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Flood Risk Assessment & Drainage Strategy

Jefferson Park, Whitehaven – Phase 2

Thomas Armstrong and Home Group

Ref: K38379.DS/001

Version	Date	Prepared By	Checked By	Approved By
Original	26th August 2021	C. Abram	T. Melhuish	T. Melhuish

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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 Thomas Armstrong on behalf of Home Group in support of their proposal to construct 14 new properties within the existing residential development at Jefferson Park, Whitehaven.

RGP has been appointed to undertake a Flood Risk Assessment and Outline 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 in terms of number of properties (14 No.), it is classed as major development (over 10 dwellings) in accordance with The Town and Country Planning Order 2015 [3].

The area covered by the application is 0.34 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.

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 site is accessible directly from Low Road (B5345) and is located within the curtailment of the existing residential development of Jefferson Park (Figure 8.1), situated approx. 1.5km south of Whitehaven town centre, at National Grid Co-ordinates 297400E 516780N.

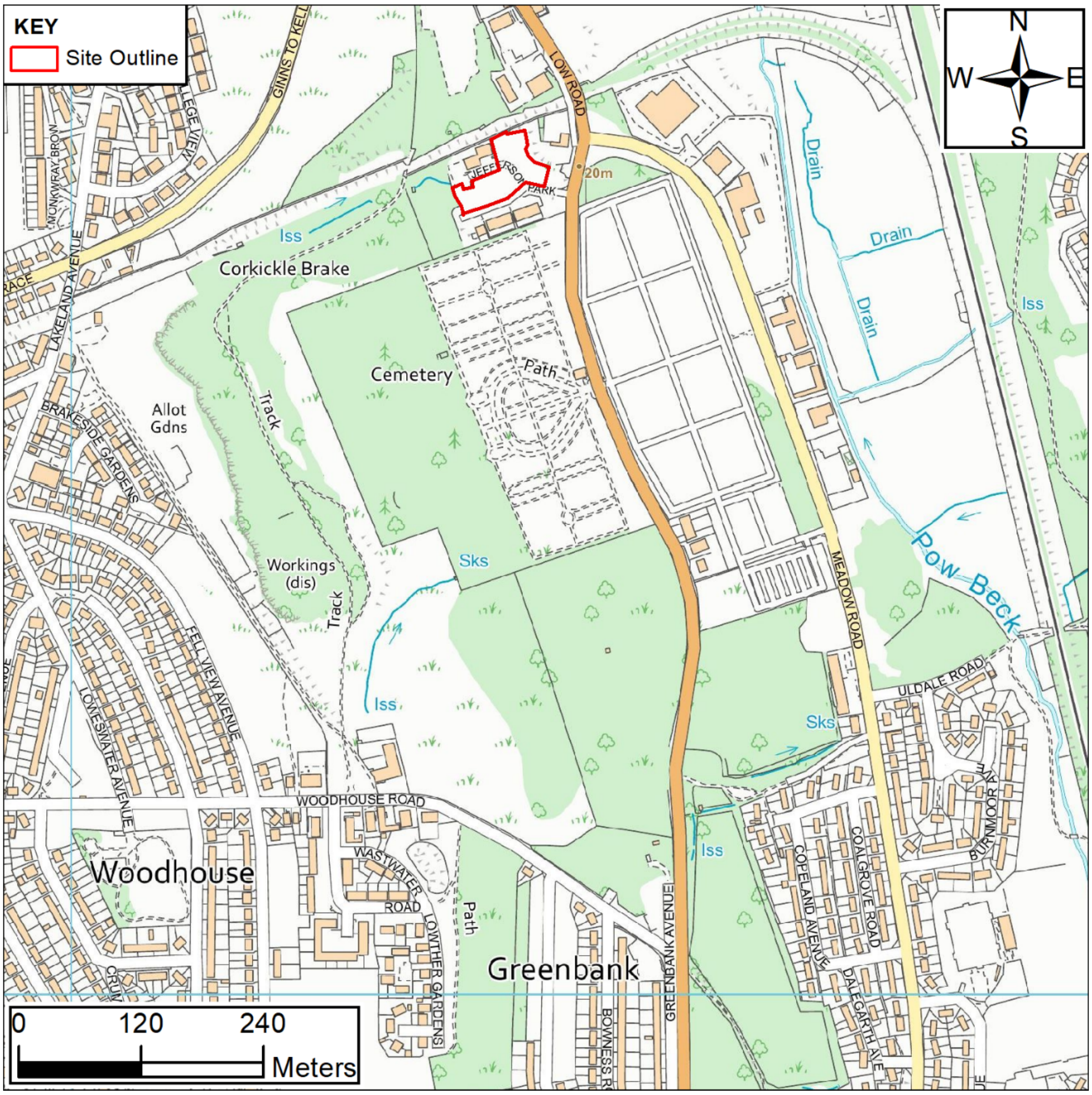

Figure 8.1 Site Location

8.2 SITE DESCRIPTION

The area of the site proposed for development covers approximately 0.34 ha. As the site already consists of existing dwellings and site access roads, it is classed as a mixture of both Greenfield and Brownfield.

Historically the site is known to have been used for a variety of industrial purposes since the late 1800's with uses including a brick field excavation, laundry and refuse tip evident within different areas of the Jefferson Park site boundary shown at various time periods on historical maps up to approximately 1994. Thereafter the site was cleared of all structures in preparation for future development.

At present, the existing housing development comprises of 24 properties including houses and apartment blocks constructed circa. 2009-2010. The site access road is already in place to service the existing properties, with the areas outlined for new development currently forming an informal greenspace between the existing dwellings on the site in areas left from the initial phase of the residential development. It is bounded by a Cemetery to the south, the long since abandoned former Corkickle Brake railway line to the north with wooded areas to the west and Low Road and a few private dwellings directly to the east.

Topographically levels vary significantly, with the survey indicating the site is located on a relatively steep hill with levels typically falling from west to east towards Low Road.

Levels in the developable area range from approximately 32.00 mAOD at the western extent to around 23.5 mAOD in the east where they then steeply drop off again to around 19.00 mAOD towards the boundary with the existing private dwellings facing Low Road.

Access to the site is via a junction with Low Road.

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 4 km southeast of the site. The development site overlies a secondary aquifer with 'Medium-High' vulnerability.

Table 8.1 Site Geological Summary

Geological Unit	Classification	Description	Aquifer Classification
Soil	Soilscape 6	Freely draining slightly acid loamy soils	N/A
Drift	Till, Devensian-Diamicton	Clay, silt sand and gravels	Secondary (undifferentiated)
Solid	Pennine Middle Coal Measures Formation	Mudstone, Siltstone, Sandstone	Secondary A

8.4 HYDROLOGY

The closest surface water feature is Pow beck located approximately 20m to the east of the development site. This is classed as a 'main' river by the EA.

There is an existing culverted ordinary watercourse running through the site. This was diverted along the northern site boundary as part of the initial phase of the Jefferson Park development and continues northeast before eventually discharging to Pow Beck.

8.5 EXISTING SEWERS

Reference to United Utilities Sewer Records indicate the presence of an adopted 150mm and 225mm diameter foul sewer servicing the existing Jefferson Park development, which discharges into a combined sewer via. a manhole connection located at the junction between Low Road and Meadow Road.

Similarly, the existing UU records indicate a surface water system servicing the existing development residences is evident, this is understood to be private and maintained by Home Group.

To verify the size and condition of the existing drainage a CCTV survey including cleaning of the pipes, was commissioned by Thomas Armstrong and undertaken by SK Drainage Solutions on the 27th May 2021.

