
Area A - Full Developed Site [6.088 Ha]
Area B - Phase 1 [0.550 Ha]
Area B - Full Developed Site [2.252 Ha]
BASIN B HYDROBRAKE FLOWCONTROL CHAMBER:
Phase 1: Greenfield QBAR Design Flow = 4.9 l/s
Full Developed Site: Greenfield QBAR Design Flow = 20.0 l/s
Incoming site drainage
Controlled discharge to outfall to existing drainage ditch.
DETENTION BASIN B:
Indicative Size of Detention Basin required for full developed western site area SW attenuation based on assumed total impermeable area of 65% (Area B).
Storage Volume required = 1028m^3
Surface Area required = 985m^2 x 1.5m deep
Max side slope gradient 1:3
Equivalent Greenfield QBAR Design Flow = 20 l/s
A Drainage updated 11/05/22 CA TM TM
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
Client: John Swift Homes
Project: Summergrove, Whitehaven
Drawing Title: Outline Drainage Layout
Scale @ A1: 1/750
First Issue: 08/04/2022
Office of Origin: Kendal
Drawn by: CA
Checked by: OS
Approved: OS
Project No: K39183
Drawing No: 100
Rev:
BIM No:

APPENDIX B

CALCULATIONS

PRE-DEVELOPMENT RUNOFF

SOURCE CONTROL ATTENUATION

SURFACE WATER TREATMENT

R G PARKINS 97 King Street | Lancaster | LA1 1RH Tel:01524 32548 Email: office@rgparkinslancaster.co.uk	**Wallingford Runoff Estimation**	Job Number K9183	Page Number 1 of 4
		Calc by CA	Check by OS
	Summergrove Whitehaven	Date 01/04/2022	Revised

DESIGN BASIS MEMORANDUM - PEAK RATE OF RUN-OFF CALCULATION FULL SITE

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 Brownfield Site

Results Summary

Rate of Run-Off (l/s)			
Event	**Greenfield**	[redacted]	**Post-Development**
Q1	64.5		64.5
QBAR	74.2		74.2
Q10	102.4		102.4
Q30	126.1		126.1
Q100	154.3		154.3
Q100 + 40% CC	216.0		216.0

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		Calc by CA	Check by OS
	Summergrove Whitehaven	Date 01/04/2022	Revised

SITE AREAS (LAND COVER AREAS)

Existing Impermeable & Permeable Land Cover

Total Site Area: **8.34** ha **83400** m²

Existing Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total impermeable area	83400.0	8.340	100%
Remaining permeable area	0.0	0.000	0%

Proposed Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total impermeable area	41700.0	4.170	50%
Remaining permeable area	41700.0	4.170	50%

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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	=	**1180**	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. This seems reasonable based on ground investigation.

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

$QBAR_{rural}$	=	0.445	m^3/s
	=	444.7	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	=	**83400**	m^2
	=	0.083	km^2
	=	8.340	ha
$QBAR_{rural\ site}$	=	0.07417	m^3/s
	=	**74.17**	l/s

Excluding significant open space which would remain disconnected from the positive drainage system during flood events- Layout unkown at this stage

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Wallingford Runoff Estimation	Job Number K9183	Page Number 4 of 4
	Calc by CA	Check by OS
Summergrove Whitehaven	Date 44652	Revised 0

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	64.53
2	0.93	68.98
5	1.19	88.27
10	1.38	102.36
25	1.64	121.65
30	1.7	126.10
50	1.85	137.22
100	2.08	154.28
200	2.32	172.09
500	2.73	202.50
1000	3.04	225.49

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

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DESIGN BASIS MEMORANDUM - PEAK RATE OF RUN-OFF CALCULATION PHASE 1

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 Brownfield Site

Results Summary

Rate of Run-Off (l/s)			
Event	Greenfield	[redacted]	Post-Development
Q1	25.9		25.9
QBAR	29.8		29.8
Q10	41.1		41.1
Q30	50.7		50.7
Q100	62.0		62.0
Q100 + 40% CC	86.8		86.8

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Wallingford Runoff Estimation	Job Number K39183	Page Number 2 of 4
	Calc by CA	Check by OS
Summergrove - Phase 1 Whitehaven	Date 01/04/2022	Revised

SITE AREAS (LAND COVER AREAS)

Existing Impermeable & Permeable Land Cover

Total Site Area: 3.35 ha 33500 m²

Existing Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total impermeable area	33500.0	3.350	100%
Remaining permeable area	0.0	0.000	0%

Proposed Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total impermeable area	16750.0	1.675	50%
Remaining permeable area	16750.0	1.675	50%

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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	=	**1180**	mm	From FEH Web Service (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.

