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Technical Note

Station Road, Great Shelford

Flood Risk & Drainage Technical Note

Project No.	1281
Revision	B
Date	10th November 2021
Client	Churchill Retirement Living
Prepared	L Blackmore
Checked	T Gilbert
Authorised	C Yalden
File Ref.	**P:\1281 Station Road, Great Shelford\C Documents\1281 - Station Road, Great Shelford - Flood Risk & Drainage Technical Note**

1 Introduction

Introduction & Background

1.1 Awcock Ward Partnership has been commissioned by Churchill Retirement Living to prepare a Flood Risk and Drainage Technical Note in support of a full planning application for the redevelopment of the former 'The Stables', 'The Maltings' and 'Granary House' commercial offices and associated car park at Station Road, Great Shelford, Cambridge, CB22 5LR.

1.2 The redevelopment is proposed to provide 39 retirement apartments and associated communal areas, parking and amenities.

1.3 The site fronts on to Station Road to the west and is bound to the north by residential properties. The site is bound to the east by the Cambridge to London railway line. The adjacent site to the south has received planning permission redevelopment into a care home (S/2809/19/FL, Sep 2020).

1.4 The site is located outside of the designated conservation area boundary.

1.5 The location of the site in relation to its surroundings can be seen within Figure 1.1. Shelford railway station is 100m due north of the site.

Figure 1.1 – Site Location on Station Road, Great Shelford

1.6 This Technical Note has been prepared in accordance with the National Planning Policy Framework (NPPF) and Cambridgeshire County Council's Flood and Water Supplementary Planning Document (July, 2016) and Surface Water Planning Guidance (June, 2021).

1.7 This document sets out the existing baseline conditions in Section 2, the development proposal in Section 3. The proposed surface water management plan and foul water strategy that will serve the development is discussed in Sections 4 and 5 respectively, with Section 6 providing the Ownership and Maintenance information before concluding in Section 7.

2 Existing Baseline Conditions

Existing Site

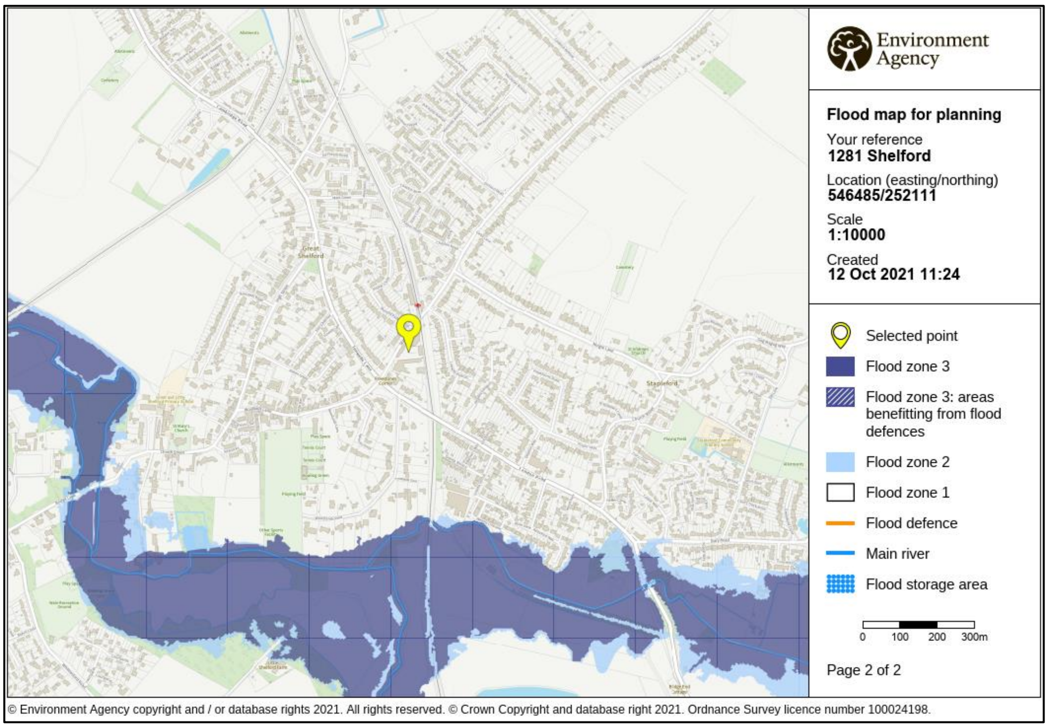
2.1 The existing brownfield site comprises existing commercial offices and car park with access from Station Road. The topographic survey confirms the site generally falls from north to south and west to east at a very shallow grade. An approximately average slope of 1 in 210 was calculated from the northern boundary to the southeast corner.

2.2 A copy of the topographic survey for the site can be seen as Appendix A.

Existing Flood Risk

2.3 An extract of the 'Flood Map for Planning' has been reproduced as Figure 2.1 and shows the site as being entirely within 'Flood Zone 1', as land assessed as having less than 1 in 1,000 annual probability of flooding from fluvial sources (<0.1%).

Figure 2.1 – EA Flood Map for Planning



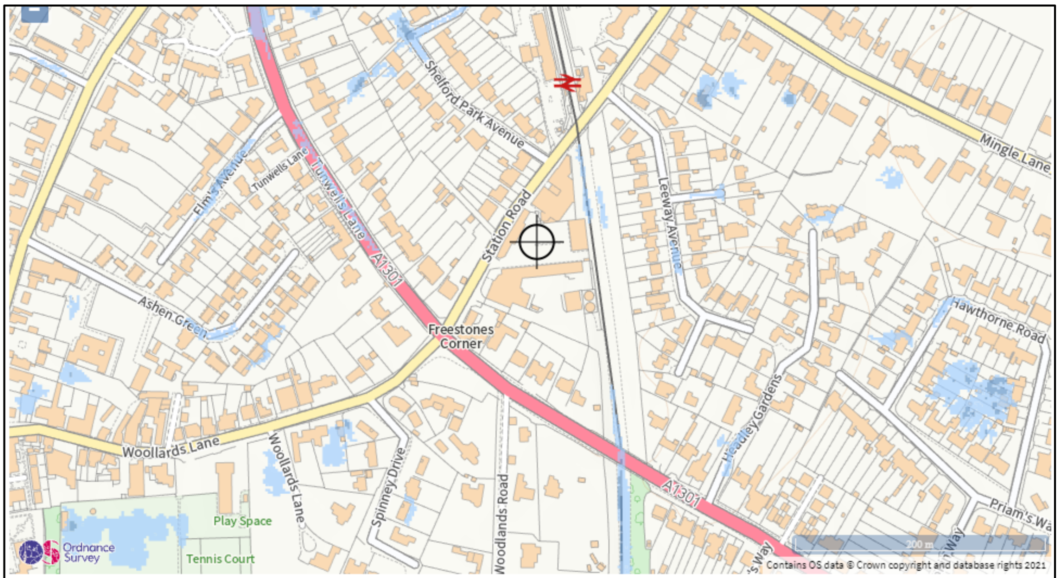
<u>Pluvial sources (surface water flooding)</u>

2.4 An extract of the EA's 'Flooding from Surface Water' maps for low and medium risk from surface water flooding are shown in Figures 2.2 and 2.3 respectively. The mapping is based on LIDAR data and indicates the typical conveyance routes of surface water runoff.

2.5 Figure 2.2 shows that the site is not at risk of flooding in up to the 1% annual probability, this being the typical lifetime for a residential development.

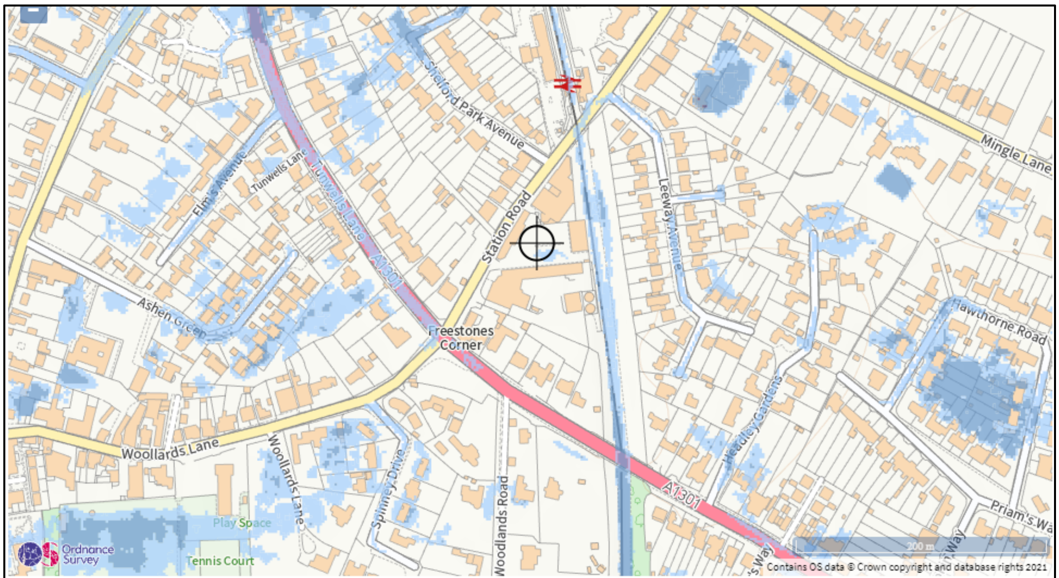
2.6 Figure 2.3 indicates a localised area within the site which would be at risk of surface water flooding between the 1 in 100 and 1 in 1,000 year return period storms (annual probability 0.1-1%), however this mapping does not account for existing drainage infrastructure and in this case the existing site drainage would assist in mitigating any concentration of runoff within the property.

