The Sizewell C Project

SZC Co.'s Response to the Secretary of State's Request for Further Information dated 18 March 2022: Appendix 3 - The Drainage Strategy

Part 4 of 12

Revision: 2.0

April 2022

APPENDIX A:

A.1. Suffolk County Council Briefing Note

08 February 2021

The Sizewell C Project

Suffolk County Council briefing note to SZC Co. regarding its concerns on surface water drainage

Background

1. At the meeting between Suffolk County Council (the Council) and SZC Co. on 1 February 2021, the Council raised that it had concerns regarding the surface water drainage proposals not only for the Land East of Eastland Industrial Estate (LEEIE), but also for most of the other sites. This note set out the concerns and expectations of the Council as the Lead Local Flood Authority (referred to in the document as the LLFA) for these other sites.

Current progress

2. To date, good progress has been made on surface water drainage strategies for both the Two Village Bypass and the Sizewell Link Road. There are regular monthly meetings between the design team and key stakeholders to assess progress on multiple topics, determine potential solutions to any problems that have arisen and a good mechanism of feedback between meetings.
3. Whilst surface water drainage strategies are coming together for both schemes these are at quite a high level with no calculations or technical drawings provided to date.
4. The Land East of Eastlands Industrial Estate (LEEIE) is the only other site to have seen any progress with regards to surface water drainage strategies. The LLFA has significant concerns with the proposed approach to surface water management at this site.
5. No progress on surface water drainage strategies has been communicated to the Council for the other sites (Main Development Site, North & South Park & Rides, Freight Management Facility, Yoxford Roundabout). It is noted that the changes consultation includes a temporary outfall pipe from Main Development Site onto the beach. The need for this is a clear indication that more detailed work has been undertaken to assess surface water drainage strategies for this site that is yet to be communicated to the Council.

Specific concerns

6. Concerns regarding the approach taken for LEEIE have been made previously. This position remains unchanged. Two points worth highlighting from this site are;
    a. SZC Co.'s notion that there is 'insufficient space for SuDS'. This argument is not accepted by the LLFA on any of the proposed development sites. NPS EN-1 is clear that priority should be given to the use of SuDS.
    b. SZC Co.'s notion that the site is only temporary, and on that basis, local and national design guidance as usually applied, and thus a SuDS approach, is not necessarily applicable. This argument is not accepted by the LLFA on any of the proposed development sites. NPS EN-1 does not differentiate between temporary and permanent. The potential surface

water flood risk presented by a development is no less important just because it is temporary. If anything, temporary developments for construction purposes have the potential to further increase surface water flood risk compared to permanent development due to suspended sediments etc associated with construction works.

7. Due to the site's location within a sensitive environment, the Main Development Site (MDS) is of particular concern to the LLFA. This is all the more relevant given the proposed site uses. Very high-level details have been provided as part of the submission documentation, but no further information has been received to date.
8. Proposals for any site need to have a deliverable surface water drainage strategy. This infrastructure is essential to prevent an increase in flood risk and in this instance (MDS) to prevent any potential harm to the surrounding sensitive environment. The space required for SuDS should be assessed, determined, and allocated ***prior*** to space for other site uses being allocated; to ensure there is sufficient space for this essential infrastructure. The LLFA is concerned that this approach may not have been followed by SZC Co.
9. As previously stated, no information has been received for any site, with the exception of LEEIE, Two Village Bypass and Sizewell Link Road.
10. All the points raised in the Issues Tracker, shared in 2019 with the applicant, remain outstanding with little if any progress made on the vast majority of issues raised to date.

What we expect

11. The LLFA expect to see (ideally before, or at worst during, the Examination process) a deliverable surface water drainage strategy for each site of proposed development. Whilst the LLFA understand that at that stage detailed design is not essential, we do expect to see evidence that the strategy can be delivered, i.e. the measures can sufficiently manage surface water runoff generated by the proposed development, within the Order Limits and whilst complying with current local and national design guidance.
12. The LFFA understands that infiltration testing has been undertaken at all sites. However, the LLFA is yet to see the results of this testing in most instances. If a design is solely based on infiltration, SZC Co. needs to evidence that a suitable infiltration rate has been achieved. If a suitable infiltration rate has not been evidenced, an alternative method of surface water disposal (as per surface water disposal hierarchy) should be identified and an appropriate design developed on this basis.
13. If infiltration testing has not been undertaken for a particular site, then it must be demonstrated that;
    a. An infiltration strategy can be achieved with a minimum infiltration rate of 10mm/hr and an appropriate Factor of Safety; and
    b. An alternative method of surface water disposal is achievable (as per surface water drainage hierarchy).

    It must be demonstrated that both options are deliverable within the Order Limits if no infiltration testing is available.

