Bellway Homes (East Midlands) Ltd

Proposed Residential Development
Land off Ashland Road West
Sutton in Ashfield
Nottinghamshire

Flood Risk Assessment
Prepared by EWE Associates Ltd
Final RevA February 2020

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CLIENT DETAILS

Bellway Homes (East Midlands) Ltd

CONTRACT

This report describes work commissioned by Bellway Homes (East Midlands) Ltd following written instruction by their representative during May 2019. Bellway Homes Ltd representative for the contract was Jo Althorpe of DLP Planning Ltd. Lea Favill of EWE Associates Ltd carried out the work.

Date:	24th February 2020
Prepared by:	………………………………………… … Lea Favill Director

REVISION HISTORY

Draft Report Rev0 issued 22nd February 2020 - 1No copy issued to Jo Althorpe (DLP)
Final Report RevA issued 24th February 2020 - 1No copy issued to Jo Althorpe (DLP)

EXECUTIVE SUMMARY

The existing development site consists of a single large agricultural field which has a single hedgerow running south to north through the centre. The site is located to the north of Ashland Road West and to the north west of the centre of Sutton in Ashfield which is to the north of Mansfield. The ground levels in this area are at a level of between 159.64mOD at the north east corner of the site adjacent to the local watercourse up to 182.72mOD at the south west corner of the site adjacent to Ashland Road West. There is existing residential development located to the south, east and west of the site. Brierley Forest Park and a local watercourse are located to the north. The total area of the site is 10.41 hectare and is considered to be 100% permeable as the whole of the site is grassland/agricultural. There are no roofed or paved areas within the site. The site presently drains in a northerly direction towards the local watercourse. The proposal is to construct a residential housing development which also includes associated driveways, access courts and access roads within the site. It is estimated that the impermeable area following completion of the development will be increased to approximately 4.23 hectares which is 41% of the total site.

The existing site is elevated sufficiently above the nearest Main River watercourses, as such, lies within Flood Zone 1 of the Environment Agency Flood Map (version 2.8.2). However, there is a local watercourse to the north of the site which is controlled by a culvert under a large soil heap, which, if it became blocked could result in localised flooding to the north of the site. It is considered that the flood water could overtop the headwall and reach a level of 160.159mOD. As such, it is recommended that the internal ground floor levels of the dwellings within the site are elevated at least 600mm above the estimated flood level, hence a level of 160.759mOD. The area where the dwellings are proposed are at 161.50mOD and above. As such, the raising of ground floor levels will be easily achieved within the site. It is also recommended that the internal ground floor levels of all the dwellings within both parcels of land are elevated at least 150mm above the adjacent proposed roads within the site to reduce the risk of flooding from overland flows.

Consideration has been given to the hierarchy for surface water disposal which recommends the SUDs approach which includes infiltration as the first tier. Further investigation is required to confirm that infiltration drainage will be a practical solution for the site.

However, other SUDs techniques can be used within the site and they have been considered. The second tier is to discharge to a watercourse. The existing site is considered to be 100% permeable. Following the proposed development, the impermeable area will be significantly increased to approximately 41% of the total site area. It is considered that the site currently discharges runoff via a combination of infiltration, evaporation and overland flow to the local watercourse to the north of the site.

Using software developed by Microdrainage the required attenuation has been calculated for the 1 in 100 year plus climate change (40%) event. The site will discharge into the existing local watercourse system to the north of the site at a peak discharge rate of 46.9l/s. The primary attenuation will be provided within a single balancing pond. The balancing pond will be 1.5m deep with a top area of approximately 4000m².

The balancing pond will be used to accommodate the storage during 1 in 1 year, 30 year, 100 year and 100 year +CC storms (worst case scenario).

The proposal is to provide a hydro-brake to restrict flows from the site. The hydro-brake will reduce the runoff from the development site during higher return periods, hence, there will be a significant reduction in runoff and as such the development will provide significant betterment in terms of runoff being passed forward from the site into the local watercourse.

It is concluded that the proposed development lies within flood zone 1 low risk and the current drainage feasibility study utilises sustainable drainage techniques where practically possible.

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1. INTRODUCTION

Terms of Reference

This report was commissioned by Bellway Homes (East Midlands) Ltd to support a planning application for the development of an area of land to the north of Ashland Road West, Sutton in Ashfield to provide residential units. The site can presently be accessed from the south off Ashland Road West. The location of the site is shown on Table 2-1.

The whole of the development site lies within Zone 1 of the Environment Agency Flood Map (version 2.8.2), being the zone with risk of 1 in 1,000 year (0.1% AEP) or less for river flooding. The overall size of the development is greater than 1 hectare.

It is usual for the Agency to raise an objection to development applications within the floodplain or Zone 2 or 3 of the flood map until the question of flood risk has been properly evaluated. The Agency will also object to developments where the total site area is in excess of 1 hectare until suitable consideration has been given to surface water runoff.

Approach to the Assessment

As there two sources of flood risk – Local Watercourses and Surface water runoff – it is necessary to determine flood water levels at the site for the desired return periods emanating from this source. Consideration has also been given to the site flooding from either overland flow or ponding of localised rainfall within the site.

Ashfield District Council undertook a level 1 in-house Strategic Flood risk Assessment (SFRA). The work was completed in February 2009.

There is a single local watercourse located to the north of the site within the Brierley Forest Park. In line with the site the watercourses enter a series of culverts. There are no modelled flood levels which may assist in predicting the design flood level for the watercourses adjacent to the proposed development site. An assessment of the flood level will be made by simple channel capacity calculations using the Manning's 'n' equation and pipe tables.

There is a single local watercourse located within the western boundary of the site which flows south to north into the local watercourse located within Brierley Forest Park. The watercourse is small with a bed width of 0.5m and depth of 0.4m with a bed slope of 1 in 19. The capacity of the watercourse is estimated at approximately 800l/s (0.8m^3/s) using the Manning's Equation. The catchment area is approximately 1.2 hectares of domestic gardens and a small part of the site. Based on a 1 in 100 year plus climate change greenfield runoff rate of 25 l/s/ha the peak flow rate entering the watercourse is only 30 l/s. it is therefore considered that the capacity of the local watercourse is adequate to convey the peak 1 in 100 year plus climate change flow. The developer should not encroach onto the watercourse or impede its conveyance route. As such, no further consideration will be given to this watercourse.