Figure 2.2 – EA Flooding from surface water (medium risk, 1%)



Surface water flood risk: water depth in a medium risk scenario
Flood depth (millimetres)

Figure 2.3 – EA Flooding from surface water (low risk, <0.1%)



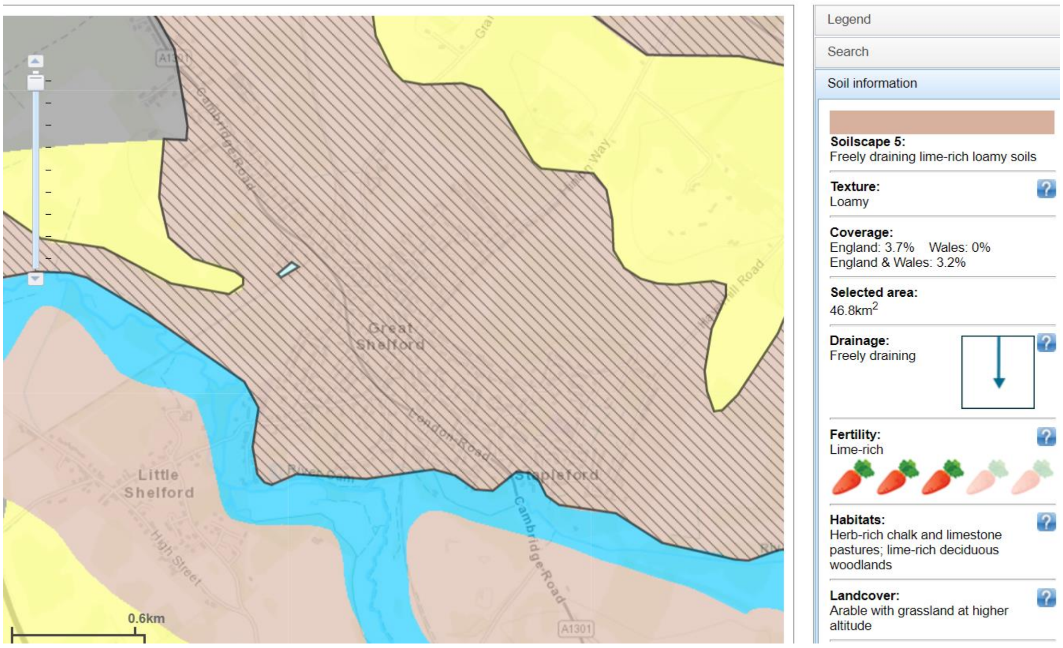
Surface water flood risk: water depth in a low risk scenario
Flood depth (millimetres)

2.7 The site does not fall within a groundwater flood risk area or lie within the maximum extent of flooding from any reservoirs and there are no known on-site flood risks associated with infrastructure failure.

Ground Conditions

2.8 The Soilscape dataset (Figure 2.4) suggests that the site lies within an area typically underlain with freely draining soils, however it should be considered that the existing site is entirely brownfield and will therefore comprise made ground which, subject to depths, may preclude use of infiltration.

Figure 2.4 – Soilscape Dataset



Geotechnical information conducted on the adjacent site, available from planning application S/3809/19/FL document 'Fuel Depot, 2 Station Road, Great Shelford, Detailed Quantitative Risk Assessment' compiled by SLR (S_3809_19_FL-PHASE_2_CONTAMINATION_REPORT_ORIGINAL _SUBMISSION -5529367), advised that ground conditions consist of *"Made Ground - comprising sandy clay to gravely sand of brick and flint to 1.5m depth."*

The sub-soils for the adjacent site were found to be West Melbury Marly Chalk Formation - recovered as light grey chalk bedrock to 9m depth, with groundwater levels within the underlying chalk bedrock at elevations of 14.5-16.0mOD, which would be as shallow as 2.0m below ground level (BGL).

2.9 Extracts from the SLR Report (2019) summarising conditions within the adjacent site can be found in Appendix B of this report.

2.10 It is considered likely that the application site will comprise similar baseline conditions and therefore the risk of elevated groundwaters may preclude the use of infiltration within this development.

2.11 It is recommended that a ground investigation is undertaken to verify the existing conditions and establish the feasibility of soakaway drainage as required by the approved drainage hierarchy.

Existing Site Drainage

2.12 The site is bordered by existing Anglian Water (AW) foul sewers to the west in Station Road. There are no nearby adopted surface water sewers.

2.13 An extract from Anglian Water's asset records can be seen as Figure 2.3.

Figure 2.3 – Anglian Water Asset Records

2.14 Survey information confirms that the existing site comprises separate storm and foul drainage.

2.15 The storm system drains to the south-east corner of the site and continues southwards off-site via a single 225mm diameter pipe. No existing attenuation is identified on any drainage surveys however, the flow is likely limited by the 150mm diameter outflow pipe linking to the existing 225mm storm drain.

2.16 The existing foul system drains to the south-west corner of the site and discharges to the adopted foul network beneath Station Road via a single 150mm (6inch) diameter pipe.

2.17 The above arrangements are identified on the utility survey drawing preferred to support the adjacent site (S/3809/19/FL).

2.18 Copies of the Anglian Water records and the Drainage Plan for application S/3809/19/FL (showing the Utility Survey information) are included within Appendix C and D of this report respectively.

Existing surface water runoff

2.19 The existing site comprises hard paved parking areas and roof space, with no landscaped/planting areas (100% impermeable catchment). Runoff generated by the existing site would be limited by the existing 150mm diameter surface water sewer that discharges off-site via the existing stormwater network. Excess flows would overwhelm the system and pond within the site or sheet flow overland.

2.20 The existing brownfield rates have been estimated based on the Modified Rational Method (HR Wallingford, 1990) and are included in Table 2.1, with a copy of the calculation sheet included as Appendix E.

Table 2.1 – Estimated Brownfield Runoff Rates (0.302ha)

Return Period	Brownfield Rate (l/s)
2 year	43.0
30 years	123.3
100 years	172.2

2.21 Cambridgeshire County Council's (CCC) 'Sustainable Drainage Design & Evaluation Guide' (SuDS Guidance) states "*On Brownfield sites (also known as Previously Developed Land), if infiltration of the 1 in 100 year rainfall event is not possible, the rate of discharge should be reduced to greenfield runoff rates.*"

2.22 The equivalent greenfield runoff rates for the site have been calculated using FEH, with the results summarised within Table 2.2 and the calculation sheet included within Appendix F of this report.

Table 2.2 – Equivalent Greenfield Runoff Rates (0.302ha)

Return Period	Greenfield Rate (l/s)
2 year	0.1
30 years	0.4
100 years	0.6

2.23 It is proposed that peak flows from the site are limited to greenfield rates as far as is practicable. In this instance limiting peak flows well below 1 l/s would require an impractically small control with increased risk of blockage, instead it is proposed to limit flows based on a minimum vortex flow control diameter with 100mm. This follows Sewer Sector Guidance where it recommends 100mm minimum control diameter where there is a risk of debris passing through the control.

3 Development Proposal

3.1 The development proposes to demolish the existing buildings and car parking area to enable the construction of a new apartment building which comprises 39 retirement apartments and associated facilities, parking and landscaping.

3.2 A copy of the proposed site layout has been included within Appendix G of this report.

4 Surface Water Management Plan

4.1 The site is less than 1ha (0.302ha) and is located within Flood Zone 1, therefore a Flood Risk Assessment is not required. This technical note has been prepared to assess any relevant flood risks and drainage constraints and to identify an appropriate drainage strategy for the proposed development.

4.2 To ensure the development is safe throughout its lifetime, the surface water strategy accounts for runoff in up to the 100 year return period.

4.3 The strategy also safeguards against the upper end allowances for climate change (40%) providing betterment over existing conditions, where the rate and volume of runoff would continue to increase due to climate change.

