Flood Risk Assessment

Proposed Residential Development
Rendlesham, Suffolk
AMA647 Rev 0
31 May 2018

Prepared for: Capital Community Developments Ltd

amazi

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1 Introduction

1.1 Amazi Consulting Ltd has been instructed by Capital Community Developments Ltd to prepare this Flood Risk Assessment (FRA) associated with the proposed residential development at land north of Garden Square, Rendlesham, Woodbridge, Suffolk IP12 2GW.

1.2 This FRA has been prepared in accordance with National Planning Policy Framework (NPPF), March 2012, and its accompanying gov.uk Planning Practice Guidance (PPG): *Flood Risk and Coastal Change*. It is expected that this report will be reviewed by the relevant authorities as part of the documentation submitted for planning permission, and the reader will have some understanding of the technical issues relating to development and flood risk.

1.3 This FRA report has been prepared for the sole use of Capital Community Developments Ltd and its contents cannot be copied or relied upon by others without the written authority of Amazi Consulting Ltd.

2 Site Description

2.1 Existing

2.1.1 The development site is centred at approximate Ordnance Survey (OS) national grid reference 633740 mE, 253800 mN. The boundary of the site and its surroundings are shown on the plan in Appendix A. A typical view of the site is shown in Photograph 1 (all photographs in this report taken on 28 September 2017).

Photograph 1 – Panoramic view across site from north boundary

2.1.2 This 5.05 hectare (ha) site is located on arable agricultural greenfield land to the north of the existing developed area of Rendlesham. The site is bounded to the north by woodland, fields and a sewage treatment works. To the south and east are the existing residential areas and to the west is woodland and further farmland. The site is situated within the jurisdiction of Suffolk Coastal District Council and Suffolk County Council.

2.1.3 The topographical survey of the site is given in Appendix B. Contours are shown at 0.5 m centres and the levels shown on the survey relate to OS GPS datum (mAOD). The levels within the site vary from 25.95 mAOD at the centre of the north of the site, to 24 mAOD at the south west corner.

2.1.4 The entire site is located well within Flood Zone 1*, the area considered to be at low risk of fluvial flooding from significant watercourses. The existing site use is classified as *less vulnerable* in accordance with Table 2 of the NPPF Planning Practice Guidance (Reference ID: 7-066-20140306).

*Refer to NPPF Table in Appendix J

2.2 Proposed

2.2.1 The proposed site layout is given on the drawing in Appendix C and comprises 75 residential units, vehicular access and public open spaces.

2.2.2 The site and buildings have been designed according to Vastu, a complete system of architecture to create built environments that support the health and well-being of people living in them. This ancient knowledge includes a comprehensive and well-established set of design principles aimed at restoring the connections between the individual, the building, nature and the cosmos. Around the world there is a revival of interest in vastu design and its implementation, as the system becomes more widely available under the inspiration and guidance of His Holiness Maharishi Mahesh Yogi.

2.2.3 With Maharishi Vastu architecture, buildings are oriented to face East to receive the maximum nourishing influence of the rising sun. Careful placement of rooms within a home aligns the activities of the occupants with specific qualities of the Sun's energy at different times of the day. The proportions, measurements and symmetry of a home are used in an integrated way to strengthen the connection between the individual and the cosmos. These design principles can be adapted to vernacular architecture in any part of the world, by using natural and non-toxic materials and styles suitable to local conditions, selected to protect the health of the occupants and to preserve the environment.

2.2.4 The proposed site levels will generally remain as existing, with localised changes to suit site infrastructure and building accesses. The site design (see section 2.2.2) requires proposed finished floor levels that are 450 mm above surrounding ground levels. Ramp accesses are included to building entrances to ensure level thresholds, complying with the Building Regulations.

2.2.5 The Suffolk Coastal Site Allocations and Area Specific Policies Development Plan Document (January 2017) allocates this site for housing, under site reference SSP12.

2.2.6 Current best practice indicates that the design life of a residential development should be 100 years. The year 2118 is therefore considered appropriate in assessing the possible impacts of climate change upon flood risk.

2.2.7 The proposed site use is classified as *more vulnerable* in accordance with Table 2 of the Planning Practice Guidance (Reference ID: 7-066-20140306). An extract from Table 3 of the Planning Practice Guidance (Reference ID: 7-067-20140306) is given below and indicated in red is the site's classification, confirming the appropriate nature of the proposed development.

Flood Zones	Flood Risk Vulnerability Classification				
	Essential infrastructure	Highly vulnerable	More vulnerable	Less vulnerable	Water compatible
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception Test required	✓	✓	✓
Zone 3a †	Exception Test required †	X	Exception Test required	✓	✓
Zone 3b *	Exception Test required *	X	X	X	✓*

Key:

✓ Development is appropriate

X Development should not be permitted.

3 Local Hydrology

3.1 General

3.1.1 Figure 01 shows ground levels in surrounding areas. These contours indicate that the general direction of falling local topography is north to south on site and east to west to the west of the site. This map has been created from 1m and 2 m composite DTM lidar data.

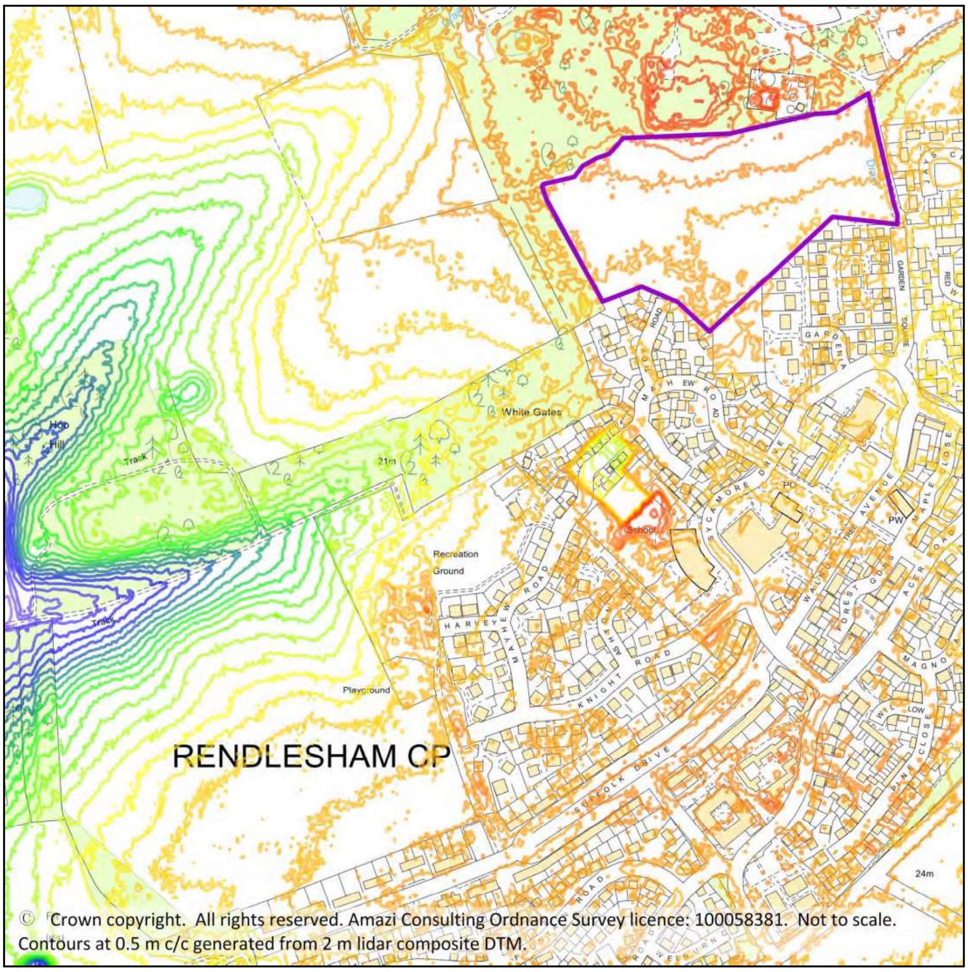


Figure 01 - Local Topography

3.1.2 The site is near to a high point in local topography which is a just north of the site. The general direction of fall is westwards towards tributaries to the River Deben. The hydrological catchment in which the site lies is shown on the Flood Estimation Handbook (FEH) map extract in Figure 02.

Figure 02 – FEH hydrological catchment

3.1.3 There is what appears to be an open watercourse in the vicinity of the site which forms the site's east boundary. This is shown in photograph 2 and was dry during our site visit. Neither the topographical mapping, the site survey, nor our thorough site observations revealed any piped in/outflow to this watercourse, or any continuation of the watercourse up or down stream of the site (was quite overgrown at the upstream end). The watercourse is not situated within a local valley. The surveyed low point in this linear feature is 1/3 from its southern extent. At its downstream end (south east corner of the site) we were able to clearly see the end to the channel at which there was no pipe or other outfall. So the watercourse is assumed to be a local soakaway feature, possibly historically associated with a former track which is shown parallel to this location on historic mapping.

Photograph 2 – Watercourse at east of site – view upstream

3.1.4 The only other feature near the site which could have a drainage function was a large ditch like area in the woodland just beyond the south west of the site. This dip in topography was very overgrown and appeared to head southwards towards the gardens of number 5 Tidy Road, but it does not seem to go anywhere.

3.2 Public Sewers

3.2.1 The Anglian Water mapping in Appendix E shows that there are several existing piped systems on site. These comprise:

- Three foul water rising mains (probably all 150 mm diameter) from the residential areas east of the site, flowing in to the Anglian water sewage treatment works at the site's north boundary.
- A 750 mm diameter public gravity surface water sewer flowing west through the site from its south east corner.
- An effluent pipe from the sewage treatment works passing through the site to the west, into which the aforementioned surface water sewer connects in the centre of the site at MH7700. This is 150 mm diameter as it exits the treatment site, but then enters the 750 mm pipe at the centre of the site.

