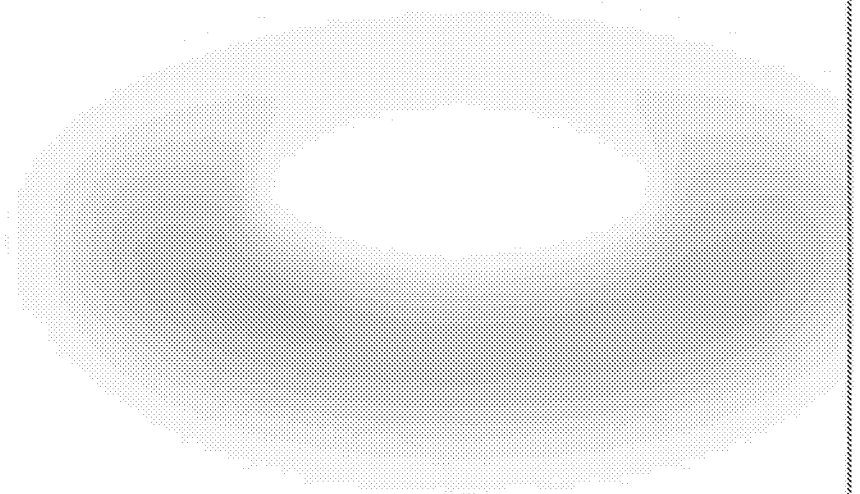




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Project: Surface Water Drainage Assessment Report (SWDS)
Prepared for: Parker Dann
Reference: 2322
Date: Dec 2016
Version: Draft v4.0



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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Contents

Contents	3
1. Introduction	4
Development Proposal	4
2. Development Description and Site Area	5
Existing Drainage Infrastructure	5
Existing Ground Conditions	6
Groundwater	8
Nearby Watercourses and Drainage	8
3. Runoff Rates and Storage Volumes	9
Interception Storage	10
Long Term Storage	10
Total Storage Required	10
4. SuDS Suitability Analysis	11
Source Control Components	12
Swales and Conveyancing Channel Components	12
Filtration Components	13
Retention and Detention Components	13
Infiltration Components	14
Wetlands	15
Other Structures	15
5. Conclusions	16
Appendix 1 – Figures	17
Appendix 2 – Information	20
Surface Water Runoff Calculation Method	20
Appendix 3 – Calculations	21
Appendix 4 – Surface water drainage strategy	27

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)

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.

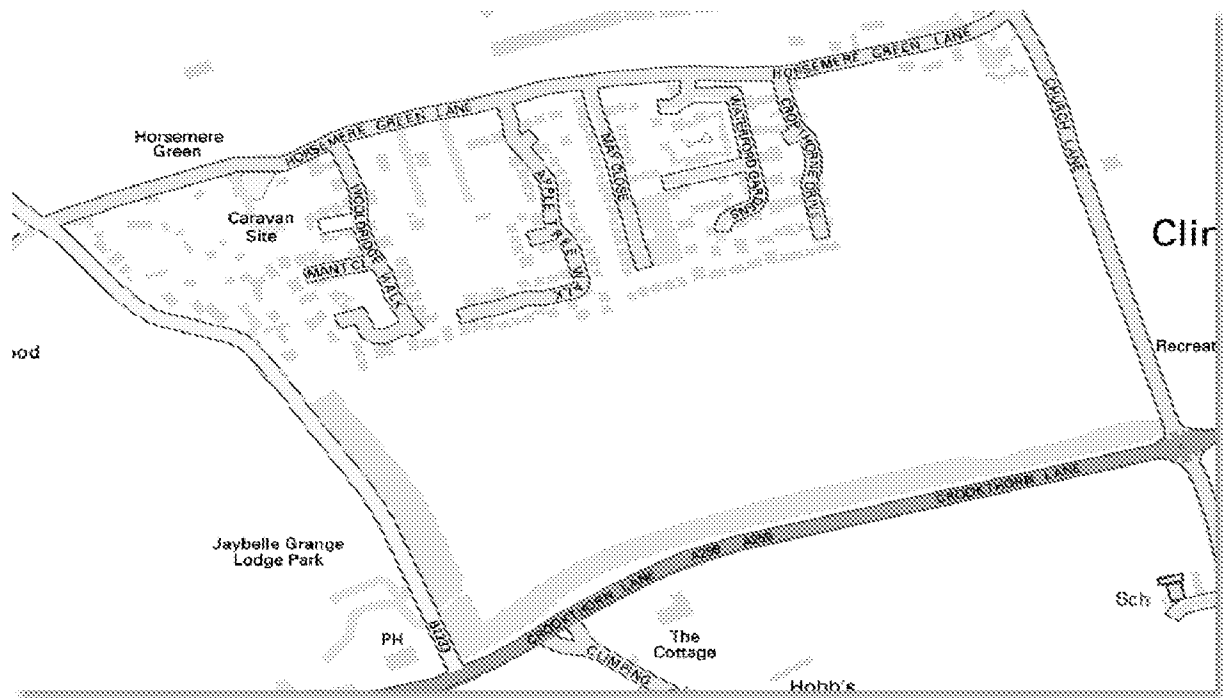


Figure 2: Wider site location (Source: OS)

- 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.

- 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 ditch located in this area.

