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Project: Surface Water Drainage Assessment Report (SWDS)

Prepared for: Parker Dann

Reference: 2322

Date: Dec 2016

Version: Draft v4.0

ARUN DISTRICT COUNCIL CM/1/17/OUT

Document Issue Record

Project: Surface Water Drainage Assessment Report

Prepared for: Parker Dann

Reference: 2322

Site Location: Land adjacent to Church Lane, Clymping, West Sussex

Proposed Development: Construction of a new area of Clymping, to include residential development, commercial facilities, allotments as well as amenity spaces.

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1. Introduction

1.1 This Surface Water Drainage Assessment has been prepared by Ambiental Technical Solutions, to provide a first stage understanding of the surface water drainage requirements on the proposed development site post development.

1.2 It aims to provide the developer with a basic understanding of the drainage requirements for the site based on the initial development masterplan, desktop study of the ground condition of the site, existing flow pathways across the site, as well as runoff rate and storage requirements.

1.3 The runoff rates and storage requirements detailed within this report are based on the masterplans provided to Ambiental by Parker Dann at the time of undertaking the works. As such these runoff rates and storage requirements should be used for information only, and not part of more detailed works.

Development Proposal

1.4 It is understood that at this time the proposed development is for the construction of a new area of Clymping, to include residential development, community facilities as well as amenity spaces.

1.5 The number of units, for both residential and commercial uses has not been finalised at this point of the development, although it will be up to 300 residential units. Nor has the final layout of public spaces and buildings been determined.

1.6 This study is based on the masterplan provided to Ambiental, identified as drawing number P14 4899 01 rev c and dated May 2016.



Figure 1: Development masterplan (Source: jrp)

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2. Development Description and Site Area

2.1 The site is approximately 21.5ha in size and located to the south of Clymping Village in West Sussex. The site is bounded to the south by Crookthorn Lane, to the east by Church Lane, to the west by Yapton Road (B2233) and to the north by existing residential development within Clymping Village itself and Horsemere Green Lane.

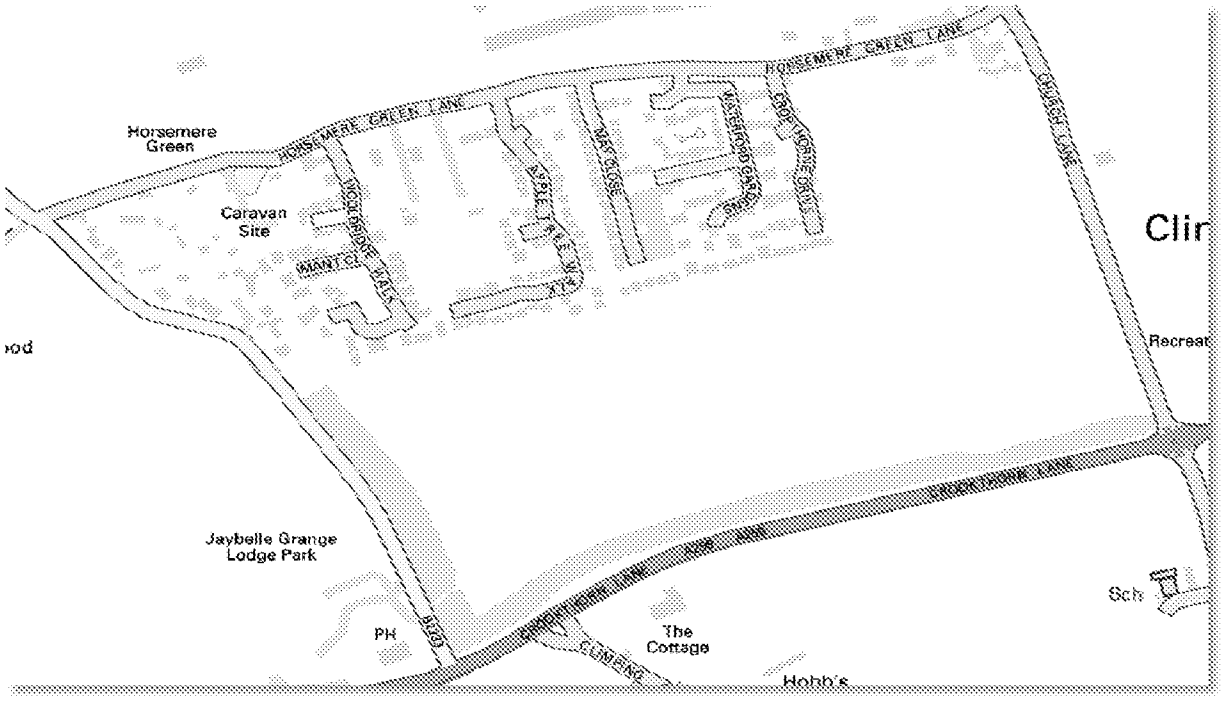


Figure 2: Wider site location (Source: OS)

2.2 The existing site is undeveloped land currently used for agriculture, as such the site has no impermeable surfaces and is deemed to have a 100% permeability.

2.3 Following development the increase in impermeable surfaces on the site will be significantly increased from 0ha to approximately 4.73ha, based on the masterplan.

2.4 The general topography of the site slopes from west to east. LIDAR data for the site shows a level of 7.1m AOD at the southwest of the site falling to 4.7m AOD at the north east corner.

Existing Drainage Infrastructure

2.5 Existing drainage infrastructure on site has not been addressed as part of this scoping works, however the general direction of surface water flow across the site has been determined based on 2m LiDAR topography.