4.4 The existing site comprises made ground and is likely to be at risk of elevated groundwater which might preclude the use of infiltration drainage. For the purposes of this Surface Water Management Plan (SWMP) it is considered that surface water runoff will be attenuated on-site and discharged to the nearest and most appropriate receiving system.

4.5 At the discharge of conditions stage and to inform detailed design of the final drainage scheme, it is recommended that a ground investigation is completed and wherever practicable infiltration drainage is promoted.

4.6 There are no nearby watercourses or other surface water features therefore the proposed scheme looks to reuse the existing on-site storm network, which continues south off-site. It is noted that the adjacent development to the south has applied the same principles within their SWMP (ref. S/3809/19/FL).

4.7 Runoff generated by the proposed buildings, western access road and external hard paving will be collected and drained towards a new cellular attenuation tank beneath the amenity space to the south of the building.

4.8 All chambers immediately upstream of the tank will include silt traps, whilst the tank itself will include vented covers or a high-level vent pipe to mitigate air-locks.

4.9 Runoff from the central and eastern extents of access road will be directed towards areas of under-drained permeable paving. The use of permeable paving will be limited to the proposed parking bays within the eastern parking court. The permeable paving is included as a pollution control measure and forms part of the attenuation system.

4.10 Runoff from the tank and under-drained permeable paving will pass through a new flow control chamber prior to discharging to the existing network via the existing site connection. This will be subject to a CCTV condition survey to verify that the existing connection is suitable for reuse, or whether it requires any remedial works etc.

4.11 The peak rates of runoff will be limited as close to greenfield as practicable, based on a minimum 100mm diameter flow control, with a design flow peak rate of 5.9l/s.

4.12 The MicroDrainage source control module has been used to determine the storage requirements for the development. The output of this exercise has been summarised within Table 4.1, with copies of the modelling outputs included within Appendix H.

Table 4.1 – Attenuation Storage Volumes Requirements

Attenuation Feature	Attenuation Volume
Cellular Tank	$82m^3$
Permeable paving	$9m^3$
TOTAL	**91m3**

4.13 The proposed development achieves a substantial betterment compared to existing site conditions, as peak rates of discharge are limited to just 5.9 l/s peak in the 100 year return period storm with 40% climate change, compared to over 172 l/s from the existing brownfield site (97% betterment).

4.14 The proposed under-drained permeable paving and cellular attenuation will offer sufficient SuDS mitigation to offset the pollution indices for the site, in accordance with CIRIA C753.

Exceedance Measures

4.15 Beyond the 100-year critical storm, exceedance runoff will be directed towards the permeable paving in the car park and any residual areas of open space and/or the proposed car park, where any aboveground capacity can be utilised.

4.16 Beyond the limits of the site, exceedance flows would continue to the natural low point along the eastern boundary, as per existing conditions.

4.17 A copy of the preliminary drainage layout can be found on drawing 1281-01-PDL-1001 included within Appendix I.

Long-Term Storage

4.18 For previously developed sites, the 'Surface Water Planning Guidance' (June, 2021) recommend: "*The runoff volume from the development site to any surface water body or sewer in the 1% AEP (1 in 100), 6 hour rainfall event must be constrained to a value as close to the greenfield runoff volume for the same event, but should never exceed the runoff volume from the existing site.*"

4.19 The proposed scheme identifies a significant amount more permeable green space than the existing site, reducing the drained catchment from 0.302ha (100% imp.) to 0.185ha (61% imp.). Therefore, the runoff volume for the new development is a betterment on the volume from the existing site.

4.20 Long term storage would be required for the additional volume of surface water, above the greenfield runoff volume. However, outflow at a discharge rate of 2 l/s/ha (thus 0.6 l/s) will require an inadequate flow control size (too small) and long term storage has therefore been disregarded.

5 Foul Water Strategy

5.1 Foul flows generated by the proposed development will drain through a new private gravity foul network and will utilise the sites existing foul connection to the Anglian Water (AW) adopted foul sewer network, located in Station Road. If the existing foul system receives live flow from the adjacent site, the private sewer would be diverted.

5.2 A foul capacity enquiry has been submitted to AW to confirm connection to their existing network beneath Station Road, and capacity has been confirmed.

5.3 The proposed foul drainage arrangements can be seen on the preliminary drainage layout drawing 1281-01-PDL-1001 within Appendix I.

6 Ownership & Maintenance

6.1 All on-site piped drainage will remain private and will be designed in accordance with Building Regulations Part H and will become the responsibility of the building operator.

6.2 The proposed attenuation will be retained under private ownership and will be operated and maintained by the operator in accordance with CIRIA C753 and any manufacturer specific guidance.

6.3 At the detailed design stage, a 'Drainage Maintenance Plan' will be prepared. The Plan will set out maintenance tasks, responsibilities and frequencies for the entire drainage network.

7 Conclusion

7.1 The proposed development has been assessed in line with the National Planning Policy Framework, to allow the planning application to be progressed and to show that the development can be undertaken in an acceptable manner from a flood risk perspective.

7.2 The proposed development is located within Flood Zone 1 and is not known to be susceptible to flooding from pluvial, groundwater, infrastructure or artificial sources.

7.3 To ensure the development is safe throughout its lifetime, the surface water strategy accounts for runoff in up to the 1 in 100 year return period.

7.4 The strategy also safeguards against climate change (40%), providing betterment over existing conditions, where the rate and volume of runoff would continue to increase due to climate change.

7.5 The existing site comprises made ground and is likely to be a risk of elevated groundwater which might preclude the use of infiltration drainage. For the purposes of this SWMP it is considered that surface water runoff will be attenuated on-site and discharged to the nearest and most appropriate receiving system.

7.6 At the discharge of conditions stage and to inform detailed design of the final drainage scheme, it is recommended that a ground investigation is completed and wherever practicable infiltration drainage is promoted.

7.7 The peak rates of runoff will be limited as close to greenfield as practicable, based on a minimum 100mm diameter flow control.

7.8 Runoff from the tank and under-drained permeable paving will pass through a new flow control chamber prior to discharging to the existing network via the existing site connection. This will be subject to a CCTV condition survey.

7.9 The proposed development achieves a substantial betterment compared to existing site conditions, as peak rates of discharge are limited to just 5.9 l/s peak in the 100 year return period storm with 40% climate change, compared to over 172 l/s from the existing brownfield site (97% betterment).

7.10 The proposed under-drained permeable paving and cellular attenuation will offer sufficient SuDS mitigation to offset the pollution indices for the site, in accordance with CIRIA C753.

7.11 The impermeable drained catchment will reduce through the development, also reducing the volume of runoff from the site.

7.12 Beyond the 100-year critical storm, exceedance runoff will be directed towards any residual areas of open space and/or car parking, where any aboveground storage can be utilised.

7.13 Foul flows generated by the proposed development will be served by a new private gravity network, tying into an existing connection to the Anglian Sewer foul sewer network.

7.14 All on-site proposed drainage will remain private and will be designed in accordance with Building Regulations Part H and CIRIA C753 and will become the responsibility of the building operator.

7.15 As the development will be safe from flooding throughout its lifetime and will actively reduce the flood risk to properties within the downstream catchment, it is recommended that the Local Planning Authority confirm they have no objections to the proposed development.

Appendix A Topographic Survey


NO.1
STATION ROAD
THE STABLES
THE STABLES
NO.6
NO.4
THE MALTINGS
tarmac
LINK HOUSE
GRANARY HOUSE
BARLEY HOUSE
SURGERY
45
TANK
TANK

Appendix B Ground Investigation (Extracts)

3.0 Previous Assessment Work

3.1 Previous Assessments

The Site has been subject to several phases of previous assessment work, which are listed in Table 3-1 and summarised below.

Table 3-1: Previous Assessment Reports

Date	Document Title and Author
August 2008	Environmental Site Investigation Report, Shelford Energy Ltd, 2 Station Road, Great Shelford, Cambridge, CM3 5LT. REC Report 50740/report 1.1.
November 2008	Site Specific Controlled Waters Risk Assessment, Shelford Energy Ltd, 2 Station Road, Shelford, Cambridge, CM3 5LT. REC Report 50740/report 3.1.
September 2014	Groundwater Remediation Verification Report, Shelford Energy, for Mr Paul Davies. OHES Report Ref: R001MT – 14.7441.
February 2015	Environmental Site Assessment and DQRA, Station Road Great Shelford, for Mr Paul Davies. OHES Report Ref: R001MT – 15.8182.
February 2016	Watson Oil Depot, Great Shelford, Environmental Site Assessment Report for FH Great Shelford Limited. SLR Consulting Report Reference 404.05952.00001.
June 2016	Groundwater Monitoring Results (SLR Consulting email)
January 2018	2 Station Road, Great Shelford, Land Quality Assessment Report for FH Great Shelford Limited. SLR Consulting Report Reference 404.05952.00001.
October 2018	2 Station Road, Great Shelford, Phase 1 Data Review and Preliminary Land Quality Assessment Report for FH Great Shelford Limited. SLR Consulting Report Reference 416-05952-00003-PLQRA
October 2018	2 Station Road, Great Shelford, Groundwater Monitoring Report for FH Great Shelford Limited. SLR Consulting Report Reference 416-05952-00003-GWMON

3.2 Summary of Previous Environmental Assessments

The details from the various phases of investigation are provided below; for further detail, the reports listed in Table 3-1 should be read in full.