Direction of Flow

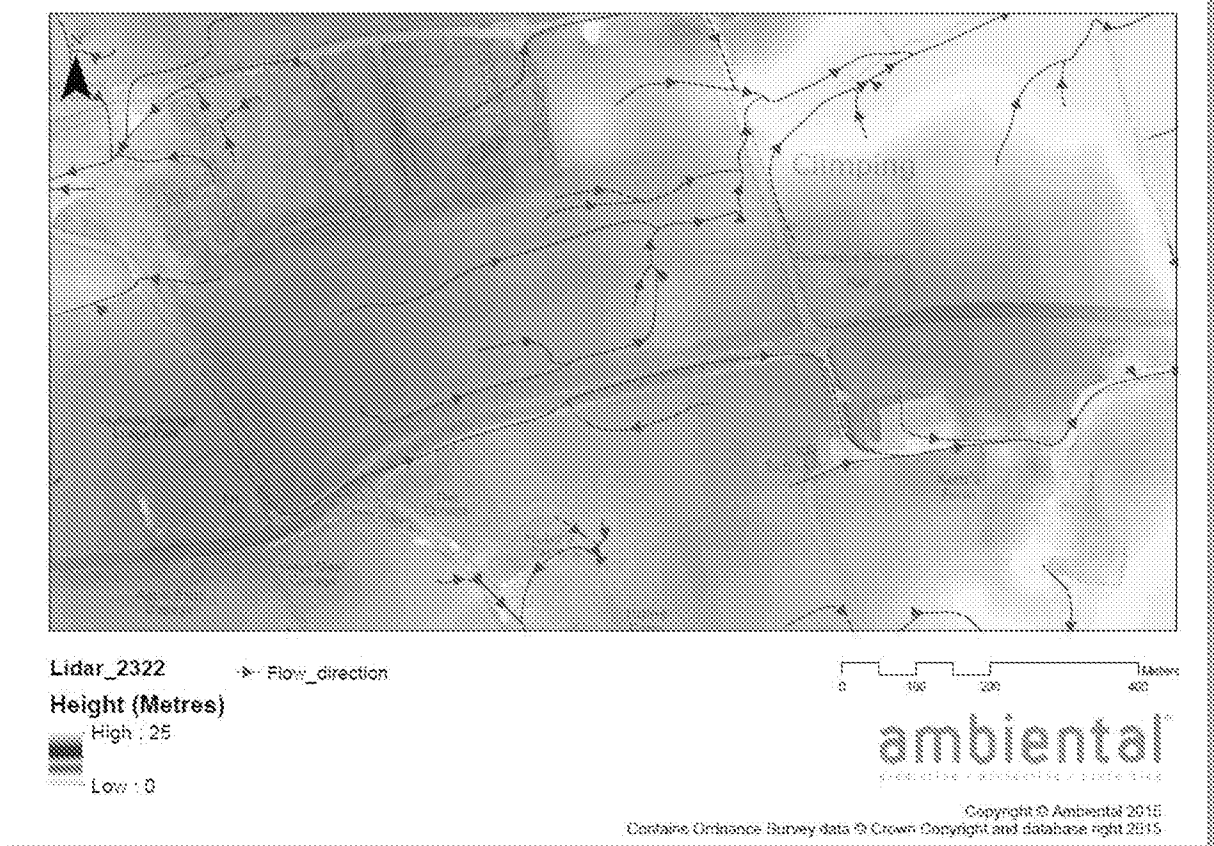


Figure 3: General surface water flow directions across the development site (Source: Ambiental, LIDAR)

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.

- 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.

