R. G. PARKINS & PARTNERS LTD
CONSULTING CIVIL & STRUCTURAL ENGINEERS

FLOOD RISK ASSESSMENT AND DRAINAGE STRATEGY

BETTY BARN
WADDINGTON
LANCASHIRE

Revision	Date	Purpose
A	17/04/2019	Planning

Our ref: K35392/01/FRA/RH

R. G. PARKINS & PARTNERS LTD
CONSULTING CIVIL & STRUCTURAL ENGINEERS

DOCUMENT CONTROL

Project:	Betty Barn, Waddington
Project Number:	K35392
Project Description:	Flood Risk Assessment and Drainage Strategy

Issue	Date	Prepared by	Checked by	Approved by
Original	27/03/2019	Rachel Heron MSc	Oliver Sugden MEng	Oliver Sugden MEng
A	17/04/2019 Site Layout Amendments	Rachel Heron MSc	Oliver Sugden MEng	Oliver Sugden MEng

Disclaimer

This report was produced by R. G. Parkins & Partners Ltd for The Trustees of the Colthurst Estate for the specific purpose of providing a Flood Risk Assessment and Drainage Strategy for the Proposed Redevelopment of Betty Barn, Waddington, Lancashire.

This report is for the sole use of The Trustees of the Colthurst Estate. R. G. Parkins & Partners Ltd will not be held responsible for any actions taken or decisions made by any third party as a result of this report.

R. G. Parkins & Partners Ltd,
Meadowside,
Shap Road,
Kendal,
Cumbria,
LA9 6NY
Reg No. 4107150

Tel: 01539 729 393
Email: mail@rgparkins.com
Web: http://www.rgparkins.com/

CONTENTS

FIGURES

TABLES

APPENDICES

GLOSSARY OF TERMS

Term	Description
AEP	Annual Exceedance Probability
AOD	Above Ordnance Datum
BGL	Below Ground Level
BGS	British Geological Survey
CC	Climate Change
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	R G Parkins and Partners Ltd
SFRA	Strategic Flood Risk Assessment
SuDS	Sustainable Drainage System
UU	United Utilities

1.0 INTRODUCTION

1.1 Background

This following report has been prepared by R. G. Parkins & Partners Ltd (RGP) for The Trustees of the Colthurst Estate in support of proposals for the conversion of Betty Barn to a single dwelling, with associated parking and landscaped areas at Waddington, Lancashire.

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 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.

1.2 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.”

Due to the development being residential and smaller than 0.5 ha, the development is classed as minor development in accordance with The Town and Country Planning Order 2015 [3].

1.3 The Development in the Context of Planning Policy

The area covered by the application is 0.2 ha (hectares) and by reference to the Environment Agency Flood Map, the site lies in Flood Zone 1. The latest site layout plan by John Coward Architects (drawing number 18102-02A) is included in Appendix A for reference.

Table 2 of the NPPF’s Planning Practice Guidance [1] classifies each development into a vulnerability class, depending on the type of development, which are outlined in Table 1.1. As a residential dwelling, the site is classified as ‘more vulnerable’. ‘More Vulnerable’ development classes are deemed acceptable in terms of flood risk within Flood Zone 1.

Table 1.1 Flood Risk 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.

2.0 SITE CHARACTERISATION

2.1 Site Location

The proposed site is located approximately 650m north west of Waddington village centre off Slaidburn Road (B6478) at National Grid Co Ordinates 372371E 444378N. The sites location is shown in Figure 2.1.