$QBAR_{rural}$	=	0.445	m^3/s
	=	444.7	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	=	**33500**	m^2
		0.034	km^2
	=	3.350	ha
$QBAR_{rural\ site}$	=	0.02979	m^3/s
	=	**29.79**	l/s

Excluding significant open space which would remain disconnected from the positive drainage system during flood events - Layout unkown at this stage

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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	25.92
2	0.93	27.71
5	1.19	35.46
10	1.38	41.12
25	1.64	48.86
30	1.7	50.65
50	1.85	55.12
100	2.08	61.97
200	2.32	69.12
500	2.73	81.34
1000	3.04	90.58

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	88.450	0.450	54.0	827.2	O K
30 min Summer	88.615	0.615	54.2	1154.3	O K
60 min Summer	88.793	0.793	54.2	1519.7	O K
120 min Summer	88.901	0.901	54.2	1751.3	O K
180 min Summer	88.955	0.955	54.2	1868.5	O K
240 min Summer	88.984	0.984	54.2	1931.5	O K
360 min Summer	89.005	1.005	54.2	1978.1	O K
480 min Summer	89.010	1.010	54.2	1989.4	O K
600 min Summer	89.006	1.006	54.2	1979.6	O K
720 min Summer	88.996	0.996	54.2	1957.3	O K
960 min Summer	88.964	0.964	54.2	1887.2	O K
1440 min Summer	88.883	0.883	54.2	1712.1	O K
2160 min Summer	88.762	0.762	54.2	1456.0	O K
2880 min Summer	88.653	0.653	54.2	1230.1	O K
4320 min Summer	88.479	0.479	54.1	884.0	O K
5760 min Summer	88.371	0.371	53.2	674.1	O K
7200 min Summer	88.309	0.309	51.8	558.0	O K
8640 min Summer	88.282	0.282	48.8	506.9	O K
10080 min Summer	88.264	0.264	45.4	474.5	O K
15 min Winter	88.595	0.595	54.2	1112.5	O K
30 min Winter	88.811	0.811	54.2	1558.8	O K
60 min Winter	89.045	1.045	54.2	2066.3	O K
120 min Winter	89.199	1.199	54.2	2415.8	O K
180 min Winter	89.281	1.281	54.2	2605.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	116.700	0.0	828.9	22
30 min Summer	82.484	0.0	1183.8	36
60 min Summer	55.690	0.0	1634.1	66
120 min Summer	33.889	0.0	1992.0	124
180 min Summer	25.361	0.0	2237.8	182
240 min Summer	20.645	0.0	2429.9	240
360 min Summer	15.430	0.0	2725.5	318
480 min Summer	12.536	0.0	2953.1	382
600 min Summer	10.660	0.0	3139.3	446
720 min Summer	9.331	0.0	3297.6	512
960 min Summer	7.548	0.0	3556.3	648
1440 min Summer	5.588	0.0	3946.3	914
2160 min Summer	4.152	0.0	4425.8	1304
2880 min Summer	3.373	0.0	4792.3	1676
4320 min Summer	2.523	0.0	5366.2	2380
5760 min Summer	2.069	0.0	5890.7	3056
7200 min Summer	1.789	0.0	6367.2	3744
8640 min Summer	1.599	0.0	6824.5	4416
10080 min Summer	1.462	0.0	7267.0	5144
15 min Winter	116.700	0.0	1115.1	22
30 min Winter	82.484	0.0	1587.4	36
60 min Winter	55.690	0.0	2183.9	64
120 min Winter	33.889	0.0	2660.8	122
180 min Winter	25.361	0.0	2988.3	180

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	89.329	1.329	54.2	2720.8	O K
360 min Winter	89.376	1.376	54.2	2831.6	O K
480 min Winter	89.386	1.386	54.2	2856.1	O K
600 min Winter	89.377	1.377	54.2	2835.8	O K
720 min Winter	89.368	1.368	54.2	2814.3	O K
960 min Winter	89.334	1.334	54.2	2733.0	O K
1440 min Winter	89.233	1.233	54.2	2495.6	O K
2160 min Winter	89.045	1.045	54.2	2067.1	O K
2880 min Winter	88.845	0.845	54.2	1630.5	O K
4320 min Winter	88.527	0.527	54.2	977.3	O K
5760 min Winter	88.345	0.345	52.7	625.6	O K
7200 min Winter	88.282	0.282	48.8	507.3	O K
8640 min Winter	88.257	0.257	43.9	460.9	O K
10080 min Winter	88.240	0.240	40.3	430.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	20.645	0.0	3244.3	238
360 min Winter	15.430	0.0	3638.1	348
480 min Winter	12.536	0.0	3941.3	456
600 min Winter	10.660	0.0	4189.2	540
720 min Winter	9.331	0.0	4399.7	570
960 min Winter	7.548	0.0	4743.6	726
1440 min Winter	5.588	0.0	5259.6	1038
2160 min Winter	4.152	0.0	5904.4	1476
2880 min Winter	3.373	0.0	6394.8	1852
4320 min Winter	2.523	0.0	7164.9	2552
5760 min Winter	2.069	0.0	7857.0	3168
7200 min Winter	1.789	0.0	8493.3	3744
8640 min Winter	1.599	0.0	9104.8	4416
10080 min Winter	1.462	0.0	9699.6	5144

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 299997 515506 NX 99997 15506
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	1.000
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 3.960

Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)
0	4	1.980	4	8	1.980

Model Details

Storage is Online Cover Level (m) 89.500

Tank or Pond Structure

Invert Level (m) 88.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	1735.9	1.500	2464.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0299-5420-1500-5420
Design Head (m)	1.500
Design Flow (l/s)	54.2
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	299
Invert Level (m)	88.000
Minimum Outlet Pipe Diameter (mm)	375
Suggested Manhole Diameter (mm)	2100