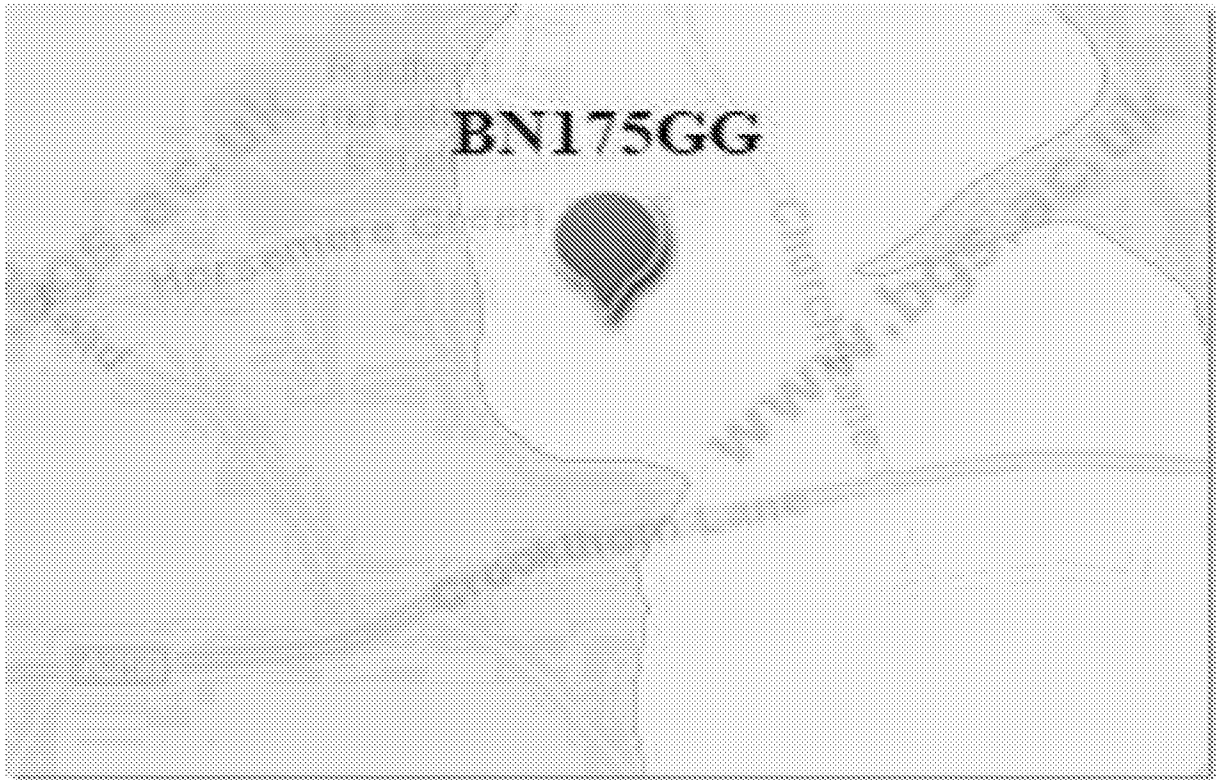


Figure 4: Geology of site; orange shows river terrace deposits, yellow shows beach deposits (Source: BGS)

- 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.

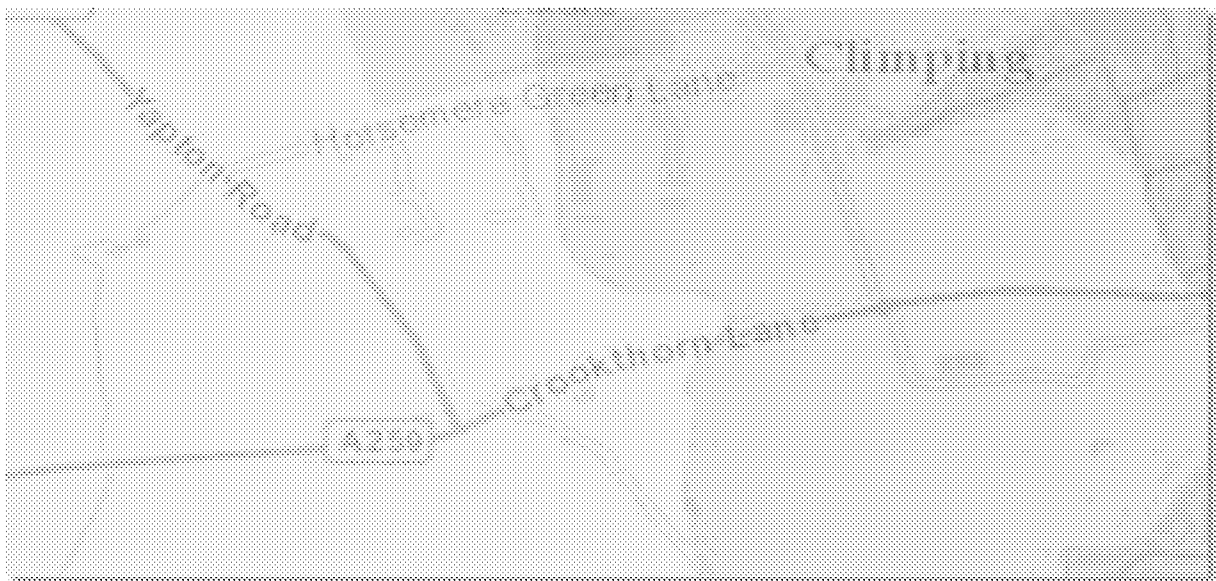


Figure 5: Soils map of the site; green shows sand to loam and blue shows sand to sandy loam (Source: UKSO)

- 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.

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
10	106.05	174.46	Runoff rate is increased	68.41
30	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

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
10	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 260m³ 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 3143m³ 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

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 2050m² 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.

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 Roofs	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 runoff is installed on the network prior to outfall to an unlined swale or channel.

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 contaminants 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

- 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 conditions
Infiltration Trench	Suitable	Suitable	Good	Poor	Unsuitable	Potentially unsuitable due to ground conditions.
Infiltration Basin	Suitable	Suitable	Good	Good	Unsuitable	Potentially unsuitable due to ground conditions
Rain Gardens	Suitable	Suitable	Medium	Good	Unsuitable	Potentially unsuitable due to ground conditions

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 runoff is installed on the network prior to outfall to an infiltration medium.
- 4.10 **An outline drainage strategy drawing is included in Appendix 4.**

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.