In line with the site the watercourses enter a series of culverts. There are no modelled flood levels which may assist in predicting the design flood level for the watercourses adjacent to the proposed development site. An assessment of the flood level will be made by simple channel capacity calculations using the Manning's 'n' equation and pipe tables.

The proposed development will significantly increase the paved and roofed area within the site. As such, the existing method of draining the site will be appraised. EWE Associates Ltd has undertaken a drainage feasibility study for the site and will provide indicative storage volumes estimated using Micro Drainage Source Control Software.

The storage volumes needed to attenuate surface water flow from the development to accommodate the required 1 in 100 year plus 40% climate change event, have therefore been calculated, using the proposed drainage strategy, as outlined above. However, the volume balance requirements should be recalculated during the detailed design stage to reflect the actual development proposal, the extent of impermeable areas and runoff to be generated.

A walk over of the site was conducted by Mr Lea Favill, Principal Engineer during February 2020; during the visit a photograph survey of the site was undertaken. During the site visit an investigation into the existing drainage route from the site was also undertaken. A topographical survey of the development site was provided by the client's representative. The survey has been calibrated to OS GPS. The surveyed levels have been used within this report.

The requirements for flood risk assessments are generally as set out in National Planning Policy Framework (NPPF). The detail and complexity of the study required should be appropriate to the scale and potential impact of the development. For the purposes of this study, the following have been considered: -

- Available information on historical flooding in the area.
- Site level information.
- Details of structures, which may influence hydraulics of the watercourse and consideration of the effect of blockage of structures.
- Estimates of design levels, equivalent to a 200-year (coastal/tidal) and a 100-year (fluvial) return period flood event.
- Allowances for increased flows resulting from the effects of climate change.
- Allowances for sea level rise resulting from the effects of climate change.

Assess the existing runoff characteristics and the potential impact the proposed development will have on the runoff.

Further guidance is also provided in the CIRIA Research Project 624 "Development and Flood Risk: Guidance for the Construction Industry".

Application of Sequential & Exceptions Test

The whole of the development site lies within Zone 1 of the Environment Agency Flood Map (version 2.8.2), being the zone with risk of 1 in 1,000 year (0.1% AEP) or less for tidal/river flooding. The SFRA completed in 2009 estimates that the whole of the site is within flood zone 1, low risk. The Environment Agency flood maps also show the whole of the site to be within flood zone 1, low risk. The proposed development is residential, as such, is more vulnerable.

Table 1: Flood Risk Vulnerability and Flood Zone 'Compatibility'

Flood Risk Vulnerability classification		Essential Infrastructure	Water compatible	Highly Vulnerable	More Vulnerable	Less Vulnerable
Flood Zone	Zone 1	✓	✓	✓	✓	✓
	Zone 2	✓	✓	Exception Test required	✓	✓
	Zone 3a	Exception Test required	✓	×	**Exception Test required**	✓
	Zone 3b	Exception Test required	✓	×	×	×

✓ Development is appropriate

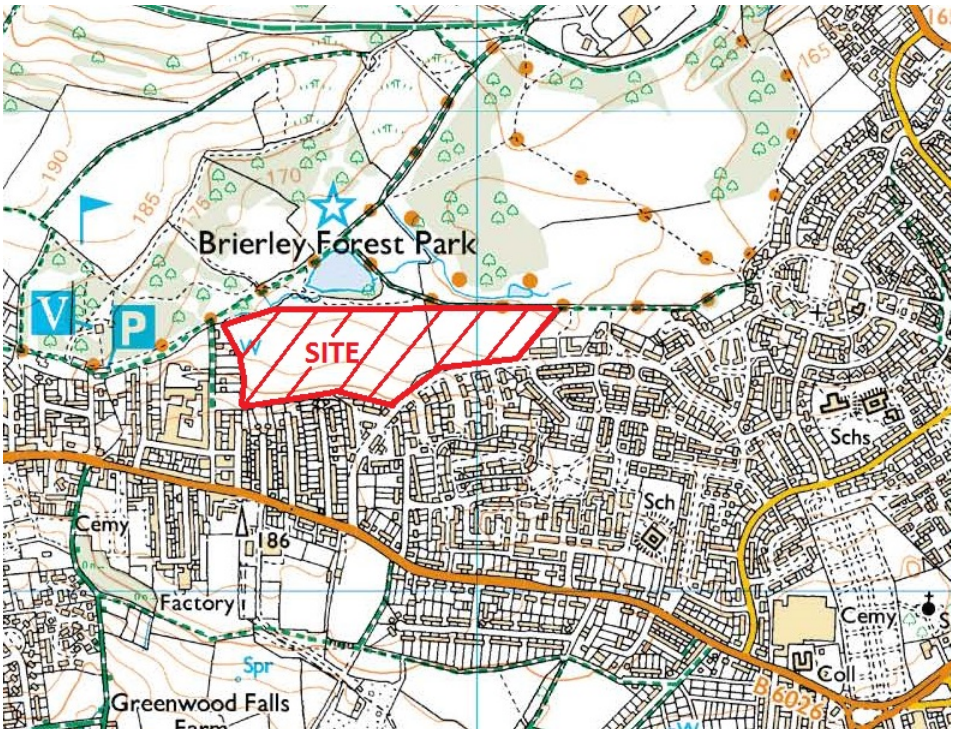
× Development should not be permitted

It is therefore considered that the exceptions test is not required for the development. Furthermore, it is considered that the site is sequentially preferable as there are no alternative sites available at a lower flood risk within the district.

2. DETAILS OF THE SITE

Site Location

Table 2-1: Location Plan



Ordnance Survey Licence no.: WL1005160

Site Details

Table 2-2: Site Details	
Site Name	Land off Ashland Road West, Sutton in Ashfield
Existing Land Use	Agricultural
Proposed Development	Residential Development
Grid Reference (centre of site)	SK 48193 59641
County	Nottinghamshire
Local Planning Authority	Ashfield District Council
Internal Drainage Board	Not Applicable
Post code	NG17 2GG

Site Description

The existing development site consists of a single large agricultural field which has a single hedgerow running south to north through the centre. The site is shown on the aerial photograph in Figure 2.1 below. The site is located to the north of Ashland Road West and to the north west of the centre of Sutton in Ashfield which is to the north of Mansfield.

The ground levels in this area are at a level of between 159.64mOD at the north east corner of the site adjacent to the local watercourse up to 182.72mOD at the south west corner of the site adjacent to Ashland Road West. The ground levels within the site are illustrated at Appendix A of this report. There is existing residential development located to the south, east and west of the site. Brierley Forest Park and a local watercourse are located to the north.