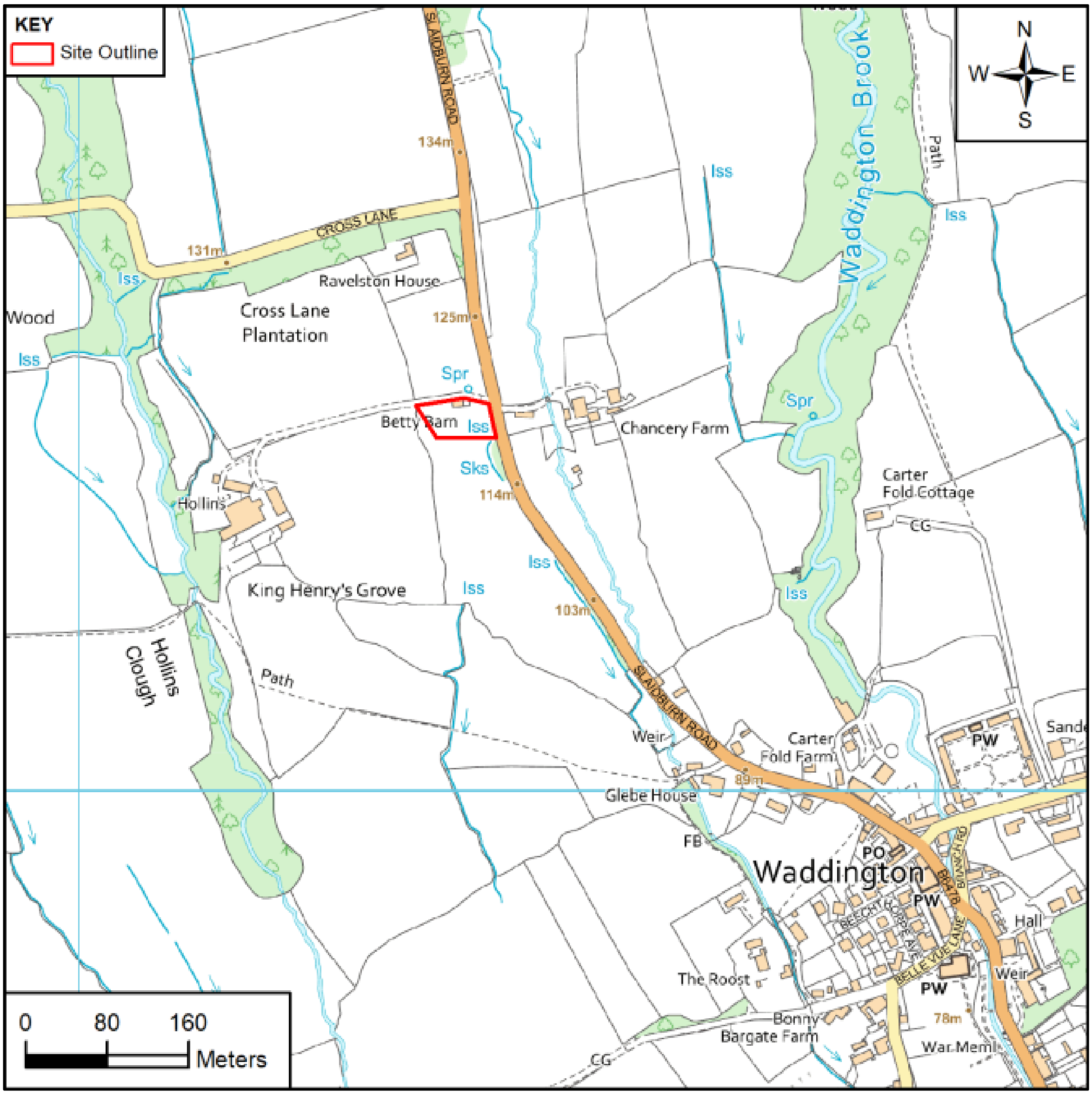


Figure 2.1 Site Location

2.2 Site Description

The site is predominately greenfield and covers approximately 0.2 ha (2000 m²). Betty Barn is a disused cattle barn, located on the northern extent of the site, within a 2.48 ha field. The site is bounded to the north by a gravel access track, agricultural land lies further north, to the south and west. Slaidburn Road lies to the east.

Topographically, the site is relatively flat with levels across the site at approximately 118.00-119.60mAOD. However, ground levels rise to the north-west corner towards the access road.

2.3 Development Proposals

The latest development proposals involve the conversion of Betty Barn to a single dwelling with related external works including landscaping and alterations to the access.

2.4 Geology & Hydrogeology

British Geological Survey (BGS) [4] and Land Information Systems (LandIS) [5] mapping indicates the site is underlain by the geological sequences outlined in Table 2.1. The EA Groundwater Vulnerability Map [6] indicates the nearest Groundwater Source Protection Zone is a Zone 1 which is situated approximately 3.0km north of the site. The development site overlies a minor aquifer with 'Low' vulnerability.

Table 2.1 Site Geological Summary

Geological Unit	Classification	Description	Aquifer Classification
Soil	Soilscape 17	Slowly permeable seasonally wet acid loamy and clayey soils	N/A
Drift	Till, Devensian-Diamicton	Clay, silt, sand and gravel	**Summary:** Secondary Undifferentiated
Solid	Clitheroe Limestone Formation & Hodder Mudstone Formation	Mudstone	**Summary:** Secondary A

2.5 Existing Watercourses

There are numerous minor watercourses in the vicinity of the site. The nearest watercourse/drainage ditch forms the west boundary of the site. It connects into a larger watercourse south of the site, ultimately discharging into the River Ribble.

2.6 Existing Sewers

Reference to the United Utilities sewer records (included in Appendix B) indicates that the nearest public sewer is a 150 mm diameter combined, which lies approximately 185 m south of the site. A manhole lies within the field the barn currently sits in, however it falls outside of the site boundary. Distance to the sewer is greater than the 30m mandatory distance per plot to require connection.

2.7 Ground Conditions

Two trial pits were dug within the site on 27th February 2019 with TP1 south west of the barn and TP2 located south east of the existing barn. The soil in TP1 can be described as light brown sandy gravelly clay overlaying dark brown firm boulder clay. In TP2 the soil was characterised as light brown firm stoney clay. Infiltration testing was undertaken at depths of 1.4 m and 0.8 m in TP1 and TP2 respectively.

TP01 failed to drain with no discernible decrease in water level over a one hour period. There was a decrease in water level in TP02, however after 2 hours, this also failed to drain and both tests were subsequently abandoned and the soil can be described as very impermeable. There was also seepage of groundwater in the deeper trial hole.

It Is therefore concluded that infiltration drainage is not viable at the site.

Figure 2.2 Percolation Testing

3.0 ASSESSMENT OF FLOOD RISK

3.1 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 Lead Local Flood Authority that the existing flood risk or the flood risk associated with the proposed development can be satisfactorily managed.

3.2 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 3.1 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 3.1 Flood Return Periods and 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%

3.3 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
- Development layout plan provided by John Coward Architects (Appendix A)
- Topographic survey provided by D2R Survey

3.4 Environment Agency Flood Map for Planning

The Environment Agency Flood Map for Planning [6] (Figure 3.1) 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 flood 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.