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.500	54.2	Kick-Flo®	1.081	46.3
Flush-Flo™	0.518	54.2	Mean Flow over Head Range	-	45.7

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	9.2	1.200	48.7	3.000	75.8	7.000	114.5
0.200	30.8	1.400	52.4	3.500	81.7	7.500	118.4
0.300	51.6	1.600	55.9	4.000	87.2	8.000	122.2
0.400	53.6	1.800	59.2	4.500	92.3	8.500	125.9
0.500	54.2	2.000	62.3	5.000	97.1	9.000	129.4
0.600	54.0	2.200	65.2	5.500	101.8	9.500	132.9
0.800	52.6	2.400	68.0	6.000	106.2		
1.000	49.3	2.600	70.7	6.500	110.4		

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	87.496	0.496	20.0	303.9	O K
30 min Summer	87.668	0.668	20.0	424.3	O K
60 min Summer	87.845	0.845	20.0	557.7	O K
120 min Summer	87.950	0.950	20.0	641.2	O K
180 min Summer	88.002	1.002	20.0	683.8	O K
240 min Summer	88.028	1.028	20.0	705.3	O K
360 min Summer	88.040	1.040	20.0	715.4	O K
480 min Summer	88.037	1.037	20.0	712.5	O K
600 min Summer	88.027	1.027	20.0	704.1	O K
720 min Summer	88.013	1.013	20.0	692.4	O K
960 min Summer	87.974	0.974	20.0	661.0	O K
1440 min Summer	87.883	0.883	20.0	587.5	O K
2160 min Summer	87.752	0.752	20.0	486.7	O K
2880 min Summer	87.633	0.633	20.0	399.5	O K
4320 min Summer	87.441	0.441	20.0	266.9	O K
5760 min Summer	87.320	0.320	19.6	188.3	O K
7200 min Summer	87.250	0.250	18.9	144.8	O K
8640 min Summer	87.211	0.211	18.2	121.0	O K
10080 min Summer	87.193	0.193	17.1	110.8	O K
15 min Winter	87.648	0.648	20.0	409.8	O K
30 min Winter	87.866	0.866	20.0	574.4	O K
60 min Winter	88.094	1.094	20.0	761.2	O K
120 min Winter	88.235	1.235	20.0	884.9	O K
180 min Winter	88.308	1.308	20.0	951.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	116.700	0.0	315.3	22
30 min Summer	82.484	0.0	447.3	36
60 min Summer	55.690	0.0	608.9	66
120 min Summer	33.889	0.0	741.5	124
180 min Summer	25.361	0.0	832.6	182
240 min Summer	20.645	0.0	903.8	242
360 min Summer	15.430	0.0	1013.4	338
480 min Summer	12.536	0.0	1097.8	396
600 min Summer	10.660	0.0	1167.0	462
720 min Summer	9.331	0.0	1225.8	526
960 min Summer	7.548	0.0	1322.0	662
1440 min Summer	5.588	0.0	1467.7	924
2160 min Summer	4.152	0.0	1639.4	1316
2880 min Summer	3.373	0.0	1775.5	1680
4320 min Summer	2.523	0.0	1990.6	2380
5760 min Summer	2.069	0.0	2179.7	3056
7200 min Summer	1.789	0.0	2356.5	3744
8640 min Summer	1.599	0.0	2526.6	4408
10080 min Summer	1.462	0.0	2692.8	5144
15 min Winter	116.700	0.0	421.7	22
30 min Winter	82.484	0.0	597.5	36
60 min Winter	55.690	0.0	812.5	66
120 min Winter	33.889	0.0	989.2	122
180 min Winter	25.361	0.0	1110.7	180

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	88.350	1.350	20.0	989.9	O K
360 min Winter	88.387	1.387	20.0	1024.5	O K
480 min Winter	88.391	1.391	20.0	1028.2	O K
600 min Winter	88.378	1.378	20.0	1016.3	O K
720 min Winter	88.365	1.365	20.0	1004.5	O K
960 min Winter	88.328	1.328	20.0	969.2	O K
1440 min Winter	88.228	1.228	20.0	878.1	O K
2160 min Winter	88.059	1.059	20.0	731.2	O K
2880 min Winter	87.842	0.842	20.0	555.1	O K
4320 min Winter	87.492	0.492	20.0	301.1	O K
5760 min Winter	87.293	0.293	19.4	171.4	O K
7200 min Winter	87.206	0.206	18.1	118.4	O K
8640 min Winter	87.185	0.185	16.4	105.7	O K
10080 min Winter	87.171	0.171	15.0	97.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	20.645	0.0	1205.6	238
360 min Winter	15.430	0.0	1351.7	348
480 min Winter	12.536	0.0	1464.3	456
600 min Winter	10.660	0.0	1556.4	542
720 min Winter	9.331	0.0	1634.7	570
960 min Winter	7.548	0.0	1762.9	726
1440 min Winter	5.588	0.0	1956.4	1038
2160 min Winter	4.152	0.0	2186.3	1492
2880 min Winter	3.373	0.0	2368.1	1876
4320 min Winter	2.523	0.0	2655.6	2552
5760 min Winter	2.069	0.0	2906.7	3168
7200 min Winter	1.789	0.0	3142.5	3744
8640 min Winter	1.599	0.0	3369.6	4408
10080 min Winter	1.462	0.0	3591.8	5144