This found the existing foul sewerage system to be in good condition and working order. Some minor defects were identified in the existing surface water system, including partial blockage of a trash screen at the point where the watercourse diversion enters the site. This will need clearing to ensure functionality and a regular maintenance schedule is recommended to mitigate against future blockages.

One major defect in the form of a deformed pipe was identified at the head of the surface water system in the southwestern site road area. Although the location of this pipe identified as requiring remedial attention is outside the proposed developable area and scope of this report, it is recommended that these repairs are incorporated as part of the construction phase of the development to minimise future disturbance.

RGP have independently conducted recent investigations (February 2020) including CCTV surveys on the sewers near the site location within Low Road. This confirmed that the private drainage system comprised of a 900mm and 600mm diameter attenuation pipe system designed by another engineer specifically for the original development. As such, RGP do not consider this existing attenuation system a suitable discharge point for the next phase of development due to a lack of storage capacity. It Is therefore concluded that surface water run-off from any new development will connect independently to the existing culverted watercourse located within the site.

These independent investigations also confirmed the presence of this culverted watercourse running in a north easterly direction from Jefferson Park under Low Road (at a depth of approx. 5m). Dye testing confirmed that this culvert runs freely and outfalls downstream to Pow Beck.

Pre-development correspondence with the local Copeland Borough Council Drainage Engineer has identified that an existing field drain from Whitehaven cemetery to the south is connected to the existing drainage within the site. Historic council investigations have found this connection to be in poor condition causing restriction to the flows resulting in surcharges and overland flooding upstream to occur. It is therefore recommended that full investigations to determine the condition of the connection are undertaken and repairs conducted where necessary as part of this second phase of development.

8.6 GROUND INVESTIGATION

Numerous ground investigations have been undertaken at the site in line with the phased nature of the overall development.

Sub Surface North West Ltd conducted the original ground investigations for the initial phase of the Jefferson Park development in mid-2007.

More recently a Phase 1 and Phase 2 geo-environmental site investigation was carried out by E3P in January 2015 when the second phase of the Jefferson Park development was being explored.

Intrusive ground investigations comprising of 13 No. trial pits, 3 No. cable percussive probeholes and 6 No. window sample probeholes, completed as environmental monitoring stations were undertaken.

Made ground deposit comprising of sandy / gravelly clay of brick, ash, concrete, clinker and timber fragments underlain by a clayey sandy gravel of mixed lithology to a variable proven depth of 6.9m below ground level was encountered in the northeast quadrant of the site.

Drift deposits comprising of natural deposits of firm to very stiff gravelly and/or sandy Clay with cobbles of sub-rounded sandstone to a depth of 10.1m bgl were proven.

Below this depth a solid geology of Coal was encountered.

Groundwater was not encountered in any shallow trial pits and was only encountered in two borehole locations at depths of 7.5m and 6.9m below ground level.

With respect to potential drainage via. infiltration. The report states that "It is considered the predominantly cohesive soil matrix underlying the made ground is unlikely to provide a high degree of soakage potential for drainage systems in this instance".

Also included was a recommendation for further site investigation in areas identified as potential shallow mine areas linked to the abandoned coal mines in the wider area.

E3P subsequently revisited the site in June 2021 to undertake a coal mining risk assessment comprising of 3 No. deep rotary boreholes.

The risk assessment confirms the presence of coal and historical mining activity/voids under the proposed site area with recommendations for proof drilling grids to be undertaken under proposed construction areas with pressure grouting and stabilisation of any workings identified to ensure future stability (subject to the Coal Authority's specification for consolidation of abandoned mine workings).

For further information and full details refer to E3P ground investigation report references:

- 10-365-L1 - Coal Mining Risk Assessment (Issued 19th July 2021)
- 10-365-R1 – Phase 1 & Phase 2 Geo-environmental Site Investigation (Issued January 2015)

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 Error! Reference source not found. 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%

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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 Final Draft Report Level 1 Strategic Flood Risk Assessment (SFRA) Error! Reference source not found.[7] in 2018 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.

However, since the original SFRA's were undertaken, the understanding of flood mechanisms and risks has developed, and the EA has carried out new modelling to define flood risks. The SFRA maps are therefore regarded as superseded by the new EA Flood Maps for Planning in any case which have been used to determine the sites flood risk for the basis of this report.

9.5 ENVIRONMENT AGENCY FLOOD MAP FOR PLANNING

Figure 9.2 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.