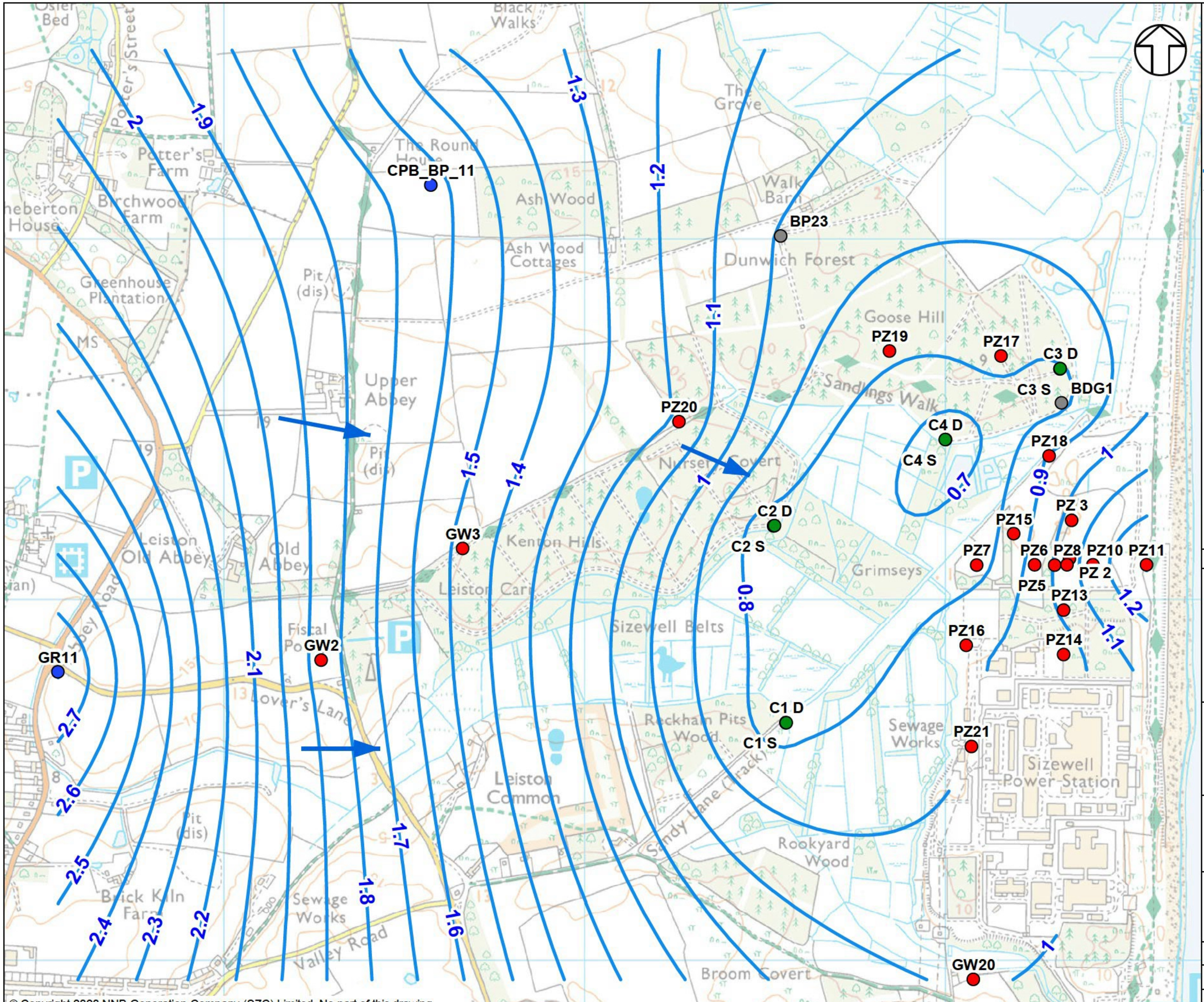
14. Any drainage strategy must consider flood risk as identified in the relevant Flood Risk Assessments.
15. It is important that all design development considers maximum water levels, total basin depths, side slope gradients and the potential for each SuDS component to deliver surface water treatment. All these aspects will affect the sizing of SuDS components which ultimately impact land take and in turn, deliverability (especially on constrained sites).

Recommended next Steps

16. The LLFA strongly recommends that the joint working approach with the LLFA taken for the design development of Two Village Bypass and Sizewell Link Road is replicated for each of the other development sites. This transparent and integrated approach has given the LLFA assurance that design is progressing in the right direction, even if more detailed information is not yet available for these two schemes.
17. As design development progresses further, additional details should be provided to the LLFA. This should include but is not limited to, results of ground investigation testing, dimensioned plans of proposed SUDS, design assumptions, results of rainfall simulations for proposed design and details of future maintenance arrangements.

APPENDIX B:

B.1. Groundwater Contour Drawing



NOTES

PEAT CONTOURS ARE CONSTRAINED TO THE EXTENT OF THE PEAT DEPOSITS

KEY

- ● 2010 BOREHOLES
- ● 2013 BOREHOLES
- ● 2014 BOREHOLES
- ● 2015 BOREHOLES
- — GROUNDWATER CRAG CONTOURS (MAOD) - WINTER 2018
- → GROUNDWATER FLOW DIRECTION

NOT PROTECTIVELY MARKED

Project Title:
SIZEWELL C PROJECT
ENVIRONMENTAL STATEMENT
VOLUME 2
CHAPTER 19B1
GROUNDWATER AND SURFACE WATER APPENDIX B1

DRAWING TITLE:
GROUNDWATER CONTOURS IN CRAG AQUIFER DURING WINTER 20 8 (9 2 2018)