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 299997 515506 NX 99997 15506
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	1.000
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.464

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	0.732	4	8	0.732

Model Details

Storage is Online Cover Level (m) 88.500

Tank or Pond Structure

Invert Level (m) 87.000

Depth (m)	**Area (m²)**	**Depth (m)**	**Area (m²)**
0.000	548.1	1.500	985.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0192-2000-1500-2000
Design Head (m)	1.500
Design Flow (l/s)	20.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	192
Invert Level (m)	87.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	**Head (m)**	**Flow (l/s)**	**Control Points**	**Head (m)**	**Flow (l/s)**
Design Point (Calculated)	1.500	20.0	Kick-Flo®	0.984	16.4
Flush-Flo™	0.452	20.0	Mean Flow over Head Range	-	17.3

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	6.7	1.200	18.0	3.000	27.8	7.000	41.9
0.200	17.7	1.400	19.3	3.500	30.0	7.500	43.3
0.300	19.4	1.600	20.6	4.000	32.0	8.000	44.7
0.400	19.9	1.800	21.8	4.500	33.8	8.500	46.0
0.500	20.0	2.000	22.9	5.000	35.6	9.000	47.3
0.600	19.7	2.200	24.0	5.500	37.3	9.500	48.6
0.800	18.8	2.400	25.0	6.000	38.9		
1.000	16.5	2.600	26.0	6.500	40.4		

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	88.216	0.216	22.2	384.7	O K
30 min Summer	88.297	0.297	24.1	535.1	O K
60 min Summer	88.387	0.387	24.8	704.6	O K
120 min Summer	88.444	0.444	24.9	814.8	O K
180 min Summer	88.473	0.473	24.9	871.7	O K
240 min Summer	88.489	0.489	24.9	902.8	O K
360 min Summer	88.505	0.505	24.9	933.7	O K
480 min Summer	88.512	0.512	24.9	948.8	O K
600 min Summer	88.515	0.515	24.9	953.9	O K
720 min Summer	88.514	0.514	24.9	951.8	O K
960 min Summer	88.505	0.505	24.9	933.6	O K
1440 min Summer	88.474	0.474	24.9	873.3	O K
2160 min Summer	88.423	0.423	24.9	774.7	O K
2880 min Summer	88.376	0.376	24.7	684.5	O K
4320 min Summer	88.301	0.301	24.2	541.9	O K
5760 min Summer	88.251	0.251	23.5	449.3	O K
7200 min Summer	88.222	0.222	22.9	395.6	O K
8640 min Summer	88.206	0.206	21.2	366.3	O K
10080 min Summer	88.193	0.193	19.8	343.9	O K
15 min Winter	88.286	0.286	24.0	514.6	O K
30 min Winter	88.395	0.395	24.8	719.9	O K
60 min Winter	88.515	0.515	24.9	953.3	O K
120 min Winter	88.596	0.596	24.9	1115.1	O K
180 min Winter	88.641	0.641	24.9	1206.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	116.700	0.0	356.7	22
30 min Summer	82.484	0.0	517.6	36
60 min Summer	55.690	0.0	737.3	66
120 min Summer	33.889	0.0	901.0	124
180 min Summer	25.361	0.0	1013.4	182
240 min Summer	20.645	0.0	1101.2	240
360 min Summer	15.430	0.0	1236.1	302
480 min Summer	12.536	0.0	1339.8	366
600 min Summer	10.660	0.0	1424.6	432
720 min Summer	9.331	0.0	1496.4	500
960 min Summer	7.548	0.0	1613.5	638
1440 min Summer	5.588	0.0	1788.4	910
2160 min Summer	4.152	0.0	2024.5	1300
2880 min Summer	3.373	0.0	2191.3	1676
4320 min Summer	2.523	0.0	2448.1	2384
5760 min Summer	2.069	0.0	2701.7	3104
7200 min Summer	1.789	0.0	2919.1	3752
8640 min Summer	1.599	0.0	3126.5	4496
10080 min Summer	1.462	0.0	3324.0	5240
15 min Winter	116.700	0.0	486.6	22
30 min Winter	82.484	0.0	700.6	36
60 min Winter	55.690	0.0	989.1	64
120 min Winter	33.889	0.0	1207.3	122
180 min Winter	25.361	0.0	1357.0	180