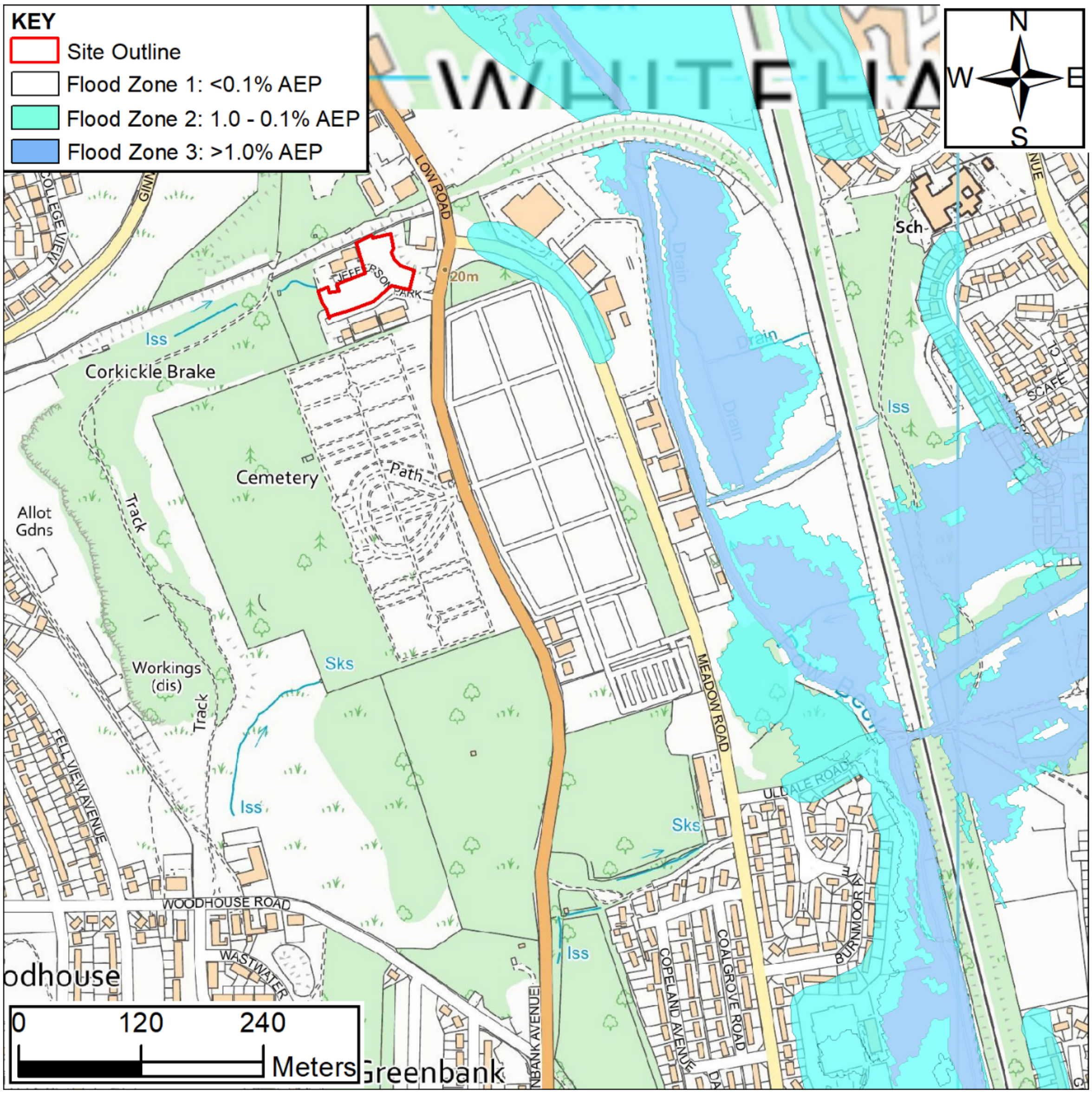

Figure 9.1 Environment Agency Flood Map for Planning

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 area proposed for development and the overall Jefferson Park site is predominantly at 'Very Low' risk of surface water flooding with the risk of flooding being less than 0.1% AEP (1 in 1000 year). There are however some 'Low' risk areas shown on the existing site road, this could be due to the run-off from the steep sloping topography above the west of the site over an area believed to be impervious soil.

Whilst not shown as extending into the site there is a small area of 'Medium' risk probability of surface water flooding shown at the northern end of the western site boundary along the line of an existing watercourse which then enters into a culvert that is routed through the site along the northern boundary before heading northeast under Low Road towards Pow Beck, this culvert is not shown on the EA surface water maps, however no incidences of surface water flooding relating to this area have been reported by residents of Jefferson Park and the development is therefore not considered to be at risk of flooding via. this method.

Outside the site boundary an area of 'High Probability' flooding is shown adjacent to Low Road downstream of the site along the line of the existing culverted watercourse. Pre-development correspondence with Copeland Borough District Council's Drainage Engineer confirms that the area shown as 'High' risk dates to an incident recorded in 2009 prior to upgrade works on the culvert being undertaken as part of the initial phase of the Jefferson Park development. These works have resolved this surface water flooding issue and as no incidents have been recorded since, the surface water flood map is therefore considered out of date for the purpose of this report and any future connection to this culvert is not considered to increase the risk of flooding downstream.

However, as surface water run-off from the site is currently directed towards Low Road due to the sloping topography, 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.

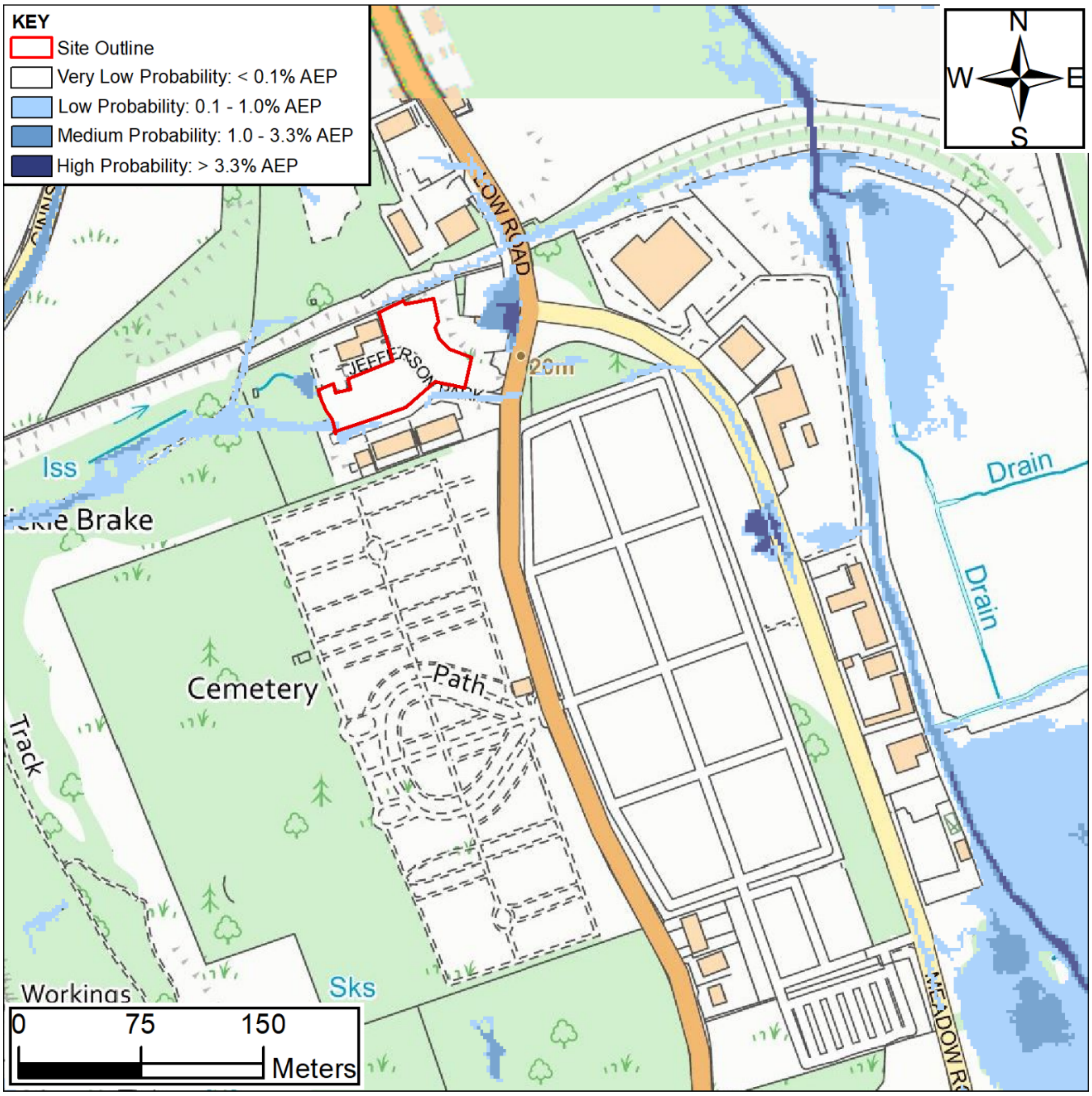

Figure 9.2 Environment Agency Surface Water Flood Map

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.

British Geological Survey (BGS) records (Figure 9.3) indicate the majority of the site has 'Potential for Groundwater Flooding to Occur at the Surface'. The dataset shows areas susceptible to groundwater flooding, but it does not indicate the likelihood of it occurring.

Groundwater was not encountered during any shallow trial pits conducted as part of the ground investigations (Section 8.6); with groundwater only encountered in boreholes at depths of 6.9m and 7.5m bgl respectively. However, it is likely that groundwater levels will fluctuate, but given the site's raised position it is unlikely to be significantly affected and no flooding via. this method has been reported on the existing development.

Nevertheless, no below ground development is proposed and groundwater would not pose a risk of flooding to the site.