The total area of the site is 10.41 hectare and is considered to be 100% permeable as the whole of the site is grassland/agricultural. There are no roofed or paved areas within the site. The site presently drains in a northerly direction towards the local watercourse.

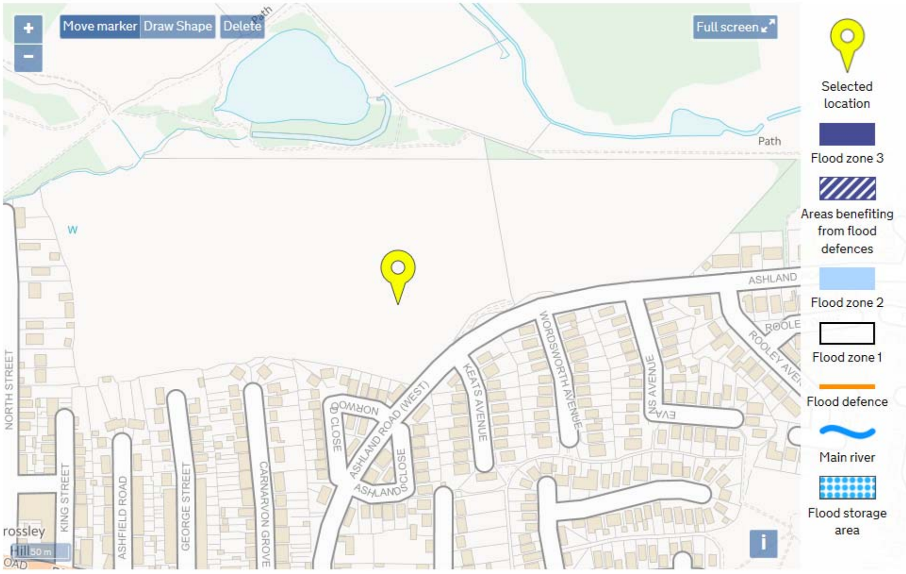
The proposal is to construct a residential housing development which also includes associated driveways, access courts and access roads within the site. Indicative layout plan for the development is shown at Appendix B of this report. It is estimated that the impermeable area following completion of the development will be increased to approximately 4.23 hectares which is 41% of the total site.

Site Photographs

Figure 2-1: Aerial Photograph of Existing Site

3. INITIAL ASSESSMENT

Environment Agency Flood Map



Environment Agency Reservoir Flood Map

Environment Agency Surface Water Flood Map

Past Flooding History

A search on the British Hydrological Society Chronology of British Hydrological Events website[1] found no records of past flooding within Sutton in Ashfield local to the site.

Undertaking an internet based search for flooding in the area provided no further information.

Environment Agency Flooding History

The Environment Agency did not provide any references to flooding close to the site.

SFRA Flooding History

The SFRA did not provide any references to flooding close to the site.

Possible Flooding Mechanisms

As there two sources of flood risk – Local watercourses and Surface water runoff – it is necessary to determine flood water levels at the site for the desired return periods emanating from these sources.

The Local watercourse is not defended by earth embankments or flood walls which extend above the natural ground level. As such, consideration will only need to be given to overtopping.

[1] http://www.dundee.ac.uk/geography/cbhe/

The proposed development will significantly increase the impermeable area and subsequently the runoff from the site will also be increased. An initial drainage strategy has been completed as part of this assessment based on available data.

Ashland Road West is elevated above the site. Therefore, this flood mechanism has been considered further.

There are no depressed areas within the site which could promote ponding. Therefore, this flood mechanism has not been considered further.

Information on groundwater flooding is limited within the Ashfield District Council. The SFRA makes no comment regards the potential for ground water flooding in the district. In addition, reference to the Groundwater Vulnerability Map and Source Protection Zones produced by the Environment Agency indicate that the district is not underlain by an aquifer, as such, groundwater is unlikely to be source of significant flood risk.

Severn Trent Water is the statutory water undertaker and is responsible for the public sewer systems within the Sutton in Ashfield area. Severn Trent Water maintains a register of historical sewer flooding events (DG5 Register) within the area. There are no report incidents close to the site.

4. FLOOD RISK ASSESSMENT

Requirements of the Environment Agency

The Environment Agency, as part of its development control procedures, generally require finished floor levels to be set above the 1% AEP plus climate change flood water level at the site. The development is residential in nature, as such, it is considered that access and egress from the development site will be essential during times of extreme floods.

The Environment Agency will request that the runoff from the proposed development is restricted to the existing peak runoff rate. None of the existing site is considered to be 'brownfield development' hence a further 30% reduction will not need to be applied to the estimated peak run off rate in order to accommodate climate change over the lifetime of the development. The whole of the site is Greenfield development which will not require a 30% reduction. They will further insist that the proposed 1 in 2 year runoff can be maintained and also insist that the 1 in 30 year event is not allowed to flood the surface; hence, the water must remain within the pipes, manholes, and storage systems (the proposed balancing ponds). The 1 in 100 year plus climate change event will be allowed to flood the surface but flood water will not be permitted to enter any of the buildings within the site. The 1 in 100 year plus climate change flood must also be limited to the development boundary and must not be allowed to migrate to adjacent properties.

Local Watercourse

There is a local watercourse which flows from the west to the east through the Brierley Forest Park approximately 30m to the north of the site. Upstream and in line within the site the local watercourse passes through the Brierley Forest Park collecting flows from within the park and the urbanised area of Huthwaite. Within the Brierley Forest Park there is a large pond which is used for fishing which is controlled by a weir and a 900mm diameter culvert directly downstream which conveys flows beneath an access track before discharging into an open channel section for the local watercourse. At the downstream end of the site the local watercourse passes through a 1300mm diameter culvert which flows under a soil heap before emerging in a small valley to the south of the Brierley Park Industrial Estate. The watercourse generally collects flows from a catchment area of 0.87 km^2.

The Environment Agency was unable to provide any estimated or historical flood data which may assist in predicting the 1 in 100 year flood at the site. Therefore, an estimate of the 1 in 100 year flood level has been made simplistically.