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	88.669	0.669	24.9	1263.8	O K
360 min Winter	88.699	0.699	24.9	1324.1	O K
480 min Winter	88.708	0.708	24.9	1344.0	O K
600 min Winter	88.707	0.707	24.9	1342.1	O K
720 min Winter	88.706	0.706	24.9	1338.5	O K
960 min Winter	88.692	0.692	24.9	1309.5	O K
1440 min Winter	88.642	0.642	24.9	1208.0	O K
2160 min Winter	88.555	0.555	24.9	1033.5	O K
2880 min Winter	88.471	0.471	24.9	868.4	O K
4320 min Winter	88.335	0.335	24.5	606.7	O K
5760 min Winter	88.250	0.250	23.4	447.9	O K
7200 min Winter	88.212	0.212	21.8	378.9	O K
8640 min Winter	88.194	0.194	19.8	344.5	O K
10080 min Winter	88.181	0.181	18.3	321.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	20.645	0.0	1473.9	236
360 min Winter	15.430	0.0	1653.5	348
480 min Winter	12.536	0.0	1791.5	454
600 min Winter	10.660	0.0	1904.1	504
720 min Winter	9.331	0.0	1999.6	568
960 min Winter	7.548	0.0	2155.1	722
1440 min Winter	5.588	0.0	2386.5	1024
2160 min Winter	4.152	0.0	2703.5	1448
2880 min Winter	3.373	0.0	2927.4	1844
4320 min Winter	2.523	0.0	3274.3	2552
5760 min Winter	2.069	0.0	3605.3	3224
7200 min Winter	1.789	0.0	3896.2	3824
8640 min Winter	1.599	0.0	4174.6	4504
10080 min Winter	1.462	0.0	4442.1	5248

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 299997 515506 NX 99997 15506
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	1.000
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 1.820

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.910	4	8	0.910

Model Details

Storage is Online Cover Level (m) 89.500

Tank or Pond Structure

Invert Level (m) 88.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	1735.9	1.500	2464.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0212-2490-1500-2490
Design Head (m)	1.500
Design Flow (l/s)	24.9
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	212
Invert Level (m)	88.000
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.500	24.9	Kick-Flo®	1.000	20.5
Flush-Flo™	0.455	24.9	Mean Flow over Head Range	-	21.4

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	7.2	1.200	22.4	3.000	34.7	7.000	52.3
0.200	20.5	1.400	24.1	3.500	37.4	7.500	54.0
0.300	24.2	1.600	25.7	4.000	39.9	8.000	55.8
0.400	24.8	1.800	27.2	4.500	42.2	8.500	57.4
0.500	24.8	2.000	28.6	5.000	44.4	9.000	59.0
0.600	24.6	2.200	29.9	5.500	46.5	9.500	60.6
0.800	23.6	2.400	31.2	6.000	48.5		
1.000	20.6	2.600	32.4	6.500	50.4		

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	87.134	0.134	3.9	75.8	O K
30 min Summer	87.185	0.185	4.3	105.6	O K
60 min Summer	87.241	0.241	4.5	139.2	O K
120 min Summer	87.276	0.276	4.6	161.3	O K
180 min Summer	87.295	0.295	4.7	172.7	O K
240 min Summer	87.305	0.305	4.7	179.1	O K
360 min Summer	87.315	0.315	4.7	185.1	O K
480 min Summer	87.320	0.320	4.7	188.2	O K
600 min Summer	87.321	0.321	4.7	189.3	O K
720 min Summer	87.321	0.321	4.7	189.1	O K
960 min Summer	87.316	0.316	4.7	186.1	O K
1440 min Summer	87.299	0.299	4.7	175.4	O K
2160 min Summer	87.271	0.271	4.6	158.0	O K
2880 min Summer	87.245	0.245	4.5	141.8	O K
4320 min Summer	87.201	0.201	4.4	115.1	O K
5760 min Summer	87.169	0.169	4.2	96.3	O K
7200 min Summer	87.147	0.147	4.0	83.3	O K
8640 min Summer	87.131	0.131	3.9	74.2	O K
10080 min Summer	87.121	0.121	3.8	68.2	O K
15 min Winter	87.178	0.178	4.2	101.4	O K
30 min Winter	87.245	0.245	4.5	142.0	O K
60 min Winter	87.319	0.319	4.7	188.0	O K
120 min Winter	87.369	0.369	4.8	220.0	O K
180 min Winter	87.397	0.397	4.8	238.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	116.700	0.0	71.7	22
30 min Summer	82.484	0.0	103.4	36
60 min Summer	55.690	0.0	146.0	66
120 min Summer	33.889	0.0	178.2	124
180 min Summer	25.361	0.0	200.3	182
240 min Summer	20.645	0.0	217.6	240
360 min Summer	15.430	0.0	244.2	308
480 min Summer	12.536	0.0	264.6	372
600 min Summer	10.660	0.0	281.3	436
720 min Summer	9.331	0.0	295.5	506
960 min Summer	7.548	0.0	318.6	644
1440 min Summer	5.588	0.0	353.1	914
2160 min Summer	4.152	0.0	398.9	1320
2880 min Summer	3.373	0.0	431.9	1704
4320 min Summer	2.523	0.0	482.9	2428
5760 min Summer	2.069	0.0	531.9	3168
7200 min Summer	1.789	0.0	574.7	3888
8640 min Summer	1.599	0.0	615.7	4576
10080 min Summer	1.462	0.0	655.0	5248
15 min Winter	116.700	0.0	97.3	22
30 min Winter	82.484	0.0	139.4	36
60 min Winter	55.690	0.0	195.5	64
120 min Winter	33.889	0.0	238.5	122
180 min Winter	25.361	0.0	268.0	180