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

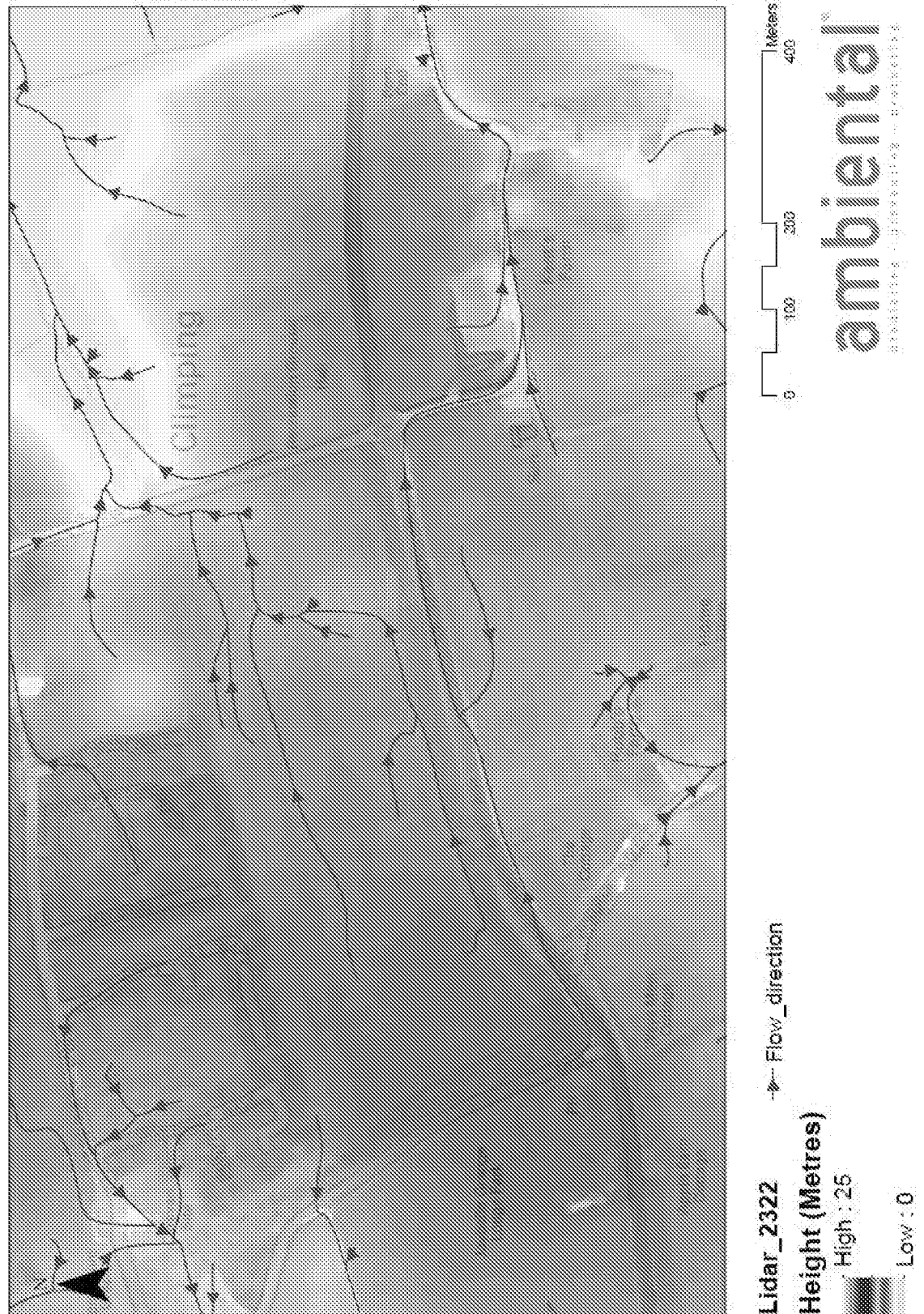
Appendix 1 – Figures

Appendix A; Figure 1 - Masterplan (Source: jrp)	18
Appendix A; Figure 2 - Direction of surface water (Source: Ambiental)	19



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

Direction of Flow



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

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.

Appendix 3 – Calculations

Residential		Page 1
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ICR S008 Mean Annual Flood

Input

Return Period (years) 1 Soil: 8.400

Area (ha): 21.500 Urban: 8.000

Area (km²): 0.0215 ICR Region: Region 1

Results 1/s

Mean Annual Flood: 78.6

Urban: 78.6

25 year: 88.1

50 year: 95.1

100 years: 108.6

200 years: 124.3

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Brighton	Initial Storage Assessment
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Summary of Results for 100 Year Return Period (+40s)

Storm Event	Max Level (m)	Max Depth (m)	Max Control Volume (l/s)	Max Volume (m³)	Status
15 min Summer	3.153	0.853	78.6	1838.9	O E
20 min Summer	3.134	0.838	78.6	1878.1	O E
30 min Summer	3.114	1.034	78.6	1963.0	O E
120 min Summer	3.083	1.183	78.6	1973.6	O E
180 min Summer	3.032	1.132	78.6	2004.3	O E
240 min Summer	3.038	1.438	78.6	2078.0	O E
360 min Summer	3.024	1.224	78.6	2052.4	O E
480 min Summer	3.001	1.201	78.6	2066.4	O E
600 min Summer	3.008	1.108	78.6	1996.3	O E
720 min Summer	3.001	1.131	78.6	1874.3	O E
960 min Summer	3.052	1.052	78.6	1734.1	O E
1440 min Summer	3.032	0.832	78.6	1451.8	O E
2160 min Summer	3.184	0.684	78.6	1081.7	O E
3600 min Summer	3.028	0.528	78.1	833.0	O E
4320 min Summer	2.801	0.384	72.3	548.6	O E
5760 min Summer	2.757	0.297	81.8	427.1	O E
7200 min Summer	2.764	0.264	53.1	406.0	O E
8640 min Summer	2.741	0.241	45.4	368.3	O E
10080 min Summer	2.703	0.203	43.3	343.3	O E
15 min Winter	3.238	0.738	78.6	1169.1	O E
20 min Winter	3.148	0.848	78.6	1349.3	O E

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	124.633	0.0	1871.7	20
20 min Summer	83.785	0.0	1648.3	30
30 min Summer	52.779	0.0	1892.3	60
100 min Summer	33.337	0.0	2336.8	100
180 min Summer	24.932	0.0	2611.3	180
240 min Summer	20.366	0.0	2885.7	240
360 min Summer	14.768	0.0	3054.8	360
480 min Summer	11.818	0.0	3316.3	480
600 min Summer	9.966	0.0	3485.9	600
720 min Summer	8.662	0.0	3647.8	720
960 min Summer	6.341	0.0	3857.2	960
1440 min Summer	5.071	0.0	4057.9	1440
2160 min Summer	3.196	0.0	4391.3	2160
3600 min Summer	2.548	0.0	4878.9	3600
4320 min Summer	2.143	0.0	5418.1	4320
5760 min Summer	1.703	0.0	5762.8	5760
7200 min Summer	1.403	0.0	6079.4	7200
8640 min Summer	1.238	0.0	6256.3	8640
10080 min Summer	1.063	0.0	6450.8	10080
15 min Winter	124.633	0.0	1202.9	20
20 min Winter	83.785	0.0	1414.8	30