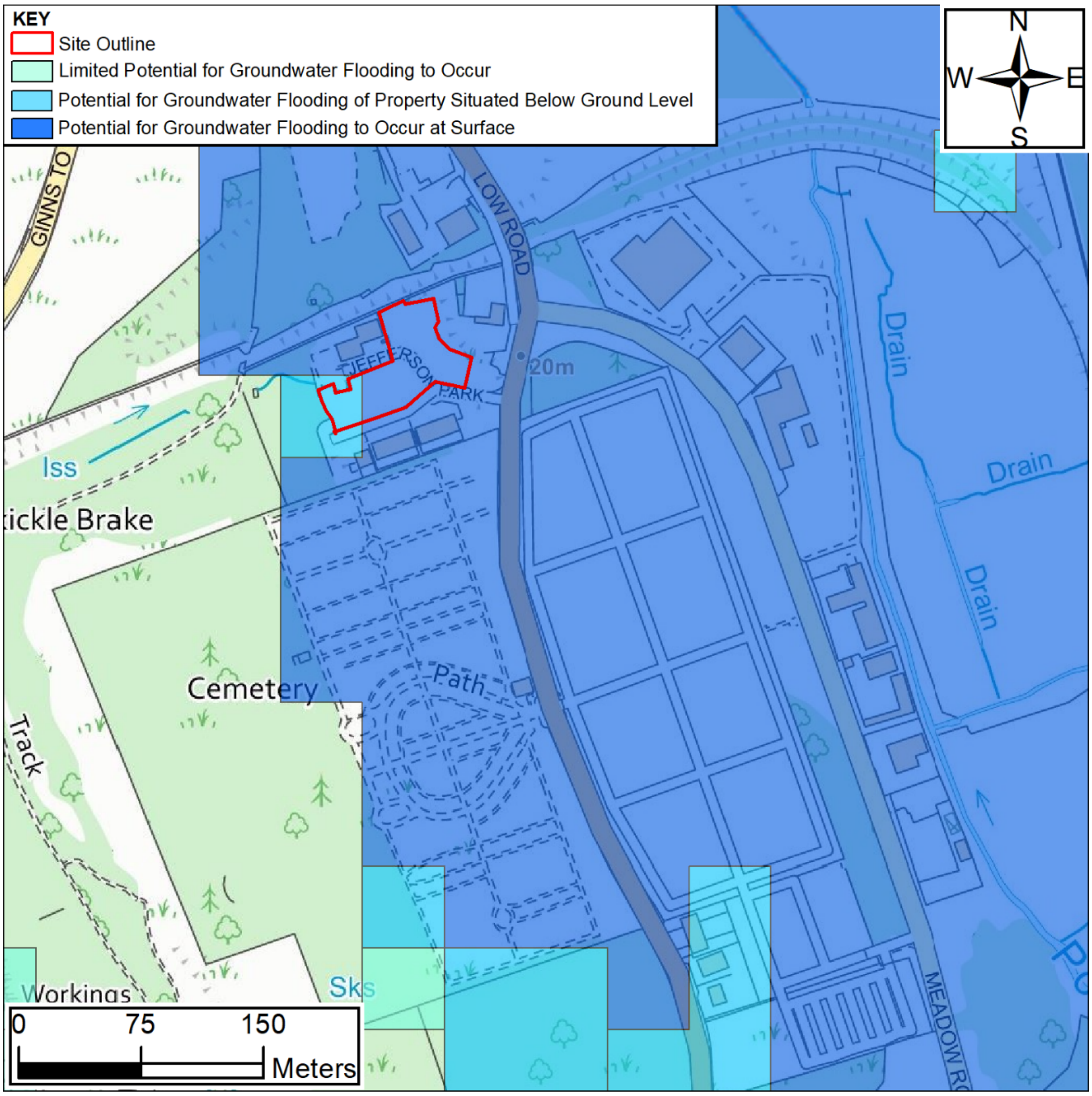

Figure 9.3 BGS Ground Water Flood Map

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. It is therefore concluded 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 [8]
- Code of Practice for Surface Water Management, BS8582:2013, November 2013[9]
- Rainfall Runoff Management for Developments, Defra/EA, SC030219, October 2013[10]
- Designing for Exceedance in Urban Drainage – Good Practice, CIRIA Report C635, 2006[11]
- Flood Estimation Handbook (FEH)[12]
- Flood Studies Report (FSR), Volume 1, Hydrological Studies, 1993[13]
- Flood Studies Supplementary Report No 14 (FSSR14), Review of Regional Growth Curves, 1983[14]
- Flood Estimation for Small Catchments, Marshall & Bayliss, Institute of Hydrology, Report No. 124 (IoH 124), 1994 [15]
- Non-Statutory technical Standards for Sustainable Drainage Systems, Defra, March 2015[16]

The following assessment and drainage strategy are based on the latest site layout plan. Any alterations to the site plan resulting in changes to impermeable areas will require the drainage strategy to be revisited.

10.2 SITE AREAS

To support the exploration of options for site drainage, the spatial extent of different types of proposed land cover on the site have been measured. Table 10.1 shows the measured proposed land cover areas. The highest percentage is green and landscaped areas at 55% of the total site area. Roof areas cover 18%, road area 4% and parking areas 24%

Table 10.1 Land Cover Areas

Land Cover	Area		Percentage of total site area
	m²	Ha	
Total Housing Roof Area	599	0.060	18%
Total Parking & Paved Areas	799	0.080	24%
Total Road Area	119	0.012	4%
Green & Landscaped Areas	1863	0.186	55%

The site can be subdivided into land cover that could be permeable and that which could be impermeable. Potential impermeable areas are regarded as housing, parking, roads, driveways and walkways. All other areas (principally public open space) are regarded as having a permeable surface. Table 10.2 gives the areas of potentially permeable and impermeable land cover, and this shows that impermeable areas could cover 45% of the site and permeable areas 55%.

Table 10.2 Area of Potentially Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	Ha	
Total Impermeable Area	1517	0.152	45%
Remaining Permeable Area	1863	0.186	55%

10.3 SURFACE WATER DRAINAGE DESIGN PARAMETERS

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

10.3.1 CLIMATE CHANGE

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

Climate change guidance is issued by the Environment Agency and outlines the anticipated changes to extreme rainfall intensity. Table 10.3 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. No properties are located immediately downstream of the site and therefore the site poses low risk to neighbouring property.

Table 10.3 Peak Rainfall Intensity Allowance in Small and Urban Catchments

(use 1961 to 1990 baseline)

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%

10.3.2 URBAN CREEP

BS 8582:2013 [9] 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) has been applied to the contributing area.

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

10.3.3 PERCENTAGE IMPERMEABILITY (PIMP)

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

10.3.4 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 0.75 for summer and 0.84 for winter and are used for design.

10.3.5 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.

10.4 PRE-DEVELOPMENT RUNOFF ASSESSMENT

As the site covers an area of less than 200 ha, (0.34 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.

Full details of the calculations and the methodology for deriving the Peak Rate of Runoff are in included in Appendix B. A summary of the results is included in Table 10.4. Without attenuation or infiltration, the proposed development would not alter the Rate of Runoff from the developed areas of the site.

Table 10.4 Pre-Development Peak Runoff Rates

Rate of Runoff (l/s)		
Event	Greenfield	Brownfield Post-Development
Q1	1.1	12.7
QBAR	1.2	18.6
Q10	1.7	25.5
Q30	2.1	31.1
Q100	2.6	39.9
Q100+ 40% CC	3.6	55.8

Without attenuation, the proposed development would 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 site before being released at a controlled rate to the existing culverted watercourse, at the pre-development Greenfield Qbar rate.

10.5 SURFACE WATER DISPOSAL

Surface water disposal has been considered in line with the hierarchy outlined in the SuDS Manual [8]. 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 (e.g. permeable paving).
- **Site Control**: Management of water in the local area (e.g. swales, detention basins).
- **Regional Control**: Management of runoff from sites (e.g. balancing ponds, wetlands).