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	87.415	0.415	4.8	249.4	O K
360 min Winter	87.433	0.433	4.8	261.5	O K
480 min Winter	87.439	0.439	4.8	265.6	O K
600 min Winter	87.439	0.439	4.8	265.6	O K
720 min Winter	87.439	0.439	4.8	265.3	O K
960 min Winter	87.432	0.432	4.8	260.5	O K
1440 min Winter	87.405	0.405	4.8	242.8	O K
2160 min Winter	87.357	0.357	4.7	212.2	O K
2880 min Winter	87.312	0.312	4.7	183.4	O K
4320 min Winter	87.235	0.235	4.5	135.9	O K
5760 min Winter	87.182	0.182	4.2	103.7	O K
7200 min Winter	87.146	0.146	4.0	82.9	O K
8640 min Winter	87.124	0.124	3.8	70.0	O K
10080 min Winter	87.113	0.113	3.5	63.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	20.645	0.0	291.0	236
360 min Winter	15.430	0.0	326.4	348
480 min Winter	12.536	0.0	353.6	454
600 min Winter	10.660	0.0	375.8	502
720 min Winter	9.331	0.0	394.6	568
960 min Winter	7.548	0.0	425.3	722
1440 min Winter	5.588	0.0	470.8	1026
2160 min Winter	4.152	0.0	532.5	1452
2880 min Winter	3.373	0.0	576.6	1852
4320 min Winter	2.523	0.0	645.4	2600
5760 min Winter	2.069	0.0	709.6	3336
7200 min Winter	1.789	0.0	767.0	3968
8640 min Winter	1.599	0.0	821.9	4592
10080 min Winter	1.462	0.0	874.9	5344

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 299997 515506 NX 99997 15506
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	1.000
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.358

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	0.179	4	8	0.179

Model Details

Storage is Online Cover Level (m) 88.500

Tank or Pond Structure

Invert Level (m) 87.000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	548.1	1.500	985.1

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0097-4900-1500-4900
Design Head (m)	1.500
Design Flow (l/s)	4.9
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	97
Invert Level (m)	87.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.500	4.9	Kick-Flo®	0.867	3.8
Flush-Flo™	0.424	4.8	Mean Flow over Head Range	-	4.2

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	3.2	1.200	4.4	3.000	6.8	7.000	10.1
0.200	4.3	1.400	4.7	3.500	7.3	7.500	10.5
0.300	4.7	1.600	5.0	4.000	7.8	8.000	10.8
0.400	4.8	1.800	5.3	4.500	8.2	8.500	11.1
0.500	4.7	2.000	5.6	5.000	8.6	9.000	11.4
0.600	4.7	2.200	5.9	5.500	9.0	9.500	11.7
0.800	4.2	2.400	6.1	6.000	9.4		
1.000	4.1	2.600	6.3	6.500	9.8		

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		Whitehaven	Revision		Initial	CA
	Title	Sustainable Drainage - Treatment			Checked	OS

DESIGN BASIS MEMORANDUM - SUSTAINABLE DRAINAGE TREATMENT OF SURFACE WATER

Design Brief

The following calculations outline the recommended treatment requirements for a sustaionable drainage system as outlined in the SuDS Manual 2015. The method used is the simple index approach outlined in section 26.
The requirement for oil interceptors has been assessed in line with the now withdrawn Pollution Prevention Guidance document PPG3, produced by the Environment Agency. An oil interceptor is not required for the proposed development.

Treatment within SuDS components is affected by the flow rate and volume of water which passes through the component. It is not reasonable or practical to treat the entirety of the runoff for infrequent greater intensity design storms. In any case the majority of the pollutants are removed from surfaces by the more frequent rainfall events and in the first flush resulting from the initial runoff from the larger events.
and to a certain capacity.

The following references have been used in the preparation of these calculations:

- SUDS Manual, CIRIA Report C753, 2015
- Pollution Mitigation Indicies provided by Hydro International

Results Summary

Roof Area:

Treatment component 1 Detention basin
Treatment component 2 None

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.2	0.2	0.05
Pollution Mitigation	0.5	0.5	0.6
Treatment Suitability	Adequate	Adequate	Adequate

Residential Parking:

Treatment component 1 Detention basin
Treatment component 2 None

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.5	0.6
Treatment Suitability	Adequate	Adequate	Adequate

Residential Roads

Treatment component 1 Detention basin
Treatment component 2 None

Indices	Suspended Solids	Metals	Hydrocarbons
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.5	0.6
Treatment Suitability	Adequate	Adequate	Adequate

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POLLUTION HAZARD INDEX

Source of Runoff	Pollution Hazard	Pollution Hazard Indices: Suspended Solids	Metals	Hydro-carbons
Residential roofing	Very low	0.2	0.2	0.05

POLLUTION MITIGATION INDEX

The receiving water body shall be: Surface Water

Suds Component		Pollution Mitigation Indices: Suspended Solids	Metals	Hydro-carbons
1	Detention basin	0.5	0.5	0.6
2	None	0	0	0
3	None	0	0	0
4	None	0	0	0
Total Pollution Mitigation Index		**0.5**	**0.5**	**0.6**