3.2.2 When visiting site there was no visual indication of the manholes associated with the above systems, so the exact routes were initially not shown on the site topographical survey in Appendix B. Indeed, the survey only resulted in identifying a manhole in the location indicated in blue on Figure 03 which is different to those on the Anglian Water records.

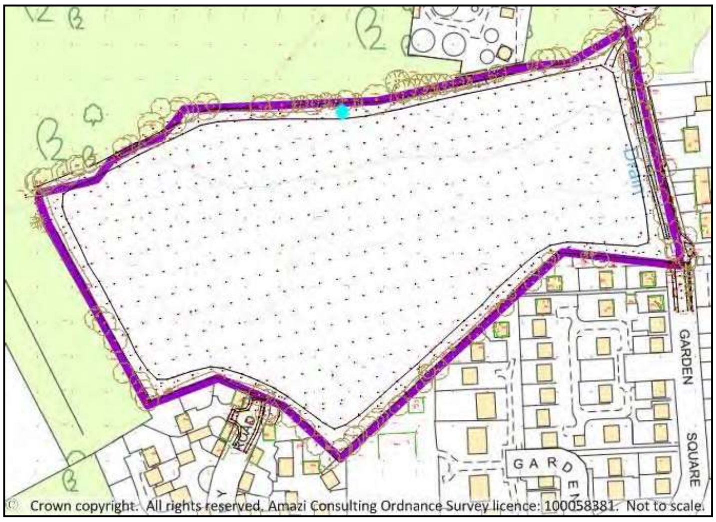


Figure 03 – Location of manhole on site survey

3.2.3 Subsequently our Client commissioned us to manage an additional surveys to establish the route of the existing gravity piped systems. In close liaison with Anglian Water a survey was undertaken on 15 February 2018. There were 7 people assisting with this survey, including: CCTV cameras, metal detectorists, excavator and representative from Anglian Water, who kindly assisted with keeping us informed about when the effluent from the treatment works would be running to etc..

3.2.4 This survey successfully found the Anglian Water manholes which were buried. The results of this survey are shown on the MK Surveys drawing in Appendix F.

3.2.5 We note that the site geophysical survey (undertaken by Suffolk Archaeology in November 2017) was used to assist with locating the chambers and pipe routes. Extracts from this survey report are given in Appendix K. The lines indicated as 'possible ferrous service' were also investigated by the team on site in February 2018. The two main lines shown as ferrous were confirmed to be small diameter cast iron pipes, which end abruptly. These do not seem to tie in with any of the public sewer or water records and are thought likely to be redundant water supply pipes or redundant mains that were routed to the former treatment plant situated north east of the site (now gone). This will need to be confirmed by further investigations prior to construction.

3.2.6 The survey in Appendix F identified a sewer running along the north of the site (150 mm/225 mm diameter). It was not physically possible to fully trace this system due to debris and water in the pipes. It was not flowing. It is concluded that this is likely to be a former outfall pipe from the treatment plant or possibly some form of overflow from the effluent system, or even a completely separate redundant pipe. The Anglian Water representative agreed that there are no obvious signs of this having a current function. This will need to be fully investigated before construction commences. Despite excavations and detections, public sewer manhole 8802 could not be found (and may not exist) to check if this other system is connected.

3.2.7 All existing sewers, rising mains and other pipework should be fully investigated prior to construction work on site. The proposed site drainage systems have been designed to avoid the existing rising mains, although positions should be checked for detailed design purposes. The site layout will require the diversion of the existing 750 mm diameter public surface water sewer. The site design also includes for the Anglian Water requirement for a 70 m zone from the south of the existing sewage treatment works where no housing is proposed.

3.3 Surface Water Flooding

3.3.1 As indicated by the topographical information on Figure 01, Figure 04 and Appendix B, there are no particularly obvious valleys, or flow routes in the vicinity of the site. This is expected since the site is situated at the high point in local topography.

Figure 04 – Risk of flooding from surface water (gov.uk, 20 October 2017)

3.3.2 Apart from an insignificantly small area at the east of the south boundary, there is no high or medium risk of flooding at the site. The extent of the low flood risks (1:1,000 year return period) are shown in Figure 04 and these are again limited to isolated areas at the south east of the site.

3.3.3 It is noted that the proposed site access road at the south east of the site crosses over the southern part of the existing site watercourse (which falls within the site boundary). As noted in section 3.1, this watercourse appears to have no direct land drainage function. It will, however, need further investigation and this infilling will need prior Land Drainage consent from Suffolk County Council.

4 Existing Site Surface Water

4.1 The existing site is fully greenfield. The existing site contours are shown on Figure 05 (at 0.1 m intervals), along with the existing site topographical sub-catchments (sub-catchment divides in red). The contour shading identifies that the existing site falls from north to south (red = high, blue = low) hence runoff flows overland across the site's southern boundary. There are no watercourse or drainage systems to intercept this runoff. (Note: the contours outside the site boundary on Figure 05 are not accurate because they are at the edges of the ground model.)

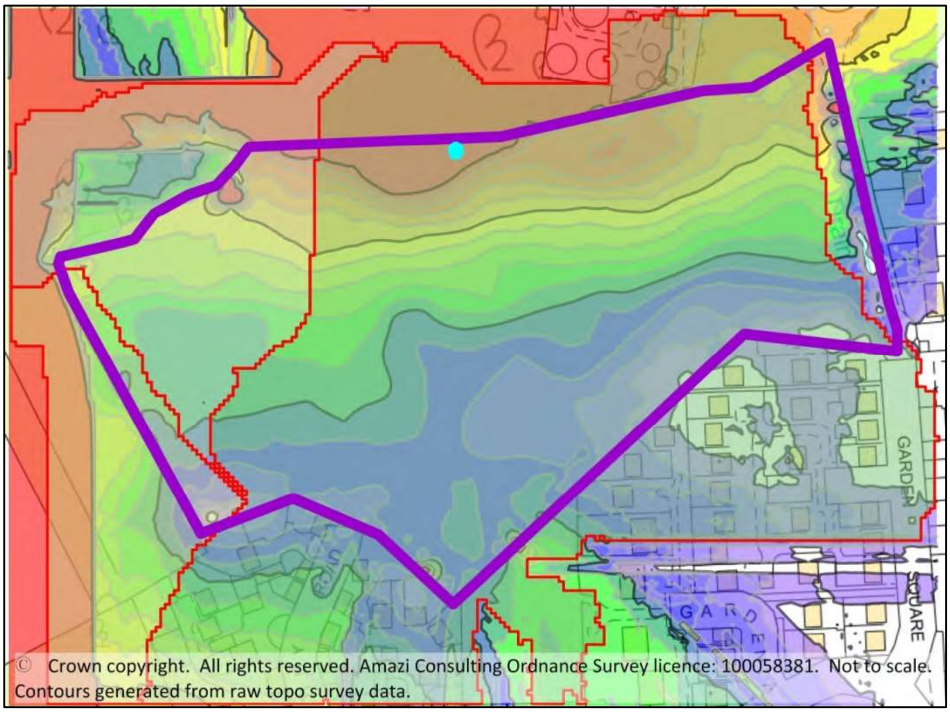


Figure 05 - Illustrative Existing Site Topography

4.2 The existing site greenfield runoff rates have been calculated using site specific rainfall data from the FEH Web Service. The resulting flow rates per unit area obtained using XP Drainage software are given in Appendix D in accordance with the methodology in the Environment Agency/DEFRA report SC030219, *Rainfall runoff management for development*, 2013. As noted in section 4.3/4/5, soil conditions at shallow depth are understood to be of low permeability, but the FEH catchment descriptors (Appendix D) indicate a soil (SPR) value of 0.23, so a value of soil value of 0.3 has been used (soil type 4) for existing site runoff purposes. The existing site runoff rates are summarised in Table 1.

Table 1 – Existing Site Peak Runoff*

Site Area (ha)	5.05
Development Area* (ha)	3.222
Q 1 year (l/s)	4.2
Q bar (l/s)	4.7
Q 30 year (l/s)	11.5
Q 100 year (l/s)	17.0
Current Outfall Description	Overland runoff to south & south east of site

*Site area minus the open grassed areas at east of site and the proposed green corridor/strip around the edge of the site.

4.3 An intrusive site investigation has been undertaken by Harrison Group Environmental Ltd and a copy of the associated report is in Appendix G. 4No soakaway test locations were installed; 2No. were installed within the shallower superficial cohesive deposits (TP01 & TP08) and 2No within the granular deposits underlying the site (TP03 & TP05). The 2no tests undertaken in cohesive soils demonstrated that there was no fall in water levels in one test, and the other initially dropped by around 50% and later stopped. This indicates that the shallow cohesive soils are unlikely to be suitable for the use of soakaways.

4.4 For the two tests undertaken in granular soils, it was not possible to measure infiltration rates at TP03 as water levels dropped too quickly, i.e. infiltration took place at greater than 1.15litres per second, which was the rate at which water is being transferred from the bowser into the test pit. The second test in granular soils was a little slower, and therefore occurred at a measurable rate. Table 02 summarises the soil infiltration rates.

Extract from Harrison's report:

Test location	Test no.	Test depth (m)	Strata	Infiltration rate (max.) (m/s)	Recommended infiltration rate (m/s)
TP01	1	1.2	Lowestoft Formation	N/A	**N/A**
TP03	1	2.5	Chillesford Church Sand Formation	*≥1.27×10^{-2}	***≥1.27×10^{-2}**
TP05	1	2.0 – 2.5	Chillesford Church Sand Formation	3.25×10^{-5}	**2.53×10^{-5}**
	2			3.96×10^{-5}	
	3			2.53×10^{-5}	
TP08	1	1.7	Lowestoft Formation	N/A	**N/A**

Table 5.1: Soakaway test results

*Soil infiltration rate is greater than or equal to, based on measured maximum infilling rate from water bowser used during testing.