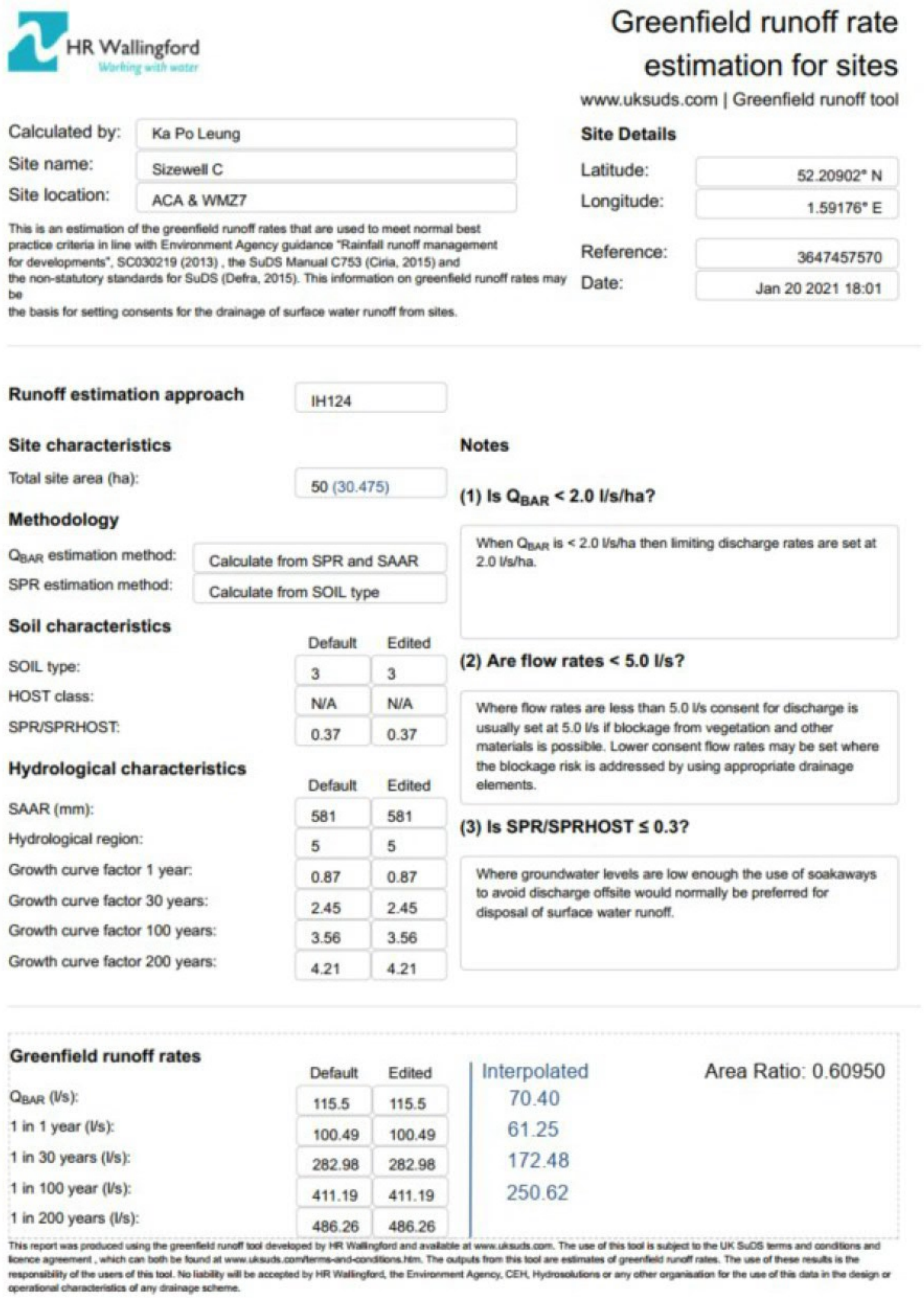
DRAWING NO:
FIGURE 19B 1.16

DATE:	DRAWN:	SCALE :
JAN 2020	S.R.H	1:10,103 @A3

SCALE BAR
0 0.1 0.2 0.3 0.4 0.5 KM

APPENDIX C:

C.1. UK SuDS Greenfield Runoff Rate



HR Wallingford
Working with water

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Ka Po Leung
Site name:	Sizewell C
Site location:	ACA & WMZ7

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:	52.20902° N
Longitude:	1.59176° E
Reference:	3647457570
Date:	Jan 20 2021 18:01

Runoff estimation approach IH124

Site characteristics

Total site area (ha): 50 (30.475)

Methodology

Q_{BAR} estimation method:	Calculate from SPR and SAAR
SPR estimation method:	Calculate from SOIL type

Soil characteristics

	Default	Edited
SOIL type:	3	3
HOST class:	N/A	N/A
SPR/SPRHOST:	0.37	0.37

Hydrological characteristics

	Default	Edited
SAAR (mm):	581	581
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

Area Ratio: 0.60950

	Default	Edited	Interpolated
Q_{BAR} (l/s):	115.5	115.5	70.40
1 in 1 year (l/s):	100.49	100.49	61.25
1 in 30 years (l/s):	282.98	282.98	172.48
1 in 100 year (l/s):	411.19	411.19	250.62
1 in 200 years (l/s):	486.26	486.26	

This report was produced using the greenfield runoff tool developed by HR Wallingford and available at www.uksuds.com. The use of this tool is subject to the UK SuDS terms and conditions and licence agreement , which can both be found at www.uksuds.com/terms-and-conditions.htm. The outputs from this tool are estimates of greenfield runoff rates. The use of these results is the responsibility of the users of this tool. No liability will be accepted by HR Wallingford, the Environment Agency, CEH, Hydrosolutions or any other organisation for the use of this data in the design or operational characteristics of any drainage scheme.

APPENDIX D:

D.1. EDF – LEEIE EWBD Drainage Design Feedback

PROTECTIVE MARKING

Minutes

SZC Project - DCO

Meeting: EDF – LEEIE EWBD Basic Drainage Design Feedback

Attendees:
[redacted]EDF) JH
[redacted](Atkins) JJ
[redacted]Atkins) NK
[redacted]Atkins) ES
[redacted]Wood) SDL
[redacted]BM
[redacted]EST
[redacted]MW
[redacted]JM
[redacted]YS
[redacted]GB
[redacted]LC

Meeting Type: MS Teams

Apologies: -

Meeting held on: 17 December 2020 1400-1530h

Meeting Agenda

1. Introductions
2. Review Objective(s) for the Meeting
3. Feedback on the enabling works basic design for drainage at LEEIE / ACA:
    a. WMA
    b. EA
    c. SCC
4. Was meeting objective(s) met?
5. AOB and next meeting date
6. Close

Meeting Minutes – 18/11/2020

Minute Ref	Actions/Comments	Who	By when
1.	**Introductions** Introductions and scene setting by [redacted] Objective 1 - Discussion around the feedback received from the EA, SCC and WMA on the technical information previously provided for the LEEIE (EW0302 ACA Drainage Strategy Technical Note), to understand concerns, points of clarity and area of common ground. Objective 2 – Time allowing, enable discussion on any other aspect of the change consultation pertaining to drainage e.g. the proposed temporary marine outfall.		
2.	YS: Questions on specific discharge rates and also queries as to why the discharge is currently proposed directly into Leiston Drain and not into the attenuation basin that would be preferable. BM: Agree – would be beneficial to go through a basin and provide improvements to the water quality. EDF should also seek to consult with Natural England for specific water quality requirements. ES: Happy to change the local strategy if all stakeholders agree it is the right approach. EST: Current EA position is that this site will not require permitting. ACTION: Potential design change - basin to be used prior to outfall to Leiston Drain. Subject to discussions with NE	JH	