- A total of 58 boreholes have been drilled across the site between July 2008 and December 2015 during the various phases of investigation that have been completed at the Site. A schedule of the boreholes advanced across the Site and adjacent land are summarised in Table 3-2 below and shown on Drawing 02.

Table 3-2: Schedule of Existing Boreholes

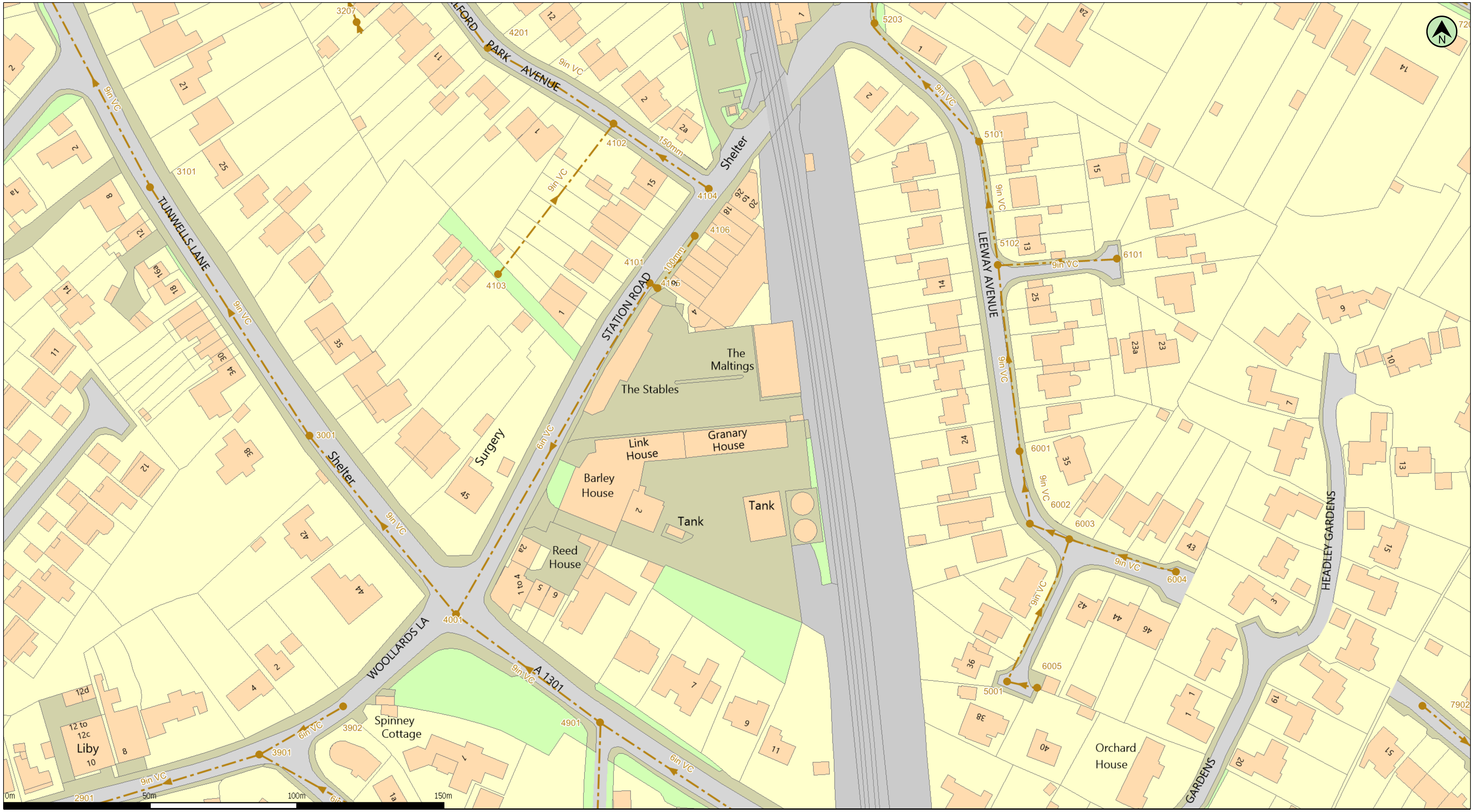
Date Drilled	Consultant	No. of BH Drilled	BH Ref.	No. of Monitoring Wells Installed	Max. Drilled Depth (mbgl)
July 2008	REC	10	CP1 to CP5, HA01 & WS1 to WS4	10	8
Sept 2008	REC	9	WS21 to WS27 (located in car park of offices to north) WS28 & WS29 (fuel depot)	9	5
July 2009	OHES	2	WS32 & WS33	2	5
Nov 2009	OHES	14	AW01 to AW14	14	9
Jan 2015	OHES	15	WS401 to WS412, & HA401 to HA403	0	5
Dec 2015	SLR	11	BH101 to BH111	11	7.3

- All boreholes were drilled using either windowless sampling, percussive or rotary coring techniques, with borehole geology descriptions based on logged drilling arisings. Compiled borehole logs are included within SLRs 2018 PLQRA report. Ground conditions were recorded to comprise the following sequence:
    - Made Ground – comprising sandy clay to gravely sand of brick and flint to 1.5m depth;
    - Superficial Deposits (River Terrace Deposits) – comprising brown gravelly Sand, with gravel of flint and chalk, typically <2m, but locally up to 3.2m in the northeast corner;
    - West Melbury Marly (WMM) Chalk Formation – recovered as light grey chalk bedrock to 9m depth. Chalk recovered in disturbed condition. Interpreted as West Melbury Marly Chalk Formation.
- Significant and widespread soil hydrocarbon impact was recorded within the boreholes drilled across the Site, typically at depths of between 2m and 5m, associated with, and below, the "smear zone" of water table fluctuation. Strong hydrocarbon odours, grey staining and elevated field headspace readings were recorded in the boreholes across the vertical profile with the highest impact typically found in the chalk bedrock between 3m and 5m depth (13m and 15mAOD).
- Numerous groundwater monitoring events have been completed at the Site associated with the following key periods of work:
    - 2009 to 2014: completed before and during groundwater remediation works;
    - 2015 to 2017: undertaken after groundwater remediation works were completed in 2014.
- Groundwater is present within the underlying chalk bedrock and has historically been recorded at depths of between 2.3m and 5m, corresponding to elevations of between 14.5m and 16mAOD with a relatively consistent seasonal fluctuation in the groundwater table of around 1 metre. The overlying Superficial Deposits are unsaturated. Groundwater depths were generally deeper in the higher elevation central parts of the Site at around 4.5m to 5.0m and shallower in the lower elevation parts of the site along the eastern and western boundaries (typically at around 3m depth).
- The 2018 groundwater monitoring data indicates an overall hydraulic gradient of 0.0024 towards the west southwest to southwest.

- Extensive and widespread hydrocarbon impact to groundwater has been recorded across the central area of the Site, including “floating oil” (LNAPL[1]), close to fuel storage and dispensing areas, historically at measured thicknesses of >350mm.
- Groundwater remediation was undertaken by OHES between 2009 and 2014, although with no regulatory involvement. This comprised a combination of oil skimming and total fluids “pump and treat” from the “AW” series of abstraction wells located in the centre of the with the objective of reducing the thickness of LNAPL floating on groundwater. OHES reported over 8 million litres (8,000 cubic metres) of groundwater was treated and 1,080 litres of oil was removed, with a corresponding reduction in LNAPL thicknesses to a few millimetres by system closure in 2014.
- Boreholes advanced by OHES and SLR in 2015 following completion of the remediation works recorded the continued presence of widespread hydrocarbon impact to soil and groundwater, generally at depths of 3m to 5m below the site.
- Numerous groundwater monitoring visits completed by SLR between 2015 and 2018 recorded low, and stable or declining, dissolved phase concentrations and continued measurable accumulations of LNAPL in only a few isolated monitoring well locations. These LNAPL accumulations corresponded to seasonal water table fluctuations with greatest LNAPL thicknesses recorded when groundwater was at its lowest level. This is likely to be attributable to gravity drainage from residual historical LNAPL previously trapped in fractures below the seasonal water table. Compiled LNAPL thickness data and plots are included as Appendix 09.
- Field permeability testing indicates the chalk to have hydraulic conductivity values of between 0.14m/d and 1.29m/d with an average value of 0.5m/d (5.77×10^{-6} m/s).
- Groundwater samples collected in 2018 recorded concentrations of petroleum hydrocarbons above water quality standards (WQS) or laboratory detection limits, with the majority of exceedances recorded in the centre of the Site within the C10 to C21 aromatic hydrocarbon bands, consistent with a weathered diesel and kerosene fuel source.
- The previous investigations identified the following historical contaminant sources as illustrated on Drawing 03:
    - Area A: The eastern area of the fuel depot yard, centred on the existing main AST farm and current fuel loading areas;
    - Area B: The area beneath the former overhead refuelling gantry and historical AST bunds.