4.5 Harrison's advice concluded that despite the poor infiltration rates determined in cohesive soils encountered on site, the granular strata have indicated favourable rates. Where tested, granular strata was confirmed at being present at between 2.0-4.0 m across the site and is likely to be a suitable strata for the disposal of surface water runoff.

Figure 06 - Soil Test locations

Extract from site investigation report (Harrison Geotechnical drawing GC21420 - DR002)

4.6 The site is not within or near a groundwater source protection zone. Harrison's report states that '*Groundwater was not encountered in any of the exploratory holes during this investigation'*, neither during the initial excavations (12-13 April 2018) or when returning for monitoring (19 April 2018). The exploratory boreholes were to a maximum depth of 5.45 m below ground. The site is at a high point in local topography and land falls away to the west of the site. We are unaware of any records of groundwater within 1 m of the base of the proposed infiltration systems, i.e. groundwater < 6 m below ground.

5 Proposed Site Surface Water Drainage

5.1 Overview

5.1.1 The development of the existing site would inevitably result in the generation of additional surface water runoff if measures are not taken to mitigate against the impact of introducing hard surfaces. The proposed site surface water drainage strategy therefore includes sustainable drainage systems (SuDS) to control runoff rates and mitigate against the impact of increased runoff volumes.

5.1.2 The key principles of the proposed site surface water drainage strategy are to manage runoff for flood risk mitigation:

- ✓ Soakaways are proposed throughout the site, so the proposed peak runoff rate in the 1:1 year rainfall event will be less than the peak greenfield runoff rate for the same event in accordance with clause S2 of the *Non-Statutory Technical Standards for Sustainable Drainage*, March 2015.
- ✓ Soakaways are proposed throughout the site, so the proposed peak runoff rate in the 1:100 year rainfall events including future climate change will not exceed the existing peak Qbar rate to satisfy clause S6 of the *Non-Statutory Technical Standards for Sustainable Drainage*, March 2015, and provide betterment in accordance with clause S2.
- ✓ Provide treatment in accordance with the simple index method outlined in Ciria report C753: The SuDS Manual, 2015.
- ✓ Allowances for climate change have been included in accordance with the Environment Agency's *Flood risk assessment: climate change allowances, April 2016.*
- ✓ Later detailed pipe network design should ensure that there is no flooding except in conveyance systems up to 1:30 year rainfall events - this will satisfy requirement S7 of the Non-Statutory Technical Standards for Sustainable Drainage (March 2015).

5.2 Strategy

5.2.1 In principle, the private areas will drain to private attenuation and soakaway systems. The Highway, to later be offered to Suffolk County Council for adoption, will discharge to separate, highway only attenuation and soakaways. These will be offered to Suffolk County Council for adoption, but we understand that they are not likely to be adopted and hence would also remain within private ownership.

5.2.2 The position of the soakaways has generally been dictated by the site topography and land ownership:

- Runoff from each dwelling will be disposed of via infiltration into shared private garden crate type soakaways. Only roof runoff will enter this system. The roof water will pass through rainwater downpipe filters (see Appendix L) and enter crate type attenuation tank before a piped outfall into a private shared soakaway.
- Runoff from private shared access roads will drain via permeable block paving with treatment and attenuation provided in tanked sub-strata beneath. These paving/attenuation stormwater controls will have a piped outfall into a private soakaway.
- Runoff from adoptable highways - It is understood that Suffolk County Council will not adopt permeable paving, so to provide the required treatment for the highway, swales with bio-filters have been included. Swales need to be kept shallow, so to prevent long, deepening pipe runs, several of these highways soakaways have been situated throughout the site. We suggested the use of kerb outlets direct into the swales (where levels allow) to eliminate the need for gulleys. Where the swale is not directly adjacent to the part of road being drained, runoff will need to be intercepted in gulleys and piped into the swale structure.

5.2.3 Because all the site surface water will outfall to soakaways, we have designed example systems that attenuation, treat and infiltrate, which will be replicated throughout the site. A summary of the example designs is given in Tables 02 (Highway) and 04 (private) and the design drawings and accompanying calculations are given in Appendices K, L & M.

Table 02 - Key features of example highway attenuation storage and soakaway structure

	Swale	**Soakaway**
Impermeable catchment (ha)	highway area = 0.07 swale area = 0.017	N/A
Inflow Source	43% of highway area enters into swale 57% of highway area enters into tank structure under the swale	Outflow from Swale via 300 mm pipe
Section	See Figure 07	
Calculations & Schematic	Appendix K	
1:100 + CC peak volume (m^3)	90.8	7.7
1:100 + CC peak water depth (m)	1.835 (from base of crate) 0.535 (ponding)	3.03

1:1 peak volume (m^3)	11.2	5
1:1 peak water depth (m)	0.28 (from base of crate)	1.98
Minimum side slopes	1:3	N/A
Size	25 m long 6.6 m wide 0.8 m deep	1 No 1.8 m diameter 3.8 m deep with infiltration up to 1 m above invert only
Software file ref	AMA647_03.xpdx	