Local watercourse is approximately 460m in length between the 1300mm diameter culvert at the downstream end of the site. It is considered that the section of the open watercourse directly in line with the downstream end of the site is the most likely point which the watercourse could flood and affect the site due to the constriction caused by the 1300mm diameter culvert. The invert level of the 1300mm diameter culvert has been estimated at 157.806mOD and the soffit of the 1300mm diameter culvert is 159.106mOD. Directly upstream of the 1300mm diameter culvert the minimum open channel cross section has a bed width is 1m and the top width of 4m. The bed slope in line with the site is 1 in 122 (3.763m fall in 460m). The culvert entrance is shown overleaf at Figure 4-1.

Due to the catchment size it is considered that the FEH technique is not appropriate for such a small catchment. The catchment is also considered to be predominantly rural and therefore the ReFH Rainfall Runoff Method has been adopted. From the site visit and ordnance survey mapping of the area it is estimated that the catchment area to the downstream end of the site is approximately 0.87km². As such a peak 1 in 100 year flow of 1.3m³/s (1300l/s) has been estimated. The peak 1 in 1,000 year flow of 2.5m³/s (2500l/s) has been estimated. The results and calculations are provided at Appendix C of this report. The Manning's calculations used to calculate the channel capacity is shown below.

Figure 4-1: 1300mm diameter culvert entrance

The channel capacity in line with the site has been estimated at 14.4m³/s. The 1 in 100 year flow has been estimated at 1.3m³/s which is considerably less than the channel capacity.

The climate change allowances were modified during February 2016 by the Environment Agency. The increases are now provided for eleven different river basins within the country and for varying confidence levels depending on the proposed development. The proposed development falls within the Humber basin and as the development is residential the 2070 to 2115 banding is relevant as the design life of the development is 100 years. As the development is residential the upper and higher central bandings should also be considered. This provides two climate change increases at 30% and 50%.

Therefore, an increase of 50% has been applied to the 1 in 100 year flow for future climate change; hence, a flow of 1.95m³/s has been estimated. The 1 in 1,000 year flow has been estimated at 2.5m³/s which is considerably less than the channel capacity. Therefore, the 1 in 100 year, 1 in 100 year plus climate change and extreme 1 in 1,000 year flows are less than the channel capacity. As such, out of bank flooding is not expected during any of the flood events.

Manning's Equation: $Q = (A*r^{2/3}*s^{1/2}) / n$

A = Cross sectional area of channel = 8.25m^2

P = Wetted perimeter of channel = 5.24m

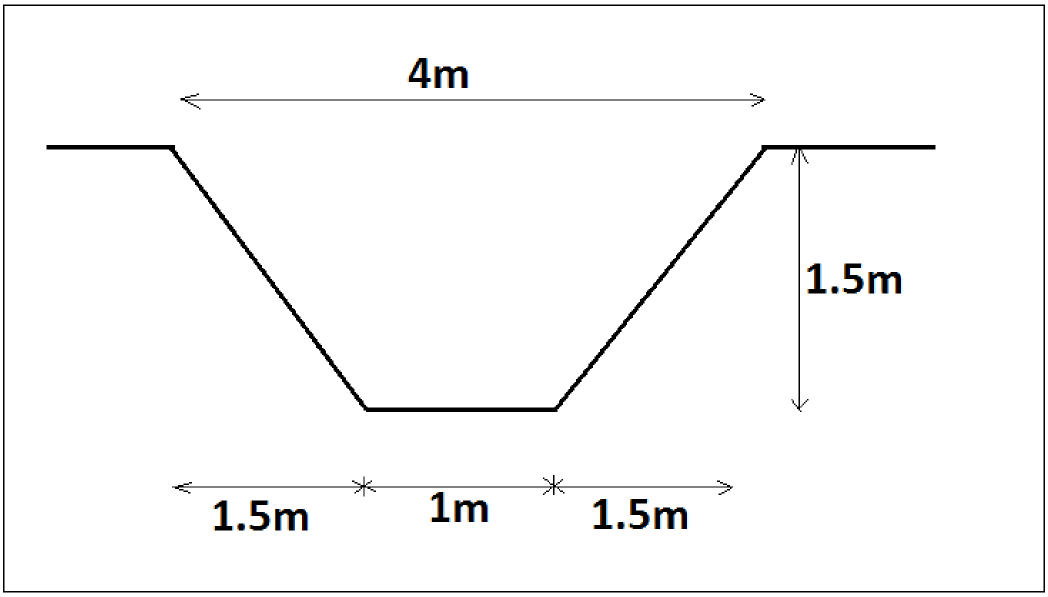
r = A/P = 1.57

s = average channel slope = 3.763m fall over 460m = 0.0082

n = Manning's coefficient of roughness = 0.070 (straight channel, with some stones and weeds).

Q = 14.4m^3/s

Figure 4-2: Cross Section In line with Site



Culvert Capacity

Directly in line with the downstream end of the site there is a 1300mm diameter culvert under the soil heap. The invert level at the culvert entrance is 157.806mOD. The culvert is several hundred metres in length. The gradient of the local watercourse upstream of the culvert entrance is 1 in 122 (0.0082). As such, a pipe gradient of 1 in 122 has been adopted for this assessment. The peak 1 in 100 year plus climate change flow has been estimated at 1.95m^3/s. The peak capacity of the culvert has been estimated using the Colebrook White pipe tables. A peak flow of 4.25m^3/s has been estimated. The calculation sheet is provided at Appendix D of this report. This is considerably greater than the estimated 1 in 100 year plus climate change flow of 1.95m^3/s. Due to the size of the culvert and the amount of vegetation located directly upstream of the headwall that it is highly probable that the culvert could become blocked. Therefore, flows will back up from the headwall and overtop the headwall during the design flood events.

Adopting the criteria set by the Environment Agency a 75% blockage is considered. Therefore, reducing the culvert capacity by 75% reduces the culvert capacity at the entrance to 1.06m³/s. This is less than the estimated 1 in 100 year plus climate change flow of 1.95m³/s and the extreme 1 in 1,000 year flow of 2.5m³/s, as such, it is considered that the water level will exceed the soffit level of the culvert (159.106mOD). It is likely that the flood water will overtop the headwall which is at a level of 159.559mOD. It is therefore estimated that during a 1 in 1,000 year extreme flood event that 1.44m³/s will overtop the headwall.

Therefore assuming the flood water is limited to a channel width of 4m it is estimated that the flood depth will not exceed 0.6m. The Manning's calculations used to calculate the overflow channel capacity is shown overleaf. Therefore, the peak flood level during the extreme 1 in 1,000 year flood event has been estimated at 160.159mOD (1559.559mOD + 0.6m).