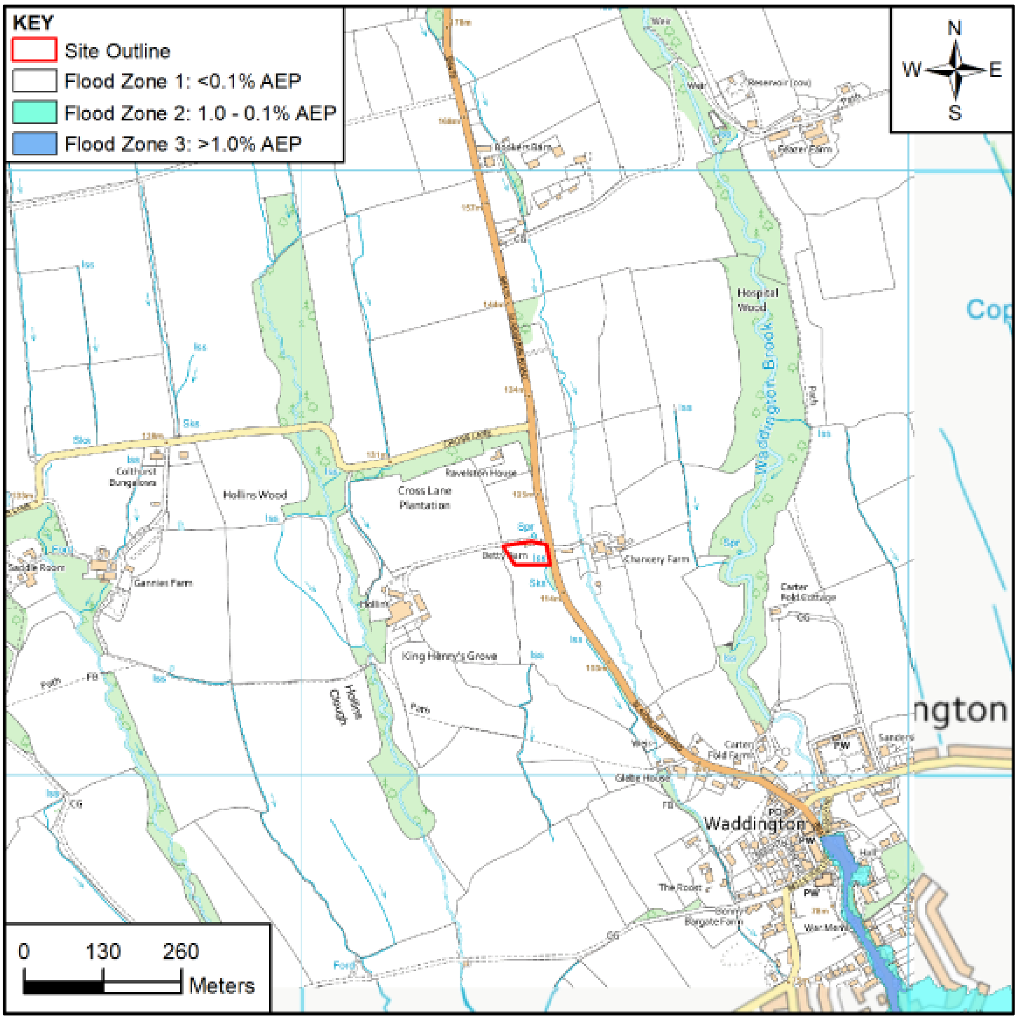


Figure 3.1 Environment Agency Flood Map for Planning

Reference to Figure 3.1 indicates the site lies within Flood Zone 1 and is considered to be at ‘Very Low’ risk of flooding from fluvial sources with an AEP of less than 0.1% (1 in 1000 year).

The watercourses in the vicinity of the site are classed as 'minor' watercourses and as such are regulated by the LLFA (Lancashire County Council) and are therefore not represented by the EA mapping (Figure 3.1), which only identifies flood risk from main rivers.

3.5 Surface Water Flood Risk

The EA have mapped areas prone to surface water flooding based on historic flooding information received from the Lead Local Flood Authority and modelling based on a LiDAR / IfSAR digital terrain model, Ordnance Survey information on urban areas and a direct rainfall approach using Flood Estimation Handbook (FEH) methodology. The critical (worst case) of the 1, 3 and 6-hour storm durations have been mapped with no areal reduction factor applied. No allowance is made for climate change, the mapping therefore indicates the current predicted flood risk.

The maps do not account for culverts / underground drainage and due to digital terrain model resolution may also underestimate or omit small drainage channels / ditches. Figure 3.2 shows the resulting predicted flood risk from surface water.

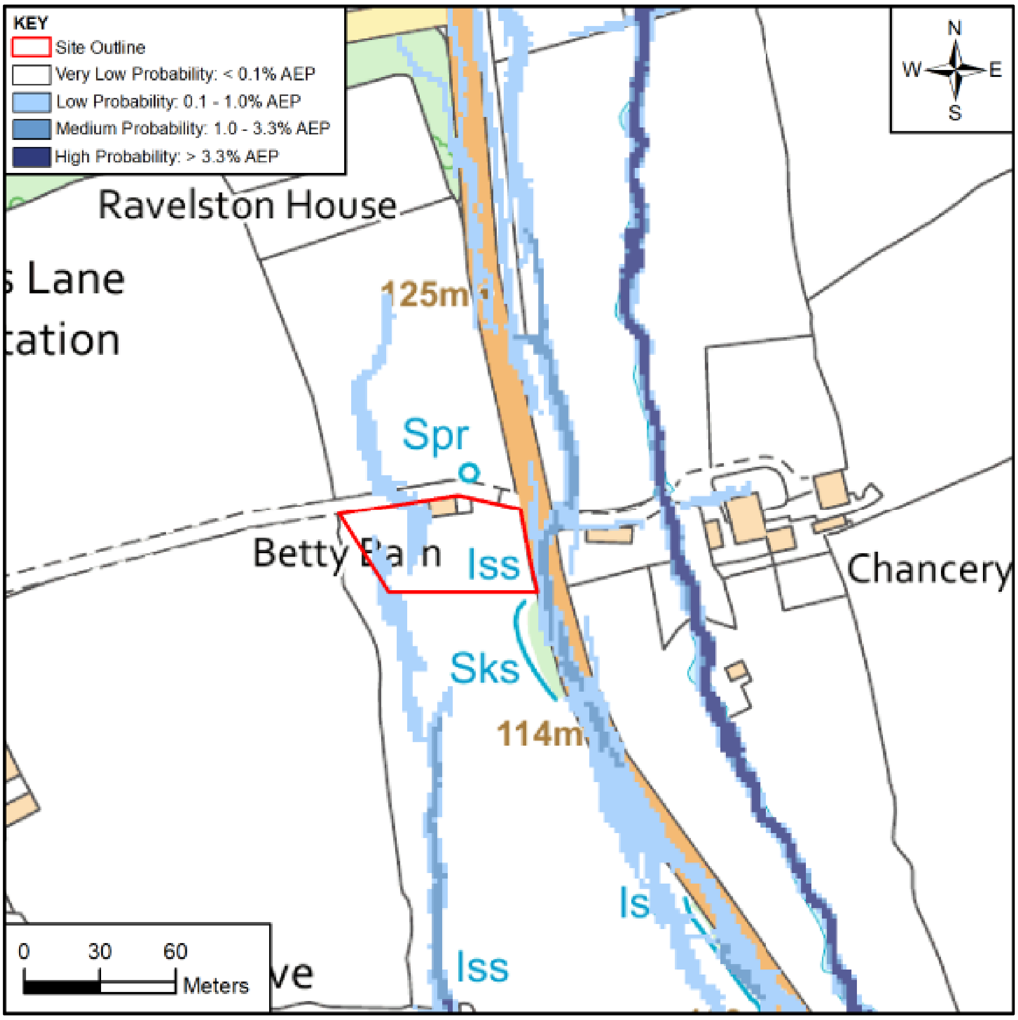


Figure 3.2 Environment Agency Surface Water Flood Map

The EA surface water flood map predicts a flow route through the site during a 0.1% AEP event (1 in 1000 year) immediately west of the barn, however, this is considered ‘Low’ risk and is associated with the topographic gradients within the site. The flow route connects into the drainage channel at the southern extent of the site c. 175 m south of the site. Further analysis of the EA mapping indicates the associated flood depth is predicted to be no greater than 300 mm, due to the velocity of runoff and gradient of the slope.