CC = climate change, 40%, see 5.2.13

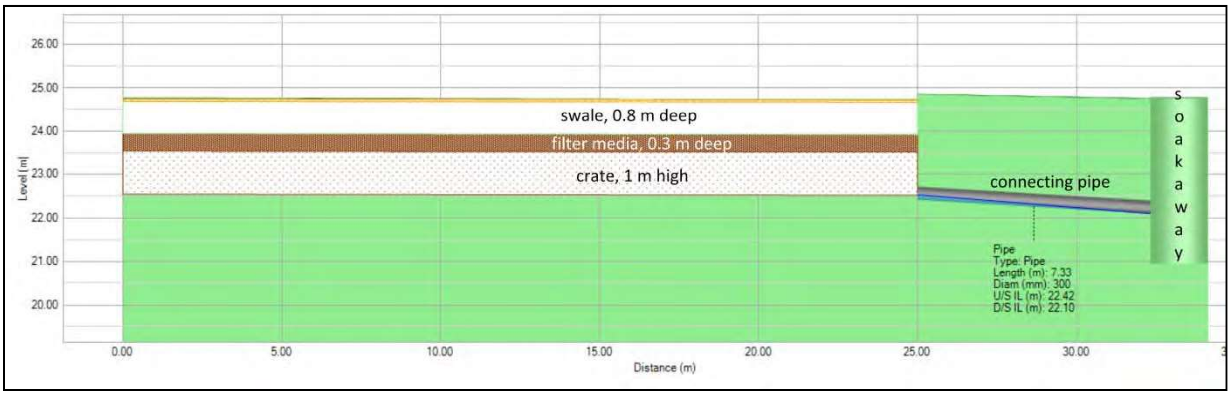


Figure 07 - Section through proposed highway swale and soakaway

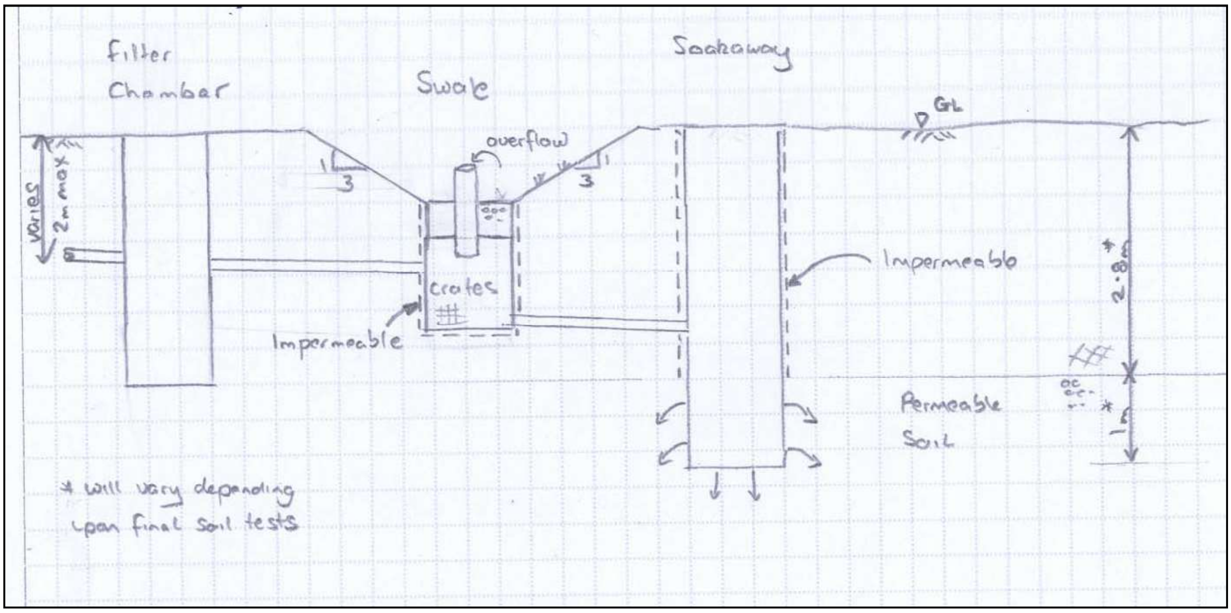


Figure 08 - Highway swale/soakaway sketch

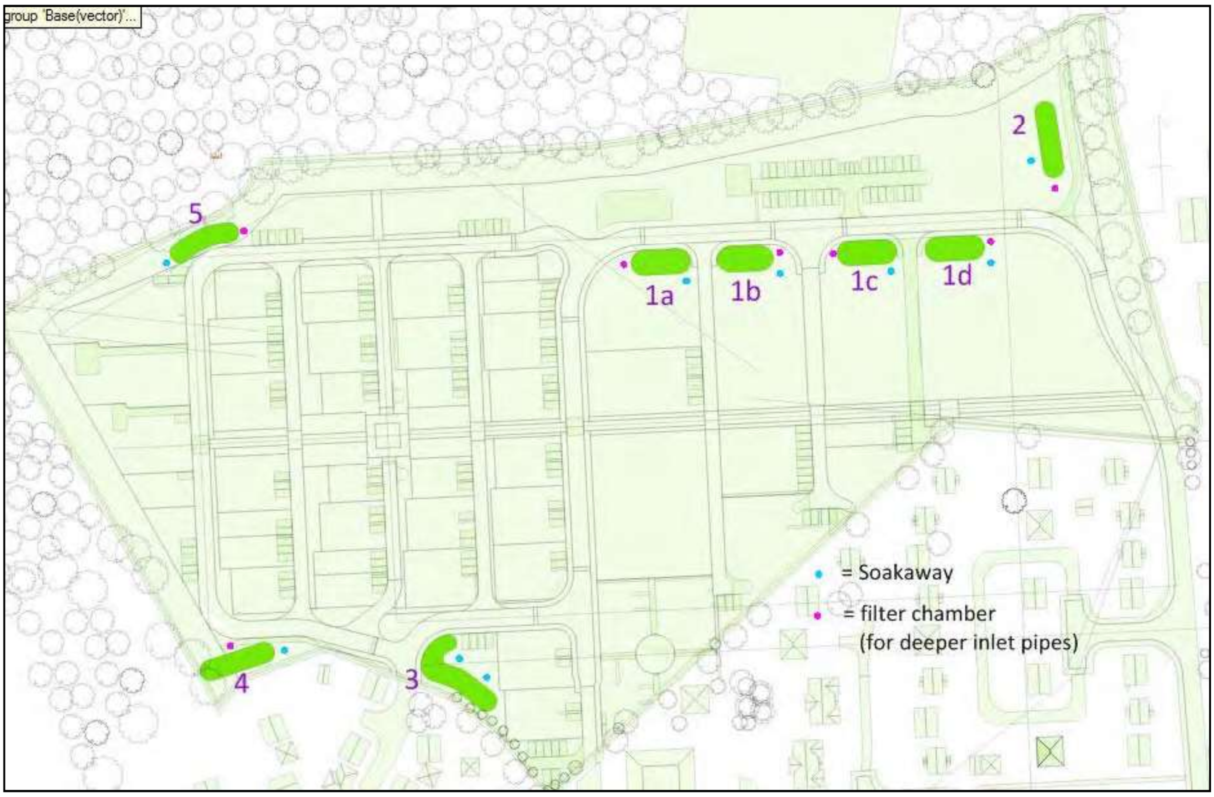


Figure 09 - Swale locations

Table 03 - Proposed Highway drainage

Proposed catchment	Total area (m^2)	Required number of swale structures	Swale reference
Blue	2,800*	4	1a, 1b, 1c, 1d
Red	336	1x half structure	5
Yellow	700	1	4
Purple	1,340	2 (combined into one structure)	3
Grey	470	1	2

*including the proposed public car park at the north of the site

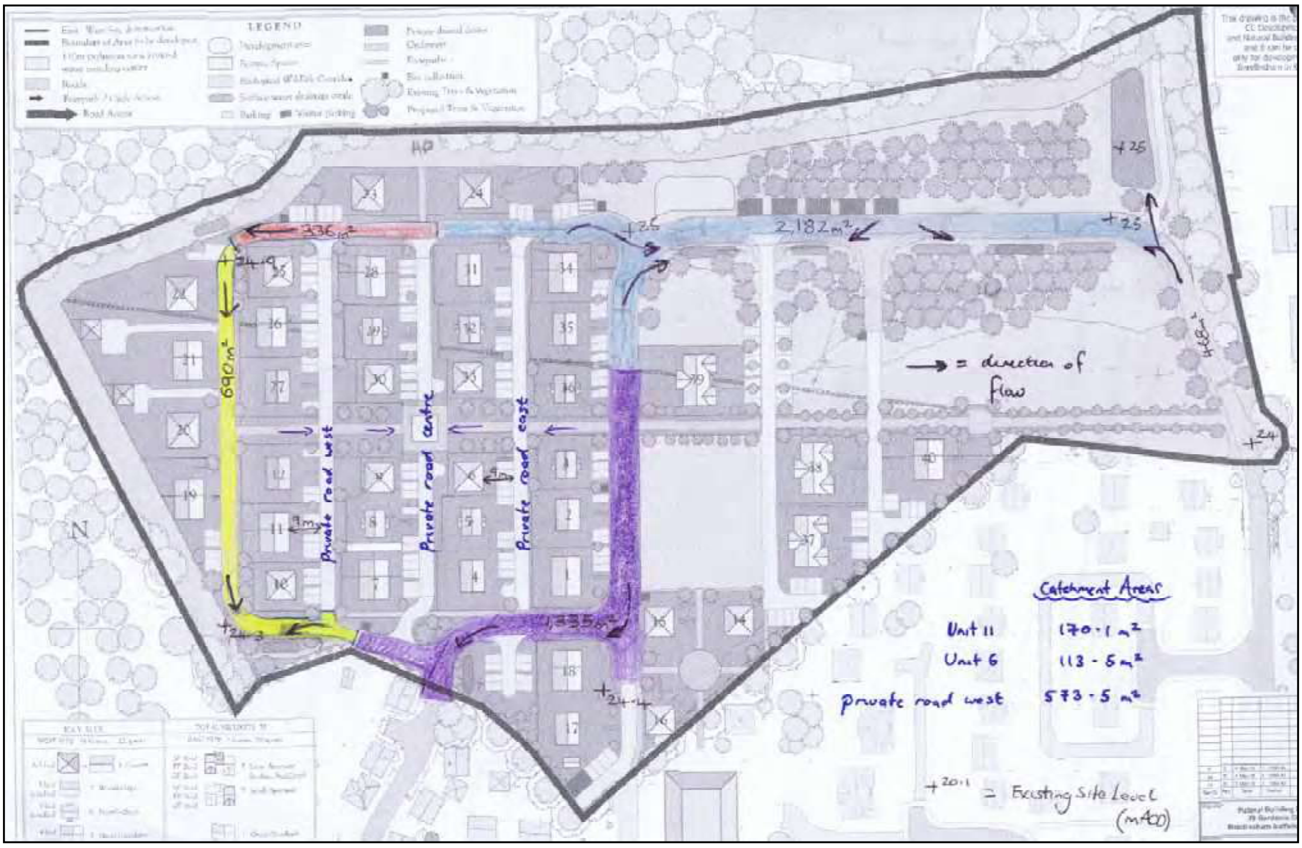


Figure 10 - Road catchments

5.2.4 Figure 10 shows the highway sub-catchments established based upon site topography. The required highway swales for these catchments are summarised in Table 03. The positions of these swales have been largely dictated by the site layout/design, but also influenced by site topography and the easements required to the public sewerage system.

5.2.5 Where runoff can enter at a shallower level, from gullies directly into the swale, it will be treated by passing through the swale and the filter blanket beneath before entering the attenuation crates. Where the road surface is more remote from the swale and the highway pipes will not be able to physically connect into the swale, treatment is provided in a treatment chamber, as or similar to the German Funke chamber (see Appendix K).

5.2.6 The highway swale example calculations have been based upon ground levels in the vicinity of swale 1b. Although the ground and structure levels will change for the swales in other positions, the general depth / cross section arrangement is not anticipated to change. The only difference may be a slight increase or decrease in depth of the soakaway depending upon the exact position of the sand encountered in the location of each structure. It is also subject to the design of the roads and gulley positions. The designs in this report have assumed a soakaway height into the permeable strata of 1 m. Also, the volume of storage required may change a little from the standard 700 m^2 assumed here once the detailed design is undertaken and gulley positions and all pipe levels have been established. What this report demonstrates is the likely number and arrangement of swales/soakaways. The detailed design will also need to include infiltration testing at each specific soakaway location.

Table 04 - Key features of example private soakaway systems

	Private Road		**Private dwellings**	
	Permeable paving attenuation structure	**Soakaway**	**Crate tank**	**Soakaway**
Impermeable catchment (ha)	0.08	-	0.023	-
1:100 peak volume (m^3)	86	8.3	20.8	2.8
1:100 peak water depth (m)	0.31	3.3	1.1	2.9
Calculations	Appendix M		Appendix L	
1:1 peak volume (m^3)	9.4	7.6	1.8	2
1:1 peak water depth (m)	0.03	3	0.089	1.84
Size	350 mm granular sub-base (no crates)	1 No 1.8 m diameter 3.8 m deep with infiltration up to 1 m above invert only	1m high crate, 22 m^2	1 No 1.8 m diameter 3.8 m deep with infiltration up to 1 m above invert only
Software file ref	AMA647_101.xpdx		AMA647_102.xpdx	

CC = climate change, 40%, see 5.2.13
The ½ drain down times are not being estimated correctly by the software

Figure 11 - Private catchment areas

(orange = catchment area)