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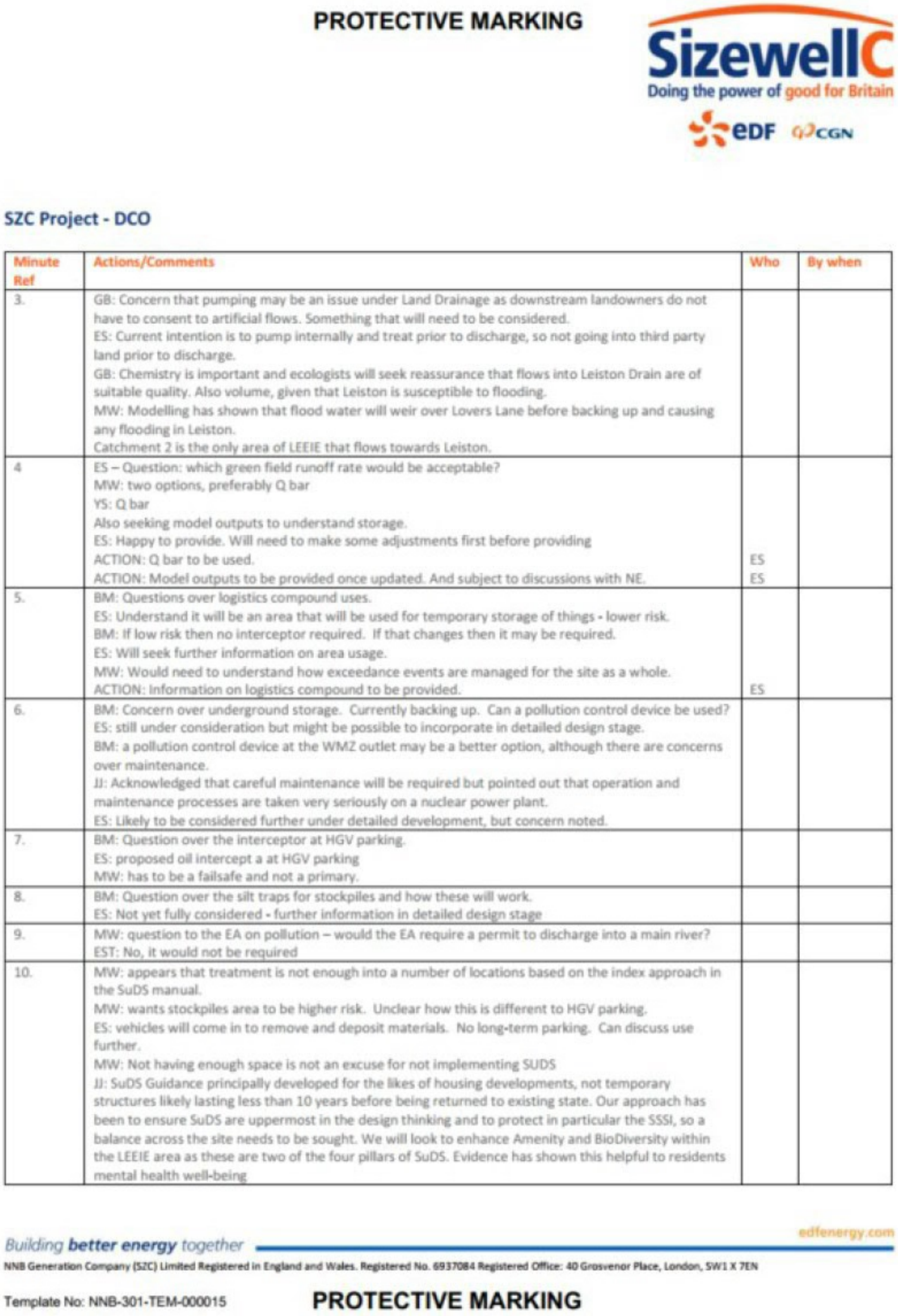
25/01/2021 Minutes Page 1 of 1



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SZC Project - DCO

Minute Ref	Actions/Comments	Who	By when
3.	GB: Concern that pumping may be an issue under Land Drainage as downstream landowners do not have to consent to artificial flows. Something that will need to be considered. ES: Current intention is to pump internally and treat prior to discharge, so not going into third party land prior to discharge. GB: Chemistry is important and ecologists will seek reassurance that flows into Leiston Drain are of suitable quality. Also volume, given that Leiston is susceptible to flooding. MW: Modelling has shown that flood water will weir over Lovers Lane before backing up and causing any flooding in Leiston. Catchment 2 is the only area of LEEIE that flows towards Leiston.		
4	ES – Question: which green field runoff rate would be acceptable? MW: two options, preferably Q bar YS: Q bar Also seeking model outputs to understand storage. ES: Happy to provide. Will need to make some adjustments first before providing ACTION: Q bar to be used. ACTION: Model outputs to be provided once updated. And subject to discussions with NE.	ES ES	
5.	BM: Questions over logistics compound uses. ES: Understand it will be an area that will be used for temporary storage of things - lower risk. BM: If low risk then no interceptor required. If that changes then it may be required. ES: Will seek further information on area usage. MW: Would need to understand how exceedance events are managed for the site as a whole. ACTION: Information on logistics compound to be provided.	ES	
6.	BM: Concern over underground storage. Currently backing up. Can a pollution control device be used? ES: still under consideration but might be possible to incorporate in detailed design stage. BM: a pollution control device at the WMZ outlet may be a better option, although there are concerns over maintenance. JJ: Acknowledged that careful maintenance will be required but pointed out that operation and maintenance processes are taken very seriously on a nuclear power plant. ES: Likely to be considered further under detailed development, but concern noted.		
7.	BM: Question over the interceptor at HGV parking. ES: proposed oil intercept a at HGV parking MW: has to be a failsafe and not a primary.		
8.	BM: Question over the silt traps for stockpiles and how these will work. ES: Not yet fully considered - further information in detailed design stage		
9.	MW: question to the EA on pollution – would the EA require a permit to discharge into a main river? EST: No, it would not be required		
10.	MW: appears that treatment is not enough into a number of locations based on the index approach in the SuDS manual. MW: wants stockpiles area to be higher risk. Unclear how this is different to HGV parking. ES: vehicles will come in to remove and deposit materials. No long-term parking. Can discuss use further. MW: Not having enough space is not an excuse for not implementing SUDS JJ: SuDS Guidance principally developed for the likes of housing developments, not temporary structures likely lasting less than 10 years before being returned to existing state. Our approach has been to ensure SuDS are uppermost in the design thinking and to protect in particular the SSSI, so a balance across the site needs to be sought. We will look to enhance Amenity and BioDiversity within the LEEIE area as these are two of the four pillars of SuDS. Evidence has shown this helpful to residents mental health well-being		