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Page 2

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Land at Clipping

Brighton

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 11/s	Max Volume (m³)	Status
60 min Winter	3.858	1.156	78.6	1822.4	O K
120 min Winter	3.854	1.334	78.6	1858.7	O K
180 min Winter	3.852	1.402	78.6	1864.0	O K
240 min Winter	3.852	1.424	78.6	1865.8	O K
360 min Winter	3.852	1.432	78.6	1867.7	O K
480 min Winter	3.852	1.439	78.6	1867.9	O K
600 min Winter	3.852	1.435	78.6	1868.2	O K
720 min Winter	3.752	1.352	78.6	1868.5	O K
840 min Winter	3.652	1.227	78.6	1870.4	O K
1440 min Winter	3.378	0.875	78.6	1825.9	O K
2160 min Winter	3.073	0.573	78.6	1804.5	O K
2880 min Winter	2.887	0.387	78.6	1800.8	O K
3600 min Winter	2.782	0.282	88.2	1834.6	O K
4320 min Winter	2.742	0.242	46.7	1711.0	O K
7200 min Winter	2.736	0.236	39.3	1311.2	O K
8640 min Winter	2.898	0.398	34.8	1002.2	O K
10080 min Winter	2.683	0.183	30.2	681.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	33.775	0.0	2159.4	64
120 min Winter	33.835	0.0	2817.7	122
180 min Winter	24.822	0.0	2925.8	178
240 min Winter	28.004	0.0	3133.7	234
360 min Winter	14.766	0.0	3647.2	340
480 min Winter	11.818	0.0	3710.4	400
600 min Winter	9.888	0.0	3919.7	458
720 min Winter	8.905	0.0	4086.8	508
840 min Winter	8.943	0.0	4268.3	472
1440 min Winter	5.071	0.0	4780.5	838
2160 min Winter	2.698	0.0	5248.2	1204
2880 min Winter	2.949	0.0	5577.8	1588
4320 min Winter	2.142	0.0	6068.9	2348
5760 min Winter	1.762	0.0	6624.8	2944
7200 min Winter	1.827	0.0	6782.9	3640
8640 min Winter	1.235	0.0	7011.1	4408
10080 min Winter	1.093	0.0	7234.1	5148

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Ambiental		Page 3
Science Park Square	Land at Olymping	
Brighton	Initial Storage Assessment	
SNI 908		
Date 05/12/2016 12:36	Designed by us	
File	Checked by	
XP Solutions	Source Control 2016.1	

Rainfall Details

Rainfall Model	FR	Wettest Month	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.845
W-60 (mm)	15.000	Shortest Storm (mins)	15
W-10 (mm)	0.350	Longest Storm (mins)	10000
Summer Storms	Yes	Climate Change %	+40

Time Area Chart

Total Area (ha): 4.700

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

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Additional		Page 4
Science Park Square Brighton BN1 9GB	Land at Clymping Initial Storage Assessment	
Date 05/12/2016 12:36	Designed by ns	
File	Checked by	
XP Solutions	Source Control 2016.1	

Model Details

Storage is to rise over level (m) 4.500

Tank or Pond Structure

Invert Level (m) 2.500

Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)	Depth (m)	Area (m ²)
0.000	1896.0	1.400	1929.1	2.800	2367.4	4.200	2872.8
0.200	1935.1	1.600	1971.3	3.000	2418.9	4.400	2931.8
0.400	1981.8	1.800	2025.0	3.200	2477.3	4.600	3003.9
0.600	1659.3	2.000	2086.4	3.400	2578.8	4.800	3107.5
0.800	1727.7	2.200	2164.9	3.600	2651.4	5.000	3237.1
1.000	1787.3	2.400	2231.4	3.800	2724.9		
1.200	1847.5	2.600	2298.5	4.000	2799.4		

Hydro-Brake Optimiser Outflow Control

Unit Reference	MD-SHS-0140-1668-0004-1860
Design Head (m)	2.000
Design Flow (l/s)	76.6
Flush-Flow	Calculation
Objective	Minimise upstream storage
Application	Surface
Segg Available	250
Diameter (mm)	340
Invert Level (m)	2.500
Minimum Joints Pipe Diameter (mm)	375
Proposed Manhole Diameter (mm)	375
	Site Specific Design (Contact Hydro International)

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.000	76.6
Flush-Flow	0.636	76.6
Back-Flow	1.381	61.0
Keep Flow over Head Range	-	65.3

The hydrological calculations have been based on the Head/Discharge relationship for the Hydro-Brake Optimiser as specified. Should another type of control device other than a Hydro-Brake Optimiser be utilized 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	18.0	1.700	70.8	3.300	65.0	7.000	140.9
0.200	24.5	1.800	64.5	3.500	100.5	7.500	145.7
0.300	62.7	1.900	68.8	3.600	107.3	8.000	150.4
0.400	73.3	2.000	72.8	3.700	112.6	8.500	154.8
0.500	76.3	2.100	76.6	3.800	116.8	9.000	159.3
0.600	76.3	2.200	80.2	3.900	120.2	9.500	163.6
0.800	76.0	2.400	83.7	4.000	123.7		
1.000	74.2	2.600	87.0	4.500	135.0		