ASSESSMENT OF TREATMENT PROPOSAL

Indices	Suspended Solids	Metals	Hydro-carbons
Pollution Hazard	0.2	0.2	0.05
Pollution Mitigation	0.5	0.5	0.6
	Adequate	**Adequate**	**Adequate**

POLLUTION HAZARD INDEX

Source of Runoff	Pollution Hazard	Pollution Hazard Indices: Suspended Solids	Metals	Hydro-carbons
Residential parking	Low	0.5	0.4	0.4

POLLUTION MITIGATION INDEX

The receiving water body shall be: Surface Water

Suds Component		Pollution Mitigation Indices: Suspended Solids	Metals	Hydro-carbons
1	Detention basin	0.5	0.5	0.6
2	None	0	0	0
3	None	0	0	0
4	None	0	0	0
	Total Pollution Mitigation Index	**0.5**	**0.5**	**0.6**

ASSESSMENT OF TREATMENT PROPOSAL

Indices	Suspended Solids	Metals	Hydro-carbons
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.5	0.6
	Adequate	**Adequate**	**Adequate**

R G PARKINS Kendal | 01539 729393 Lancaster | 01524 32548	CALCULATION		Job No.	K39183	Page	4 of 4
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		Whitehaven	Revision		Initial	CA
	Title	Sustainable Drainage - Treatment			Checked	OS

POLLUTION HAZARD INDEX

		Pollution Hazard Indices		
Source of Runoff	**Pollution Hazard**	**Suspended Solids**	**Metals**	**Hydro-carbons**
Low traffic roads (e.g. residential roads and general access roads, < 300 traffic movements/day)	Low	0.5	0.4	0.4

POLLUTION MITIGATION INDEX

The receiving water body shall be: Surface Water

		Pollution Mitigation Indices		
	Suds Component	**Suspended Solids**	**Metals**	**Hydro-carbons**
1	Detention basin	0.5	0.5	0.6
2	None	0	0	0
3	None	0	0	0
4	None	0	0	0
	Total Pollution Mitigation Index	**0.5**	**0.5**	**0.6**

ASSESSMENT OF TREATMENT PROPOSAL

Indices	**Suspended Solids**	**Metals**	**Hydro-carbons**
Pollution Hazard	0.5	0.4	0.4
Pollution Mitigation	0.5	0.5	0.6
	Adequate	**Adequate**	**Adequate**

APPENDIX C

UU CORRESPONDENCE

PRE-DEVELOPMENT ENQUIRY RESPONSE EMAIL

Chris Abram

From: Wastewater Developer Services <WastewaterDeveloperServices@uuplc.co.uk>
Sent: 05 May 2022 15:19
To: Chris Abram
Cc: Troy Melhuish; Wastewater Developer Services
Subject: RE: Pre-Development Enquiry - Land South of Summergrove Park, Whitehaven CA28 8YH - UU ref 4200049122

Good Afternoon Chris,

Pre Development Enquiry for: Land South of Summergrove Park, Whitehaven CA28 8YH - UU ref 4200049122

We have carried out an assessment of your application which is based on the information provided. This pre-development advice on your drainage strategy will be valid for 12 months. Your drainage strategy will need to be reviewed by other competent authorities as part of the planning process, and we advise that you carry out the necessary site investigations to confirm the viability of your proposals.

If your investigations require access to our public sewer network, we ask that you contact our network engineers with a request for an access certificate via our main contact telephone number 0345 6723 723 or refer to the link below:

https://www.unitedutilities.com/builders-developers/working-near-our-assets/

Foul Water

Foul flow from this site will be allowed to drain into the public foul water/combined sewer system. We would have no objections to the proposed connection to the 150mm diameter public foul sewer within Summergrove Park (UUMH0504). As the proposals involve pumping foul flows, we would ask that the pump rate is kept as minimal as possible. That being said we would recommend every effort is made to avoid the pumping station – the connection point should be checked on site to confirm levels, hopefully it is deep enough to allow for a gravity connection.

In accordance with our infrastructure plans we may ask you to change your point of connection. Therefore please contact us when you are ready to formalise your drainage proposals, we would suggest before you submit for Full Planning.

Surface Water

Thank you for outlining the surface water proposals, which will involve draining surface water to nearby open watercourses via a series of SuDS features. Whilst fundamentally this would be acceptable to United Utilities, we would (as always) like to advise the following:

All surface water flow from the proposed development should drain in-line with the drainage hierarchy, as outlined in Paragraph 80, (Reference ID: 7-080-20150323), of the National Planning Practice Guidance. We also recommend you prioritise the use of multi-functional sustainable drainage systems for the management of surface water in accordance with national planning policy.

Generally, the aim should be to discharge surface run off as high up the following hierarchy of drainage options as reasonably practicable.
This is outlined as follows, in order of priority:

1. **into the ground (infiltration);**
2. **to a surface waterbody;**
3. **to a surface water sewer or highway drain;**

4. **to a combined sewer.**

For guidance, The **North West SuDS Pro-Forma** provides information on the appropriate evidence required at each stage of the hierarchy, to demonstrate how each level has been discounted.