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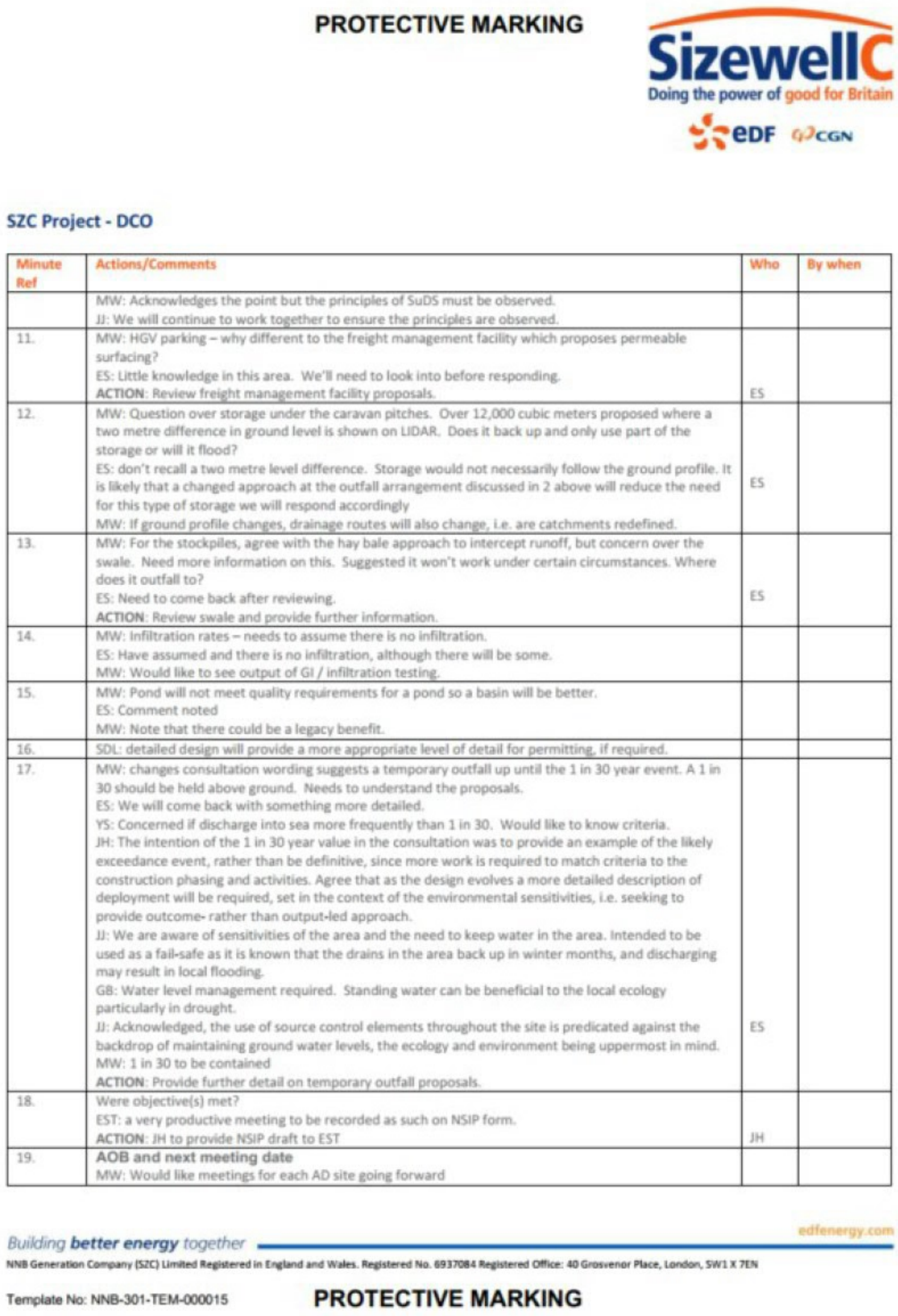

PROTECTIVE MARKING

SZC Project - DCO

Minute Ref	Actions/Comments	Who	By when
	MW: Acknowledges the point but the principles of SuDS must be observed. JJ: We will continue to work together to ensure the principles are observed.		
11.	MW: HGV parking – why different to the freight management facility which proposes permeable surfacing? ES: Little knowledge in this area. We'll need to look into before responding. **ACTION**: Review freight management facility proposals.	ES	
12.	MW: Question over storage under the caravan pitches. Over 12,000 cubic meters proposed where a two metre difference in ground level is shown on LIDAR. Does it back up and only use part of the storage or will it flood? ES: don't recall a two metre level difference. Storage would not necessarily follow the ground profile. It is likely that a changed approach at the outfall arrangement discussed in 2 above will reduce the need for this type of storage we will respond accordingly MW: If ground profile changes, drainage routes will also change, i.e. are catchments redefined.	ES	
13.	MW: For the stockpiles, agree with the hay bale approach to intercept runoff, but concern over the swale. Need more information on this. Suggested it won't work under certain circumstances. Where does it outfall to? ES: Need to come back after reviewing. **ACTION**: Review swale and provide further information.	ES	
14.	MW: Infiltration rates – needs to assume there is no infiltration. ES: Have assumed and there is no infiltration, although there will be some. MW: Would like to see output of GI / infiltration testing.		
15.	MW: Pond will not meet quality requirements for a pond so a basin will be better. ES: Comment noted MW: Note that there could be a legacy benefit.		
16.	SDL: detailed design will provide a more appropriate level of detail for permitting, if required.		
17.	MW: changes consultation wording suggests a temporary outfall up until the 1 in 30 year event. A 1 in 30 should be held above ground. Needs to understand the proposals. ES: We will come back with something more detailed. YS: Concerned if discharge into sea more frequently than 1 in 30. Would like to know criteria. JH: The intention of the 1 in 30 year value in the consultation was to provide an example of the likely exceedance event, rather than be definitive, since more work is required to match criteria to the construction phasing and activities. Agree that as the design evolves a more detailed description of deployment will be required, set in the context of the environmental sensitivities, i.e. seeking to provide outcome- rather than output-led approach. JJ: We are aware of sensitivities of the area and the need to keep water in the area. Intended to be used as a fail-safe as it is known that the drains in the area back up in winter months, and discharging may result in local flooding. GB: Water level management required. Standing water can be beneficial to the local ecology particularly in drought. JJ: Acknowledged, the use of source control elements throughout the site is predicated against the backdrop of maintaining ground water levels, the ecology and environment being uppermost in mind. MW: 1 in 30 to be contained **ACTION**: Provide further detail on temporary outfall proposals.	ES	
18.	Were objective(s) met? EST: a very productive meeting to be recorded as such on NSIP form. **ACTION**: JH to provide NSIP draft to EST	JH	
19.	**AOB and next meeting date** MW: Would like meetings for each AD site going forward		