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Appendix 4 – Surface water drainage strategy

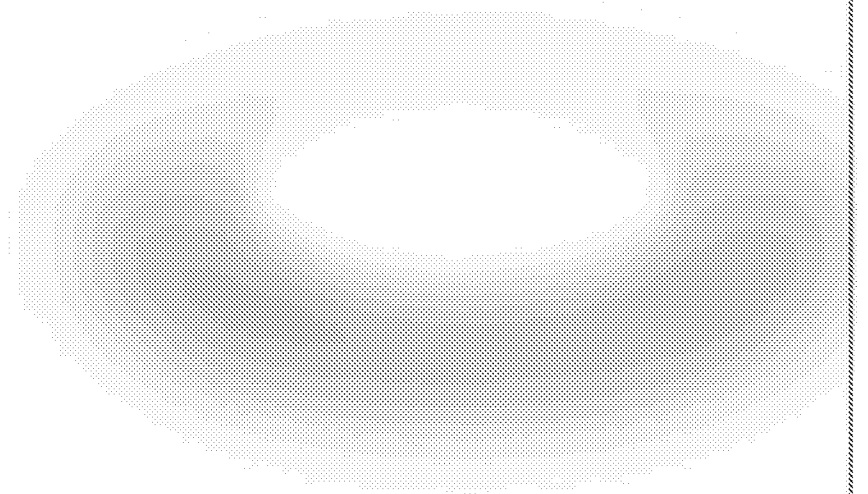




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predicting - preventing - protecting

Project: 2495
Prepared for: Parker Dann Ltd
Date: December 2016
Version: Final v2.0



Document Issue Record

Project: 2495**Prepared for:** Parker Dann Ltd**Site Location:** Land at Clymping, West Sussex

Title:		Foul Water Strategy Technical Note			
Written By:	N. Jackson	Checked By:	D. Frosoni	Authorised By:	D. Frosoni
	MSc		CEng MCIWEM MICE		CEng MCIWEM MICE

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Contents

Document Issue Record 1

 Please Note: 1

 Contact Us:..... 1

Contents.....2

1.0 Introduction 3

2.0 Foul Water Strategy 3

1.0 Introduction

- 1.1 This *Technical Note* has been prepared by Cole Easdon Consultants on behalf of Ambiental Technical Solutions, in support of a planning application for development at land at Clymping, West Sussex.

2.0 Foul Water Strategy

- 2.1 The application site comprises some 270,908m² (27.091ha) of greenfield land located to the north of Crookthorn Lane. The development proposal comprises 300 dwellings.
- 2.2 Sewer records provided by Southern Water indicate the presence of a foul water rising main and a treated effluent rising main sewer flowing in a southerly and then easterly direction across the development site. A second foul water rising main crosses the south westerly site extent in a south easterly direction. Public foul water sewers are present to the south of the site within Crookthorn Lane and to the north of the site within the existing residential development and running along Horsemere Green Lane. Refer to the sewer records enclosed with this Report.
- 2.3 It is proposed to discharge foul water from the site to the existing foul water manhole (0302) to the north of the site within Horsemere Green Lane. No topographical survey has been provided however a review of the sewer records provided indicates that a pumping station may be required in order to connect to the existing sewer. A typical pumping station will require a compound area of 8m x 12m with a 15m offset distant from any habitable buildings to ensure protection from noise, odour or nuisance. The pumping station should have a safe and reasonable vehicular access. As far as practicable, the rising main from the pumping station should be laid in highways or public open space and be laid a minimum of 3m from any building or structure. An indicative rising main route is indicated on CEC Plan 4983/501 [*Proposed Foul Water Drainage*], enclosed with this Report.
- 2.4 It is proposed that all on-site foul water sewers and pumping stations will be offered for adoption by SW. The rising main will terminate at a point to be determined within Horsemere Green Lane, allowing foul flows to gravitate to the existing public foul water sewer manhole over a minimum 5m length. The extent of rising main will be determined during detailed design.
- 2.5 A pre-capacity check was submitted in February 2016 for 140 dwellings found that there was inadequate capacity within the foul water network to accommodate post development flow (6.5 l/s). Improvements to the existing network will be required in order to provide capacity in the future. Alternatively, a direct connection could be made to the Ford Watewater Treatment Works some 920m distant from the site. Refer to the capacity check results enclosed with this Report. The extent of works required to provide capacity for the development site can be obtained via Section 98 of the Water Industry Act 1991 (sewer requisition). This provides a legal mechanism through which appropriate infrastructure can be requested (by the Developer)

and provided to drain a specific location, and includes any offsite reinforcement works in order to provide capacity for the proposed foul flow to be accommodated within the local sewerage system.

- 2.6 Alternatively, a private package treatment works could be implemented on the site. However, the suitability of such a facility would need to be assessed. A suitable discharge point to ground or watercourse would need to be identified for treated effluent. A nonmains assessment would need to accompany the planning submission. Environment Agency consent (post planning) would also be required.