2.6 This analysis shows that surface flow direction is generally from west to east, with flows travelling from south to north at the eastern end of the site. There are also flows potentially running from west to the east along Crookthorn Lane on the southern boundary of the site which could have a potential to affect the development's access.

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2.7 This analysis has also shown that the general surface water flow direction from the site exits the site primarily at the north easterly point and small portion of the site drains north westerly.

2.8 The flow pathway leaving the site at the north west corner appears to cross Yapton Road and travel towards agricultural land. Both the LIDAR data and walkover survey have indicated a dich located in this area.

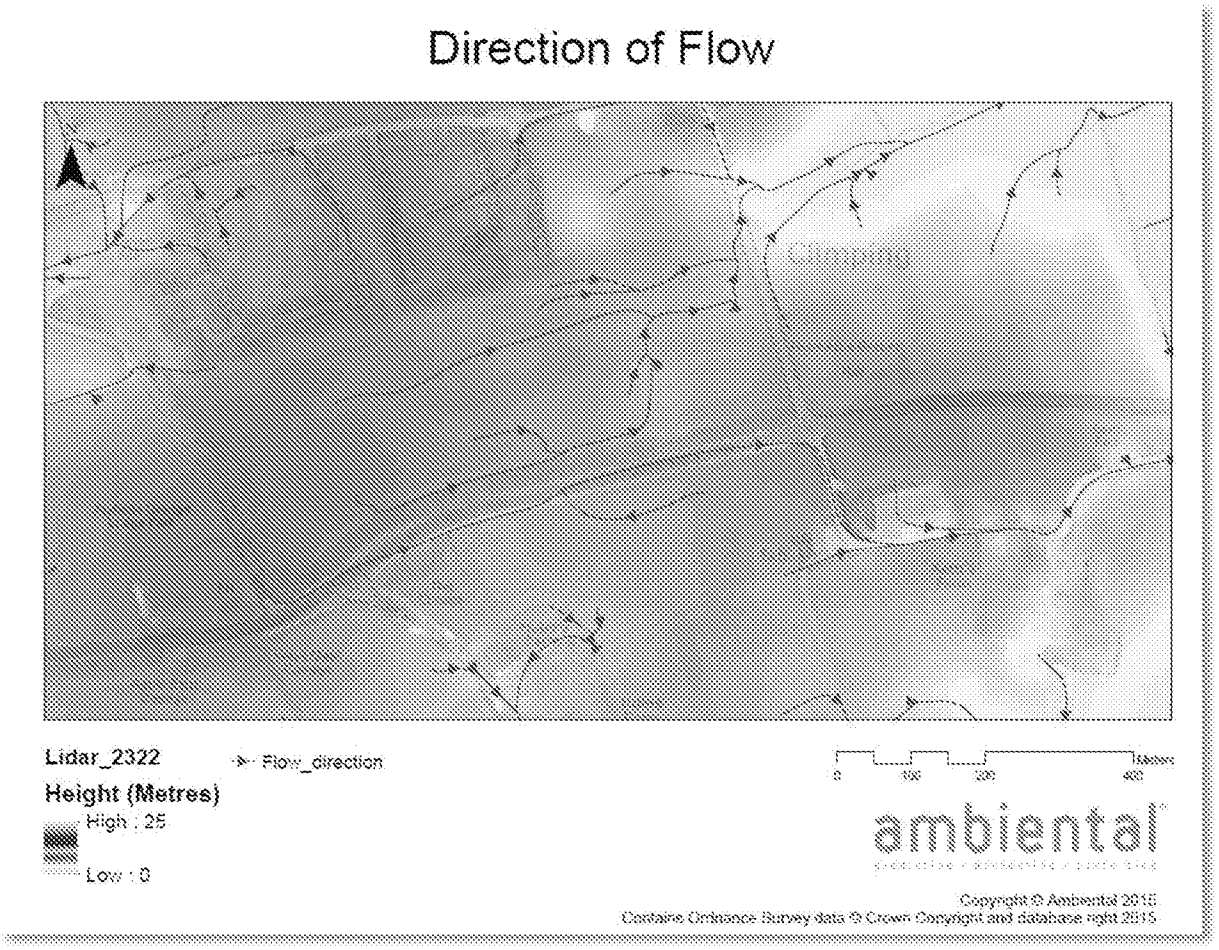


Figure 3: General surface water flow directions across the development site (Source: Ambiental, LiDAR)

Existing Ground Conditions

2.9 The British Geological Survey (BGS) Map indicates that the site geology is split into two distinct geologies.

2.10 The eastern half of the site has a bedrock geology of the Lewes Nodular Chalk Formation, Seaford Chalk Formation, Newhaven Chalk Formation and Culver Chalk Formation. With a superficial geology of raised beach deposits, formed of clayey sand and gravel. This area may provide some infiltration potential.

2.11 The western half of the site has a bedrock geology of the Lewes Nodular Chalk Formation, Seaford Chalk Formation, Newhaven Chalk Formation and Culver Chalk Formation. With a superficial geology of river terrace deposits, formed of sand, silt and clay. It is unlikely this area is suitable for infiltration.

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2.12 The BGS Map shows no boreholes to be available across the development site, however a borehole located just north of the site at Ford Airfield, drilled in August 1981 shows that the bedrock chalk appeared to be found at a depth of 7.8m below surface.