[1] light non-aqueous phase liquid also known as separate or free phase fuel

Appendix C Severn Trent Water Records



(c) Crown copyright and database rights 2021 Ordnance Survey 100022432	Date: 11/04/21	Scale: 1:1250	Map Centre: 546505,252101	Data updated: 01/04/21	Our Ref: 537194 - 1	Wastewater Plan A3

This plan is provided by Anglian Water pursuant its obligations under the Water Industry Act 1991 sections 198 or 199. It must be used in conjunction with any search results attached. The information on this plan is based on data currently recorded but position must be regarded as approximate. Service pipes, private sewers and drains are generally not shown. Users of this map are strongly advised to commission their own survey of the area shown on the plan before carrying out any works. The actual position of all apparatus MUST be established by trial holes. No liability whatsoever, including liability for negligence, is accepted by Anglian Water for any error or inaccuracy or omission, including the failure to accurately record, or record at all, the location of any water main, discharge pipe, sewer or disposal main or any item of apparatus. This information is valid for the date printed. This plan is produced by Anglian Water Services Limited (c) Crown copyright and database rights 2021 Ordnance Survey 100022432.This map is to be used for the purposes of viewing the location of Anglian Water plant only. Any other uses of the map data or further copies is not permitted. This notice is not intended to exclude or restrict liability for death or personal injury resulting from negligence.

Foul Sewer
Surface Sewer
Combined Sewer
Final Effluent
Rising Main*
Private Sewer*
Decommissioned Sewer*

Outfall*
Inlet*
Manhole*

Sewage Treatment Works
Public Pumping Station
Decommissioned Pumping Station

*(Colour denotes effluent type)

john.duncan@nrswa.net

Great Shelford

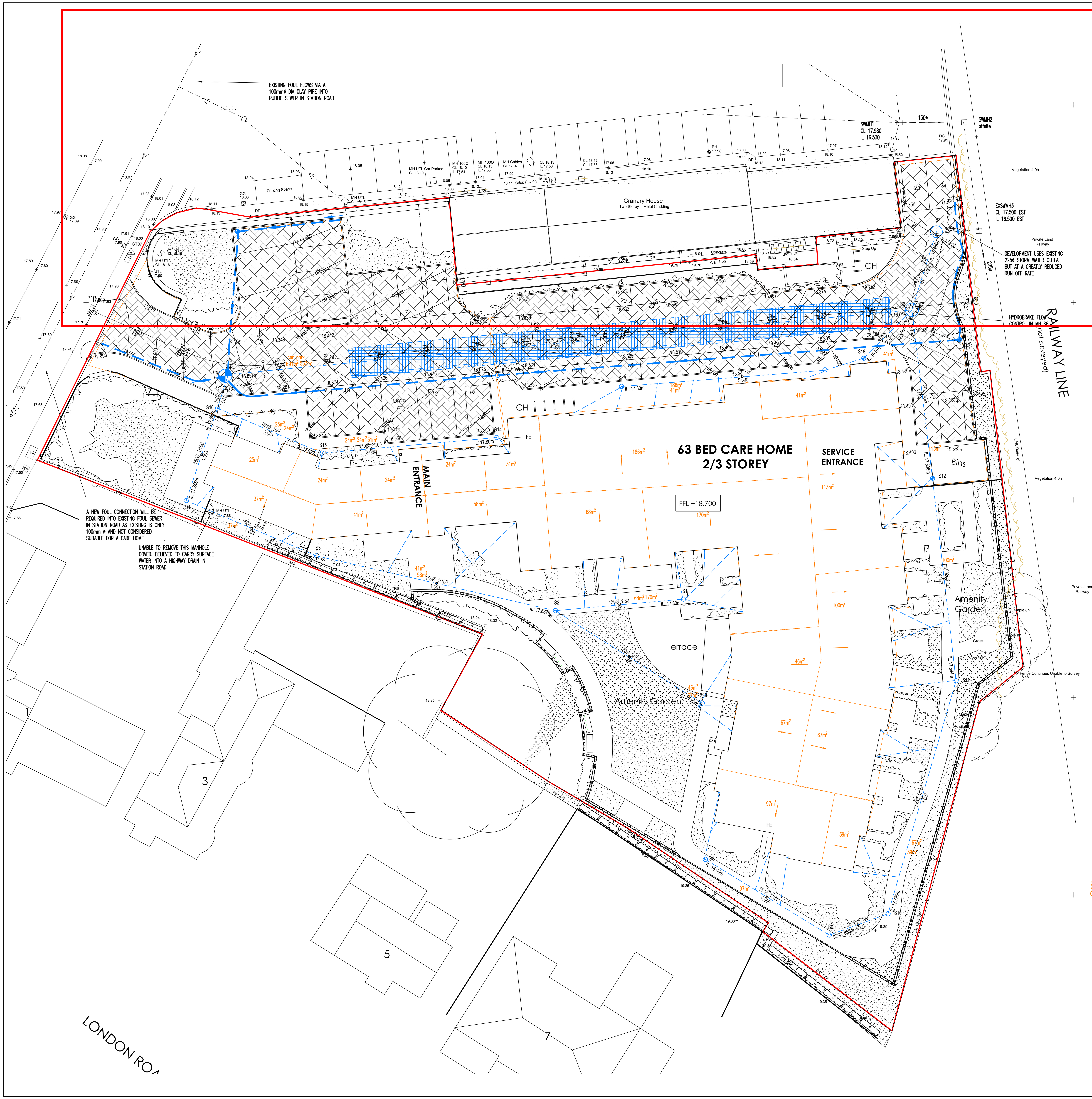
love every drop anglianwater

Manhole Reference	Liquid Type	Cover Level	Invert Level	Depth to Invert
3001	F	15.996	14.386	1.61
3101	F	15.496	13.886	1.61
3207	F	-	-	-
3901	F	16.493	14.344	2.149
3902	F	16.526	14.633	1.893
4001	F	16.776	14.776	2
4101	F	18.547	16.917	1.63
4102	F	17.316	15.536	1.78
4103	F	17.358	16.208	1.15
4104	F	18.36	15.784	2.576
4105	F	-	-	-
4106	F	-	-	-
4201	F	16.036	14.546	1.49
4901	F	18.001	16.151	1.85
5001	F	-	-	-
5101	F	-	-	-
5102	F	-	-	-
5203	F	-	-	-
6001	F	-	-	-
6002	F	-	-	-
6003	F	-	-	-
6004	F	-	-	-
6005	F	-	-	-
6101	F	-	-	-
7902	F	-	-	-

Manhole Reference	Liquid Type	Cover Level	Invert Level	Depth to Invert

Manhole Reference	Liquid Type	Cover Level	Invert Level	Depth to Invert

Appendix D Utility Survey



MH No.	MANHOLE DIAMETER (mm)	MANHOLE TYPE	COVER LEVEL (m)	INVERT LEVEL (m)	DEPTH TO SOFFIT (m)
S1	450	Type 3	18.55	17.80	0.60
S2	450	Type 3	18.55	17.637	0.763
S3	450	Type 3	18.55	17.389	1.011
S4	450	Type 3	18.55	17.246	1.154
S5	1200	Catchpit	18.147	16.857	1.065
S6	1350	Type 2	18.090	16.664	1.201
S7	1200	Type 2	17.880	16.486	1.246
S8	450	Type 3	18.55	18.00	0.40
S9	450	Type 3	18.55	17.853	0.547
S10	450	Type 3	18.55	17.790	0.610
S11	450	Type 3	18.55	17.544	0.856
S12	450	Type 3	18.55	17.336	1.064
S13	450	Type 3	18.55	17.80	0.60
S14	450	Type 3	18.55	17.80	0.60
S15	450	Type 3	18.625	17.622	0.853
S16	450	Type 3	18.32	17.147	1.023
S17	450	Type 3	18.50	17.80	0.55
S18	450	ST	18.50	16.970	1.380
S19	450	ST	18.15	17.370	0.630
S20	450	ST	18.65	17.815	0.685
S21	450	ST	18.00	17.060	0.790

NOTES

~~1. DO NOT SCALE FROM THIS DRAWING. USE FIGURED DIMENSIONS ONLY, IF IN DOUBT ASK.~~

2. ALL DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.

3. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ALL RELEVANT ENGINEERS AND ARCHITECTS DRAWINGS AND SPECIFICATIONS.