The Lead Local Flood Authority has responsibility for all surface water drainage concerns and their input to your proposal is critical. You should also consider whether it is necessary to discuss your proposal with the Environment Agency, or Internal Drainage Board (if operating in your area).

The Local Planning Authority are the determining authority for any application for planning permission and the appropriate authority for determining cost viability of a proposed drainage scheme, such assessments are outside of the jurisdiction of United Utilities.

Infiltration

Surface water runoff generated from this development should discharge to the ground via infiltration system where feasible.

A detailed evidence based feasibility assessment must be carried out in line with Chapter 25 of the CIRIA SuDS Manual 2015 to determine whether infiltration is a suitable method of surface water disposal.

Particular attention must be paid to Ground Water Source Protection Zones to ensure that the risk of pollution to these valuable resources is not compromised. Details can be obtained from the government website:

https://www.gov.uk/guidance/groundwater-source-protection-zones-spzs#find-groundwater-spzs

If your site is in a Groundwater Source Protection Zone, you should have regard to the Environment Agency's approach to Groundwater Protection. Information on this is available via the link below:

https://www.gov.uk/government/publications/groundwater-protection-position-statements

Please note that such a location could have implications for the principle of your development and the need for additional mitigating measures to protect the groundwater environment and public water supply in the detailed design of your site.

Waterbody

If an evidence based assessment has been carried out and confirms that infiltration is not feasible, surface water must discharge to a nearby watercourse as proposed. Discharge rates and consents must be discussed with all interested parties (e.g. the LLFA and/or the EA).

We would encourage you to identify and engage with any third party landowner and riparian owner to agree access and discharge rights to the water body if this is not in your ownership.

Levels

For low-lying sites, (where the ground level of the site or the level of a basement is below the ground level at the point where the drainage connects to the public sewer), care should be taken to ensure that the property is not at increased risk of flooding. If these circumstances exist, we recommend that you contact us to discuss further. It could affect the detailed design of your site and result in the need to incorporate appropriate mitigating measures in your drainage scheme.

Land drainage / Overland flows / track drainage

United Utilities have no obligation, and furthermore we do not accept land drainage, overland flows or track drainage into the pubic sewerage network under any circumstances

Existing Wastewater Assets Crossing the Site

We have reviewed our records and can confirm that there does not appear to be any charted public sewers located within the boundary of proposed development. However, due to the accuracy of the records and the public sewer transfer legislation in 2011, not all public sewers are shown on our records so we would ask that you proceed with caution and carry out your own site investigation works. If any uncharted sewers are identified while carrying out your works we would ask that you contact United Utilities at the earliest opportunity so that we can offer guidance and update our records.

Existing Water Assets Crossing the Site

It is the developer responsibility to identify utilities on-site. Where clean water assets are shown on our records, we recommend that you contact our Water Pre-Development Team, via the following email address: DeveloperServicesWater@uuplc.co.uk. Further information for this service can be found on our website via the link below:

https://www.unitedutilities.com/builders-developers/larger-developments/pre-development/water-pre-dev/

Connection Application

Although we may discuss and agree discharge points and rates in principle, please be aware that you will have to apply for a formal sewer connection. This is so that we can assess the method of construction, Health & Safety requirements and to ultimately inspect the connection when it is made. Details of the application process and the form itself can be obtained from our website by following the link below:

https://www.unitedutilities.com/builders-developers/wastewater-services/sewer-connections/sewer-connection/

We recommend that the detailed design should confirm the locations of all utilities in the area and ensure that any proposed drainage solution considers routing and clash checks where required.

If we can be of any further assistance please don't hesitate to contact us further.

Many thanks and kind regards,

Tom

United Utilities
Water for the North West

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Developer Services & Metering
Customer Services
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From: Chris Abram [mailto:chris.abram@rgparkins.com]
Sent: 22 April 2022 12:01

To: Wastewater Developer Services <WastewaterDeveloperServices@uuplc.co.uk>
Cc: Troy Melhuish <troy.melhuish@rgparkins.com>
Subject: Pre-Development Enquiry - Summergrove, Whitehaven

Good Morning,

We have been asked to undertake some feasibility studies for a foul and surface water drainage strategy for a proposed development at Summergrove in Whitehaven, Cumbria.

This proposed development is outline at this stage but is anticipated to comprise of approximately 35 dwellings.

An assumed foul flow rate of 1.62 l/s has been calculated for the proposed development on this basis using the latest Design and Construction Guidance for Foul and Surface Water Sewers.

Due to the topography, the majority of the site is sloping away from the nearest public sewer, it is therefore anticipated foul flows will be required to be pumped from the low points of the site back up towards the road to a new manhole chamber located at a higher level within the site boundary which will then connect to the existing 150mm diameter public sewer located in Summergrove Park Road via. gravity. The development drainage system will remain private.

I have attached a draft outline drainage plan Drawing No. K39183-SK01 showing the indicative outfall route and proposed connection point of the foul drainage for reference.

It is anticipated that the surface water system will remain private, and it is therefore proposed that surface water will be dealt with on site using a series of SuDS features before controlled release to existing open watercourses located near the site.

Please could you advise whether the above foul connection proposal is acceptable in principle to UU?

Best regards,

Chris Abram BEng(Hons) GMICE
Engineer

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