10.5.1 INFILTRATION

Geotechnical testing indicates soil on the site is unsuitable for the disposal of surface water by infiltration. The Geo-environmental Site Assessment indicates the underlying strata is likely to be unsuitable for drainage via infiltration. For further information refer to Section 8.6.

10.5.2 POSITIVE DRAINAGE – WATERCOURSE

The impermeable area of the site associated with the new dwellings will require a positive drainage solution. Runoff will be attenuated to pre-development Greenfield Qbar rates as far as practical, with discharge proposed to the existing culverted water course located within the Jefferson Park development.

10.6 SURFACE WATER DRAINAGE DESIGN

The proposed surface water network serving the impermeable access roads and plots has been modelled using Micro Drainage Source Control (results included in Appendix B).

The drainage design has been sized to contain a future 1% AEP event of critical duration. Future climate change (40%) and urban creep (10% to housing roof areas only) is accounted for.

It is proposed that all roof and driveway areas will drain into shared private geocellular crate attenuation systems, located within private drive areas.

Roof water and driveway runoff will connect directly into the surface water pipe network upstream of the crate systems, with inspection chambers utilised to route the new pipework and allow for future inspection and maintenance. Ground levels will be required to fall consistently around the site in order to enable gravity connections to the drainage system.

Advanced silt traps will be located upstream of each attenuation tank, which will provide surface water treatment and access for maintenance. Silt traps isolate silt and other particles by encouraging settlement into removal silt buckets, preventing ingress into the tank.

The crates will be founded at a suitable level providing a minimum depth of cover of 600 mm over the top. These tanks will be wrapped and sealed with an impermeable membrane to provide a water-tight structure.

Mini flow control chambers will restrict discharge from each shared geocellular crate system. With the cumulative discharge from the site culminating in a total discharge rate of 1.2 l/s from the new plots equalling the pre-development greenfield runoff Qbar rate for the equivalent site area.

A storage assessment has been undertaken for the Q100+40% CC storm event and has been designed with sufficient capacity to convey flows without causing flooding, the results are provided in Table 10.5.

Table 10.5 Attenuation Storage Volumes

Plot No.	Tank Plan Area (m^2)	Tank Dimensions (m)	Volume to TWL (m^3)	Discharge Rate (l/s)
1 - 8	88	8 x 11 x 0.8	65.7	0.6
9 - 12	38.5	5.5 x 7 x 0.8	28	0.4
13 - 14	30	5 x 6 x 0.8	22	0.2
			TOTAL	1.2

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

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

10.7 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 designed to attenuate a 100-year design storm including a 40% allowance for climate change. 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 geocellular systems, 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 in more permeable green space areas as is currently the case, or towards the existing site highway where they would be contained and intercepted by gullies.

10.8 OPERATIONS & MAINTENANCE RESPONSIBILITY

The drainage will remain private and will therefore be maintained by Home Group. A SuDS 'Operations & Maintenance Plan' has been prepared by RGP detailing the requirements for future maintenance of the drainage system.

11. FOUL WATER DRAINAGE STRATEGY

It is proposed that foul water from the new dwellings shall be drained via gravity within the site before being connected to the nearest existing 150mm or 225mm diameter foul sewer located within the existing Jefferson Park site road, via new connections to the nearest existing UU manholes to the properties.

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 the foul water drainage proposals.

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 [17], as shown in Table 11.1 below.

Table 11.1 Peak Foul Flow Rates

Sewerage Sector Design and Construction Guidance Clause B3.1	
Peak Load Based on Number of Dwellings – 14 No. @ 4000 L/day	56,000
Peak Foul Flow Rate from Site (l/s)	0.65

The estimated predicted peak foul flow rate from the new development is 0.65 l/s.

The outline foul water drainage pipe routes are included on drawing K38379-10, included in Appendix A.

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).
- By reference to the National Planning Policy Framework [1] on Flood Risk, More Vulnerable development is acceptable within this flood zone.
- The site is not considered to be at significant risk of flooding from surface water, groundwater, reservoirs, canals, or any artificial structures.
- It is proposed that surface water drainage shall be positively drainage and attenuated, using 3 No. shared geocellular crate systems, with individual flow control devices restricting discharge to the pre-development Greenfield Qbar rate.
- Discharge from the site shall be controlled to a maximum rate of 1.2 l/s via the 3 No. orifice flow control devices, before connecting via gravity to the existing culverted watercourse running through the development site.
- In addition to these measures, a SuDS Operations and Maintenance Plan will be made available to Home Group detailing future maintenance requirements of all sustainable drainage systems.
- Foul flows from the site shall discharge by gravity to the existing public foul sewers located in the Jefferson Park site road. A pre-development enquiry has been submitted to UU.

13. REFERENCES

[1] Ministry of Housing, Communities and Local Government, National Planning Policy Framework, July 2021.

[2] Ministry of Housing, Communities and Local Government, Planning Practice Guidance to the National Planning Policy Framework, July 2018.

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

[4] British Geological Survey, 2021. Geoindex. http://mapapps2.bgs.ac.uk/geoindex/home.html

[5] Land Information System (LANDIS)- Soilscapes viewer, Accessed August 2021. http://www.landis.org.uk/soilscapes

[6] Defra Magic Maps, 2021. https://magic.defra.gov.uk/MagicMap.aspx. Accessed August 2021.

[7] Copeland Borough Council, Draft Strategic Flood Risk Assessment (SFRA), May 2018

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

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

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

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

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

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

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

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

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

[17] Water UK, Design and Construction Guidance for Foul & Surface Water Sewers Offered for Adoption Under the Code for Adoption Agreements for Water and Sewage Companies Operating Wholly or Mainly in England, Approved Version 2.0, March 2020

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

APPENDIX A

ARCHITECT'S SITE PLAN

OUTLINE FOUL AND SURFACE WATER DRAINAGE LAYOUT

Accommodation schedule	
HT1 2B3P (70m2)	14
Total	**14**
Car parking	
Residents	21
Visitors	3
Total	**24**

Revision Notes:

Rev:	Date:	Notes:
???	???	???

Legend:

Project: Jefferson Park

Drawing Title: Site Plan

Drawn By: DM

Checked By:

Scale @A1: @1:200

Date: 23/06/2021

Drawing Number: 202

Design


PROPOSED DRAINAGE KEY
Proposed foul water sewer - 100Ø uPVC
Proposed private surface water sewer - 100 / 150Ø UPVC
Proposed geocellular attenuation tanks
Proposed linear channel drain to parking/driveway areas
450Ø polypropylene inspection chamber u.n.o.
Exceedence Flows
EXISTING DRAINAGE KEY
Existing public foul water sewer
Existing private surface water sewer
Existing culverted watercourse
Rose Cottage
38 To 43
44 To 48
JEFFERSON PARK
Silt Trap
Geocellular attenuation system: 5.5m x 7m x 0.8m deep No. of crates = 154 Min 0.6m cover
Orifice Flow Control Chamber Design Flow = 0.4 l/s
New foul connection to existing FW manhole
New internal backdrop connection to existing deep manhole chamber. Max combined controlled runoff rate = 1.2 l/s
New 1500Ø surface water PCC Manhole CL ~ 26.60 IL ~ 23.20 TBC
Orifice Flow Control Chamber Design Flow = 0.2 l/s
Exceedence flows directed towards nearest existing highway gully
New foul connection to existing FW manhole
PPIC
Geocellular attenuation system: 5m x 6m x 0.8m deep No. of crates = 120 Min. 0.6m cover
Orifice Flow Control Chamber Design Flow = 0.6 l/s
Geocellular attenuation system: 8m x 11m x 0.8m deep No. of crates = 352 Min 0.6m cover
2B3P
shared
R G PARKINS
Kendal | 01539 729393 Lancaster | 01524 32548
Scale @ A1: 1:200
First Issue: 26/08/2021
Office of Origin: Kendal
Drawn by: CA
Checked by: TM
Approved: TM
Client: THOMAS ARMSTRONG
Project: JEFFERSON PARK
Project No: K38379
Drawing No: 100
Rev:
Drawing Title: Outline Foul and Surface Water Drainage Layout
BIM No:

APPENDIX B

PRE-DEVELOPMENT RUNOFF CALCULATIONS

SOURCE CONTROL DRAINAGE CALCULATIONS

R G PARKINS & PARTNERS LTD	CALCULATION		Job No.	K38379	Page	1 of 6
Meadowside	Job	Jefferson Park	Drg No.	N/A	Date	25/08/2021
Shap Road		Whitehaven	Revision	Orig	Initial	CA
KENDAL LA9 6NY	Title	Rate of Run-Off			Checked	??