LEGEND

- FOUL DRAIN PRIVATE
- FOUL MANHOLE
- FOUL TYPE 3 INSPECTION CHAMBER PRIVATE WAVIN RANGE 450 MAX DEPTH 2.0m
- FOUL TYPE 4 INSPECTION CHAMBER PRIVATE WAVIN RANGE 225 MAX DEPTH 0.6m
- RODDING EYE PRIVATE
- SURFACE WATER TYPE 3 INSPECTION CHAMBER PRIVATE WAVIN RANGE 450 MAX DEPTH 2.0m
- HYDROBRAKE CHAMBER
- SURFACE WATER DRAIN PRIVATE
- SURFACE WATER SILT TRAP
- AQUACELL CORE OR SIMILAR ATTENUATION CRATES. LAID ON 100mm BED OF COARSE SAND. 100mm COARSE SAND TO SIDES OF EXCAVATION AND LAID DIRECTLY ON TOP OF MEMBRANE. IMPERMEABLE 2000g MEMBRANE. SYSTEM TO BE VENTED WITH 110mm PIPE USING SIDE CONNECTION TO BESPOKE VENTILATION BOX WITH BALLOON GRATING FLANGE ADAPTORS
- 150ø PERFORATED PIPE — PERMEABLE PAVING TO CAR PARK COLOUR TO ARCHITECT/CLIENT REQUIREMENTS. SEMI PERFORATED PIPE TO CARRY FLOW FROM DRAINAGE BLANKET SUMP INTO SILT TRAPS
- EXISTING FOUL SEWER
- EXISTING SURFACE WATER SEWER
- 18.000 + PROPOSED CAR PARK LEVEL +

A CONTRACTOR WHO IS ACCREDITED TO THE BRITISH PLASTICS FEDERATION PLASTIC PIPES GROUP MUST UNDERTAKE THE INSTALLATION OF STRUCTURAL WALL PLASTIC PIPES THAT ARE OFFERED FOR ADOPTION

PIPE MATERIALS
ALL PIPEWORK TO BE 100mm ø UNLESS NOTED OTHERWISE
100mmø MUST HAVE CLASS SN8 LOADING
150mmø WAVIN/ OSMA ULTRARIB SYSTEM
225mmø – 300mmø MARLEY QUANTUM SEWER
301mmø – 600mmø POLYPIPE/ RIDGISEWER TWINWALL SYSTEM
601mmø – CONCRETE OR WEHOLITE SUBJECT TO DESIGN

A SECTION 106 APPLICATION TO CONNECT MUST BE MADE TO THE WATER AUTHORITY. THE DEVELOPER SHALL GIVE 21 DAYS NOTICE PRIOR TO CONNECTION. THE WORKS MAY ONLY BE UNDERTAKEN BY AN SSIP HEALTH AND SAFETY APPROVED CONTRACTOR

REVISION / DATE		DRAWN / CHECKED
B	MINOR ADJUSTMENTS FOLLOWING sSOUTH CAMBRIDGESHIRE CC	–
12.06.20	COMMENTS	–
A	PLAN UPDATED WITH REVISED ARCHITECT LAYOUT 110–E	–
18.05.20	–	–

Client:
PORTHAVEN

Project Title:
2 STATION ROAD
GREAT SHELFORD
CAMBRIDGESHIRE

Drawing Title:
PROPOSED DRAINAGE STRATEGY

Drawn By: RN
Scales: 1:200@A1
Date: AUG 2019
Drawing Number: 10540–500
Rev: B
Checked By:
Date:
Drawing Status: PLANNING

NJP Consulting Engineers Limited
20 St Andrews Crescent
Cardiff CF10 3DD

Tel: 029 2072 9500
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Web: www.njpuk.com

Appendix E Brownfield Runoff Calculations

Project No.	*1281*	
Project Title	*Station Road, Great Shelford*	
Client	*CRL*	
Sheet Ref	*P:\1281 Station Road, Great Shelford\D Design and Analysis\SPREADSHEETS\01 Drainage\03 Sewer Design\[1281 Colebrook White Equation (pipe velocity & capacity).xlsx]Colebrook-White*	

Calcs by	*L Blackmore*
Checked by	*TG*
Approved by	*CPY*
Date	*02.11.2021*
Revision	*A*

Catchment area analysis based on Modified Rational Method equation (HR Wallingford, 1990);

$$Q_{BAR} = 2.78 \cdot i \cdot A$$

Hydrological Region: **5** *see map

Where: Q_{BAR} *Average discharge (l/s)*

i *Rainfall intensity (mm/hr)*

A *Catchment area (m^2)*

i **57.6** mm/hr *see map

Return Period	**2yr**	**30yr**	**100yr**	
Growth Factor (Q/QBAR)	**0.89**	**2.55**	**3.56**	
Critical Area (ha)	**0.2805**	**0.0979**	**0.0701**	(area that can freely drain)

Brownfield flow rate analysis based on Modified Rational Method (HR Wallingford, 1990);

Area (ha):			2yr	30yr	100yr	QBAR
Area (ha):	**0.302**	**BF flow (l/s):**	**43.04**	**123.31**	**172.16**	**48.36**

Appendix F Greenfield Runoff Rates

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Letisha Blackmore
Site name:	1281 Great Shelford
Site location:	G Shelford, Cambridgeshire

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance "Rainfall runoff management for developments", SC030219 (2013) , the SuDS Manual C753 (Ciria, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Site Details

Latitude:	52.14784° N
Longitude:	0.13956° E
Reference:	1521797951
Date:	Oct 20 2021 16:45

Runoff estimation approach FEH Statistical

Site characteristics

Total site area (ha): 0.302

Methodology

Q_{MED} estimation method:	Calculate from BFI and SAAR
BFI and SPR method:	Specify BFI manually
HOST class:	N/A
BFI / BFIHOST:	0.771
Q_{MED} (l/s):	
Q_{BAR} / Q_{MED} factor:	1.12

Hydrological characteristics	Default	Edited
SAAR (mm):	540	540
Hydrological region:	5	5
Growth curve factor 1 year:	0.87	0.87
Growth curve factor 30 years:	2.45	2.45
Growth curve factor 100 years:	3.56	3.56
Growth curve factor 200 years:	4.21	4.21

Notes

(1) Is Q_{BAR} < 2.0 l/s/ha?

When Q_{BAR} is < 2.0 l/s/ha then limiting discharge rates are set at 2.0 l/s/ha.

(2) Are flow rates < 5.0 l/s?

Where flow rates are less than 5.0 l/s consent for discharge is usually set at 5.0 l/s if blockage from vegetation and other materials is possible. Lower consent flow rates may be set where the blockage risk is addressed by using appropriate drainage elements.

(3) Is SPR/SPRHOST ≤ 0.3?

Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Greenfield runoff rates	Default	Edited
Q_{BAR} (l/s):		0.16
1 in 1 year (l/s):		0.14
1 in 30 years (l/s):		0.38
1 in 100 year (l/s):		0.56
1 in 200 years (l/s):		0.66

Appendix G Proposed Site Plan

REVISIONS

Rev. | Date | By

NOTE

Allowing for widening of existing footpath to 2.00m to Station Road frontage.

BOUNDARY TREATMENT

A-B-C	1.1m metal railings with enhanced landscaping (adjacent to public footpath)
C-D	1.1m railing around existing low level brick wall
D-E	Existing brick wall retained and made good where required
E-A	1.8M Timber Fencing with landscaping

FLAT MIX:

1 BED APARTMENTS	= 24
2 BED APARTMENTS	= 15
TOTAL	= 39

- 1 BED APARTMENTS
- 2 BED APARTMENTS
- COMMUNAL AREAS

planning issues
TOWN PLANNING AND ARCHITECTURAL DESIGN

Gideon Lemberg RIBA
Design Director
Churchill House * Alban Park
Hatfield Road * St Albans AL4 0LA
Telephone: (01727) 733650
Fax : (01727) 733651
E-mail design@planningissues.co.uk

Client

Churchill Retirement Living

Project Title

PROPOSED RETIREMENT HOUSING
Station Road,
Great Shelford,
Cambridge, CB22 5LR

Drawing Title

SITE PLAN

Scale	1:200 @ A1	Date	01.11.21
Drawn	MJS/EKS	Checked	GSL
Drawing No.		Rev.	