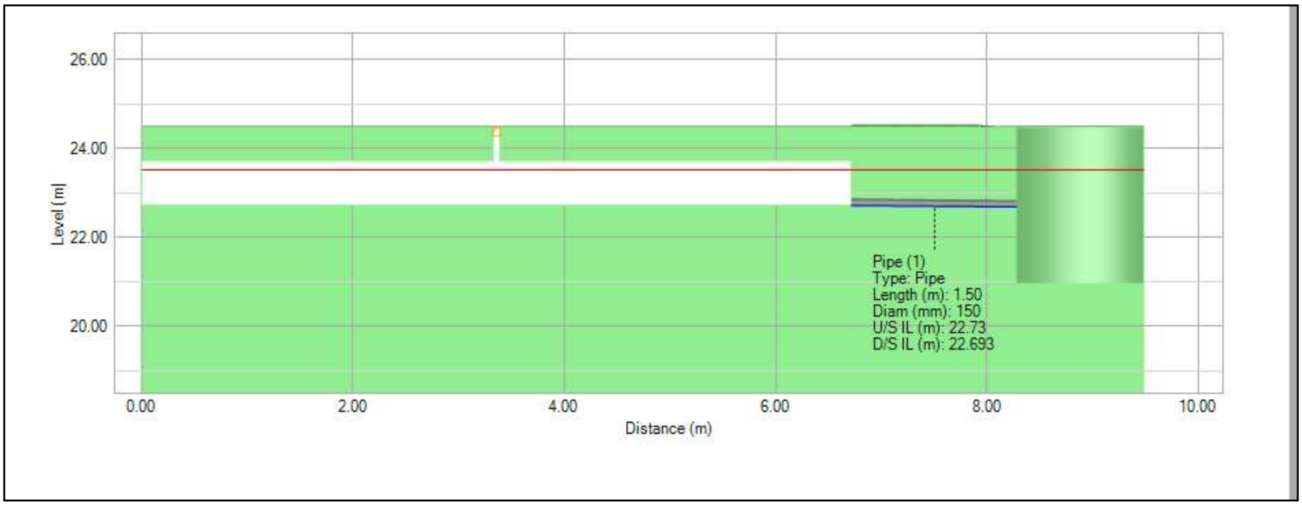


Figure 12 - Private house soakaway - section

5.2.7 The example soakaway design for private roads has been based upon the 'Central' road. The other two private roads are similar (topography, layout etc), with a slightly smaller catchment area.

5.2.8 All the parking spaces on site (private, visitor parking and the car park and access road at the north of the site) will be permeable, surfaced with a simple eco-grid type system in-filled with gravel. They have not, therefore, been included in our example soakaway designs. We have, however, included in our calculations for future increase in drained hard surfaces at the private dwellings. The actual volumetric runoff coefficient, Cv, values used in the calculations for the residential soakaways have been increased to include for 10% urban creep. XP Drainage software does not yet have a function (we are campaigning!) for adding urban creep. The catchment areas cannot be scaled up by 10% because XP Drainage has a fully scaled graphical interface, so we have agreed with the software provider (Innovyze) that we must therefore adjust the Cv values to account for 10% urban creep. The resulting Cv values used in these calculations are therefore:

Summer Cv = 0.75 x 1.1 = 0.825

Winter Cv = 0.84 x 1.1 = 0.924

5.2.9 All the soakaway calculations have been based upon the lowest recorded infiltration rate of 2.53×10^{-5} m/s. A factor of safety of 2 has then been applied to this rate, so the infiltration rate used in the calculations is 1.265×10^{-5} m/s (0.0455 m/hr). This is considered to be a conservative rate, which is appropriate at this initial stage in the design. Particularly since rates $>1.27 \times 10^{-2}$ m/s were also observed in the same strata. The ground investigation clearly shows that there is permeable sand strata across the site, although its depth below ground varies. Now that the surface water strategy has been established and soakaway locations are known, further infiltration testing should be undertaken to inform the next stage of design, i.e. the exact depth to sand and infiltration rate specific to each location.

5.2.10 Generally, at the west of the site, the permeable strata was encountered at < 3 m below ground. In the east of the site, the sand was not encountered until c 3 m below ground, and at borehole DCS04, not until 3.95 m below ground. According to the drainage hierarchy (given in The Building Regulations, The SuDS Manual report C753, other national and local best practice documents), infiltration drainage techniques should be considered as the first option for disposal of surface water and only if this is not possible, should other options be considered. Because of all this, the proposed designs rely upon deep soakaways (with inverts 3.5 to 5 m below ground level).

5.2.11 The Anglian Water report from December 2017 in Appendix E states that '*It is our understanding that the evidence to confirm your compliance with the surface water hierarchy* *is not currently available. However once the evidence has been confirmed, then a connection point may be made to manhole 6800 in the existing on site public sewer at NGR TM3363953802 at a rate of 13.7l/s'.* Since the evidence of good soil permeability is now available, there is no longer a proposal to connect into the existing public sewer on site. However, if for some reason it is later found that soakaways are not viable, say from a cost or health and safety perspective, then this there would be this alternative positive outfall option.

5.2.12 The central private access road has been used as the design example. This design has permeable block paving with 350 mm of tanked granular sub-base which provides the attenuation. This system then outfalls via a pipe into a soakaway. The final level of and connections from the road attenuation system to the soakaway will be subject to detailed design, but there is adequate fall available because infiltration does not start until c 3 m below ground.

5.2.13 Climate change has been accounted for in accordance with the published Government guidance: *Flood risk assessments: climate change allowances* (April 2016). The site drainage design should use 20% increase in rainfall and the upper end allowance of 40% to check no increase in flood risk elsewhere and site safety during the 1:100 year events. In order to demonstrate at this planning stage that the development can manage the site runoff, we have run the calculations using the higher 40% climate change allowance. The results summarised in the tables above demonstrate that even during the upper (40%) climate change analysis, the attenuation storage included in the development provides adequate flood management measures by preventing uncontrolled runoff.

5.2.14 Further design notes:

a. Piped outlets from the attenuation systems will transfer flow into the soakaway chamber. These chambers outfall via infiltration below the cohesive soil strata.
b. The key methods of runoff interception and exceedance flow routes are shown on Figure 10. The proposed hard surfaces mainly comprise permeable paving, or fall towards swales which will intercept exceedance runoff. This is a significant improvement upon the existing situation because currently the site has no interception of greenfield runoff.
c. Trees will need to be sighted away from proposed attenuation systems, and/or protection provided to prevent root damage to the drainage systems.
d. The structural design should take account of, or advise on, the detailed soakaway designs. For this report, soakaways are situated at least 5 m away from buildings.
e. During detailed design further evidence should be established to confirm that seasonally high groundwater levels do not rise to less than 1 m from the base of the deeper soakaway structure.

f. As a result of the depth of permeable soil, the soakaway structures are deep. This is not ideal from a construction or future maintenance perspective. During detailed design, thorough guidance should be provided for future maintenance and operation. All of the soakaways are situated in reasonably accessible positions. These soakaway structures could present a significant health and safety risk in future due to the risk of falling, drowning, gases or entrapment if the cover is lifted. The cover to the soakaway chambers should therefore be bolted to prevent unauthorised entry. Entry should be avoided. Only if absolutely necessary, should entry be undertaken only by personnel properly trained in confined space entry. The swales and filter systems upstream of the soakaways should ensure that very limited silt or debris can get into the chamber. The upstream systems should be maintained to prevent the need for entry to the soakaway chambers. Construction must be undertaken with the appropriate width of trenching and with properly designed shoring as required.

g. All of the soakaways in this report have been designed as ring soakaway chambers. There should be consideration in the next stage of design as to whether shallow borehole soakaways may be more appropriate. This will depend upon further soil testing and confirmation of the depth to sand at each soakaway position. This should not affect the overall strategy for the surface water drainage and its impact upon site layout, but may affect the final number of soakaways (boreholes 'v' chambers).

5.2.15 Although the piped network design will later be designed to ensure no flooding even during the 1:30 worst case duration storm event, the proposed roads will be designed to convey surface water into the swales in the event of, say, gulley blockages where temporary ponding and flow might occur above ground. This will prevent overland runoff from the site's hard surfaces discharging off site in an uncontrolled manner.

5.2.16 The sustainable drainage technique of rainwater harvesting will be included in the development. Five apartment buildings (3 buildings of 6 two or one bed apartments and 2 buildings of 3 two bed apartments) will include rainwater harvesting for re-use within the buildings. This will reduce the potable water usage and may slightly decrease the size of the required soakaway attenuation storage volume.

5.3 Treatment

5.3.1 The proposed site surface water strategy also includes treatment in accordance with the simple index method outlines in Ciria report C753: The SuDS Manual, 2015. The measures included in the drainage design to treat runoff prior to shallow infiltration are listed below and detailed in Tables 05 and 06.

1. Tanked permeable paving structure - block paving, underlain by granular fill. To provide adequate treatment, the depth of granular fill should be provided to ensure a minimum treatment depth of c350 mm (subject to final design).

2. Rainwater downpipes will be fitted with filter chambers to provide filtration prior to water entering the soakaways (see Filter Chamber Specification in Appendix L).

3. The adoptable Highway runoff will pass though, either the swale and Remedi8® filter media, or just the Remedi8® filter media within a Funke chamber (or similar), depending upon incoming invert level. Refer to details in Appendix K.

Table 05 - Proposed runoff treatment – residential dwellings

Roofs		Treatment Required[1]	Proposed Treatment individual indices[2]
Pollution Hazard[1]:	**Low**		Downpipe Filter chamber
	Total suspended solids indices	**0.3**	Actual indices are not available for this silt trap type system, but the flow will also pass through the crate tank and the geotextile membrane of the soakaways chamber, so it is considered that adequate siltation, hence treatment will occur. Rainwater downpipes should be sealed to prevent ingress of untreated substances and the property sales documentation should explain the function and maintenance needed to the house drainage systems.
	Metals indices	**0.2**	
	Hydrocarbons indices	**0.05**	

Notes:
1 - C753 table 26.2
2 - C753 table 26.3

Table 06 - Proposed runoff treatment – private roads*

Road		Treatment Required[1]	Proposed Treatment individual indices[2]
Pollution Hazard[1]:	**Low**		Permeable paving
	Total suspended solids indices	**0.5**	**0.7**
	Metals indices	**0.4**	**0.6**
	Hydrocarbons indices	**0.4**	**0.7**

Notes:
Assumed < 300 vehicle movements/day. 1 - C753 table 26.2 2 - C753 table 26.3

Table 07 - Proposed runoff treatment – Highway*

Road		Treatment Required[1]	Minimum proposed treatment individual indices[2]	
Pollution Hazard[1]:	**Low**		Swale (with Remedi8)	Funke filter chamber (with Remedi8)
	Total suspended solids indices	**0.5**	**+0.8**	**+0.8**
	Metals indices	**0.4**	**+0.8**	**+0.8**
	Hydrocarbons indices	**0.4**	**+0.8**	**+0.8**

Notes:
Assumed < 300 vehicle movements/day.