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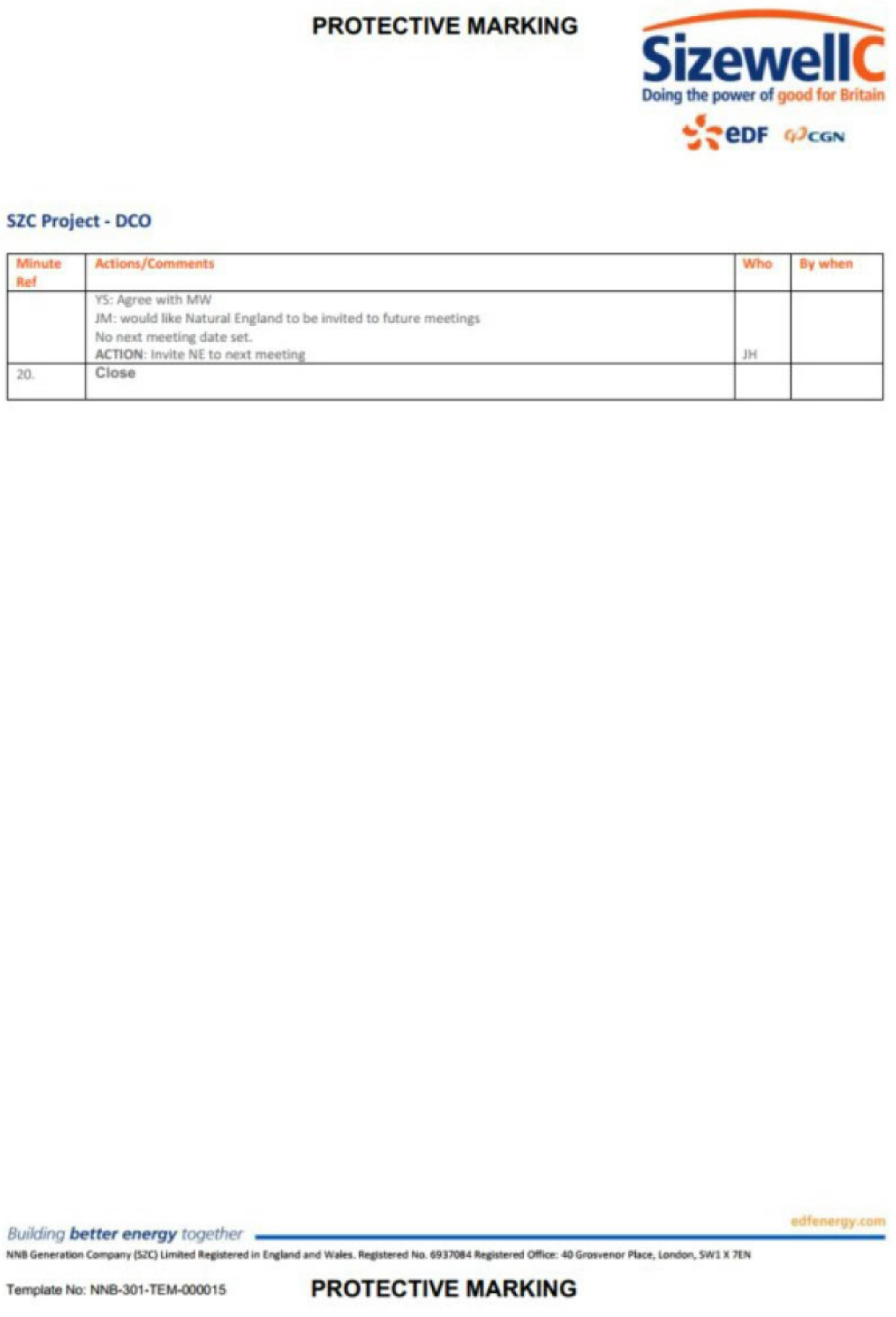

PROTECTIVE MARKING

SZC Project - DCO

Minute Ref	Actions/Comments	Who	By when
	YS: Agree with MW JM: would like Natural England to be invited to future meetings No next meeting date set. **ACTION**: Invite NE to next meeting	JH	
20.	Close		

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Template No: NNB-301-TEM-000015

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APPENDIX E:

E.1. Infiltration Rate Testing Results

TP-WMZ-24 & TP-WMZ-25 from Structural Soils Limited, July 2017 - 2016 Onshore Ground Investigation Campaign. Factual Report on Ground Investigation (SZC-SZC030-XX-000-REP-100000)

ACA_2020-1 & ACA_2020-2 from Fugro - Sizewell C Infiltration Testing Report on Ground Investigation without Geotechnical Evaluation (G200003U_GIR Rev 02)