No surface water flooding is shown for the higher probability events (3.3% AEP and 1.0% AEP). The development proposals are for a barn conversion and as such it may not be practical to raise finished floor levels significantly above existing ground levels. Threshold drainage should therefore be used as an alternative mitigation measure.

3.6 Groundwater Flood Risk

The British Geological Survey (BGS) has calculated groundwater flooding susceptibility across Britain. There are several mechanisms that increase the risk of groundwater flooding, including prolonged rainfall, high in-bank river levels and artificial structures. Figure 3.3 indicates the site has the potential for groundwater flooding to occur at the surface.

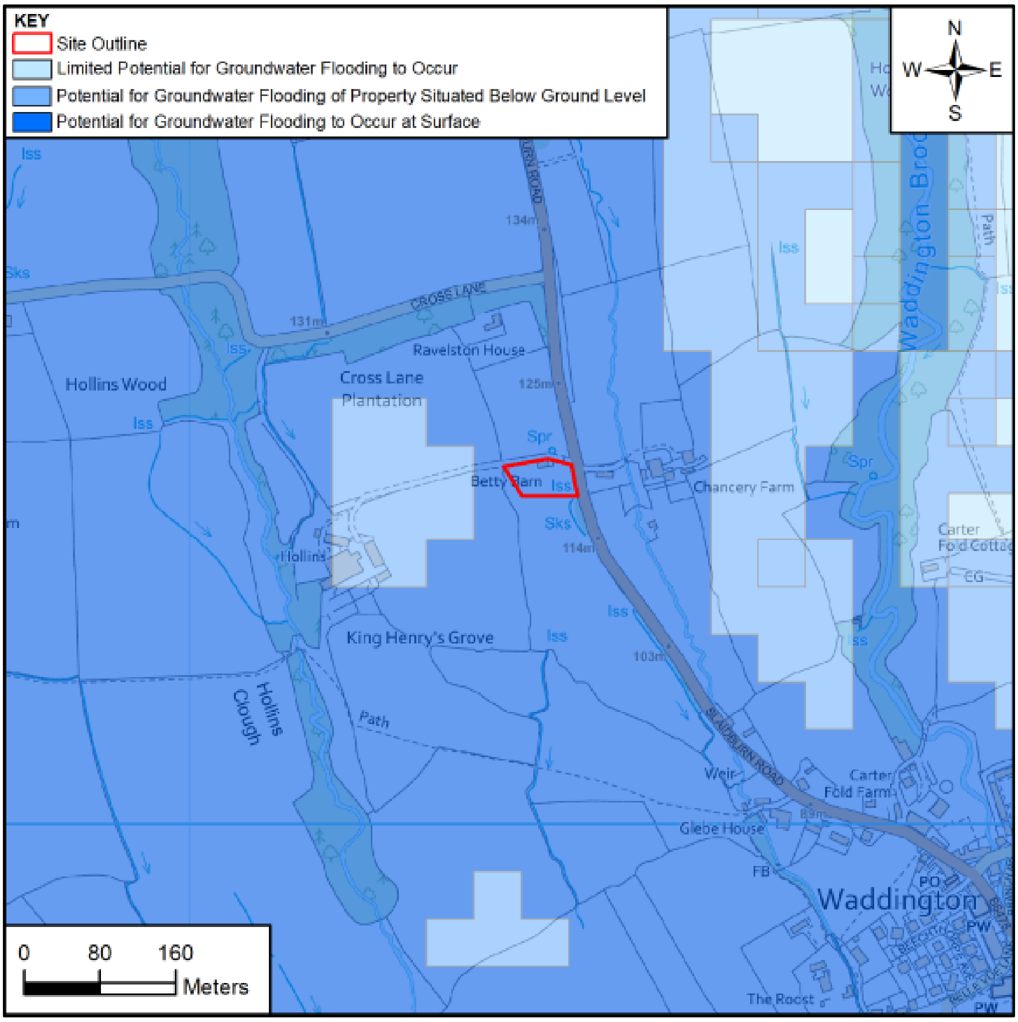


Figure 3.3 BGS Groundwater Susceptibility Flood Map

The bedrock geology is comprised of mudstone, a Secondary A aquifer, a permeable layer capable of supporting water supply. The mudstone is overlaid by Till Devensian, which has been proven by the preliminary ground investigations to be impermeable. The risk is likely to be influenced by the hydraulic continuity between groundwater and the watercourses/drainage ditches in the vicinity of the site.

3.7 Flooding from Reservoirs, Canals or Other Artificial Sources

The Ordnance Survey map indicates that there are no reservoirs [6], canals or artificial structures in the proximity of the site.

3.8 Flooding from Sewers

United Utilities (UU) do not provide information on flood risk from their assets. It is known a 150mm diameter combined sewer is located c. 185 m south of the site. However, the level and distance of this sewer is sufficiently lower and far enough away to not affect the proposed barn conversion.

4.0 FLOOD MITIGATION

4.1 Summary of Flood Risk

The risk of flooding from rivers, sewers, surface water, artificial sources and reservoirs is considered to be low. However, flooding from groundwater could cause issues on site and therefore it is recommended that mitigation measures are put in place to manage these issues. A summary of the predicted flood risk is outlined in Table 4.1.

Table 4.1 Flood Risk Summary

Source of Flood Risk	Interpreted Risk Classification	Justification
Fluvial	Low	As predicted by EA model
Surface Water	Low	As predicted by EA model
Groundwater	Medium	Indicated by BGS susceptibility maps
Artificial Sources	Extremely Unlikely	As identified by the EA
Sewer	Very Low	Engineering judgement based on sewer records

Raising finished floor level above external levels is not considered practical. Below ground development is not proposed. An alternative flood risk mitigation measure comprising threshold drainage should therefore be installed in the form of filter drains for gravel paths or channel or slot drains if paving is to be used.

4.2 Permitted Development

In terms of development within Flood Zone 1, 'more vulnerable' development is appropriate in this zone, as set out in Table 3 of NPPF Planning Practice Guidance [1], included in Table 4.2.