Cole Easdon Consultants Limited

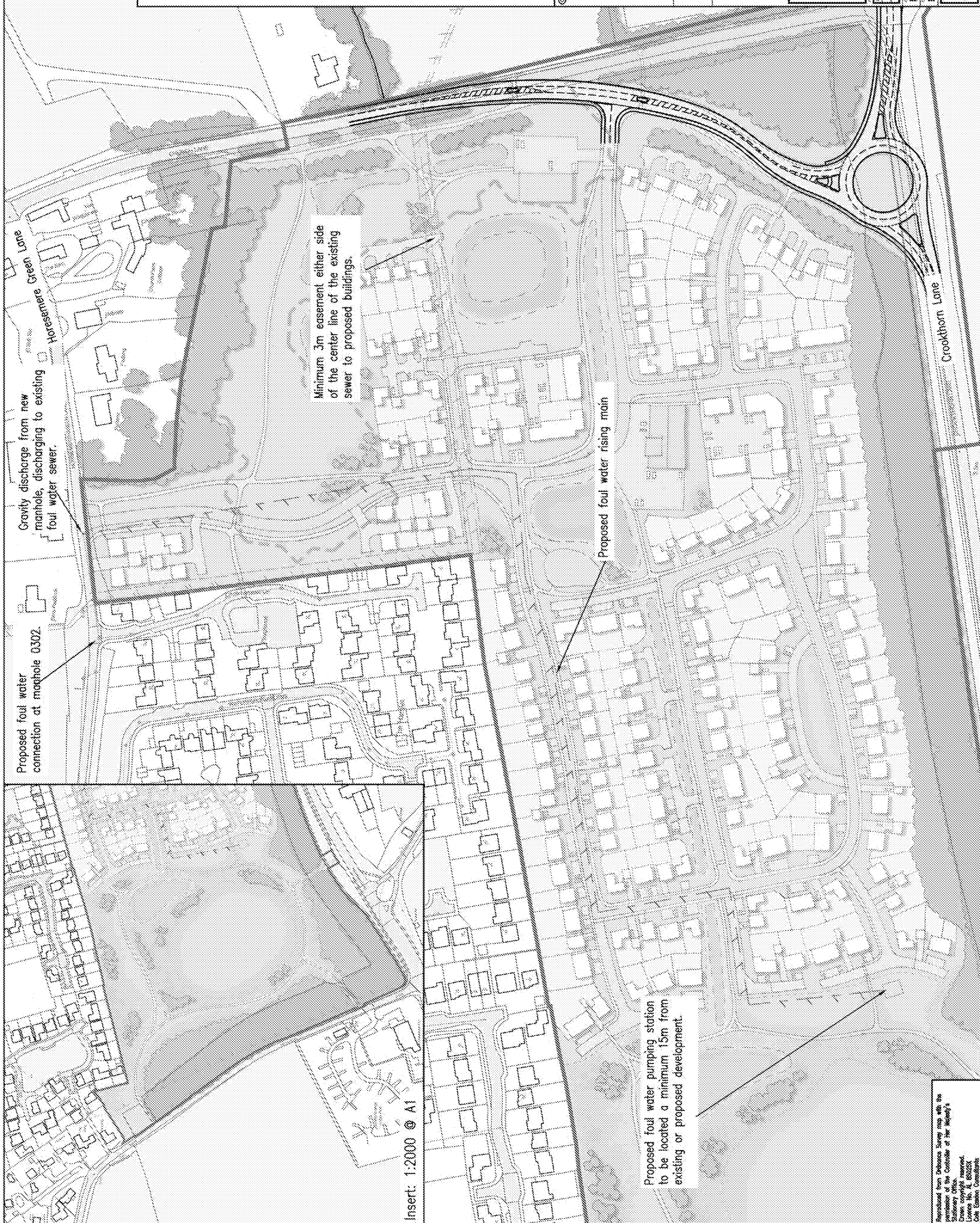
December 2016

Enclosures

CEC Plan 4983/501 Proposed Foul Water Drainage

Southern Water Sewer Records

Southern Water Capacity Check



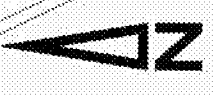
Proposed foul water connection at manhole 0302.

Gravity discharge from new manhole, discharging to existing foul water sewer.

Minimum 3m easement either side of the center line of the existing sewer to proposed buildings.

Proposed foul water rising main

Proposed foul water pumping station to be located a minimum 15m from existing or proposed development.



- Key**
- Site boundary (270808m²)
 - Existing public foul water sewer
 - Existing public foul water rising main
 - Proposed foul water rising main

Notes:

1. All levels are in metres above ordnance datum.
2. All dimensions are in millimetres unless stated otherwise.
3. Sewer records provided by Southern Water.



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**COLE EASDON
CONSULTANTS**

Ambiental Technical Solutions

Proposed Development
Land at Clymping
West Sussex

Proposed Foul Water Drainage

DATE	11/11/2016	SCALE	1:1000 @ A1
NO.	4963/501	REV.	
DATE	11/11/2016	SCALE	1:1000 @ A1
NO.	4963/501	REV.	

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Cole Easdon Consultants





4983
Rec'd 2/3/16

1/2

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Developer Services
Southern Water
Sparrowgrove House
Sparrowgrove
Otterbourne
Hampshire
SO21 2SW

Tel: **0330 303 0119**
Email: **developerservices@southernwater.co.uk**

Your Ref:

Our Ref:

CC/002924

Date:

29 February 2016

F.A.O: Ms. Jess Pockett,

Site: Land South of Horsemere Green Lane, Climping, West Sussex, BN17 5GG.

Dear Madam,

We have completed the capacity check for the above development site and the results are as follows:-

Foul capacity check:

There is currently inadequate capacity within the foul sewerage network to accommodate a foul flow of 6.5 l/s at manhole reference TQ00020302. The proposed development would increase flows to the public sewerage system, and existing properties and land may be subject to a greater risk of flooding as a result. Additional off-site sewers, or improvements to existing sewers, will be required to provide sufficient capacity to service the development.