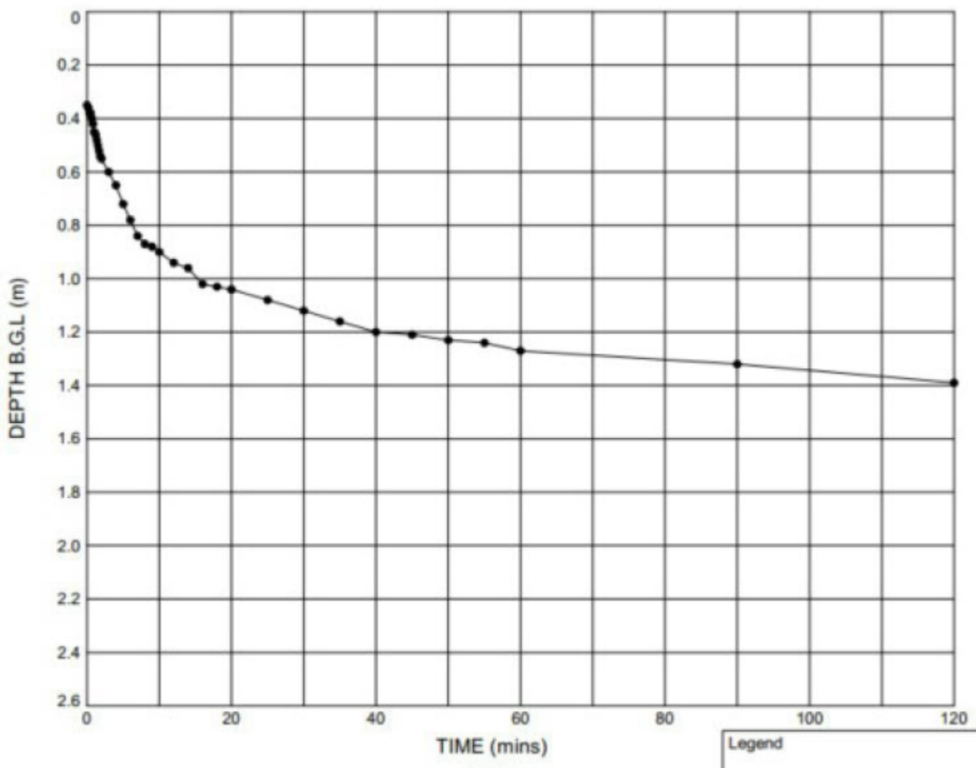
FULL SCALE SOAKAWAY TEST

Non-standard test

Soakaway Test - Position ID : **TP-WMZ-24**

Ground Level (m): **16.92** National Grid Co-ordinates: **E:645407.4 N:262892.8**

PLOT OF DEPTH OF WATER BELOW GROUND LEVEL AGAINST TIME



Test 1

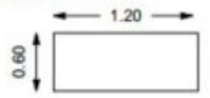
Pit start depth:	=	3.00	m
Pit final depth:	=	3.00	m
Effective depth, De	=	2.65	m
Effective storage volume, $V_{p75\text{-}25}$	=	0.9540	m³
Surface area, a_{p50}	=	5.4900	m²
Time, $t_{p75\text{-}25}$	=	30619	secs
Infiltration rate, f	=	5.68×10^{-6}	m/s

Please note test data was extrapolated to obtain tp75-tp25.

Legend

● Test 1 (14.3.17)

Plan (Not to scale)



No Bearing Taken

STRUCTURAL SOILS
The Potteries
Pottery Street
Castleford
W. Yorkshire WF10 1NJ

Compiled By	Date	Checked By	Date
[redacted]	14/6/17	[redacted]	14/6/17
Contract Sizewell Section 53 Land		Contract Ref: 764156	

GINT_LIBRARY_V8_06.GLB LibVersion: v8_06_017 PrjVersion: v8_06 - Core+Logs - 002 | Graph I - TP SOAKAWAY - 2 - FINAL REPORT - A4P | 764156 - SIZEWELL SECTION 53 LAND.GPJ - v8_06. | 14/6/17 - 10:04 | AC7 |

FULL SCALE SOAKAWAY TEST

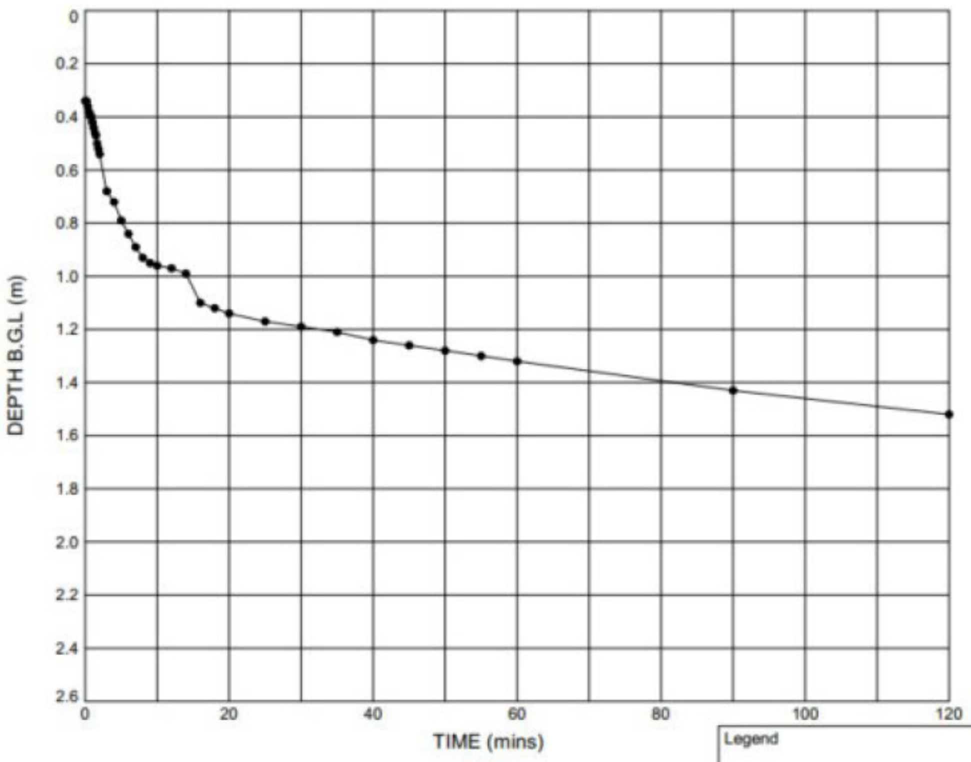
Non-standard test

Soakaway Test - Position ID : **TP-WMZ-25**

Ground Level (m): **16.44**

National Grid Co-ordinates: **E:645595.9 N:262725.8**

PLOT OF DEPTH OF WATER BELOW GROUND LEVEL AGAINST TIME



Test 1

Legend

● Test 1 (14.3.17)

Pit start depth: = **3.00** m

Pit final depth: = **3.00** m

Effective depth, De = **2.66** m

Effective storage volume,V_{p75-25} = **1.6758** m³

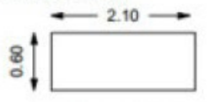
Surface area,a_{p50} = **8.4420** m²

Time, t_{p75-25} = **22861** secs

Infiltration rate, f = **8.68×10^{-6}** m/s

Please note test data was extrapolated to obtain tp75-tp25.