Table 4.2 Flood Risk Vulnerability & Flood Zones Compatibility

Flood Zones	Flood Risk Vulnerability Classification				
	Essential Infrastructure	**Highly Vulnerable**	**More Vulnerable**	**Less Vulnerable**	**Water Compatible**
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception Test Required	✓	✓	✓
Zone 3a	Exception Test Required	✗	Exception Test Required	✓	✓
Zone 3b	Exception Test Required	✗	✗	✗	✓

Key:

✓ Development is appropriate ✗ Development should not be permitted

5.0 SURFACE WATER DRAINAGE STRATEGY

5.1 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]
- Preliminary Rainfall Runoff Management for Developments, Defra/EA, W5-074/A/TR/1, Revision D [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 SuDS [17]

The following assessment and drainage strategy is based on the latest site layout plan by John Coward Architects Ltd (18102-02A) included in Appendix A.

Any alterations to the site plan resulting in changes to impermeable areas will require the drainage strategy to be revisited.

5.2 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 5.1 shows the measured proposed land cover areas. The highest percentage is garden areas at 70% of the total site area. Housing covers 6%, parking areas 6% and the access road 18%.

Table 5.1 Proposed Land Cover Areas

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Total housing roof area + 10%	121.0	0.012	6%
Total parking and paved area	125.0	0.013	6%
Total road area	360.0	0.036	18%
Garden areas	1394.0	0.139	70%

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 gardens) are regarded as having a permeable surface. Table 5.2 gives the areas of potentially permeable and impermeable land cover and this shows that impermeable areas could cover 30% of the site and permeable areas 70%.

Table 5.2 Area of Potentially Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m^2	Ha	
Total impermeable area	606.0	0.061	30%
Remaining permeable area	1394.0	0.139	70%

Currently impermeable area of the site totals 480 m^2, approximately 24% of the total site area.

5.3 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:

5.3.1 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.

In February 2016, new climate change guidance issued by the Environment Agency came into effect outlining the anticipated changes in extreme rainfall intensity.

Table 5.3 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, both the central and upper end allowances should be assessed to understand the range of impacts. A climate change allowance of 30% has been selected for the purpose of drainage design based on the 100-year anticipated design life of the proposed development. This intermediate figure has been selected for conservative design. No properties are located immediately downstream of the site and therefore the site poses low risk to neighbouring property.

Although design is undertaken considering a 30% climate change allowance, a 40% allowance has also been analysed to understand the range of impacts.

Table 5.3 Peak Rainfall Intensity Allowance in Small and Urban Catchments (using 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%

5.3.2 *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) 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 the density of the properties.

5.3.3 *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.

5.3.4 *Volumetric Runoff Coefficient, Cv*

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

For urban catchments, percentage runoff (PR) can be estimated in accordance with the Wallingford Procedure using the following equation:

$$PR = 0.829\ PIMP + 25.0\ SOIL + 0.078\ UCWI - 20.7$$

Where the Urban Catchment Wetness Index (UCWI) is a function of the 5-day antecedent precipitation index (API5) and the soil moisture deficit (SMD). UCWI can also be obtained using a best-fit graph derived from multiple catchments to correlate UCWI with Standardised Average Annual Rainfall (SAAR).

For the proposed development site:

PIMP = 100%
SOIL = 0.47
UCWI = 120 (summer), 147 (winter)
$PR_{(summer)}$ = 83, $PR_{(winter)}$ = 85

Volumetric runoff coefficient is described by the below formula:

Cv = PR / 100

$Cv_{(summer)}$ = 0.83, $Cv_{(winter)}$ = 0.85

The percentage runoff equation is known to underestimate runoff from long duration rainfall events. Winter design storms are the critical consideration for long duration events whilst summer events are likely to be critical for the shorter duration events. To fully account for this known flaw and for the purpose of conservative design, winter volumetric runoff coefficient has been uplifted to 0.925.

Therefore, for the purpose of drainage design calculations, the Cv values for summer and winter conditions have been uplifted to 0.83 and 0.925 respectively. This results in conservative design and accounts for all surfaces being in good condition with reduced infiltration as a result.

5.3.5 Rainfall Model

The calculations use the REFH2 unit hydrograph methodology in line with best practice as outlined in the SuDS manual. 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.

5.4 Surface Water Disposal

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

5.4.1 Infiltration

Infiltration testing indicates soil on the site is unsuitable for the disposal of surface water by this method. For further information refer to Section 7.

5.4.2 Positive Drainage

The entire impermeable area of the site will require a positive drainage solution. A drainage channel is located on the west boundary of the site. In line with the SuDS hierarchy for surface water disposal, discharge of surface water shall be to this drainage ditch.

5.5 Rate of Runoff Assessment

The site is less than 200 ha therefore the Greenfield calculations have been undertaken in accordance with methodology described in IoH 124 [16]. For catchments of less than 50ha the Greenfield runoff rate is scaled according to the size of the catchment in relation to a 50 ha site. Currently the site is undeveloped (greenfield) and used for grazing.

Full details of the calculations and the methodology for deriving the Peak Rate of Runoff are in included in Appendix C. A summary of the results is included in Table 5.4.

Table 5.4 Surface Water Rate of Runoff Results

Rate of Runoff (l/s)			
Event	**Greenfield**	**Existing Brownfield**	**Post Development (Proposed)**
Q1	0.5	4.8	5.9
QBAR	0.6	7.0	8.7
Q10	0.8	9.5	11.8
Q30	1.0	11.7	14.4
Q100	1.2	15.0	18.6
Q100 + 30% CC	1.6	19.6	24.2

Without attenuation or infiltration, the proposed development would increase the Rate of Runoff from the increased impermeable areas of the site. To mitigate the potential increase in runoff, a Sustainable Drainage System (SuDS) is proposed, attenuating runoff as far as is practical to the pre-development Qbar rate of 0.6 l/s.

5.6 Surface Water Drainage Design

Due to the impermeability of the soils, runoff from roof and driveway will be served by a geocellular crate system located south of the dwelling. It is recommended that a silt trap is located upstream of the inlet of the crate system to prevent the build-up of sediment to ensure the system works effectively and efficiently. Non- statutory Technical Standards [17] state runoff for new development should be attenuated to match greenfield rates if practical. Runoff rate should never exceed the existing brownfield rate. A flow control device will limit discharge to the QBAR runoff rate of 0.6 l/s with discharge to the drainage ditch west of the site.

The new road surface will comprise stone chippings to match the existing lane and concrete gutters will be installed within the road to collect and convey runoff into the drainage ditch at an unattenuated rate.