Manning's Equation: $Q = (A*r^{2/3}*s^{1/2}) / n$

A = Cross sectional area of channel = 2.4m²

P = Wetted perimeter of channel = 5.2m

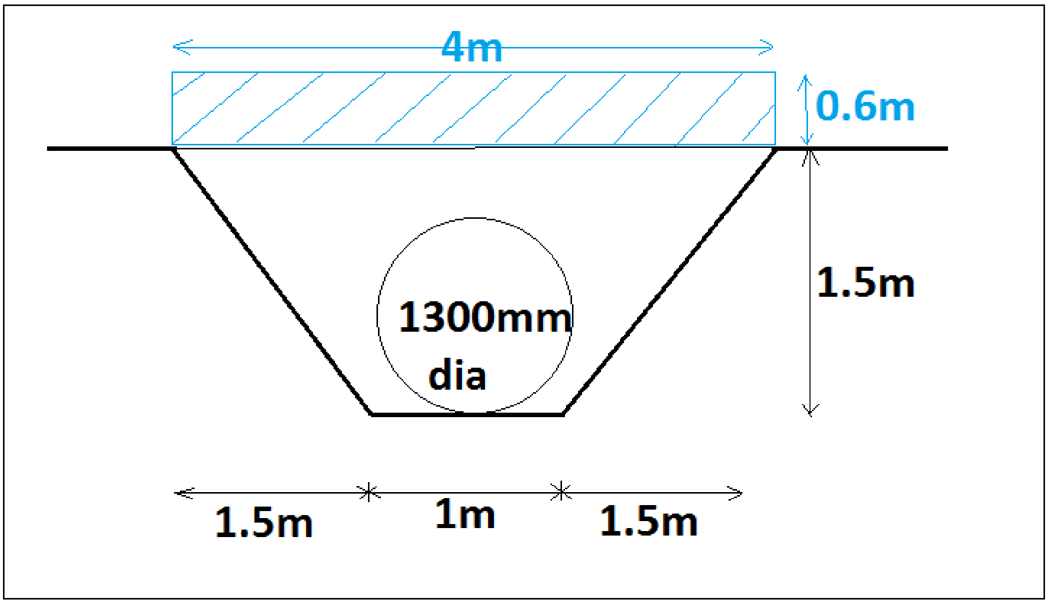
r = A/P = 0.462

s = average channel slope = 3.763m fall over 460m = 0.0082

n = Manning's coefficient of roughness = 0.070 (straight channel, with some stones and weeds).

Q = 1.55m³/s

Figure 4-3: Cross Section at 1300mm diameter culvert



Increased Runoff due the Development

Existing Drainage

The site consists of a single large agricultural field which slopes south from Ashland Road West towards the north where the local watercourse within Brierley Forest Park is located. As such, the whole of the site is considered to be 100% permeable. There is a single hedgerow running south to north approximately midway within the site. However, there are no ditches or dikes within the site.

Based on the soil maps and from experience of other developments local to Sutton in Ashfield it is considered that infiltration drainage is not a practical option. As such, at this stage it is assumed that infiltration is not a practical solution for the development site and therefore the current discharge to the local watercourse has been assessed.

For the purpose of this assessment the peak discharge rates from the site below in Table 4-1 have been conservatively adopted. The ICP SUDS Method has been used to calculate the runoff from the site. The estimate assumes a 100% permeable area. The calculation sheet is provided at Appendix E of this report. Any discharge from the site into the local watercourse will require the consent of the appropriate water authority/riparian owner and as such they will also need to be approached to agree the discharge restriction from the site. There is a 600mm diameter culvert which flows south to north past the north east corner of the site which outfalls into the local watercourse directly adjacent to the downstream end of the site. It is possible that a connection could be made into this culvert from the site if the owners are in agreement.

For the purpose of this assessment a peak discharge rate of 46.9l/s has been adopted.

Table 4-1: Runoff Estimates from site 10.41 ha

Return Period	Flow in litres per second (l/s)	Flow in litres per second per hectare (l/s/ha)
Qbar	46.9	4.50
1 in 2 year	42.0	4.04
1 in 30 year	91.9	8.83
1 in 100 year	120.5	11.58

Intrusive investigation work will be needed to determine whether any areas of the site are suitable for the use of soakaways. If soakaways are determined to be unsuitable then roof and pavement drainage will need to discharge into the existing local watercourse located to the north of the site.

Proposed Drainage Strategy

It is proposed to ultimately discharge any surface water flows generated by the development of the site which cannot drain via infiltration to the local watercourse located to the north of the site.

The proposed impermeable area for the development site has been calculated to be approximately 4.23 ha which is 41% of the total site area, with the remainder of the site proposed as gardens, landscaping and open space.

The drainage strategy utilises an appropriately sized hydro brake to restrict the flows to a peak rate of 46.9l/s. As such, it is expected that during the more extreme return periods there will be a considerable betterment as the hydro brake is likely to restrict flows to a lesser rate than estimated at present.

Based upon the assumption that the EA will agree to the maximum discharge of 46.9l/s, a preliminary surface water strategy has been developed and attenuation has been sized using MicroDrainage software. Attenuation volumes have been calculated for the 1 in 100 year plus 40% climate change event. The proposed drainage strategy is shown at Appendix F of this report.

It is proposed that a single pond is provided adjacent to the northern boundary of the site at the lowest end of the site (east corner) which will discharge either via the 600mm diameter culvert into the local watercourse are direct into the local watercourse via a new connecting sewer or open section of watercourse.

The model data for the proposed surface water drainage network has been obtained from the proposed development layout drawing and the drainage strategy drawing is provided at Appendix F of this report. A model has been developed to represent the main drainage runs within the proposed drainage network and contributing drainage areas within the development.

Overall, the hydraulic models include the following;

- 50 pipes to represent the proposed system
- 1 hydro-brake (46.9l/s 1.5m head)
- 1 attenuation pond
- 1 outfall into the ditch watercourse with no surcharging

Impermeable area contributions have been based on those supplied on the proposed layout drawing, considered to be 100% impermeable, comprising of roofed and paved areas.

The models have been set up as a fixed runoff model assuming 100% runoff coefficient for roofed and paved areas. The rainfall characteristics for Sutton in Ashfield have been utilised with a value for M5-60 given as 20mm (the depth of rain in a once in five years one hour duration event); and r given as 0.40 (the ratio of the M5-60 rainfall to the M5-2day rainfall). For durations over 60 minutes the FEH runoff data for Sutton in Ashfield has been used.