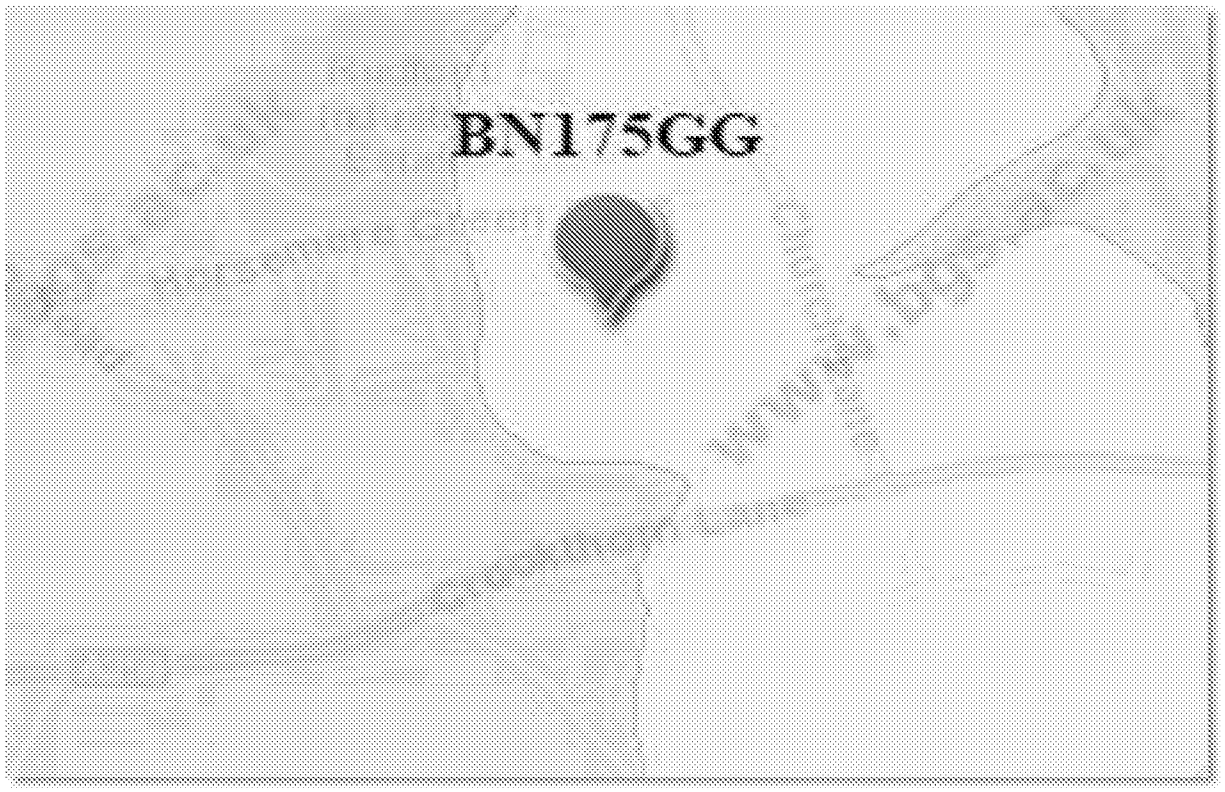


Figure 4: Geology of site; orange shows river terrace deposits, yellow shows beach deposits (Source: BGS)

2.13 The soil type across the site appears to be split based on the superficial geology, with the soils to the west of the site being classified by the UK Soil Observatory (UKSO) website as Sand to Loam, and the east of the site being classified as Sand to Sandy Loam.

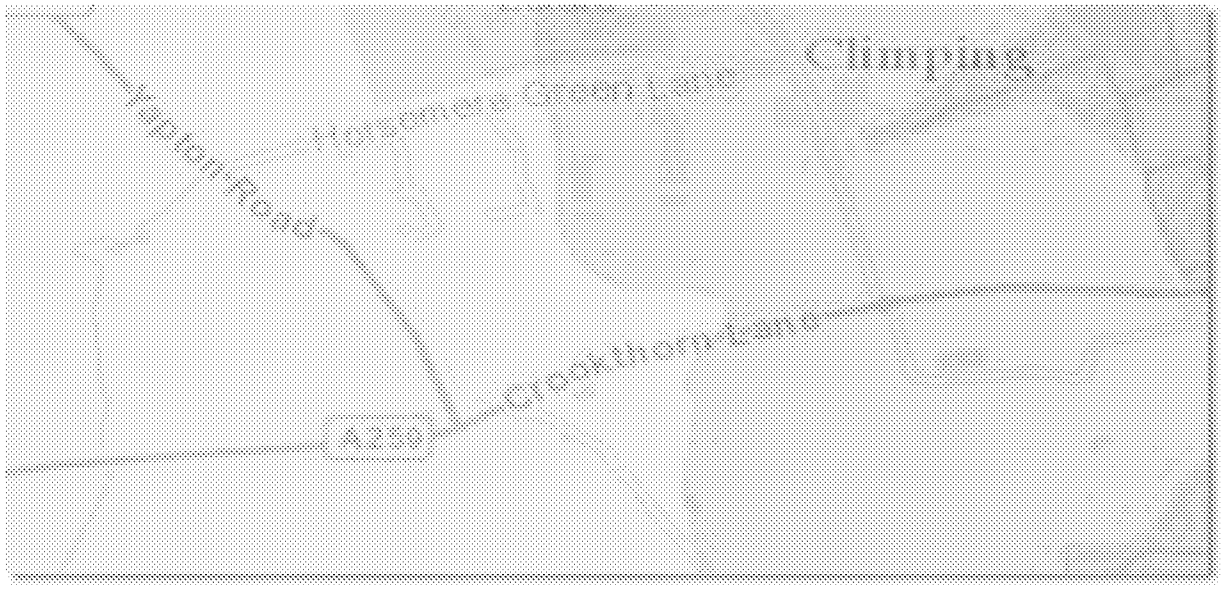


Figure 5: Soils map of the site; green shows sand to loam and blue shows sand to sandy loam (Source: UKSO)

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2.14 At the time of writing the report no site investigation has been carried out so it is assumed no infiltration will be feasible at this stage. Detailed site investigation may show infiltration is possible and it is recommended both BRE365 pit tests and bore-hole infiltration tests into the chalk are carried out to verify suitability for infiltration and ground water levels.