The nearest point where capacity is currently available is at Ford WTW which is located approximately 920 m North West of the proposed development site. Rights are not issued for a direct connection to Wastewater Treatment Works. Please note that connection to the WTW will have to be agreed by Southern Water Services before being carried out.

Section 98 of the Water Industry Act 1991 provides a legal mechanism through which the appropriate infrastructure can be requested (by the developer) and provided to drain a specific location. Alternatively, carry out off site reinforcement works in order to provide

capacity for the proposed foul flow to be accommodated within the local sewerage system. The extent of the works required to provide capacity for the development site could be obtained via a level 2 Capacity Assessment (Application forms enclosed).

Unfortunately, it may be several years before adequate infrastructure is provided via a prioritised capital expenditure programme. Therefore, this proposed development would be considered premature until such time adequate sewerage infrastructure could be provided.

Before any connections are made to foul and surface sewerage network, an application form needs to be completed and approved by Southern Water Services.

Should you require any further information, please contact us at the above mentioned phone number or address.

Yours faithfully,

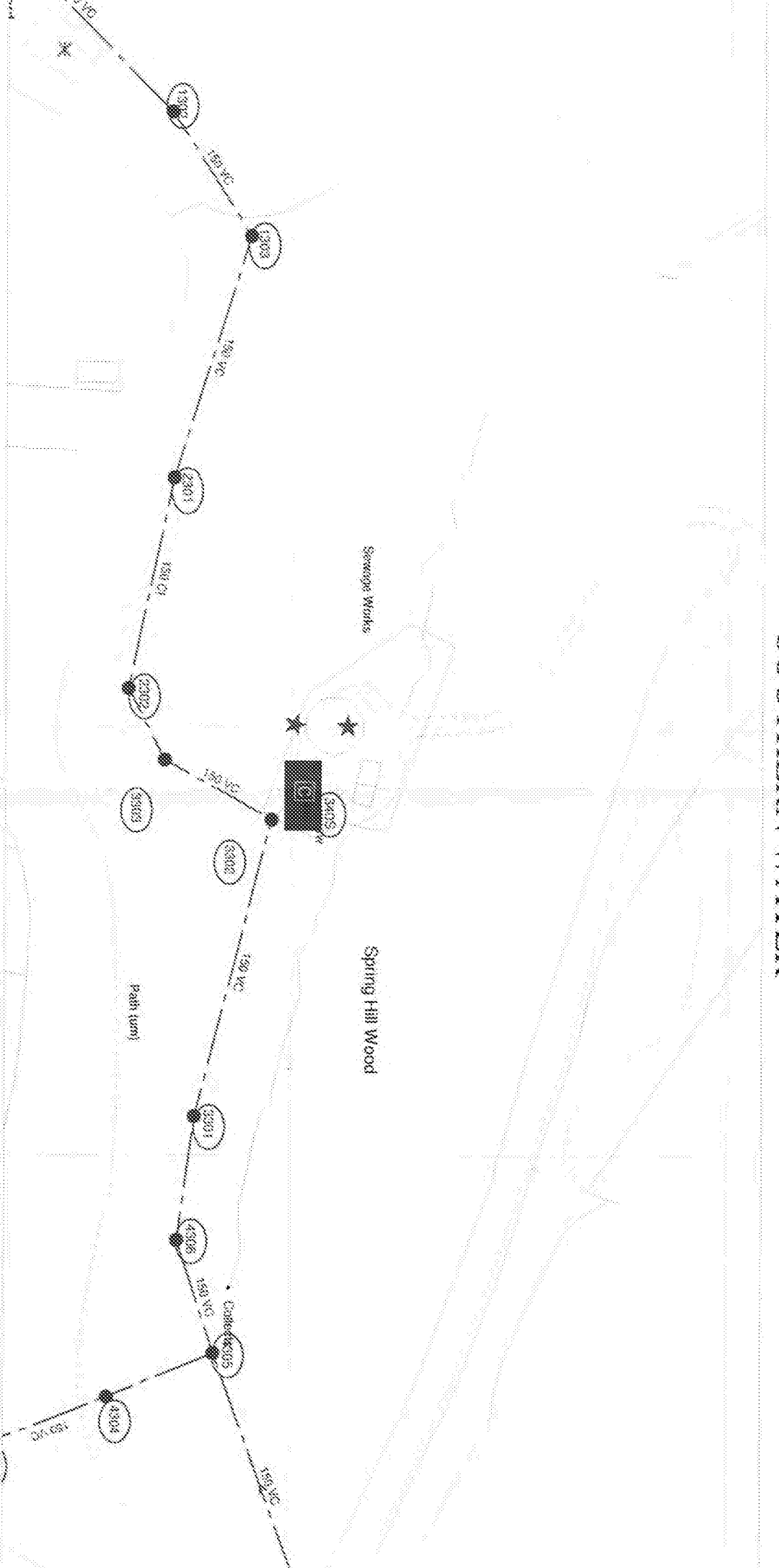


Developer services

Please note: -

The information provided above does not grant approval for any designs/drawings submitted for the capacity analysis. The results quoted above are only valid for 12 months from the date of issue of this letter.

SOUTHERN WATER



The position of pipes shown on this plan are believed to be correct, but Southern Water Systems Ltd accepts no responsibility in the event of inaccuracy. The actual position should be determined on site.

Based upon reference: Survey Report Data with the permission of the controller of HM Coastguard, Report No. W11 2005/10

O.S. REF: TOS240SW

Screen Print

Scale: 1:1250



Printed By: krishna
Nearest point of available capacity
Requested By:

Date: 29-2-2016