The volume balance requirements should be recalculated during the detailed design stage to reflect the actual development proposal, agreed discharge rate and the extent of impermeable areas and runoff to be generated.

Hydraulic Modelling Results

The proposed MicroDrainage models have been simulated with the 1 in 100 year plus climate change (40%) return period design storm events with durations of 15, 45, 60, 180, 360, 420, 480, 540, 600, 900 and 1440 minutes. At the request of the Environment Agency seven day 10080 minute duration was also undertaken. The durations were run in both Winter and Summer profiles. It was found that the Winter profile was critical.

The table overleaf shows a summary of the 1 in 100 year plus climate change model runs and the impact on the drainage system in terms of peak depth within the pond and flow through the hydro-brake.

The 480 minute duration produced the largest flow through the hydro-brake (46.3 l/s) which is less than the restricted runoff rate (46.9l/s). The modelled result for the 480 minute Winter model run is provided at Appendix G. There was no flooding during this event.

Return Period	Profile	Duration (min)	Peak water level in pond	Peak flow into ditch	Status
100 year+CC	Winter FSR	15min	160.954	46.3	OK
100 year+CC	Winter FSR	45min	161.146	46.2	OK
100 year+CC	Winter FEH	60min	161.194	46.1	OK
100 year+CC	Winter FEH	180min	161.401	46.2	OK
100 year+CC	Winter FEH	360min	161.463	46.3	OK
10.8100 year+CC	Winter FEH	420min	161.472	46.3	OK
100 year+CC	Winter FEH	480min	161.476	46.2	Surcharge
100 year+CC	Winter FEH	540min	161.474	46.2	OK
100 year+CC	Winter FEH	600min	161.473	46.3	OK
100 year+CC	Winter FEH	900min	161.461	46.3	OK
100 year+CC	Winter FEH	1440min	161.424	46.3	OK
100 year+CC	Winter FEH	10080min	160.766	29.3	OK

Sustainable Urban Drainage

With regard to water quality considerations for the site, it is recommended that 1 treatment train is provided for building roofs and 2 treatment trains are provided for roads and hard-standing areas in line with CIRIA C697 recommendations. For the building roof, it is proposed to use trapped drainage outlets to provide 1 treatment train.

It is proposed that the attenuation will be provided in the form of a single large pond, designed in accordance with the recommendations of CIRIA C697 Table 5.7. This is considered to be the most appropriate type of attenuation facility for a development of this type. The current drainage strategy is to direct the whole of the site to an online pond. As such, flows will constantly be running through the pond prior to discharging to the public sewer. The ponds have been designed to provide the following 4 treatment techniques which are all considered to be high quality.

- Detention Basin
- Shallow wet land – two areas have been provided within the pond
- Wet open channel linking the two shallow wet lands
- Filtration within the basin – although percolation of the subsoil's is poor, which results in infiltration devices failing to meet the half empty time criteria, infiltration will still occur within the wet land areas.

The volume balance requirements should be recalculated during the detailed design stage to reflect the actual development proposal, agreed discharge rate and the extent of impermeable areas and runoff to be generated.

Overland Flow

The southern boundary of the site is formed by the existing Ashland Road West which is generally elevated above the development site. The highway and the adjacent residential dwellings are all supported by a formalised drainage system which is believed to provide at least the 1 in 30 year design standard. As such, any rainfall event which exceed this are likely to flood the highway which could potentially be encouraged towards the new development. It is envisaged that the flows will be shallow, say 100mm in depth.

Foul Drainage

There is a foul sewer located within Ashland Road West to the south of the site. It is proposed that a foul pumping station will be required to lift foul sewerage into the public sewer system.

5. MITIGATION MEASURES

Raising Floor Levels/Land Raising

The existing site is elevated sufficiently above the nearest Main River watercourses, as such, lies within Flood Zone 1 of the Environment Agency Flood Map (version 2.8.2). However, there is a local watercourse to the north of the site which is controlled by a culvert under a large soil heap, which, if it became blocked could result in localised flooding to the north of the site. It is considered that the flood water could overtop the headwall and reach a level of 160.159mOD.

As such, it is recommended that the internal ground floor levels of the dwellings within the site are elevated at least 600mm above the estimated flood level, hence a level of 160.759mOD. The area where the dwellings are proposed are at 161.50mOD and above. As such, the raising of ground floor levels will be easily achieved within the site.

It is also recommended that the internal ground floor levels of all the dwellings within both parcels of land are elevated at least 150mm above the adjacent proposed roads within the site to reduce the risk of flooding from overland flows.

Emergency Access & Egress

As the development is residential, it is considered that dry access and egress from the development site will be essential during extreme flood events.

It is considered that the proposed development is located outside of the 1 in 1,000 year extreme flood envelope and will be a safe area during flood events. As such, dry access and egress will be available at all times onto Ashland Road West to the south of the site.

Control of Runoff

Consideration has been given to the hierarchy for surface water disposal which recommends the SUDs approach which includes infiltration as the first tier. Further investigation is required to confirm that infiltration drainage will be a practical solution for the site.

However, other SUDs techniques can be used within the site and they have been considered. The second tier is to discharge to a watercourse. The existing site is considered to be 100% permeable. Following the proposed development, the impermeable area will be significantly increased to approximately 41% of the total site area. It is considered that the site currently discharges runoff via a combination of infiltration, evaporation and overland flow to the local watercourse to the north of the site.

Using software developed by Microdrainage the required attenuation has been calculated for the 1 in 100 year plus climate change (40%) event. The site will discharge into the existing local watercourse system to the north of the site at a peak discharge rate of 46.9l/s. The primary attenuation will be provided within a single balancing pond. The balancing pond will be 1.5m deep with a top area of approximately 4000m².

The balancing pond will be used to accommodate the storage during 1 in 1 year, 30 year, 100 year and 100 year +CC storms (worst case scenario).

The proposal is to provide a hydro-brake to restrict flows from the site. The hydro-brake will reduce the runoff from the development site during higher return periods, hence, there will be a significant reduction in runoff and as such the development will provide significant betterment in terms of runoff being passed forward from the site into the local watercourse.

It is recommended that during the detailed phase of the development the following items are considered.

- The proposed surface water drainage system should be modelled using Micro Drainage WinDes or similar. The model should be used to analyse the possibility that the design for surface water may fail or becomes blocked and as such should design a backup plan. Overland floodwater should be routed away from vulnerable areas. Acceptable depths and rates of flow are contained in EA and Defra document FD2320/TR2 “Flood Risk Assessment Guidance for New Development Phase 2”.
- The maintenance and adoption regimes for all elements of the development should be considered for the lifetime of the development.
- Consenting will be required from the Water Authority for any connections/outfalls into the local watercourse.