Groundwater

2.15 The site is not located in a groundwater source protection zone, however the superficial deposits across the area have been classified as 'Secondary A' aquifers and the bedrock geology has been classified as 'Principle' aquifers.

2.16 Secondary A aquifers are defined by the EA as 'permeable layers capable of supporting water supplies at a local rather than strategic scale, and in some cases forming an important source of base flow to rivers.'

2.17 Principle aquifers are defined by the EA as 'layers of rock or drift deposits that have high intergranular and/or fracture permeability - meaning they usually provide a high level of water storage. They may support water supply and/or river base flow on a strategic scale'.

2.18 As such the site's groundwater may be of local or strategic importance. Whilst the site is not shown to be in a groundwater source protection zone it is likely due to the close proximity of the bedrock to the ground level (as shown in borehole holes near to the site) that potential groundwater contamination post-development should be taken into consideration.

Nearby Watercourses and Drainage

2.19 The closest watercourse to the site is the River Arun, some 735m to the north-west of the site's most easterly boundary. At this location the River Arun runs in a generally north – south direction and is tidally influenced.

2.20 Small unnamed drains are located in close proximity to the site, however none are located on site.

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3. Runoff Rates and Storage Volumes

3.1 The existing and proposed runoff rates, as well as the required volume of storage to mitigate the developments affect has been calculated based on impermeable areas determined using the masterplan included in Appendix 1.

3.2 The existing greenfield runoff rate (QBar) has been calculated based on the FSR method within Microdrainage and for the entire site (21.5ha) the runoff rate is 76.6l/s.

3.3 The runoff rates for the site have been calculated for the 6 hour storm duration for a number of return periods, including the 1:100 year with an allowance for climate change. This has shown that the development increased runoff rate for all return periods with a storm duration of 6 hours.

3.4 The pre mitigation runoff rate for the site post development for the 6 hour storm event of a 1:100 +CC return period is 400.8l/s. An increase of 150.21l/s runoff from the site post development.

Return Period	Existing Runoff Rate l/s	Pre mitigation Runoff Rate l/s	Description of Runoff Changes	Difference in Runoff Rate for Site l/s
1	56.95	78.07	Runoff rate is increased	21.12
2	57.61	100.59	Runoff rate is increased	42.99
30	106.05	174.46	Runoff rate is increased	68.41
50	161.04	249.54	Runoff rate is increased	88.50
100	208.83	334.00	Runoff rate is increased	125.17
100 + Climate Change	250.59	400.80	Runoff rate is increased	150.21

Table 1: Runoff rates for the site during a 6 hour storm event

3.5 The difference between existing and post mitigation runoff rate will need to be controlled. **The maximum limiting runoff rate for development on this site would be QBar @76.6l/s** to comply with para 24.10.1 with the Ciria SuDS Manual for Long term storage**.**

3.6 As described within Section 2 the eastern portion of the site would likely offer some infiltration potential to provide the option for partial infiltration to the final design. Intrusive investigation

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should be carried out at detailed design stage to verify infiltration potential and ground water levels to inform a final design incorporating betterment over the existing situation.

3.7 The volume of storage required to address the affect the development will have on the site has been calculated. These figures take into account the uncertainty the estimation methods have by increasing the required storage by +25% as current guidance suggests.

Return Period	Existing Runoff Volume m^3	Pre mitigation Runoff Volume m^3	Storage Required m^3
1	1230.18	1518.16	359.97
2	1244.32	1894.64	812.89
30	2290.69	3309.79	1273.87
30	3478.45	4770.72	1615.33
100	4510.67	6359.22	2310.69
100 + Climate Change	5412.81	7915.15	3127.92

Table 2: Volume of storage required

Interception Storage

3.8 In line with current best practice outlined in the SuDS manual interception storage is required to retain the initial 5mm of rainfall on site. The final volume will be subject to arrangement of the detailed architectural layout but initial assessment is for 260m^3 to be provided.

Long Term Storage

3.9 Long-term storage (LTS) is required to address the additional runoff caused by the development above that which would be contributed from the site in its Greenfield state. As stated in *CIRIA 753 'The SUDS Manual'* as the proposal is to utilise QBar as the maximum site outflow rate long term storage is not required.

Total Storage Required

3.10 It can be seen from the simple Microdrainage source control storage model in Appendix 3 that for utilising the QBar runoff rate of 76.6l/s as maximum site outflow the storage required to facilitate the proposed development would be in the order of 3143m^3 with a water depth of circa 1.4m in pond storage. The maximum storage requirement occurs during the 240min rainfall event. The microdrainage volume is slightly higher than the volume calculated through the storage methodology but as they are very similar results the results validate the storage volume

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required. For the purposes of this report as the Microdrainage volume is slightly more onerous it will be utilised for the volume required.

3.11 The proposed site layout shows approximately 2050m^2 open water storage on plan. The site ground levels are in the order of 4.5m AOD or higher and the proposed outfall location has an invert level of circa 2.5m AOD as indicated on the LIDAR ground level information. The initial calculations shows that the volume of water required to be stored can be accommodated within the area proposed as open water.