DESIGN BASIS MEMORANDUM - PEAK RATE OF RUN-OFF CALCULATION

Design Brief

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

Background Information & References

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

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

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

Proposed Land Use Changes

Changes to the existing site are as follows:

Greenfield Site to Brownfield Site

Results Summary

Rate of Run-Off (l/s)			
Event	Greenfield	[redacted]	Post-Development
Q1	1.1		12.7
QBAR	1.2		18.6
Q10	1.7		25.5
Q30	2.1		31.1
Q100	2.6		39.9
Q100 + 40% CC	3.6		55.8

SITE AREAS (LAND COVER AREAS)

Existing Impermeable & Permeable Land Cover

Total Site Area: **0.338** ha **3380** m^2

Existing Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m^2	ha	
Total impermeable area	261	0.026	8%
Remaining permeable area	3119	0.312	92%

Proposed Land Cover Areas

Land Cover	Area		Percentage of total site area
	m^2	ha	
Total housing roof area	599	0.060	18%
Total parking and paved area	799	0.080	24%
Total road area	119	0.012	4%
Garden & landscaped areas	1863	0.186	55%

Proposed Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m^2	ha	
Total impermeable area	1517	0.152	45%
Remaining permeable area	1863	0.186	55%

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 = **1102** mm From UKSuds website (point data)

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

Where:			
S1	=		%
S2	=		%
S3	=		%
S4	=	**100**	%
S5	=		%
		100	%

UK Suds website provides a value of 2 based on the equivalent Host value. This seems reasonable based on ground investigation.

So, SOIL = **0.47**

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

$QBAR_{rural}$ = 0.410 m^3/s
= 410.5 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 = **1517** m^2
= 0.002 km^2
= 0.152 ha

Excluding significant open space which would remain disconnected from the positive drainage system during flood events.

$QBAR_{rural\ site}$ = 0.00125 m^3/s
= **1.25** l/s

R G PARKINS & PARTNERS LTD	CALCULATION		Job No.	K38379	Page	4 of 6
Meadowside	Job	Jefferson Park	Drg no.	N/A	Date	25/08/2021
Shap Road		Whitehaven	Revision	Orig	Initial	CA
KENDAL LA9 6NY	Title	Rate of Run-Off			Checked	??

GREENFIELD RETURN PERIOD ORDINATES

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

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

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

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

Region = **10**

Use Figure A1.1 to determine region

GREENFIELD RETURN PERIOD FLOW RATES

Return Period	Ordinate	Q (l/s)
1	0.87	1.08
2	0.93	1.16
5	1.19	1.48
10	1.38	1.72
25	1.64	2.04
30	1.7	2.12
50	1.85	2.30
100	2.08	2.59
200	2.32	2.89
500	2.73	3.40
1000	3.04	3.79

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

ESTIMATE OF BROWNFIELD RUNOFF

Total site impermeable area, A = **1517** m²

M5-60 rainfall depth **17** mm *[Flood Studies Report (NERC, 1975)]*

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

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

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

M5-15 rainfall depth = 10.0 mm

Return period ratio, Z2

M1-15	0.61
M10-15	1.22
M30-15	1.49
M100-15	1.91

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

Rainfall

	Depth (mm)	Intensity, i (mm/hr)
M1-15	6.1	24
M10-15	12.2	49
M30-15	14.9	60
M100-15	19.2	77

Peak discharge, Qp = Cv Cr i A

Where:

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

Cv = 0.95
Cr = 1.3

Peak Runoff

	l/s
Q1	12.7
Q10	25.5
Q30	31.1
Q100	39.9

R G PARKINS & PARTNERS LTD	CALCULATION		Job No.	K38379	Page	6 of 6
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KENDAL LA9 6NY	Title	Rate of Run-Off			Checked	??

ESTIMATION OF QBAR (BROWNFIELD RUNOFF RATE)

See Table 2.39 for region curve ordinates

Use FSSR2 Growth Curves to estimate Qbar

Region = **10**

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

Use Figure A1.1 to determine region

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

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

Ordinate used	Qbar l/s
10 year	18.5
30 year	18.3
100 year	19.2

Proposed Brownfield Runoff, Qbar = 18.64 l/s

Using the average Qbar derived from three ordinates.

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

Half Drain Time : 780 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	8.842	0.242	0.0	0.2	0.2	8.9	O K
30 min Summer	8.941	0.341	0.0	0.2	0.2	12.5	O K
60 min Summer	9.055	0.455	0.0	0.3	0.3	16.6	O K
120 min Summer	9.136	0.536	0.0	0.3	0.3	19.6	O K
180 min Summer	9.183	0.583	0.0	0.3	0.3	21.3	O K
240 min Summer	9.212	0.612	0.0	0.3	0.3	22.4	O K
360 min Summer	9.246	0.646	0.0	0.3	0.3	23.6	O K
480 min Summer	9.261	0.661	0.0	0.3	0.3	24.2	O K
600 min Summer	9.267	0.667	0.0	0.3	0.3	24.4	O K
720 min Summer	9.271	0.671	0.0	0.3	0.3	24.6	O K
960 min Summer	9.274	0.674	0.0	0.3	0.3	24.7	O K
1440 min Summer	9.268	0.668	0.0	0.3	0.3	24.4	O K
2160 min Summer	9.249	0.649	0.0	0.3	0.3	23.7	O K
2880 min Summer	9.225	0.625	0.0	0.3	0.3	22.8	O K
4320 min Summer	9.175	0.575	0.0	0.3	0.3	21.0	O K
5760 min Summer	9.131	0.531	0.0	0.3	0.3	19.4	O K
7200 min Summer	9.096	0.496	0.0	0.3	0.3	18.1	O K
8640 min Summer	9.066	0.466	0.0	0.3	0.3	17.1	O K
10080 min Summer	9.041	0.441	0.0	0.3	0.3	16.1	O K
15 min Winter	8.872	0.272	0.0	0.2	0.2	9.9	O K
30 min Winter	8.982	0.382	0.0	0.3	0.3	14.0	O K
60 min Winter	9.111	0.511	0.0	0.3	0.3	18.7	O K
120 min Winter	9.204	0.604	0.0	0.3	0.3	22.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	117.428	0.0	8.7	23
30 min Summer	83.122	0.0	12.1	37
60 min Summer	56.245	0.0	17.2	66
120 min Summer	34.142	0.0	20.9	126
180 min Summer	25.424	0.0	23.3	184
240 min Summer	20.592	0.0	25.2	244
360 min Summer	15.253	0.0	27.9	362
480 min Summer	12.318	0.0	30.0	480
600 min Summer	10.429	0.0	31.7	540
720 min Summer	9.099	0.0	33.0	598
960 min Summer	7.328	0.0	34.9	722
1440 min Summer	5.399	0.0	36.8	988
2160 min Summer	3.998	0.0	44.2	1408
2880 min Summer	3.240	0.0	47.7	1820
4320 min Summer	2.417	0.0	53.3	2636
5760 min Summer	1.975	0.0	58.3	3408
7200 min Summer	1.699	0.0	62.7	4184
8640 min Summer	1.509	0.0	66.8	4928
10080 min Summer	1.370	0.0	70.7	5656
15 min Winter	117.428	0.0	9.8	22
30 min Winter	83.122	0.0	13.3	37
60 min Winter	56.245	0.0	19.3	66
120 min Winter	34.142	0.0	23.4	124