6. CONCLUSION

It is concluded that the proposed development lies within flood zone 1 low risk and the current drainage feasibility study utilises sustainable drainage techniques where practically possible.

Appendix A: - Existing Ground Levels within Site

Appendix B: - Proposed Layout Drawing

Appendix C: - ReFH Calculation Sheet

Revitalised FSR/FEH rainfall runoff method

Spreadsheet application report

User name	Lea Favill	**Catchment name**		**Date/time modelled**	27-Oct-2014 12:35
Company name	EWE Associates Ltd	**Catchment easting**	448050	**Version**	1.4
Project name	Ashland Road West Suttor	**Catchment northing**	359750		
		Catchment area	0.87		

Summary of model setup

Design rainfall parameters		Loss model parameters		Routing model parameters		Baseflow model parameters	
Return period (yr)	100	**C_{max} (mm)**	401	**T_p (hr)**	0.93	**BL (hr)**	19.2
Duration (hr)	1.8	**C_{ini} (mm)**	126	**U_p**	0.65	**BR**	1.23
Timestep (hr)	0.2	**α factor**	0.83	**U_k**	0.8	**BF_0 (m^3/s)**	0
Season	Winter						

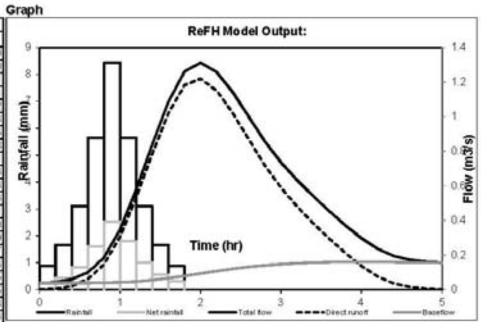
Summary of results

FEH DDF rainfall (mm)	51.1	**Peak rainfall (mm)**	8.4
Design rainfall (mm)	31	**Peak flow (m^3/s)**	1.3

Results

Series	Design Rainfall	Net rainfall	Direct runoff	Baseflow	Total flow
Unit	mm	mm	m^3/s	m^3/s	m^3/s
0.0	0.9	0.2	0.0	0.0	0.0
0.2	1.7	0.4	0.0	0.0	0.0
0.4	3.1	0.8	0.0	0.0	0.1
0.6	5.7	1.6	0.1	0.0	0.1
0.8	8.4	2.5	0.1	0.0	0.2
1.0	5.7	1.8	0.3	0.0	0.3
1.2	3.1	1.0	0.5	0.0	0.6
1.4	1.7	0.6	0.8	0.1	0.8
1.6	0.9	0.3	1.0	0.1	1.1
1.8	0.0	0.0	1.2	0.1	1.3
2.0	0.0	0.0	1.2	0.1	1.3
2.2	0.0	0.0	1.2	0.1	1.3
2.4	0.0	0.0	1.0	0.1	1.1
2.6	0.0	0.0	0.9	0.1	1.0
2.8	0.0	0.0	0.7	0.1	0.9
3.0	0.0	0.0	0.6	0.1	0.7
3.2	0.0	0.0	0.5	0.2	0.6
3.4	0.0	0.0	0.4	0.2	0.5
3.6	0.0	0.0	0.3	0.2	0.5
3.8	0.0	0.0	0.2	0.2	0.4
4.0	0.0	0.0	0.1	0.2	0.3
4.2	0.0	0.0	0.1	0.2	0.2
4.4	0.0	0.0	0.0	0.2	0.2
4.6	0.0	0.0	0.0	0.2	0.2
4.8	0.0	0.0	0.0	0.2	0.2
5.0	0.0	0.0	0.0	0.2	0.2
Total (mm)	31.0	9.3	9.3	2.3	11.6

Graph



Audit comments

Model run with ReFH dll version 1.4.0005

Catchment
Catchment descriptors imported from file
Catchment descriptor file = 'ashland road west csv.csv'
Catchment decriptor file exported from CD ROM version 1
BFIHOST value of 0.505 used
PROPWET value of 0.35 used
SAAR value of 745 used
DPLBAR value of 0.87 used
DPSBAR value of 41.1 used
URBEXT value of 0.176 used
C value of -0.026 used
D1 value of 0.343 used
D2 value of 0.392 used
D3 value of 0.29 used
E value of 0.304 used
F value of 2.405 used

Rainfall
Recommended season is Summer, as URBEXT => 0.125
Recommended season overridden by the user
ReFH design standard Seasonal Correction Factor of 0.63 applied
ReFH design standard Areal Reduction Factor of 0.97 applied

Loss Model
C_{Max} derived from catchment descriptors
ReFH design standard C_{ini} used
ReFH design standard α factor used

Routing Model
T_p derived from catchment descriptors
ReFH design standard used for U_p
ReFH design standard used for U_k

Baseflow Model
BL derived from catchment descriptors
BR derived from catchment descriptors
ReFH design standard BF_0 used

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Revitalised FSR/FEH rainfall runoff method

Spreadsheet application report

User name Lea Favill
Company name EWE Associates Ltd
Project name Ashland Road West 1,000

Catchment name
Catchment easting 448050
Catchment northing 359750
Catchment area 0.87

Date/time modelled 27-Oct-2014 12:49
Version 1.4

Summary of model setup

Design rainfall parameters		Loss model parameters		Routing model parameters		Baseflow model parameters	
Return period (yr)	1000	**C_{max} (mm)**	401	**T_p (hr)**	0.93	**BL (hr)**	19.2
Duration (hr)	1.8	**C_{ini} (mm)**	126	**U_p**	0.65	**BR**	1.23
Timestep (hr)	0.2	**α factor**	0.7	**U_k**	0.8	**BF_0 (m^3/s)**	0
Season	Winter						

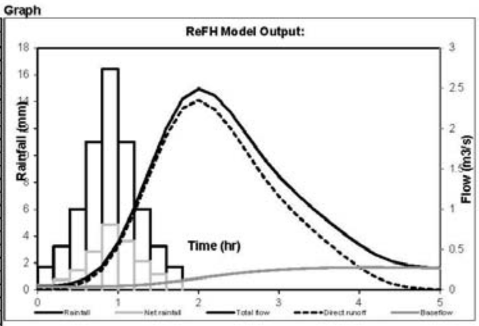
Summary of results

FEH DDF rainfall (mm) 99.6
Design rainfall (mm) 60.4
Peak rainfall (mm) 16.4
Peak flow (m^3/s) 2.5