3.12 As described within Section 2 the eastern portion of the site would likely offer some infiltration potential to provide the option for partial infiltration to the final design. Intrusive investigation should be carried out at detailed design stage to verify infiltration potential and ground water levels to inform a final design incorporating betterment over the existing situation and a potential reduction in the volume of storage required.

3.13 The storage areas are subject to final design but should be designed in accordance with best practice, minimum side slopes, fencing where appropriate, etc, to mitigate any health and safety issues associated with wetlands in the urban environment.

3.14 **An outline drainage strategy drawing is included in Appendix 4.**

4. SuDS Suitability Analysis

4.1 Following analysis of the sites geology, soils and the proposed development the suitability of SuDS components has been analysed.

4.2 SuDS components work in a number of ways, the predominant being:

- Infiltration
- Conveyance into a watercourse or sewer
- Storage
- Attenuation

4.3 It is usual for a SuDS scheme to combine a number of components and a number of these processes across the site to provide a drainage strategy for the whole site. As discussed in paragraph 2.14 at this early stage an intrusive site investigation has not been carried out to verify potential site contamination, groundwater levels or infiltration potential therefore an assumption has been made that infiltration is not feasible.

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Source Control Components

Component	Suitability for Development Type		Water Quality Treatment Potential	Amenity Potential	Suitability for Sites above Vulnerable Groundwater	Suitability for Site based on Geology and Soils
	Residential	Commercial / Industrial				
Green Roof	Suitable	Suitable	Good	Good	Suitable	Unsuitable for residential applications.
Rainwater Harvesting	Suitable	Suitable	Poor	Poor	Suitable	Generally Suitable for multi-residential units
Pervious Surfaces	Suitable	Suitable	Good	Poor	Suitable	Suitable; although limitations as to types of roads and traffic volumes apply

Table 3: Source Control Components

4.4 At this site source control SuDS components would be suitable given the correct design. The use of pervious surfaces for roads, driveways and pavements where suitable can significantly reduce the areas on site deemed to be impermeable and as such can help to reduce the attenuation storage required for the overall site.

Swales and Conveyancing Channel Components

Component	Suitability for Development Type		Water Quality Treatment Potential	Amenity Potential	Suitability for Sites above Vulnerable Groundwater	Suitability for Site based on Geology and Soils
	Residential	Commercial / Industrial				
Swales	Suitable	Suitable	Good	Medium to Good depending on design	Suitable with liner in place	Suitable; although liners may be required to protect potential groundwater beneath site
Channels or Rills	Suitable	Suitable	Good to Very Good depending on design	Good to Very Good depending on design	Suitable with liner in place	Suitable; although liners may be required to protect potential groundwater beneath site

Table 4: Swales and Conveyancing Channel Components

4.4 The site has not at this stage been shown to be above a sensitive groundwater source, however due to the geology of the site groundwater contamination is a potential concern post-development. As such it is advised that should swales or channels be used within the development appropriate liners are used unless appropriate treatment to water runuff is installed on the network prior to outfall to an unlined swayle or channel.

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Filtration Components

Component	Suitability for Development Type		Water Quality Treatment Potential	Amenity Potential	Suitability for Sites above Vulnerable Groundwater	Suitability for Site based on Geology and Soils
	Residential	Commercial / Industrial				
Filter Strips	Suitable	Suitable	Medium	Medium	**Unsuitable**	Potentially **unsuitable** due to groundwater vulnerability potential
Bio-retention Areas	Suitable	Suitable	Good	Good	Suitable	Suitable on areas of site without steep slopes
Filter Trench	Suitable	Suitable	Good	Poor	Suitable	Suitable; although requires high levels of regular maintenance

Table 5: Filtration Components

4.5 Filtration SuDS components appear to be largely suitable for the proposed development site, although by their nature they pose a potential risk of introducing contaminates into the groundwater beneath the site. As such appropriate levels of treatment should be undertaken before water is released through further drainage components into the ground beneath the site.

Retention and Detention Components

Component	Suitability for Development Type		Water Quality Treatment Potential	Amenity Potential	Suitability for Sites above Vulnerable Groundwater	Suitability for Site based on Geology and Soils
	Residential	Commercial / Industrial				
Detention Basin	Suitable	Suitable	Medium	Good	Suitable, with liner	Suitable
Retention Pond	Suitable	Suitable	Good	Good	Suitable, with liner	Suitable; potential H&S concerns of permanent water on site
Geocellular drainage	Suitable	Suitable	Poor	Poor	Suitable	Suitable

Table 6: Retention and Detention Components

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4.6 Retention and detention SuDS components appear to be suitable for the proposed development site. These components are designed to store water and slow its release, either into further SuDS components or into watercourses or the sewers. Detention basins are designed to be dry for most of the time, whilst retention ponds are designed to contain some water all year round. As such the amenity potential for both of these options is high within a development such as the proposed works in Clymping.

4.7 The existing masterplan incorporates ponds as part of the amenity spaces, which provides potential to use these features within the drainage system on the site.