1 - C753 table 26.2

2 - C753 table 26.3

5.4 Maintenance

5.4.1 A draft surface water maintenance plan is given in Appendix N. It is important that the permeable paving is installed and maintained as suggested by the manufacturer. Also that the downpipe filters, treatment channel and highway gullies are regularly maintained (and filter media replaced) as recommended by the manufacturers and in accordance with published best practice guidance. This is required to treat runoff and intercept silt prior to discharge into attenuation systems.

5.4.2 During construction the contractor will also be responsible for preparing appropriate method statements and ensuring compliance with all relevant legislation and industry best practice in regards to managing site surface water and construction of the SuDS systems. This is necessary to manage water on site, prevent environmental pollution elsewhere and ensure the SuDS are properly constructed. Some of the relevant documents that they may use are:

CIRIA report C698, Site Handbook for Construction of SuDS, 2007
CIRIA report C741, Environmental Good Practice on Site Ed 4, 2015
The former Pollution Prevention Guidance note 6 also provides useful reference.

We anticipate the following will need consideration and notes will be made on final design drawings accordingly to provide clear information to the contractor. The contract should allow for regular monitoring of the SuDS construction. Controlling silt during construction will include:

- Construction runoff will be controlled in the low areas on site – the location of the final permanent attenuation system where applicable.
- Construct temporary stilling ponds – these should include filtration prior to outfall (presumably to temporary soakaways).
- Cover stockpiles where possible.
- Obviously turbid water should not leave the site and additional settlement or filtration measures should be put in place if necessary.
- Piped network to be cleaned out if necessary post construction and before final basin commissioning.
- Rock baffles to be placed at the inlet to any temporary basin or stilling pond to dissipate energy and reduce velocity.

6 Foul

6.1 Anglian Water has confirmed its acceptance to receive the foul outflow from the development into its public sewerage system. The Anglian Water report in Appendix E states the required connection into their existing system at location NGR TM33830,53919, as shown on Figure 1, page 6 of the attached Anglian Water Pre-Development report.

6.2 This connection point is within the Anglian Water treatment works site and hence we have not been able to establish a level of this connection. The site levels at proposed properties vary from 25.5 mAOD (unit 24) to 24 mAOD (unit 40). The ground level at the site's northern boundary near the treatment site is 25.2 mAOD. We have requested for confirmation of the level at the connection point, but have not yet received this. It is therefore assumed that there will need to be a pumping station on site to lift the site foul water to the connection point at a higher level. The architectural layout of the proposed development in Appendix C shows the position of the pumping station at the north of the site, with vehicular access off the site's main entrance road.

6.3 The foul water drainage will drain from the properties by gravity in piped systems to the pumping station.

6.4 It is noted that the Anglian Water plan in Appendix E shows existing rising mains at the east of the site. These will need to be surveyed and their position taken into consideration during the design of the site infrastructure.

7 Conclusions

7.1 The proposed development comprises the construction of 75 residential units on the existing greenfield site.

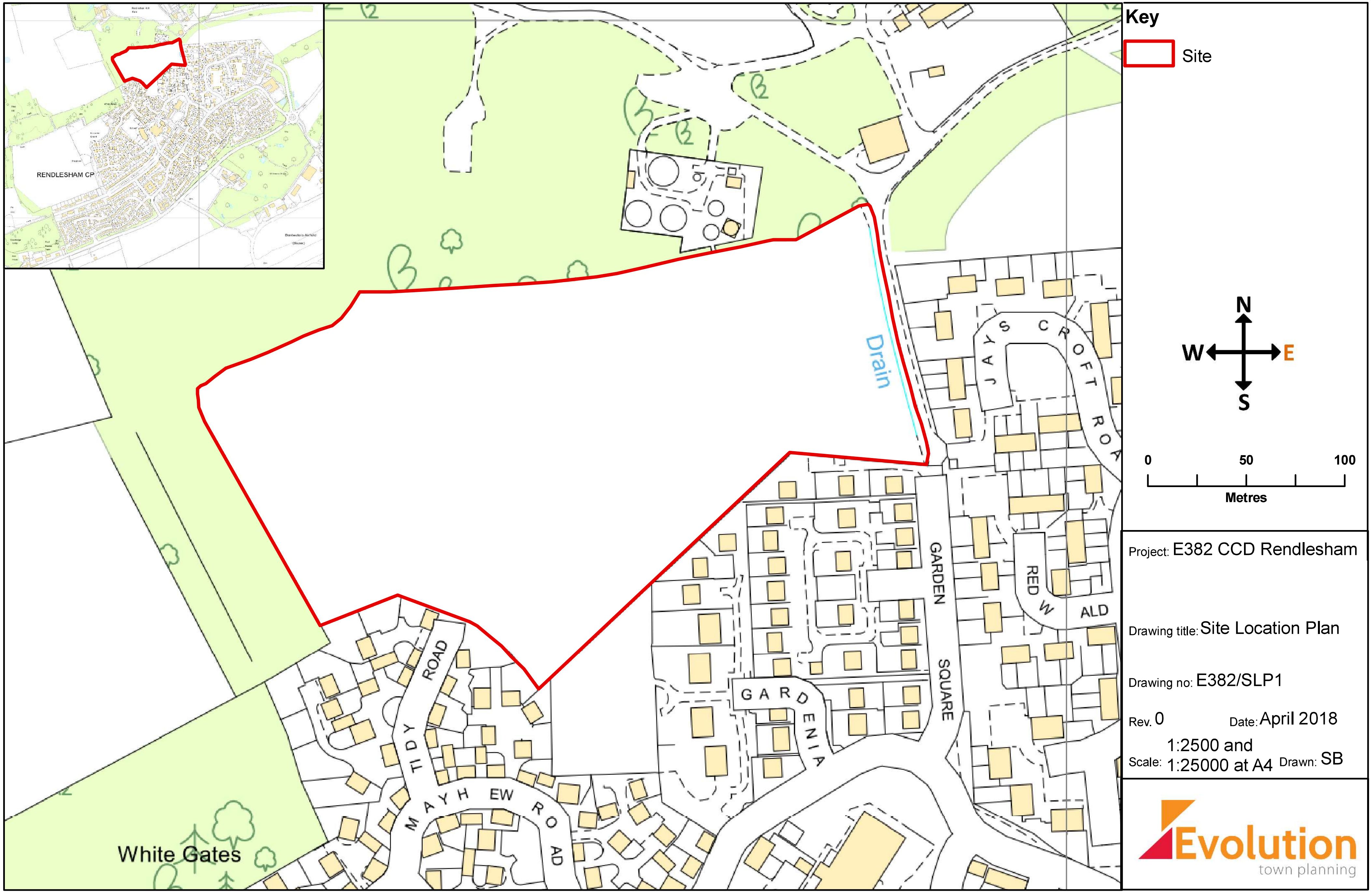
7.2 The entire site is located fully within flood zone 1 and the proposed site is at low risk of flooding from all sources.

7.3 The proposed development includes soakaways as an outfall for surface water runoff. These will be located across the site. Example designs have been included in this report.

7.4 Treatment to surface water runoff is proposed, in accordance with CIRIA report C753, *The SuDS Manual,* prior to discharge into the soakaways.

7.5 The proposed surface water drainage strategy ensures compliance with the *Non-Statutory Technical Standards for Sustainable Drainage (*2015).

A
Location Plan



Project: E382 CCD Rendlesham

Drawing title: Site Location Plan

Drawing no: E382/SLP1

Rev. 0 Date: April 2018

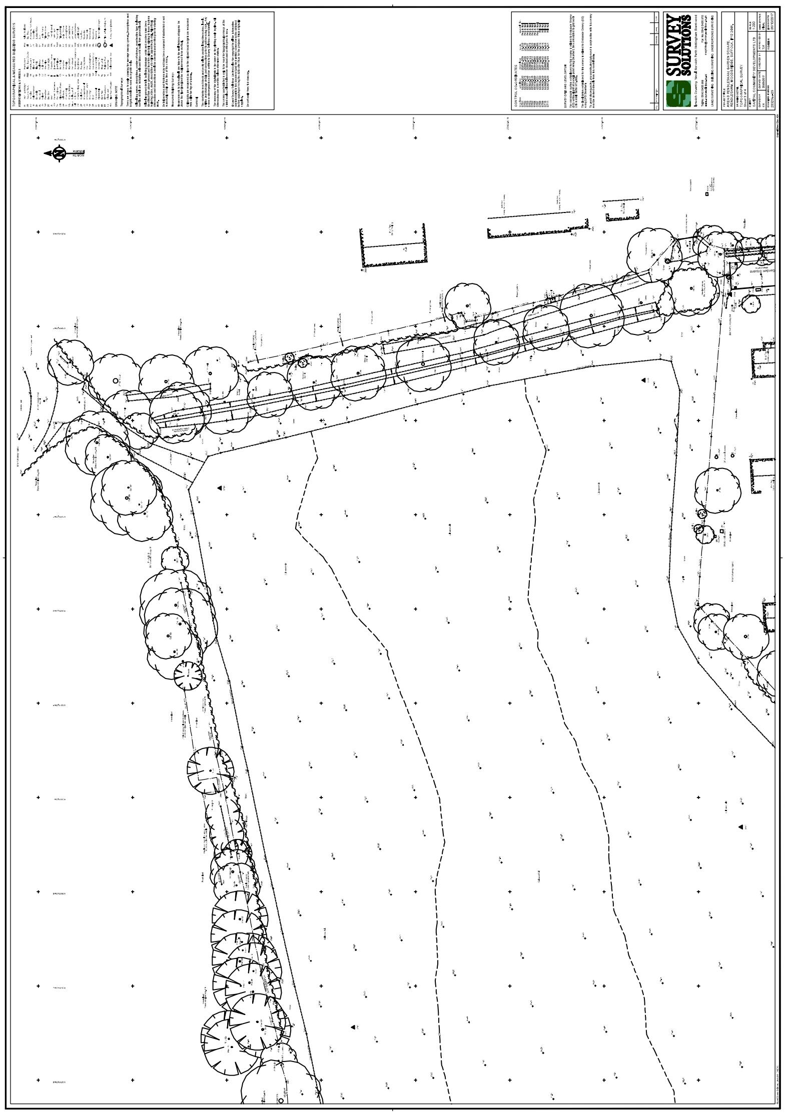
Scale: 1:2500 and 1:25000 at A4 Drawn: SB