Results

Series	Design Rainfall	Net rainfall	Direct runoff	Baseflow	Total flow
Unit	mm	mm	m^3/s	m^3/s	m^3/s
0.0	1.7	0.4	0.0	0.0	0.0
0.2	3.2	0.7	0.0	0.0	0.0
0.4	6.0	1.4	0.0	0.0	0.1
0.6	11.0	2.9	0.1	0.0	0.1
0.8	16.4	4.9	0.2	0.0	0.3
1.0	11.0	3.6	0.5	0.0	0.6
1.2	6.0	2.1	1.0	0.1	1.0
1.4	3.2	1.2	1.4	0.1	1.5
1.6	1.7	0.6	1.9	0.1	2.0
1.8	0.0	0.0	2.2	0.1	2.4
2.0	0.0	0.0	2.4	0.1	2.5
2.2	0.0	0.0	2.2	0.2	2.4
2.4	0.0	0.0	2.0	0.2	2.2
2.6	0.0	0.0	1.7	0.2	1.9
2.8	0.0	0.0	1.4	0.2	1.6
3.0	0.0	0.0	1.2	0.2	1.4
3.2	0.0	0.0	0.9	0.3	1.2
3.4	0.0	0.0	0.8	0.3	1.0
3.6	0.0	0.0	0.6	0.3	0.9
3.8	0.0	0.0	0.4	0.3	0.7
4.0	0.0	0.0	0.3	0.3	0.6
4.2	0.0	0.0	0.2	0.3	0.4
4.4	0.0	0.0	0.1	0.3	0.4
4.6	0.0	0.0	0.0	0.3	0.3
4.8	0.0	0.0	0.0	0.3	0.3
5.0	0.0	0.0	0.0	0.3	0.3
Total (mm)	**60.4**	**17.9**	**17.9**	**3.7**	**21.6**

Graph



Audit comments

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C value of -0.026 used
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D3 value of 0.29 used
E value of 0.304 used
F value of 2.405 used

Rainfall
Recommended season is Summer, as URBEXT => 0.125
Recommended season overridden by the user
ReFH design standard Seasonal Correction Factor of 0.63 applied
ReFH design standard Areal Reduction Factor of 0.97 applied

Loss Model
C_{Max} derived from catchment descriptors
ReFH design standard C_{ini} used
ReFH design standard α factor used

Routing Model
T_p derived from catchment descriptors
ReFH design standard used for U_p
ReFH design standard used for U_k

Baseflow Model
BL derived from catchment descriptors
BR derived from catchment descriptors
ReFH design standard BF_0 used

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Appendix D: - Culvert Capacity Calculation Sheet

COLEBROOK WHITE

Roughness	1.5	mm	**U/S level**	161.569	m	
Diam(mm)	1300	mm	**D/S level**	157.806	m	
Length	460	m	**Gradient**	0.00818043		122.242891

PROPOR'N DEPTH	WETTED PERIMETER	AREA OF FLOW	HYDRAULIC MEAN DEPTH	VELOCITY (m/s)	DISCHARGE (l/s)	DEPTH (mm)	SURFACE WIDTH (mm)
FULL	4.08407045	1.327322896	0.3250000	3.20	4,240.80	1300	
0.01	0.2604353	0.002246561	0.0086262	0.28	0.63	13	259
0.02	0.36893234	0.006335011	0.0171712	0.46	2.92	26	364
0.03	0.45261583	0.011602714	0.0256348	0.61	7.07	39	444
0.04	0.52353059	0.017808777	0.0340167	0.74	13.16	52	509
0.05	0.58633486	0.024811696	0.0423166	0.86	21.26	65	567
0.1	0.83655144	0.069079218	0.0825762	1.33	92.08	130	780
0.15	1.03401848	0.12484826	0.1207408	1.71	212.87	195	928
0.2	1.20548378	0.18898223	0.1567688	2.02	380.99	260	1,040
0.25	1.36135682	0.259493099	0.1906136	2.28	592.42	325	1,126
0.3	1.50706332	0.334904522	0.2222233	2.52	842.62	390	1,191
0.35	1.64593477	0.414017008	0.2515391	2.72	1,126.54	455	1,240
0.4	1.78026993	0.495794973	0.2784943	2.90	1,438.30	520	1,274
0.45	1.91181758	0.579302493	0.3030114	3.06	1,771.51	585	1,293
0.5	2.04203522	0.663661448	0.3250000	3.20	2,120.40	650	1,300
0.55	2.17225287	0.748020403	0.3443524	3.31	2,478.19	715	1,293
0.6	2.30380052	0.831527923	0.3609375	3.41	2,837.17	780	1,274
0.65	2.43813568	0.913305888	0.3745919	3.49	3,188.35	845	1,240
0.7	2.57700712	0.992418374	0.3851050	3.55	3,525.07	910	1,191
0.75	2.72271363	1.067829797	0.3921932	3.59	3,836.71	975	1,126
0.8	2.87858667	1.138340667	0.3954512	3.61	4,110.55	1040	1,040
0.85	3.05005197	1.202474636	0.3942473	3.60	4,333.72	1105	928
0.9	3.24751901	1.258243678	0.3874477	3.57	4,486.90	1170	780
0.95	3.49773559	1.3025112	0.3723870	3.48	4,531.44	1235	567
1	4.08407045	1.327322896	0.3250000	3.20	4,240.80	1300	0

Appendix E: - ICP SUDs Calculation Sheet

EWE Associates Ltd		Page 1
Windy Ridge Barn Thealby Lane Winterton DN15 9TG		Micro Drainage
Date 03/11/2014 11:30 File	Designed By Lea Checked By	
Micro Drainage	Source Control W.12.4	

ICP SUDS Mean Annual Flood

Input

Return Period (years)	2	Soil	0.450
Area (ha)	10.410	Urban	0.000
SAAR (mm)	715	Region Number	Region 4

Results	**l/s**
QBAR Rural	46.9
QBAR Urban	46.9
Q2 years	42.0
Q1 year	38.9
Q30 years	91.9
Q100 years	120.5

Appendix F: - Drainage Strategy Drawing

Appendix G: - WINDES 100yr+CC 480min