Infiltration Components

Component	Suitability for Development Type		Water Quality Treatment Potential	Amenity Potential	Suitability for Sites above Vulnerable Groundwater	Suitability for Site based on Geology and Soils
	Residential	Commercial / Industrial				
Soakaways	Suitable	Suitable	Good	Poor	**Unsuitable**	Potentially **unsuitable** due to ground contitions
Infiltration Trench	Suitable	Suitable	Good	Poor	**Unsuitable**	Potentially **unsuitable** due to ground contitions.
Infiltration Basin	Suitable	Suitable	Good	Good	**Unsuitable**	Potentially **unsuitable** due to ground contitions
Rain Gardens	Suitable	Suitable	Medium	Good	**Unsuitable**	Potentially **unsuitable** due to ground contitions

Table 7. Infiltration Components

4.8 Infiltration post-development has the potential to be unsuitable due to the potential groundwater contamination concerns at the sites location. As discussed in Section 2.0 at the time of writing the report no site investigation has been carried out so it is assumed no infiltration will be feasible at this stage. Detailed site investigation may show infiltration is possible and it is recommended both BRE365 pit tests and bore-hole infiltration tests into the chalk are carried out to verify suitability for infiltration and ground water levels.

4.9 Given potential groundwater vulnerability post development it is advised that should infiltration be used within the development appropriate treatment to water runuff is installed on the network prior to outfall to an infiltration medium.

4.10 **An outline drainage strategy drawing is included in Appendix 4.**

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Wetlands

Component	Suitability for Development Type		Water Quality Treatment Potential	Amenity Potential	Suitability for Sites above Vulnerable Groundwater	Suitability for Site based on Geology and Soils
	Residential	Commercial / Industrial				
Wetlands	Suitable	Suitable	Good	Good	Suitable, with liner	Suitable

Table 8: Wetlands

4.11 Wetlands are likely to be suitable to use within the development, and provide both good water quality treatment potential but can be used to increase local amenity. The combination of ponds and wetlands can provide a SuDS component train whilst increasing amenity within the development.

4.12 As the proposed masterplan includes ponds, wetlands could be used to provide a further ecosystem, increase biodiversity, improve amenity whilst providing water treatment as part of the drainage scheme.

Other Structures

Component	Suitability for Development Type		Water Quality Treatment Potential	Amenity Potential	Suitability for Sites above Vulnerable Groundwater	Suitability for Site based on Geology and Soils
	Residential	Commercial / Industrial				
Inlet, Outlet and Control structures	Suitable	Suitable	Medium, depending on design	Poor	Suitable	Suitable
Vortex Flow Controls	Suitable	Suitable	Low	Low	Suitable	Suitable

Table 9: Other Structures

4.13 These other components are often used between SuDS components, as well as at any end points of the drainage where water is released from the site. These components can control flow rates, provide sediment filtration, as well as petrochemical control.

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5. Conclusions

5.1 The existing site is greenfield and currently used for agriculture. Post development there will be an increase of impermeable surfaces on the site, based on calculations derived from the masterplan the proposed development's impermeable footprint will be approximately 4.73ha.

5.2 Based on the 1:100 year with an allowance for climate change 6 hour storm event the proposed development will be required to provide 3143m³ of storage to mitigate against the increase in hard surface. The proposed maximum offsite flow rates will be limited to the existing site QBar rate at 76.6l/s.

5.3 Initial calculations included in this report show that the required storage volume can be accommodated in the areas designated for water storage on the proposed masterplan.

5.4 The site is located on largely permeable superficial and bedrock geology and as such infiltration components are suitable for the site, given that appropriate treatment can be provided to address the potential groundwater contamination concerns. For the purposes of this report as no infiltration testing is available and the local area is drain to watercourses a scheme based on outfall to a local watercourse has been considered.

5.5 The proposed SuDS treatment train of bio-retention and filter trenches followed by ponds will offer adequate water quality treatment prior to outfall. This train can be augmented by proprietary items for removal of sediments and further oil removal as necessary.

The findings and recommendations of this report are for the use of the client who commissioned the assessment, and no responsibility or liability can be accepted for the use of the report or its findings by any other person or for any other purpose.

Dr. J. B. Butler
B.Sc., M.Phil., PhD.
Ambiental Technical Solutions Ltd. August 2015

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Appendix 1 – Figures

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Appendix A; Figure 2 - Direction of surface water (Source: Ambiental) 19