Evolution town planning

B

Topographical Survey

20323se-01 to 04 Topographical Survey (4 sheets)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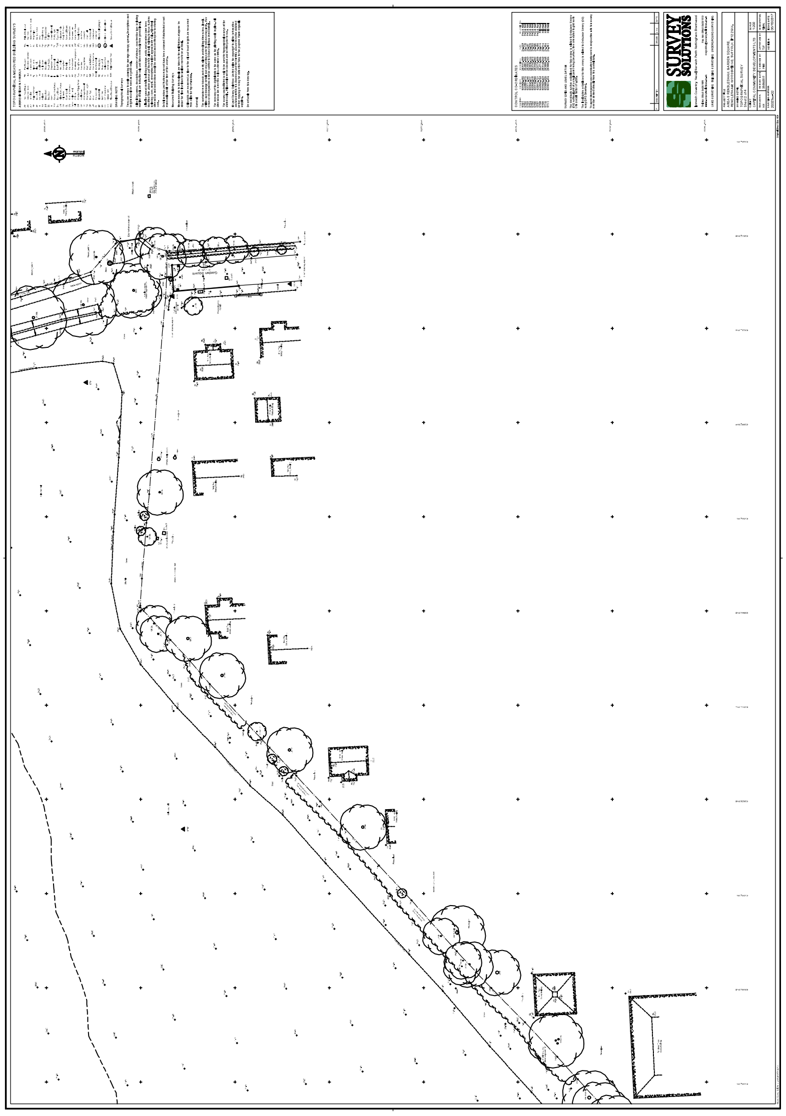
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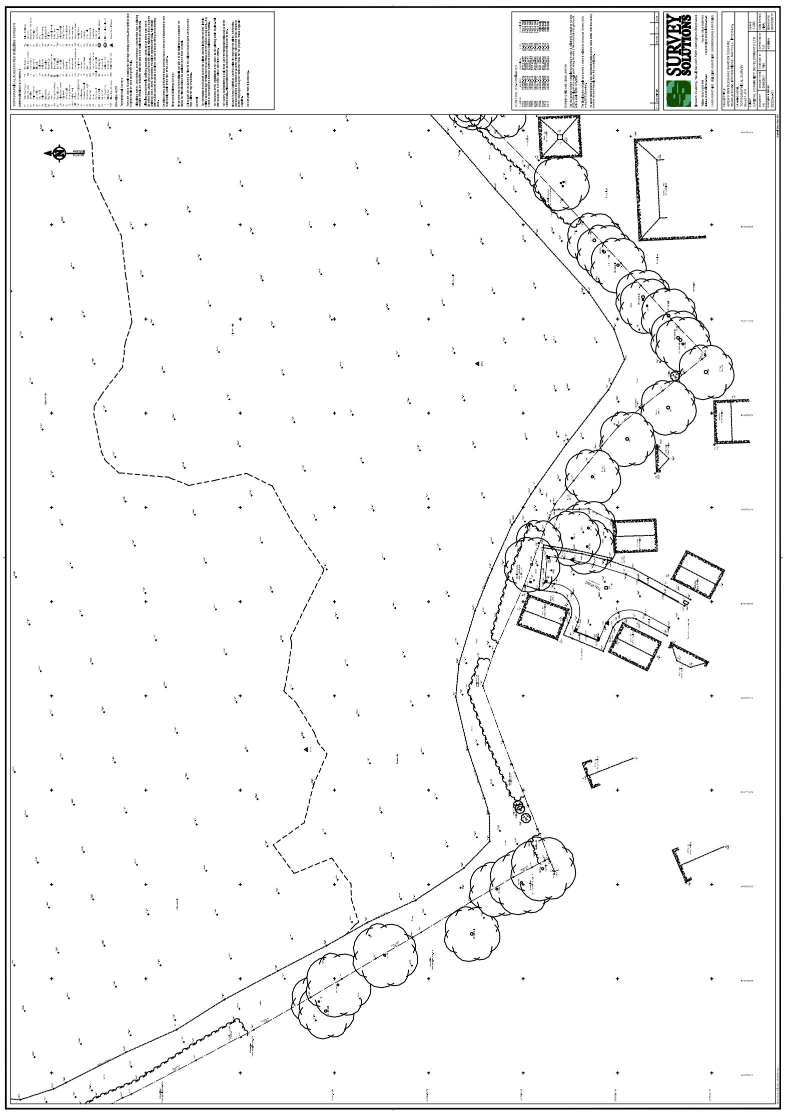
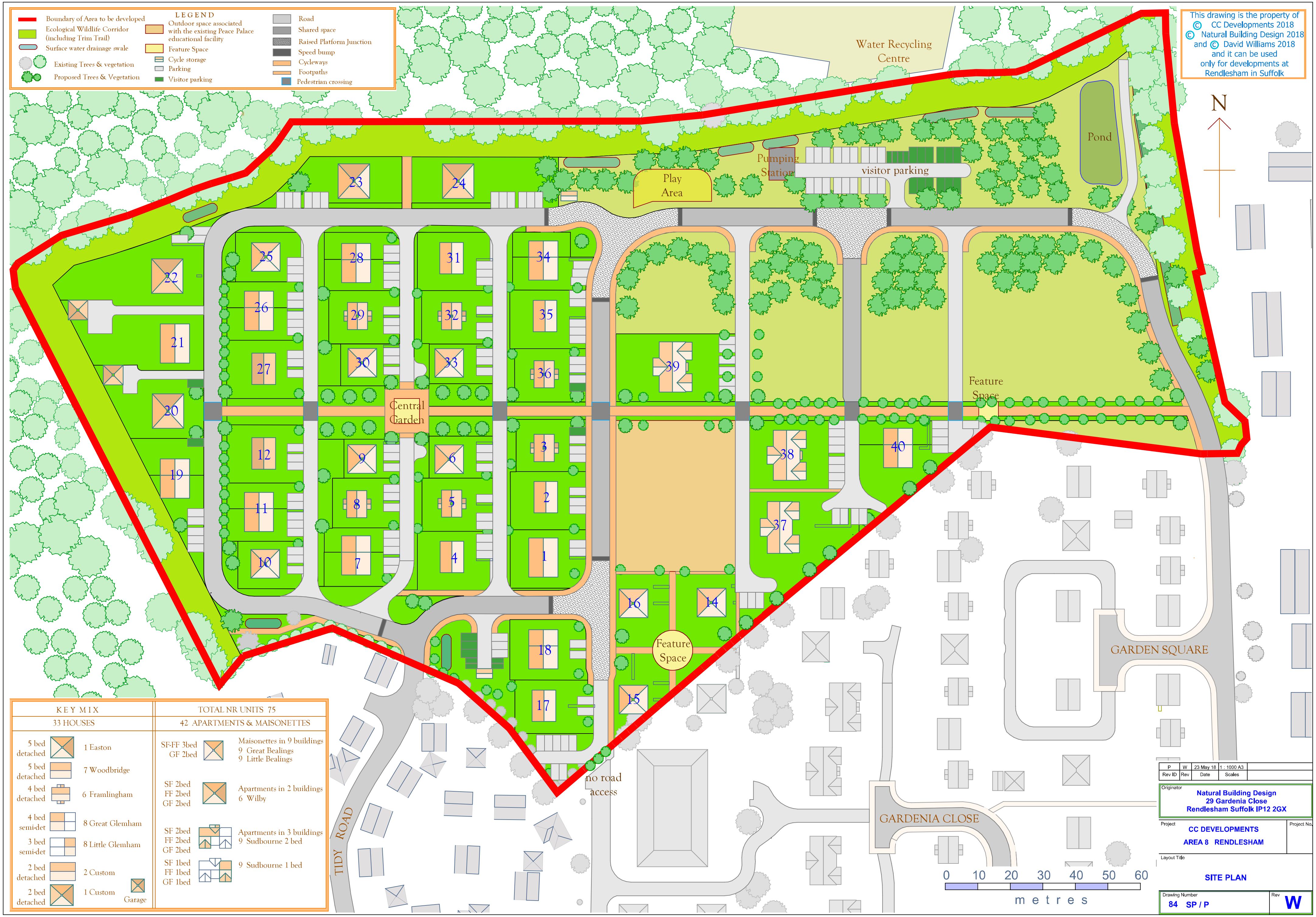