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

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control Σ (l/s)	Max Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	9.221	0.621	0.0	0.5	0.5	51.9	O K
240 min Winter	9.260	0.660	0.0	0.5	0.5	55.1	O K
360 min Winter	9.310	0.710	0.0	0.5	0.5	59.3	O K
480 min Winter	9.341	0.741	0.0	0.5	0.5	62.0	O K
600 min Winter	9.361	0.761	0.0	0.5	0.5	63.6	O K
720 min Winter	9.374	0.774	0.0	0.5	0.5	64.7	O K
960 min Winter	9.385	0.785	0.0	0.5	0.5	65.6	O K
1440 min Winter	9.386	0.786	0.0	0.5	0.5	65.7	O K
2160 min Winter	9.380	0.780	0.0	0.5	0.5	65.2	O K
2880 min Winter	9.363	0.763	0.0	0.5	0.5	63.8	O K
4320 min Winter	9.315	0.715	0.0	0.5	0.5	59.8	O K
5760 min Winter	9.268	0.668	0.0	0.5	0.5	55.8	O K
7200 min Winter	9.226	0.626	0.0	0.5	0.5	52.3	O K
8640 min Winter	9.189	0.589	0.0	0.5	0.5	49.3	O K
10080 min Winter	9.157	0.557	0.0	0.4	0.4	46.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	25.424	0.0	51.1	182
240 min Winter	20.592	0.0	54.1	240
360 min Winter	15.253	0.0	58.3	356
480 min Winter	12.318	0.0	61.0	472
600 min Winter	10.429	0.0	63.0	584
720 min Winter	9.099	0.0	64.4	696
960 min Winter	7.328	0.0	66.2	912
1440 min Winter	5.399	0.0	67.1	1144
2160 min Winter	3.998	0.0	102.9	1604
2880 min Winter	3.240	0.0	109.3	2072
4320 min Winter	2.417	0.0	112.8	2944
5760 min Winter	1.975	0.0	136.8	3808
7200 min Winter	1.699	0.0	147.1	4616
8640 min Winter	1.509	0.0	156.7	5448
10080 min Winter	1.370	0.0	165.6	6256

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 297410 516791 NX 97410 16791
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.086

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.043	4	8	0.043

Model Details

Storage is Online Cover Level (m) 10.000

Cellular Storage Structure

Invert Level (m) 8.600 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	88.0	88.0	0.801	0.0	118.4
0.800	88.0	118.4			

Orifice Outflow Control

Diameter (m) 0.017 Discharge Coefficient 0.600 Invert Level (m) 8.600

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

Half Drain Time : 1176 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	8.821	0.221	0.0	0.1	0.1	6.3	O K
30 min Summer	8.912	0.312	0.0	0.1	0.1	8.9	O K
60 min Summer	9.018	0.418	0.0	0.1	0.1	11.9	O K
120 min Summer	9.097	0.497	0.0	0.1	0.1	14.2	O K
180 min Summer	9.145	0.545	0.0	0.2	0.2	15.5	O K
240 min Summer	9.178	0.578	0.0	0.2	0.2	16.5	O K
360 min Summer	9.219	0.619	0.0	0.2	0.2	17.6	O K
480 min Summer	9.244	0.644	0.0	0.2	0.2	18.4	O K
600 min Summer	9.259	0.659	0.0	0.2	0.2	18.8	O K
720 min Summer	9.267	0.667	0.0	0.2	0.2	19.0	O K
960 min Summer	9.274	0.674	0.0	0.2	0.2	19.2	O K
1440 min Summer	9.279	0.679	0.0	0.2	0.2	19.3	O K
2160 min Summer	9.277	0.677	0.0	0.2	0.2	19.3	O K
2880 min Summer	9.268	0.668	0.0	0.2	0.2	19.0	O K
4320 min Summer	9.241	0.641	0.0	0.2	0.2	18.3	O K
5760 min Summer	9.212	0.612	0.0	0.2	0.2	17.4	O K
7200 min Summer	9.187	0.587	0.0	0.2	0.2	16.7	O K
8640 min Summer	9.165	0.565	0.0	0.2	0.2	16.1	O K
10080 min Summer	9.147	0.547	0.0	0.2	0.2	15.6	O K
15 min Winter	8.848	0.248	0.0	0.1	0.1	7.1	O K
30 min Winter	8.950	0.350	0.0	0.1	0.1	10.0	O K
60 min Winter	9.069	0.469	0.0	0.1	0.1	13.4	O K
120 min Winter	9.159	0.559	0.0	0.2	0.2	15.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	117.428	0.0	5.6	23
30 min Summer	83.122	0.0	7.2	37
60 min Summer	56.245	0.0	12.0	68
120 min Summer	34.142	0.0	14.4	126
180 min Summer	25.424	0.0	15.9	186
240 min Summer	20.592	0.0	16.9	244
360 min Summer	15.253	0.0	18.2	364
480 min Summer	12.318	0.0	19.2	482
600 min Summer	10.429	0.0	19.8	602
720 min Summer	9.099	0.0	20.3	720
960 min Summer	7.328	0.0	20.9	838
1440 min Summer	5.399	0.0	21.3	1086
2160 min Summer	3.998	0.0	31.2	1492
2880 min Summer	3.240	0.0	33.5	1904
4320 min Summer	2.417	0.0	35.1	2728
5760 min Summer	1.975	0.0	41.2	3528
7200 min Summer	1.699	0.0	44.3	4328
8640 min Summer	1.509	0.0	47.2	5112
10080 min Summer	1.370	0.0	50.0	5944
15 min Winter	117.428	0.0	6.1	23
30 min Winter	83.122	0.0	7.8	37
60 min Winter	56.245	0.0	13.4	66
120 min Winter	34.142	0.0	15.9	124