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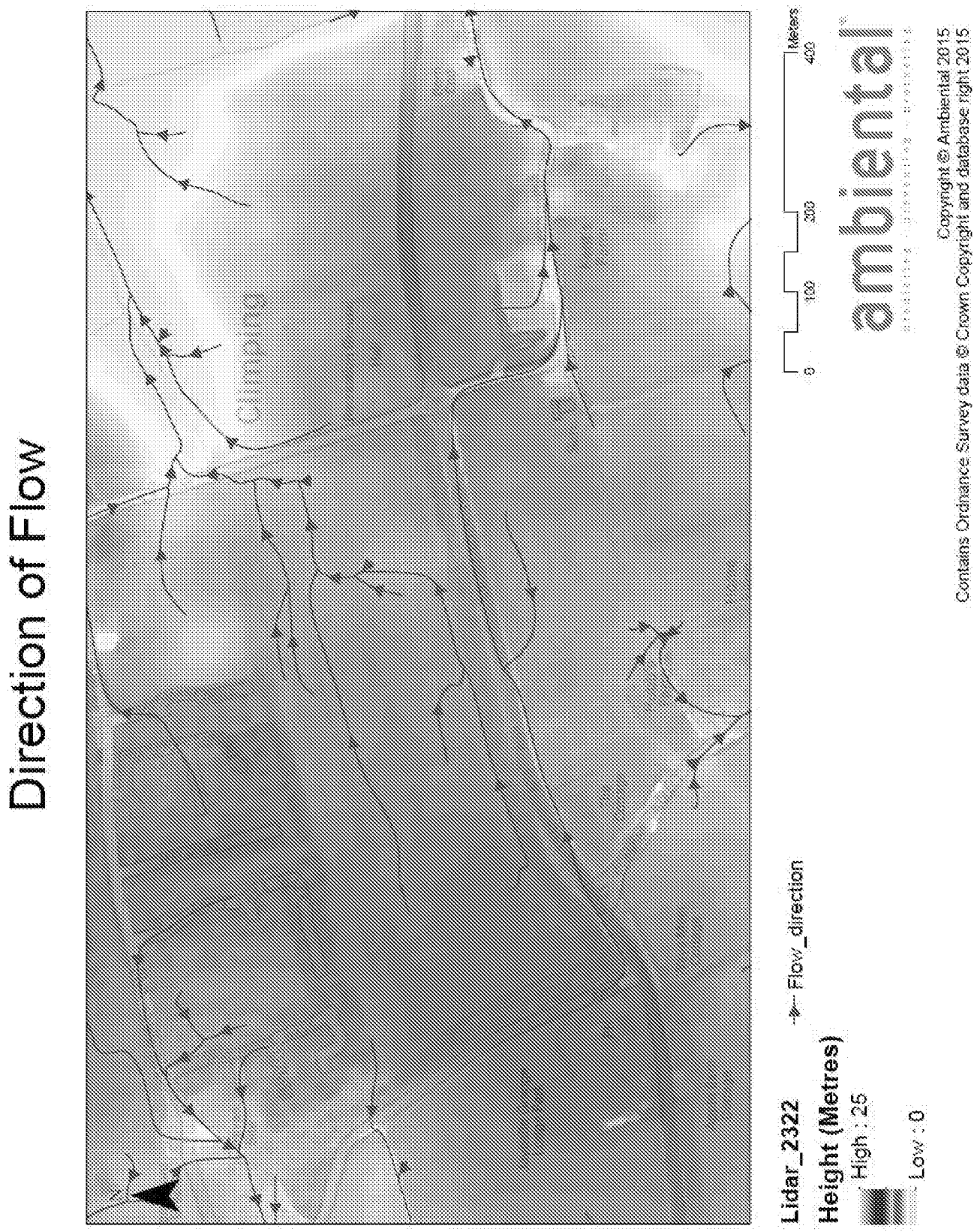
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Appendix A; Figure 1 - Masterplan (Source: jrp)

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Appendix A; Figure 2 - Direction of surface water (Source: Ambiental)

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Appendix 2 – Information

Surface Water Runoff Calculation Method

Rainfall data has been extracted from the FEH CD-ROM for the several storm duration events for a number of return periods, including 1:1.01 year, 1:10 year and 1:100 year storm events. These return periods are industry standard, however it is important to be aware that return periods less than 1:2 years are not considered reliable and should not be used in detailed design calculations.

The 1:100 year with an allowance for climate change has been based on a 30% increase to the 1:100 year rainfall intensity and not the rainfall depth. This is to provide the most conservative runoff rates for the site possible.

Greenfield runoff rates have been calculated using The Institute of Hydrology Report 124 Marshall and Bayliss, 1994 method, as recommended in the *CIRIA 697 'The SUDS Manual'*. In keeping with standard practice, the calculations are based on calculating the Greenfield runoff rates for a 50 Ha site and then factored to account for the actual site size.

Impermeable runoff rates have been calculated using the Modified Rational Method for the impermeable surfaces on site only.

Throughout the calculations a weighted co-efficient has been used, allowing different materials of surface covering on site to be taken into account.

These runoff rates have then been combined to provide the most accurate runoff rate possible for both the existing and proposed site.

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Appendix 3 – Calculations

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Date 05/12/2016 12:37 File	Designed by mark.kaumann Checked by	
XP Solutions	Source Control 2016.1	

ICP SUDS Mean Annual Flood

Input

Return Period (years): 1 Soil 0.400
Area (ha): 31.899 Urban 0.900
SAAR (mm): 728 Region Number Region 7

Results	l/s
QBAR Rural	76.6
QBAR Urban	76.6
Q1 year	66.1
Q1 year	65.1
Q30 years	173.6
Q100 years	248.3

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Science Park Square Brighton BN1 9SB	Land at Clympling Initial Storage Assessment	
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Summary of Results for 100 year Return Period (+40%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	3.153	0.653	76.6	1038.6	O K
30 min Summer	3.348	0.848	76.6	1370.1	O K
60 min Summer	3.534	1.034	76.6	1702.8	O K
120 min Summer	3.683	1.183	76.6	1973.6	O K
180 min Summer	3.782	1.282	76.6	2064.3	O K
240 min Summer	3.838	1.338	76.6	2076.6	O K
360 min Summer	3.926	1.426	76.6	2062.4	O K
480 min Summer	3.981	1.481	76.6	2086.4	O K
600 min Summer	4.018	1.518	76.6	1946.3	O K
720 min Summer	4.031	1.531	76.6	1878.2	O K
960 min Summer	3.992	1.492	76.6	1734.1	O K
1440 min Summer	3.832	1.332	76.6	1461.6	O K
2160 min Summer	3.588	1.088	76.6	1081.7	O K
2880 min Summer	3.408	0.908	76.1	832.0	O K
4320 min Summer	3.214	0.714	72.3	548.6	O K
5760 min Summer	3.097	0.597	61.8	457.1	O K
7200 min Summer	3.026	0.526	53.1	406.8	O K
8640 min Summer	2.991	0.491	46.4	368.3	O K
10080 min Summer	2.973	0.473	41.3	341.6	O K
15 min Winter	3.238	0.738	76.6	1163.1	O K
30 min Winter	3.448	0.948	76.6	1549.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	124.603	0.0	1071.7	22
30 min Summer	83.789	0.0	1548.2	34
60 min Summer	53.779	0.0	1882.2	64
120 min Summer	33.337	0.0	2336.9	122
180 min Summer	24.432	0.0	2611.5	182
240 min Summer	20.006	0.0	2805.7	228
360 min Summer	14.708	0.0	3054.8	284
480 min Summer	11.818	0.0	3310.3	346
600 min Summer	9.966	0.0	3495.9	416
720 min Summer	8.662	0.0	3647.8	482
960 min Summer	6.943	0.0	3897.2	616
1440 min Summer	5.071	0.0	4767.9	876
2160 min Summer	3.698	0.0	4681.3	1260
2880 min Summer	2.949	0.0	4878.9	1588
4320 min Summer	2.162	0.0	5416.1	2268
5760 min Summer	1.745	0.0	5762.6	2944
7200 min Summer	1.477	0.0	6024.4	3672
8640 min Summer	1.295	0.0	6258.8	4408
10080 min Summer	1.163	0.0	6455.8	5136
15 min Winter	124.603	0.0	1307.9	23
30 min Winter	83.789	0.0	1884.5	36