SURVEY SOLUTIONS

C

Proposed Site

84 SP / P Rev W Site Plan


LEGEND
Boundary of Area to be developed
Ecological Wildlife Corridor (including Trim Trail)
Surface water drainage swale
Existing Trees & vegetation
Proposed Trees & Vegetation
Outdoor space associated with the existing Peace Palace educational facility
Feature Space
Cycle storage
Parking
Visitor parking
Road
Shared space
Raised Platform Junction
Speed bump
Cycleways
Footpaths
Pedestrian crossing
This drawing is the property of © CC Developments 2018 © Natural Building Design 2018 and © David Williams 2018 and it can be used only for developments at Rendlesham in Suffolk
N
Water Recycling Centre
Pond
Pumping Station
visitor parking
Play Area
Central Garden
Feature Space
Feature Space
no road access
TIDY ROAD
GARDEN SQUARE
GARDENIA CLOSE
0 10 20 30 40 50 60
metres
KEY MIX
33 HOUSES
5 bed detached 1 Easton
5 bed detached 7 Woodbridge
4 bed detached 6 Framlingham
4 bed semi-det 8 Great Glemham
3 bed semi-det 8 Little Glemham
2 bed detached 2 Custom
2 bed detached 1 Custom Garage
TOTAL NR UNITS 75
42 APARTMENTS & MAISONETTES
SF-FF 3bed GF 2bed Maisonettes in 9 buildings 9 Great Bealings 9 Little Bealings
SF 2bed FF 2bed GF 2bed Apartments in 2 buildings 6 Wilby
SF 2bed FF 2bed GF 2bed Apartments in 3 buildings 9 Sudbourne 2 bed
SF 1bed FF 1bed GF 1bed 9 Sudbourne 1 bed
P W 23 May 18 1 : 1000 A3
Rev ID Rev Date Scales
Originator
Natural Building Design
29 Gardenia Close
Rendlesham Suffolk IP12 2GX
Project
CC DEVELOPMENTS
AREA 8 RENDLESHAM
Project No.
Layout Title
SITE PLAN
Drawing Number
84 SP / P
Rev
W

D
Existing Runoff Calculations

Unit runoff rates

FEH catchment descriptors

ICP SUDS Mean Annual Flood

Input

Return Period(years)	100
Area (ha)	1.000
SAAR (mm)	587.000
	0.300
Urban	0.000
Region	Region 5

Results

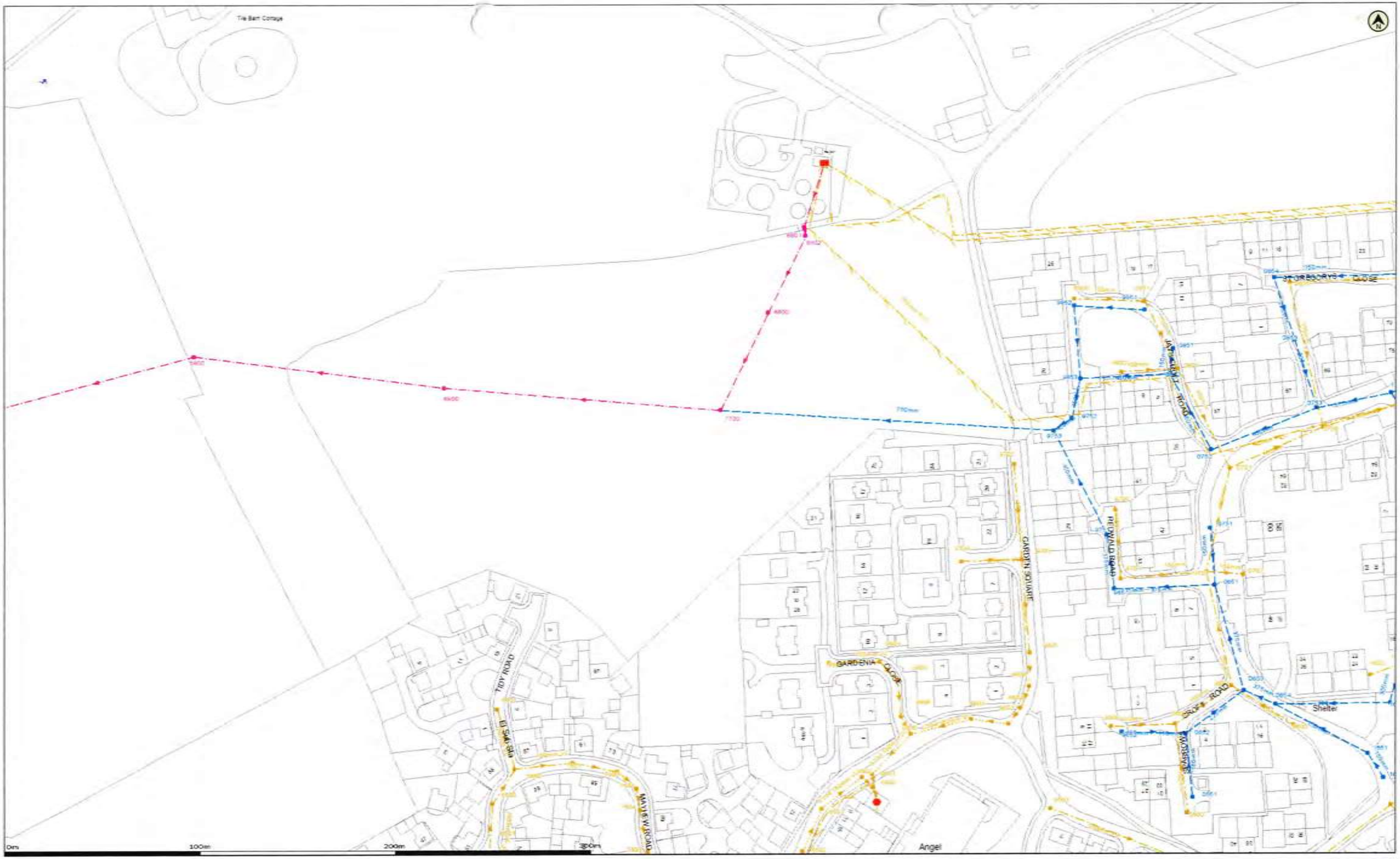
QBAR Rural (L/s)	1.4832
QBAR Urban (L/s)	1.4832
Q 100 (years)	5.2802
Q 1 (years)	1.2904
Q 30 (years)	3.5634
Q 100 (years)	5.2802

VERSION	"FEH CD-ROM"	Version		3	exported at	17:19:45	GMT	Wed	20-Sep-17
CATCHMENT	GB	633250	253450	TM 33250 53450					
CENTROID	GB	633595	253601	TM 33595 53601					
AREA	0.535								
ALTBAR	23								
ASPBAR	256								
ASPVAR	0.51								
BFIHOST	0.777								
DPLBAR	0.58								
DPSBAR	8.2								
FARL	1								
FPEXT	0.1542								
FPDBAR	0.444								
FPLOC	0.696								
LDP	1.06								
PROPWET	0.26								
RMED-1H	10.9								
RMED-1D	30.9								
RMED-2D	39.5								
SAAR	587								
SAAR4170	602								
SPRHOST	23.83								
URBCONC1990	0.659								
URBEXT1990	0.1379								
URBLOC1990	1.171								
URBCONC2000	0.75								
URBEXT2000	0.0374								
URBLOC2000	1.568								
C	-0.02								
D1	0.29162								
D2	0.2729								
D3	0.23626								
E	0.309								
F	2.54876								
C(1 km)	-0.02								
D1(1 km)	0.293								
D2(1 km)	0.27								
D3(1 km)	0.239								
E(1 km)	0.309								
F(1 km)	2.551								

E
Anglian Water Information

Sewer mapping (Ref:204685-2)

Pre-planning Assessment Report (Ref: 00025173, December 2017)



 Date: 14/11/16 Scale: 1:1250 Map Centre: 633770,253780 Data updated: 24/10/16 Our Ref: 204685 - 2 Wastewater Plan A2

Foul Sewer
Surface Sewer
Combined Sewer
Final Effluent
Rising Main (Colour denotes effluent type)
Private Sewer (Colour denotes effluent type)
Decommissioned Sewer (Colour denotes effluent type)
Outfall (Colour denotes effluent type)
Inlet (Colour denotes effluent type)
Manhole (Colour denotes effluent type)
Sewage Treatment Works
Pumping Station

stat.payments@atkinsglobal.com
49951

love every drop
anglianwater