For further detail refer to the Drainage Layout Plan (K35392/A1/01) included in Appendix A.

5.6.1 *Storage Volume*

The proposed surface water network serving the impermeable areas of the site has been modelled using Microdrainage Source Control. A total storage volume of 17 m^3 is proposed which has the capacity to attenuate the critical 720-minute duration winter design storm. A geocellular crate system is the preferred method of attenuation as this will provide the necessary storage in a cost-effective, readily maintainable structure.

Further analysis has determined that an increase in rainfall of 40% due to climate change would also be contained within the geocellular crate system with freeboard reduced to 124 mm below the emergency overflow. Discharge from the surcharged flow control would increase to 0.9 l/s.

5.7 Designing for Local Drainage System Failure

In accordance with the general principles discussed in CIRIA Report C635 – Designing for Exceedance in Urban Drainage [12] the proposed surface water drainage, where practical, should be designed to ensure there is no increased risk of flooding to the buildings on the site or elsewhere as a result of extreme rainfall, lack of maintenance, blockages or other causes.

5.7.1 *Blockage*

Any overland flow from the site resulting from blockage or exceedance of the drainage system capacity would spill from the cover of the flow control device and discharge to the drainage ditch west of the development.

The dwelling would not be at risk due to the topography of the site and careful design of the access road / parking areas, falling away from property.

5.7.2 *Exceedance*

The site drainage will be designed to attenuate a 100-year design storm including a 30% allowance for climate change. The drainage system will also provide capacity for lower probability (greater design storm events) which are not critical duration. Exceedance flows shall be retained on site within the drainage system as far as practical however for storms of a greater return period it may be necessary to pass forward more flow or spill flows. In the first instance this shall be achieved by passing forward more flow to the drainage ditch via the flow control device due to surcharge of this device.

Following this and should the freeboard within the drainage system be used, flow will discharge from the flow control chamber cover into the drainage ditch with the flow route as outlined in Section 5.7.1.

The following general measures will be implemented as part of the detailed drainage design:

Surface Storage & External Levels – where possible parking areas should be designed to offer additional surface storage volume and conveyance of flood water should the attenuation system fail, flood or exceed capacity. Where appropriate, the kerb lines should be raised via half batter kerbs (0.1m) to channel surface water runoff back into the drainage system.

Overland Flow Route – The overland flow route in the highly unlikely event of exceedance / blockage/ exceedance of cellular storage would be towards existing drainage ditch west of the site.

Drainage Contingency – the proposed surface water system will be designed to provide adequate storage volume against flooding including a 30% allowance to account for climate change. A 40% allowance has also been assessed, with the additional volume stored within the freeboard.

Building Layout & Detail – the finished floor and threshold levels will be set above the external levels and wherever possible external levels will fall away from the building, ensuring that any flood water runs away from, rather than towards, the property.

5.8 Maintenance

The plot drainage is to be maintained by the property owners. A *SuDs Operations & Maintenance Plan* will be made available to the site owners detailing the requirements for future maintenance of the drainage system.

6.0 FOUL WATER DRAINAGE STRATEGY

Foul water from the site shall connect to an existing 150 mm diameter combined sewer 185 m south of the site, running west to south east of the field. It is believed the sewer crosses land within the client's ownership, the developer could form therefore a connection without the need to obtain permission / consent from third party land owners. A pre-development enquiry will be submitted to UU to ascertain if a connection is feasible.

Preliminary foul water discharge calculations have been undertaken in accordance with Sewers for Adoption 7th Edition (Table 6.1). The estimated predicted peak foul water flow rate (6 x Dry Weather Flow) from the development is 0.046 l/s.

Table 6.1 Foul Runoff Results

Sewers for Adoption 7th Edition, Clause B5.1	
Peak Load Based on Number of Dwellings – 1 No. @ 4000 L/day	4000
Total Foul Flow Rate from Site 6DWF (L/s)	0.046

Further drainage investigations are required to determine the level of the combined sewer at the proposed connection. Should the level be insufficient it may be necessary to drain part or all of the development via a pumping station, or a private packaged treatment plant.

7.0 CONCLUSIONS AND RECOMMENDATIONS

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

- The site is located within Flood Zone 1 and is at Low risk of flooding from fluvial sources. The site is also identified as being at Low risk of flooding from surface water, groundwater, sewers and artificial sources.
- By reference to the National Planning Policy Framework [1] on Flood Risk, More Vulnerable development is acceptable within these flood zones.
- Surface water drainage for the site shall be positively drained and attenuated prior to discharge at an attenuated rate to match greenfield runoff Qbar. Discharge shall be to an existing drainage ditch west of the site.
- Foul flows from the site shall be connected by gravity if levels allow to an existing 150mm combined sewer south of the site. Should levels prevent a gravity connection a foul pumping station or private packaged treatment plant will be required. It is recommended investigations are undertaken to determine the exact line and level of this sewer.
- The site layout and drainage systems will be designed to ensure that there is no increased risk of flooding on or off site as a result of extreme rainfall, lack of maintenance, blockages or other causes. The measures that will be implemented comprise additional flow conveyance capacity on the access road and car parking areas, a series of contingency and safety features for the surface water drainage system and the careful design of building layouts and details.
- In addition to the above measures, a *SuDs Operations & Maintenance Plan* will be made available to the site owners detailing future maintenance requirements of all sustainable drainage systems.

8.0 REFERENCES

[1] Ministry of Housing, Communities and Local Government, *National Planning Policy Framework*, February 2019.

[2] Ministry of Housing, Communities and Local Government, *Planning Practice Guidance to the National Planning Policy Framework*, Accessed March 2019.

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

[4] British Geological Survey (BGS), GeoIndex Onshore, Superficial Deposits and Bedrock Geology, 1: 50,000.

[5] Land Information System (LANDIS)- Soilscapes viewer, Accessed March 2019. http://www.landis.org.uk/soilscapes

[6] Environment Agency Spatial Data Catalogue, WMS Links, March 2019. http://environment.data.gov.uk/ds/catalogue/#/catalogue

[7] Defra/Environment Agency, Flood and Coastal Defence R & D Programme- Flood Risks to People Phase 2, FD2321/TR2, March 2006.

[8] Defra/Environment Agency, Flood and Coastal Defence R & D Programme- Flood Risk Assessment Guidance for New Development Phase 2, FD2320/TR2, October 2005.