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

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
60 min Winter	3.858	1.156	76.6	1827.4	O K
120 min Winter	3.834	1.334	76.6	2250.7	O K
180 min Winter	3.802	1.402	76.6	2394.8	O K
240 min Winter	3.771	1.423	76.6	2416.8	O K
360 min Winter	3.703	1.402	76.6	2383.7	O K
480 min Winter	3.658	1.359	76.6	2307.6	O K
600 min Winter	3.620	1.309	76.6	2208.2	O K
720 min Winter	3.592	1.252	76.6	2108.5	O K
960 min Winter	3.557	1.123	76.6	1876.4	O K
1440 min Winter	3.578	0.879	76.6	1425.6	O K
2160 min Winter	3.673	0.573	76.6	904.5	O K
2880 min Winter	3.687	0.387	73.3	606.8	O K
4320 min Winter	3.782	0.282	58.2	434.6	O K
5760 min Winter	3.732	0.242	46.7	371.0	O K
7200 min Winter	3.716	0.216	39.3	331.2	O K
8640 min Winter	3.858	0.196	34.6	303.2	O K
10080 min Winter	3.689	0.185	30.2	281.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	53.779	0.0	2109.4	64
120 min Winter	32.337	0.0	2819.7	122
180 min Winter	24.832	0.0	2925.9	178
240 min Winter	20.566	0.0	3138.7	234
360 min Winter	14.796	0.0	3807.2	340
480 min Winter	11.818	0.0	3715.8	382
600 min Winter	9.968	0.0	3916.7	498
720 min Winter	8.685	0.0	4086.9	528
960 min Winter	6.943	0.0	4368.5	672
1440 min Winter	5.071	0.0	4782.5	939
2160 min Winter	3.696	0.0	5246.2	1284
2880 min Winter	2.949	0.0	5577.9	1588
4320 min Winter	2.142	0.0	6058.6	2248
5760 min Winter	1.702	0.0	6686.6	2944
7200 min Winter	1.427	0.0	6738.9	3680
8640 min Winter	1.235	0.0	7011.1	4404
10080 min Winter	1.093	0.0	7236.1	5144

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Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.000	Shortest Storm (mins)	15
Ratio R	0.350	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 4.700

Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)
0	4	2.350	4	8	2.350

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Model Details

Storage is Online Cover Level (m) 4.500

Tank or Pond Structure

Invert Level (m) 0.500

Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	1500.0	1.300	1803.1	3.000	2367.6	4.700	2874.9
0.100	1535.4	1.400	1871.5	3.600	2416.9	4.800	2951.8
0.300	1631.6	1.800	2035.0	3.200	2507.4	4.900	3029.0
0.600	1668.8	2.000	2066.4	3.400	2578.8	4.900	3107.5
0.800	1727.7	2.200	2164.9	3.600	2651.4	5.000	3187.1
1.000	1787.8	2.400	2231.4	3.800	2726.3		
1.200	1847.6	2.600	2298.7	4.000	2799.8		

Hydro-Brake Optimum® Outflow Control

Unit Reference	MD-SHE-0340-7600-2000-7600
Design Head (m)	2.000
Design Flow (l/s)	76.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	340
Invert Level (m)	0.500
Minimum Outlet Pipe Diameter (mm)	375
Suggested Manhole Diameter (mm)	Site Specific Design (Contact Hydro International)

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	76.6
Flush-Flo™	0.616	75.6
Kick-Flo®	1.381	64.0
Mean Flow over Head Range	-	65.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake Optimum® as specified. Should another type of control device other than a Hydro-Brake Optimum® be utilised then these storage routing calculations will be invalidated

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	38.0	1.200	70.9	3.000	95.2	7.000	140.8
0.200	54.5	1.400	64.5	3.500	100.5	7.500	145.7
0.300	62.7	1.600	68.8	4.000	107.1	8.000	150.4
0.400	73.4	1.800	72.8	4.500	113.6	8.500	154.8
0.500	75.8	2.000	76.6	5.000	119.5	9.000	159.3
0.600	76.6	2.200	80.2	5.500	125.3	9.500	163.6
0.800	76.0	2.400	83.7	6.000	130.7		
1.000	74.3	2.600	87.0	6.500	135.9		

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Appendix 4 – Surface water drainage strategy

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