40040GS/PA01

Appendix H MicroDrainage Calculations

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

Half Drain Time : 157 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	98.636	0.636	0.0	4.5	4.5	49.6	O K
30 min Summer	98.797	0.797	0.0	4.5	4.5	62.1	O K
60 min Summer	98.915	0.915	0.0	4.5	4.5	71.2	O K
120 min Summer	99.518	1.518	0.0	5.5	5.5	78.7	O K
180 min Summer	99.542	1.542	0.0	5.5	5.5	79.8	O K
240 min Summer	99.528	1.528	0.0	5.5	5.5	79.2	O K
360 min Summer	98.977	0.977	0.0	4.5	4.5	76.1	O K
480 min Summer	98.907	0.907	0.0	4.5	4.5	70.6	O K
600 min Summer	98.836	0.836	0.0	4.5	4.5	65.1	O K
720 min Summer	98.765	0.765	0.0	4.5	4.5	59.6	O K
960 min Summer	98.615	0.615	0.0	4.5	4.5	47.9	O K
1440 min Summer	98.363	0.363	0.0	4.5	4.5	28.3	O K
2160 min Summer	98.184	0.184	0.0	4.3	4.3	14.4	O K
2880 min Summer	98.121	0.121	0.0	4.0	4.0	9.4	O K
4320 min Summer	98.088	0.088	0.0	2.9	2.9	6.9	O K
5760 min Summer	98.075	0.075	0.0	2.3	2.3	5.8	O K
7200 min Summer	98.067	0.067	0.0	2.0	2.0	5.2	O K
8640 min Summer	98.062	0.062	0.0	1.7	1.7	4.8	O K
10080 min Summer	98.059	0.059	0.0	1.6	1.6	4.6	O K
15 min Winter	98.718	0.718	0.0	4.5	4.5	55.9	O K
30 min Winter	98.900	0.900	0.0	4.5	4.5	70.1	O K
60 min Winter	99.556	1.556	0.0	5.5	5.5	80.4	O K
120 min Winter	99.766	1.766	0.0	5.9	5.9	89.9	O K
180 min Winter	99.790	1.790	0.0	5.9	5.9	91.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	154.100	0.0	52.6	18
30 min Summer	99.217	0.0	67.9	33
60 min Summer	60.715	0.0	83.4	62
120 min Summer	38.326	0.0	105.5	110
180 min Summer	28.794	0.0	119.0	142
240 min Summer	23.270	0.0	128.3	174
360 min Summer	16.940	0.0	140.1	252
480 min Summer	13.365	0.0	147.4	322
600 min Summer	11.059	0.0	152.4	390
720 min Summer	9.444	0.0	156.2	460
960 min Summer	7.324	0.0	161.5	596
1440 min Summer	5.085	0.0	168.0	820
2160 min Summer	3.529	0.0	174.8	1144
2880 min Summer	2.736	0.0	180.5	1472
4320 min Summer	1.941	0.0	191.7	2204
5760 min Summer	1.543	0.0	202.9	2936
7200 min Summer	1.309	0.0	215.0	3672
8640 min Summer	1.157	0.0	227.6	4336
10080 min Summer	1.050	0.0	240.8	5128
15 min Winter	154.100	0.0	59.0	18
30 min Winter	99.217	0.0	76.2	32
60 min Winter	60.715	0.0	93.5	60
120 min Winter	38.326	0.0	118.3	116
180 min Winter	28.794	0.0	133.4	146

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

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
240 min Winter	99.771	1.771	0.0	5.9	5.9	90.1	O K
360 min Winter	99.649	1.649	0.0	5.7	5.7	84.6	O K
480 min Winter	99.510	1.510	0.0	5.4	5.4	78.4	O K
600 min Winter	98.918	0.918	0.0	4.5	4.5	71.5	O K
720 min Winter	98.817	0.817	0.0	4.5	4.5	63.7	O K
960 min Winter	98.592	0.592	0.0	4.5	4.5	46.1	O K
1440 min Winter	98.260	0.260	0.0	4.5	4.5	20.2	O K
2160 min Winter	98.115	0.115	0.0	3.8	3.8	9.0	O K
2880 min Winter	98.090	0.090	0.0	3.0	3.0	7.0	O K
4320 min Winter	98.070	0.070	0.0	2.1	2.1	5.5	O K
5760 min Winter	98.061	0.061	0.0	1.7	1.7	4.7	O K
7200 min Winter	98.055	0.055	0.0	1.4	1.4	4.3	O K
8640 min Winter	98.051	0.051	0.0	1.3	1.3	4.0	O K
10080 min Winter	98.049	0.049	0.0	1.2	1.2	3.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	23.270	0.0	143.7	184
360 min Winter	16.940	0.0	157.0	262
480 min Winter	13.365	0.0	165.2	336
600 min Winter	11.059	0.0	170.8	422
720 min Winter	9.444	0.0	175.1	498
960 min Winter	7.324	0.0	181.0	636
1440 min Winter	5.085	0.0	188.3	836
2160 min Winter	3.529	0.0	196.0	1124
2880 min Winter	2.736	0.0	202.4	1468
4320 min Winter	1.941	0.0	215.0	2184
5760 min Winter	1.543	0.0	227.6	2864
7200 min Winter	1.309	0.0	241.1	3656
8640 min Winter	1.157	0.0	255.3	4320
10080 min Winter	1.050	0.0	270.2	5120

Rainfall Details

Rainfall Model	FEH	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
FEH Rainfall Version	2013	Cv (Winter)	0.840
Site Location	GB 546487 252112 TL 46487 52112	Shortest Storm (mins)	15
Data Type	Point	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+40

Time Area Diagram

Total Area (ha) 0.185

Time From:	**(mins) To:**	**Area (ha)**
0	4	0.185

Model Details

Storage is Online Cover Level (m) 100.000

Complex Structure

Cellular Storage

Invert Level (m) 98.000 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	82.0	50.0	1.000	82.0	80.0	1.001	0.0	80.0

Porous Car Park

Infiltration Coefficient Base (m/hr)	0.00000	Width (m)	5.0
Membrane Percolation (mm/hr)	1000	Length (m)	30.0
Max Percolation (l/s)	41.7	Slope (1:X)	0.0
Safety Factor	2.0	Depression Storage (mm)	5
Porosity	0.30	Evaporation (mm/day)	3
Invert Level (m)	99.500	Cap Volume Depth (m)	0.420

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0100-4500-1000-4500
Design Head (m)	1.000
Design Flow (l/s)	4.5
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	100
Invert Level (m)	98.000
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.000	4.5	Kick-Flo®	0.630	3.6
Flush-Flo™	0.292	4.5	Mean Flow over Head Range	-	3.9

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)	Depth (m)	Flow (l/s)
0.100	3.3	0.800	4.1	2.000	6.2	4.000	8.6	7.000	11.2
0.200	4.4	1.000	4.5	2.200	6.5	4.500	9.1	7.500	11.6
0.300	4.5	1.200	4.9	2.400	6.8	5.000	9.6	8.000	12.0
0.400	4.4	1.400	5.3	2.600	7.0	5.500	10.0	8.500	12.3
0.500	4.2	1.600	5.6	3.000	7.5	6.000	10.4	9.000	12.7
0.600	3.8	1.800	5.9	3.500	8.1	6.500	10.8	9.500	13.0

Appendix I Preliminary Drainage Layout

Notes:

1. The proposed development has been assessed in line with the National Planning Policy Framework, to allow the planning application to be progressed and to show that the development can be undertaken in an acceptable manner from a flood risk perspective.
2. The proposed development is located within Flood Zone 1 and is not known to be susceptible to flooding from pluvial, groundwater, infrastructure or artificial sources.
3. To ensure the development is safe throughout its lifetime, the surface water strategy accounts for runoff in up to the 1 in 100 year return period.
4. The strategy also safeguards against climate change (40%), providing betterment over existing conditions, where the rate and volume of runoff would continue to increase due to climate change.
5. The existing site comprises made ground and is likely to be a risk of elevated groundwater which might preclude the use of infiltration drainage. For the purposes of this SWMP it is considered that surface water runoff will be attenuated on-site and discharged to the nearest and most appropriate receiving system.
6. At the discharge of conditions stage and to inform detailed design of the final drainage scheme, it is recommended that a ground investigation is completed and wherever practicable infiltration drainage is promoted.
7. The peak rates of runoff will be limited as close to greenfield as practicable, based on a minimum 100mm diameter flow control.
8. Runoff from the tank and under-drained permeable paving will pass through a new flow control chamber prior to discharging to the existing network via the existing site connection. This will be subject to a CCTV condition survey.
9. The proposed development achieves a substantial betterment compared to existing site conditions, as peak rates of discharge are limited to just 5.9 l/s peak in the 100 year return period storm with 40% climate change, compared to over 172 l/s from the existing brownfield site (97% betterment).
10. The proposed under-drained permeable paving and cellular attenuation will offer sufficient SuDS mitigation to offset the pollution indices for the site, in accordance with CIRIA C753.
11. The impermeable drained catchment will reduce through the development, also reducing the volume of runoff from the site.
12. Beyond the 100-year critical storm, exceedance runoff will be directed towards any residual areas of open space and/or car parking, where any aboveground storage can be utilised.
13. Foul flows generated by the proposed development will be served by the a new private gravity network, tying into an existing connection to the Anglian Sewer foul sewer network.
14. All on-site proposed drainage will remain private and will be designed in accordance with Building Regulations Part H and CIRIA C753 and will become the responsibility of the building operator.
15. As the development will be safe from flooding throughout its lifetime and will actively reduce the flood risk to properties within the downstream catchment, it is recommended that the Local Planning Authority confirm they have no objections to the proposed development.