[9] CIRIA, *The SuDS Manual*, Report C753, 2015.

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

[11] DEFRA/EA, *Preliminary Rainfall Runoff Management for Developments*, W5-074/A/TR/1, Revision D, September 2005.

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

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

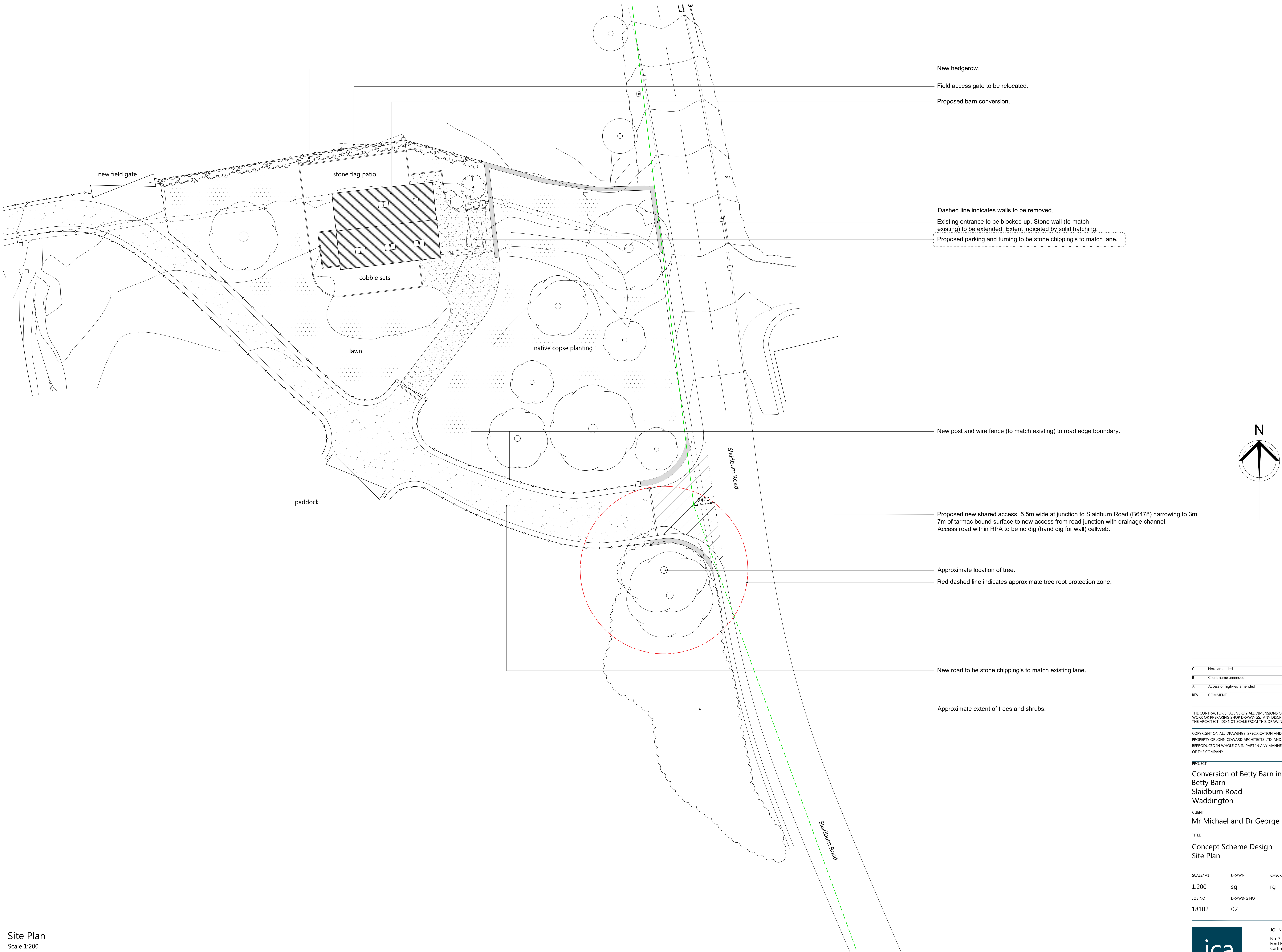
[14] Institute of Hydrology, *Flood Studies Report, Volume 1, Hydrological Studies*, 1993.

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

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

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

APPENDIX A: DEVELOPMENT PROPOSALS



Site Plan
Scale 1:200

REV	COMMENT	BY	DATE
C	Note amended	sg	16-04-19
B	Client name amended	sg	07-02-19
A	Access of highway amended	sg	09-01-19

THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS ON THE JOB BEFORE STARTING WORK OR PREPARING SHOP DRAWINGS. ANY DISCREPANCIES MUST BE REFERRED TO THE ARCHITECT. DO NOT SCALE FROM THIS DRAWING.

PROJECT

Conversion of Betty Barn into Dwelling
Betty Barn
Slaidburn Road
Waddington

CLIENT

Mr Michael and Dr George Fisher

TITLE

Concept Scheme Design
Site Plan

SCALE/ A1	DRAWN	CHECKED	DATE
1:200	sg	rg	Dec '18
JOB NO	DRAWING NO		REVISION
18102	02		C

jca architects

JOHN COWARD ARCHITECTS LIMITED

No. 3 Unsworth's Yard,
Ford Road,
Cartmel,
Cumbria, LA11 6PG

Tel : (015395) 36596

www.johncowardarchitects.co.uk

DO NOT SCALE – IF IN DOUBT ASK
USE FIGURED DIMENSIONS ONLY

Revisions	Date	Initial	Suffix

Client:
THE COLTHURST ESTATE

Project:
BETTY BARN AND TEEWOOD FARM

Drawing Title:
BETTY BARN DRAINAGE LAYOUT

R. G. PARKINS & PARTNERS LTD
CONSULTING CIVIL & STRUCTURAL ENGINEERS
Meadowside, Kendal, LA9 6NY
Tel: 01539 729393
Fax: 01539 740609
Email: mail@rgparkins.com
97 King Street, Lancaster, LA1 1RH
Tel: 01524 32548
Fax: 01524 843998
Email: office@rgparkinslancaster

Drawn by: JB	Checked by: OS	Issue Purpose:
Scale: 1:100	Issue Date: 22-03-19	INFORMATION

Drawing Number: K35392/A1/01

APPENDIX B: UNITED UTILITIES SEWER RECORDS

United Utilities Maps for SafeDig

Centre : **X :** 372433 **Y :** 444837	**Date :** 06/03/2019 11:46:43	**Scale Approx :** 8000

Extract from maps of United Utilities' Underground Assets

The position of the underground apparatus shown on this plan is approximate only and is given in accordance with the best information currently available. The actual positions may be different from those shown on the plan and private service pipes may be shown by a blue broken line. United Utilities Water will not accept liability for any damage caused by the actual position being different from those shown.