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

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control Σ (l/s)	Max Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	9.257	0.657	0.0	0.3	0.3	24.0	O K
240 min Winter	9.292	0.692	0.0	0.3	0.3	25.3	O K
360 min Winter	9.333	0.733	0.0	0.3	0.3	26.8	O K
480 min Winter	9.354	0.754	0.0	0.4	0.4	27.6	O K
600 min Winter	9.364	0.764	0.0	0.4	0.4	27.9	O K
720 min Winter	9.366	0.766	0.0	0.4	0.4	28.0	O K
960 min Winter	9.365	0.765	0.0	0.4	0.4	28.0	O K
1440 min Winter	9.352	0.752	0.0	0.4	0.4	27.5	O K
2160 min Winter	9.317	0.717	0.0	0.3	0.3	26.2	O K
2880 min Winter	9.277	0.677	0.0	0.3	0.3	24.8	O K
4320 min Winter	9.198	0.598	0.0	0.3	0.3	21.9	O K
5760 min Winter	9.131	0.531	0.0	0.3	0.3	19.4	O K
7200 min Winter	9.078	0.478	0.0	0.3	0.3	17.5	O K
8640 min Winter	9.034	0.434	0.0	0.3	0.3	15.9	O K
10080 min Winter	8.998	0.398	0.0	0.3	0.3	14.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	25.424	0.0	26.1	182
240 min Winter	20.592	0.0	28.2	238
360 min Winter	15.253	0.0	31.2	354
480 min Winter	12.318	0.0	33.5	464
600 min Winter	10.429	0.0	35.3	572
720 min Winter	9.099	0.0	36.6	672
960 min Winter	7.328	0.0	38.5	758
1440 min Winter	5.399	0.0	40.3	1068
2160 min Winter	3.998	0.0	49.5	1516
2880 min Winter	3.240	0.0	53.5	1960
4320 min Winter	2.417	0.0	59.6	2812
5760 min Winter	1.975	0.0	65.3	3624
7200 min Winter	1.699	0.0	70.2	4400
8640 min Winter	1.509	0.0	74.8	5184
10080 min Winter	1.370	0.0	79.2	5944

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 297410 516791 NX 97410 16791
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.041

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.020	4	8	0.021

Model Details

Storage is Online Cover Level (m) 10.000

Cellular Storage Structure

Invert Level (m) 8.600 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	38.5	38.5	0.801	0.0	58.5
0.800	38.5	58.5			

Orifice Outflow Control

Diameter (m) 0.014 Discharge Coefficient 0.600 Invert Level (m) 8.600

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

Half Drain Time : 1213 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	8.824	0.224	0.0	0.3	0.3	18.7	O K
30 min Summer	8.915	0.315	0.0	0.3	0.3	26.4	O K
60 min Summer	9.023	0.423	0.0	0.4	0.4	35.3	O K
120 min Summer	9.103	0.503	0.0	0.4	0.4	42.1	O K
180 min Summer	9.152	0.552	0.0	0.4	0.4	46.1	O K
240 min Summer	9.185	0.585	0.0	0.5	0.5	48.9	O K
360 min Summer	9.228	0.628	0.0	0.5	0.5	52.5	O K
480 min Summer	9.254	0.654	0.0	0.5	0.5	54.7	O K
600 min Summer	9.270	0.670	0.0	0.5	0.5	56.0	O K
720 min Summer	9.279	0.679	0.0	0.5	0.5	56.7	O K
960 min Summer	9.286	0.686	0.0	0.5	0.5	57.4	O K
1440 min Summer	9.292	0.692	0.0	0.5	0.5	57.8	O K
2160 min Summer	9.292	0.692	0.0	0.5	0.5	57.8	O K
2880 min Summer	9.284	0.684	0.0	0.5	0.5	57.2	O K
4320 min Summer	9.257	0.657	0.0	0.5	0.5	54.9	O K
5760 min Summer	9.229	0.629	0.0	0.5	0.5	52.6	O K
7200 min Summer	9.205	0.605	0.0	0.5	0.5	50.6	O K
8640 min Summer	9.183	0.583	0.0	0.5	0.5	48.8	O K
10080 min Summer	9.165	0.565	0.0	0.4	0.4	47.2	O K
15 min Winter	8.851	0.251	0.0	0.3	0.3	21.0	O K
30 min Winter	8.954	0.354	0.0	0.4	0.4	29.6	O K
60 min Winter	9.074	0.474	0.0	0.4	0.4	39.6	O K
120 min Winter	9.166	0.566	0.0	0.5	0.5	47.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	117.428	0.0	16.2	23
30 min Summer	83.122	0.0	20.9	37
60 min Summer	56.245	0.0	35.3	68
120 min Summer	34.142	0.0	42.4	126
180 min Summer	25.424	0.0	46.5	186
240 min Summer	20.592	0.0	49.4	244
360 min Summer	15.253	0.0	53.4	364
480 min Summer	12.318	0.0	56.1	482
600 min Summer	10.429	0.0	58.0	602
720 min Summer	9.099	0.0	59.4	720
960 min Summer	7.328	0.0	61.1	856
1440 min Summer	5.399	0.0	62.1	1098
2160 min Summer	3.998	0.0	92.1	1496
2880 min Summer	3.240	0.0	98.6	1908
4320 min Summer	2.417	0.0	103.0	2728
5760 min Summer	1.975	0.0	122.1	3568
7200 min Summer	1.699	0.0	131.3	4328
8640 min Summer	1.509	0.0	139.9	5184
10080 min Summer	1.370	0.0	147.9	5944
15 min Winter	117.428	0.0	17.6	23
30 min Winter	83.122	0.0	22.6	37
60 min Winter	56.245	0.0	39.4	66
120 min Winter	34.142	0.0	46.7	124

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

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Σ Max Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	9.213	0.613	0.0	0.2	0.2	17.5	O K
240 min Winter	9.251	0.651	0.0	0.2	0.2	18.6	O K
360 min Winter	9.300	0.700	0.0	0.2	0.2	19.9	O K
480 min Winter	9.330	0.730	0.0	0.2	0.2	20.8	O K
600 min Winter	9.349	0.749	0.0	0.2	0.2	21.4	O K
720 min Winter	9.361	0.761	0.0	0.2	0.2	21.7	O K
960 min Winter	9.371	0.771	0.0	0.2	0.2	22.0	O K
1440 min Winter	9.371	0.771	0.0	0.2	0.2	22.0	O K
2160 min Winter	9.363	0.763	0.0	0.2	0.2	21.8	O K
2880 min Winter	9.345	0.745	0.0	0.2	0.2	21.2	O K
4320 min Winter	9.296	0.696	0.0	0.2	0.2	19.8	O K
5760 min Winter	9.248	0.648	0.0	0.2	0.2	18.5	O K
7200 min Winter	9.206	0.606	0.0	0.2	0.2	17.3	O K
8640 min Winter	9.169	0.569	0.0	0.2	0.2	16.2	O K
10080 min Winter	9.138	0.538	0.0	0.2	0.2	15.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	25.424	0.0	17.4	182
240 min Winter	20.592	0.0	18.5	240
360 min Winter	15.253	0.0	19.9	356
480 min Winter	12.318	0.0	20.9	472
600 min Winter	10.429	0.0	21.6	584
720 min Winter	9.099	0.0	22.1	696
960 min Winter	7.328	0.0	22.7	908
1440 min Winter	5.399	0.0	23.0	1142
2160 min Winter	3.998	0.0	34.9	1604
2880 min Winter	3.240	0.0	37.1	2052
4320 min Winter	2.417	0.0	38.5	2944
5760 min Winter	1.975	0.0	46.2	3808
7200 min Winter	1.699	0.0	49.6	4616
8640 min Winter	1.509	0.0	52.9	5448
10080 min Winter	1.370	0.0	55.9	6248

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 297410 516791 NX 97410 16791
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.029

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	0.014	4	8	0.015

Model Details

Storage is Online Cover Level (m) 10.000

Cellular Storage Structure

Invert Level (m) 8.600 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m^2)	Inf. Area (m^2)	Depth (m)	Area (m^2)	Inf. Area (m^2)
0.000	30.0	30.0	0.801	0.0	47.6
0.800	30.0	47.6			

Orifice Outflow Control

Diameter (m) 0.010 Discharge Coefficient 0.600 Invert Level (m) 8.600