Area Summary Schedule

Exist. Impermeable Catchment	0.302 ha
Net Developable Area	0.302 ha
Prop. Impermeable Catchment	0.185 ha
Prop. Percentage Impermeable	61%

Equivalent Greenfield Runoff Rates

The greenfield runoff rates have been assessed for the net developable area using the FEH Method. The calculation excludes large areas of open space which will remain undeveloped.

Return Period	Greenfield Rate (l/s)
2yr	0.14
30yr	0.38
100yr	0.56

Attenuation Summary

Complex Attenuation Feature

Catchment	0.185 ha
Hydraulic Control	Hydrobrake @IL+0.000 Ref: MD-SHE-100-4500-1000-4500
Type	Cellular Storage with Under-drained Porous Parking
Cellular Storage Porosity	95%
Cellular Storage Dimensions	4.0 m x 20.5 m x 1.0 m deep
Porous Parking Porosity	30%
Porous Parking Dimensions	5.0 m x 30.0 m x 0.5 m deep
100yr+40% Volume Required:	**91.0 m^3**
100yr+40% Discharge Rate:	**5.9 l/s**

Key

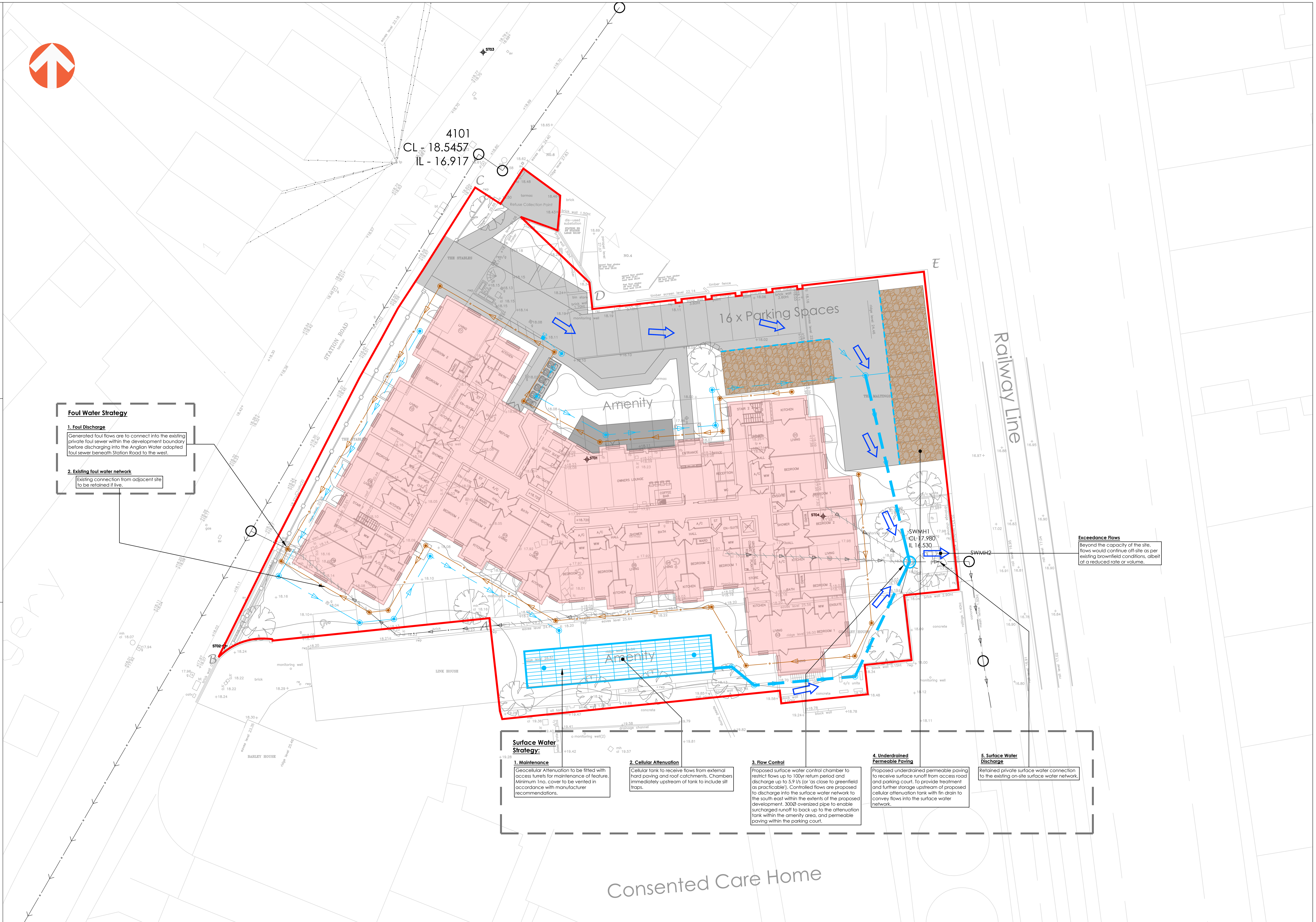
Site Boundary

Existing Utilities

- Adopted Foul Water Sewer
- Private Surface Water Sewer
- Private Foul Water Sewer

Proposed Drainage

- Impermeable Building Catchment
- Impermeable Highway Catchment
- Underdrained Permeable Paving
- Cellular Storage Tank
- Private Surface Water Sewer
- 300Ø Oversized Private Surface Water Sewer
- Private Yard Gully and Connection
- Private Surface Inspection Chamber
- Flow Control Chamber
- Fin Drain
- Private Foul Water Sewer
- Private Foul Inspection Chamber
- Private Foul Manhole Chamber
- Overland exceedance drain
- Existing Drainage to be Abandoned



Foul Water Strategy

1. Foul Discharge

Generated foul flows are to connect into the existing private foul sewer within the development boundary before discharging into the Anglian Water adopted foul sewer beneath Station Road to the west.

2. Existing foul water network

Existing connection from adjacent site to be retained if live.

Exceedance Flows

Beyond the capacity of the site, flows would continue off-site as per existing brownfield conditions, albeit at a reduced rate or volume.

Surface Water Strategy:

1. Maintenance

Geocellular Attenuation to be fitted with access turrets for maintenance of feature. Minimum 1no. cover to be vented in accordance with manufacturer recommendations.

2. Cellular Attenuation

Cellular tank to receive flows from external hard paving and roof catchments. Chambers immediately upstream of tank to include silt traps.

3. Flow Control

Proposed surface water control chamber to restrict flows up to 100yr return period and discharge up to 5.9 l/s (or 'as close to greenfield as practicable'). Controlled flows are proposed to discharge into the surface water network to the south east within the extents of the proposed development. 300Ø oversized pipe to enable surcharged runoff to back up to the attenuation tank within the amenity area, and permeable paving within the parking court.

4. Underdrained Permeable Paving

Proposed underdrained permeable paving to receive surface runoff from access road and parking court. To provide treatment and further storage upstream of proposed cellular attenuation tank with fin drain to convey flows into the surface water network.

5. Surface Water Discharge

Retained private surface water connection to the existing on-site surface water network.

REV	DATE	DESCRIPTION	BY	CHK	APD
B	10.11.2021	UPDATED LAYOUT AND DRAINAGE TO SUIT	RF	LB	CY
A	02.11.2021	INITIAL ISSUE	TG	LB	CY

CLIENT: CHURCHILL RETIREMENT LIVING

DRAWING STATUS: PLANNING APPLICATION

PROJECT: STATION ROAD, GREAT SHELFORD

TITLE: PRELIMINARY DRAINAGE LAYOUT

PROJECT No: 1281 DRAWING No: 01-PDL-1001 REV: B

SCALE @ A1: 0 1:200 10 metres

DESIGN BY: awcockward partnership

Awcock Ward Partnership, Ada House, Pynes Hill, Exeter, EX2 5TU
Tel: 01392 409007 Web: www.awpexeter.com