APPENDIX C: CALCULATIONS

R G PARKINS & PARTNERS LTD	CALCULATION		Job No.	K35392	Page	1 of 8
Meadowside	Job	Betty Barn	Drg no.	N/A	Date	22/03/2019
Shap Road		Waddington	Revision	Orig	Initial	RH
KENDAL LA9 6NY	Title	Rate of Run-Off			Checked	OS

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

3

- Flood Estimation Handbook (FEH)
- Flood Studies Report (FSR), Volume 1, Hydrological Studies, 1993
- Flood Studies Supplementary Report No 2 (FSSR2), The Estimation of Low Return Period Floods
- Flood Studies Supplementary Report No 14 (FSSR14), Review of Regional Growth Curves, 1983
- Planning Practice guidance of the National Planning Policy Framework, Recommended national precautionary sensitivity ranges for peak rainfall intensities, peak river flows, offshore wind speeds and wave heights.

Proposed Land Use Changes

Changes to the existing site are as follows:

Brownfield Site to Brownfield Site (Reduced Impermeable Area)

Results Summary

Rate of Run-Off (l/s)			
Event	Greenfield	Brownfield	Post-Development
Q1	0.5	4.8	5.9
QBAR	0.6	7.0	8.7
Q10	0.8	9.5	11.8
Q30	1.0	11.7	14.4
Q100	1.2	15.0	18.6
Q100 + 30% CC	1.6	19.6	24.2

R G PARKINS & PARTNERS LTD	CALCULATION		Job No.	K35392	Page	2 of 8
Meadowside	Job	Betty Barn	Drg no.	N/A	Date	22/03/2019
Shap Road		Waddington	Revision	Orig	Initial	RH
KENDAL LA9 6NY	Title	Rate of Run-Off			Checked	OS

SITE AREAS (LAND COVER AREAS)

Existing Impermeable & Permeable Land Cover

Total Site Area: **0.2** ha **2000** m²

Existing Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total impermeable area	480	0.048	24%
Remaining permeable area	1520	0.152	76%

Proposed Land Cover Areas

Land Cover	Area		Percentage of total site area
	m²	ha	
Total housing roof area + 10%	121	0.012	6%
Total parking and paved area	125	0.013	6%
Total road area	360	0.036	18%
Garden & landscaped areas	1394	0.139	70%

Proposed Impermeable & Permeable Land Cover

Land Cover	Area		Percentage of total site area
	m²	ha	
Total impermeable area	606	0.061	30%
Remaining permeable area	1394	0.139	70%

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

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

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	=	**1297**	mm	From UKSuds website (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.

$QBAR_{rural}$	=	0.497	m^3/s
	=	496.7	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	=	**595**	m^2
	=	0.001	km^2
	=	0.060	ha
$QBAR_{rural\ site}$	=	0.00059	m^3/s
	=	**0.59**	l/s

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

GREENFIELD RETURN PERIOD ORDINATES

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

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

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

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

Region = **10**

Use Figure A1.1 to determine region

GREENFIELD RETURN PERIOD FLOW RATES

Return Period	Ordinate	Q (l/s)
1	0.87	0.51
2	0.93	0.55
5	1.19	0.70
10	1.38	0.82
25	1.64	0.97
30	1.7	1.00
50	1.85	1.09
100	2.08	1.23
200	2.32	1.37
500	2.73	1.61
1000	3.04	1.80

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

ESTIMATE OF BROWNFIELD RUNOFF

Total site impermeable area, A = **480** m²

M5-60 rainfall depth **21** mm *[Flood Studies Report (NERC, 1975)]*

Ratio M5-60/M5-2Day, r **0.24** *[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 0.56 — *[The Wallingford Proceedure - V4 Modified Rational Method, Fig A.3b (Hydraulics Research, 1983)]*

M5-15 rainfall depth = 11.8 mm

Return period ratio, Z2

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

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

Rainfall

	Depth (mm)	**Intensity, i (mm/hr)**
M1-15	7.2	29
M10-15	14.4	58
M30-15	17.7	71
M100-15	22.8	91

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	4.8
Q10	9.5
Q30	11.7
Q100	15.0

ESTIMATION OF QBAR (BROWNFIELD RUNOFF RATE)

See Table 2.39 for region curve ordinates

Use FSSR2 Growth Curves to estimate Qbar

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

Region = **10**

Use Figure A1.1 to determine region

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

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

Qbar

Ordinate used	l/s
10 year	6.9
30 year	6.9
100 year	7.2

Proposed Brownfield Runoff, Qbar = **6.99** l/s

Using the average Qbar derived from three ordinates.

ESTIMATE OF BROWNFIELD RUNOFF

Total site impermeable area, A = **595** m²

M5-60 rainfall depth **21** mm — *[Flood Studies Report (NERC, 1975)]*

Ratio M5-60/M5-2Day, r **0.24** — *[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 0.56 — *[The Wallingford Proceedure - V4 Modified Rational Method, Fig A.3b (Hydraulics Research, 1983)]*

M5-15 rainfall depth = 11.8 mm

Return period ratio, Z2

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

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

Rainfall

	Depth (mm)	Intensity, i (mm/hr)
M1-15	7.2	29
M10-15	14.4	58
M30-15	17.7	71
M100-15	22.8	91

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	5.9
Q10	11.8
Q30	14.4
Q100	18.6

ESTIMATION OF QBAR (BROWNFIELD RUNOFF RATE)

See Table 2.39 for region curve ordinates

Use FSSR2 Growth Curves to estimate Qbar

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

Region = **10**

Use Figure A1.1 to determine region

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

Ordinate from FSSR2

Interpolation taken from Figure 24.2 (pg 515) SuDS Manual

Qbar

Ordinate used	l/s
10 year	8.5
30 year	8.5
100 year	9.0

Proposed Brownfield Runoff, Qbar = **8.67** l/s

Using the average Qbar derived from three ordinates.