Plan (Not to scale)



No Bearing Taken

STRUCTURAL SOILS
The Potteries
Pottery Street
Castleford
W. Yorkshire WF10 1NJ

Compiled By	Date	Checked By	Date
[redacted]	14/6/17	[redacted]	14/6/17

Contract: **Sizewell Section 53 Land**

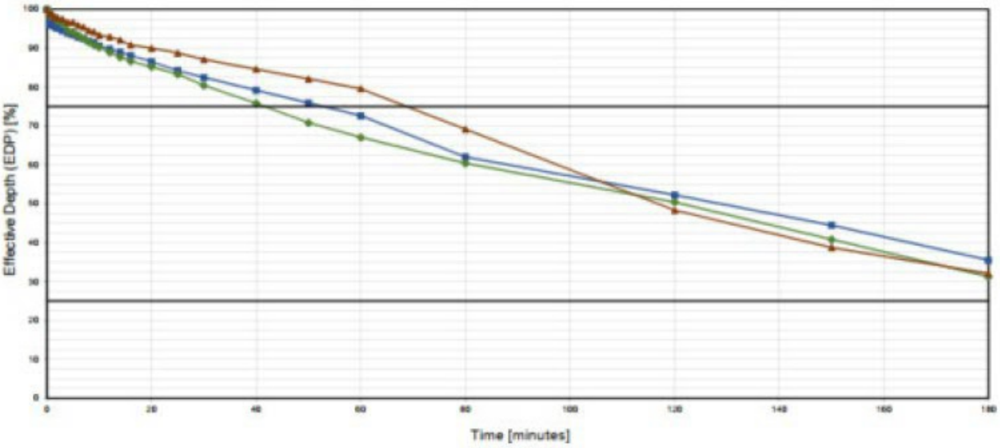
Contract Ref: **764156**

GINT_LIBRARY_V8_06.GLB LibVersion: v8_06_017 PrjVersion: v8_06 - Core+Logs - 002 | Graph I - TP SOAKAWAY - 2 - FINAL REPORT - AIP | 764156 - SIZEWELL SECTION 53 LAND.GPJ - v8_06. | 14/6/17 - 10:05 | AC7 |

NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD

Date	06/03/2020	Operator	JJL	Hole Depth	3.50 m	Hole No.	ACA_2020-1



Test 1 ■				Test 2 ◆				Test 3 ▲			
Start Time =	11:25			Start Time =	14:32			Start Time =	08:35		
Test Top =	0.45	m BGL		Test Top =	0.50	m BGL		Test Top =	0.50	m BGL	
Test Base =	2.90	m BGL		Test Base =	2.90	m BGL		Test Base =	2.90	m BGL	
75% EDP =	1.06	m BGL		75% EDP =	1.10	m BGL		75% EDP =	1.10	m BGL	
50% EDP =	1.68	m BGL		50% EDP =	1.70	m BGL		50% EDP =	1.70	m BGL	
Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
00:00:00		0.45	100	00:00:00		0.50	100	00:00:00		0.50	100
00:00:30		0.54	96	00:00:30		0.54	98	00:00:30		0.53	99
00:01:00		0.55	96	00:01:00		0.54	98	00:01:00		0.53	99
00:01:30		0.56	96	00:01:30		0.56	98	00:01:30		0.55	98
00:02:00		0.57	95	00:02:00		0.57	97	00:02:00		0.55	98
00:03:00		0.59	94	00:03:00		0.59	96	00:03:00		0.56	98
00:04:00		0.60	94	00:04:00		0.63	95	00:04:00		0.58	97
00:05:00		0.61	93	00:05:00		0.64	94	00:05:00		0.58	97
00:06:00		0.63	93	00:06:00		0.66	93	00:06:00		0.60	96
00:07:00		0.64	92	00:07:00		0.68	93	00:07:00		0.61	95
00:08:00		0.65	92	00:08:00		0.70	92	00:08:00		0.63	95
00:09:00		0.66	91	00:09:00		0.72	91	00:09:00		0.64	94
00:10:00		0.68	91	00:10:00		0.74	90	00:10:00		0.66	93
00:12:00		0.70	90	00:12:00		0.77	89	00:12:00		0.67	93
00:14:00		0.72	89	00:14:00		0.80	88	00:14:00		0.69	92
00:16:00		0.74	88	00:16:00		0.82	87	00:16:00		0.72	91
00:20:00		0.78	87	00:20:00		0.86	85	00:20:00		0.74	90
00:25:00		0.84	84	00:25:00		0.90	83	00:25:00		0.77	89
00:30:00		0.88	82	00:30:00		0.97	80	00:30:00		0.81	87
00:40:00		0.96	79	00:40:00		1.08	76	00:40:00		0.87	85
00:50:00		1.04	76	00:50:00		1.20	71	00:50:00		0.93	82
01:00:00		1.12	73	01:00:00		1.29	67	01:00:00		0.99	80
01:20:00		1.38	62	01:20:00		1.45	60	01:20:00		1.24	69
02:00:00		1.62	52	02:00:00		1.69	50	02:00:00		1.74	48
02:30:00		1.81	44	02:30:00		1.92	41	02:30:00		1.97	39
03:00:00		2.03	36	03:00:00		2.15	31	03:00:00		2.13	32

Datum (-ve denotes AGL) = 0.00 m BGL
Ground Level taken as Datum.

Input by KB 09/03/2020 Checked by JJL 12/03/2020 Approved by NHA 05/06/2020

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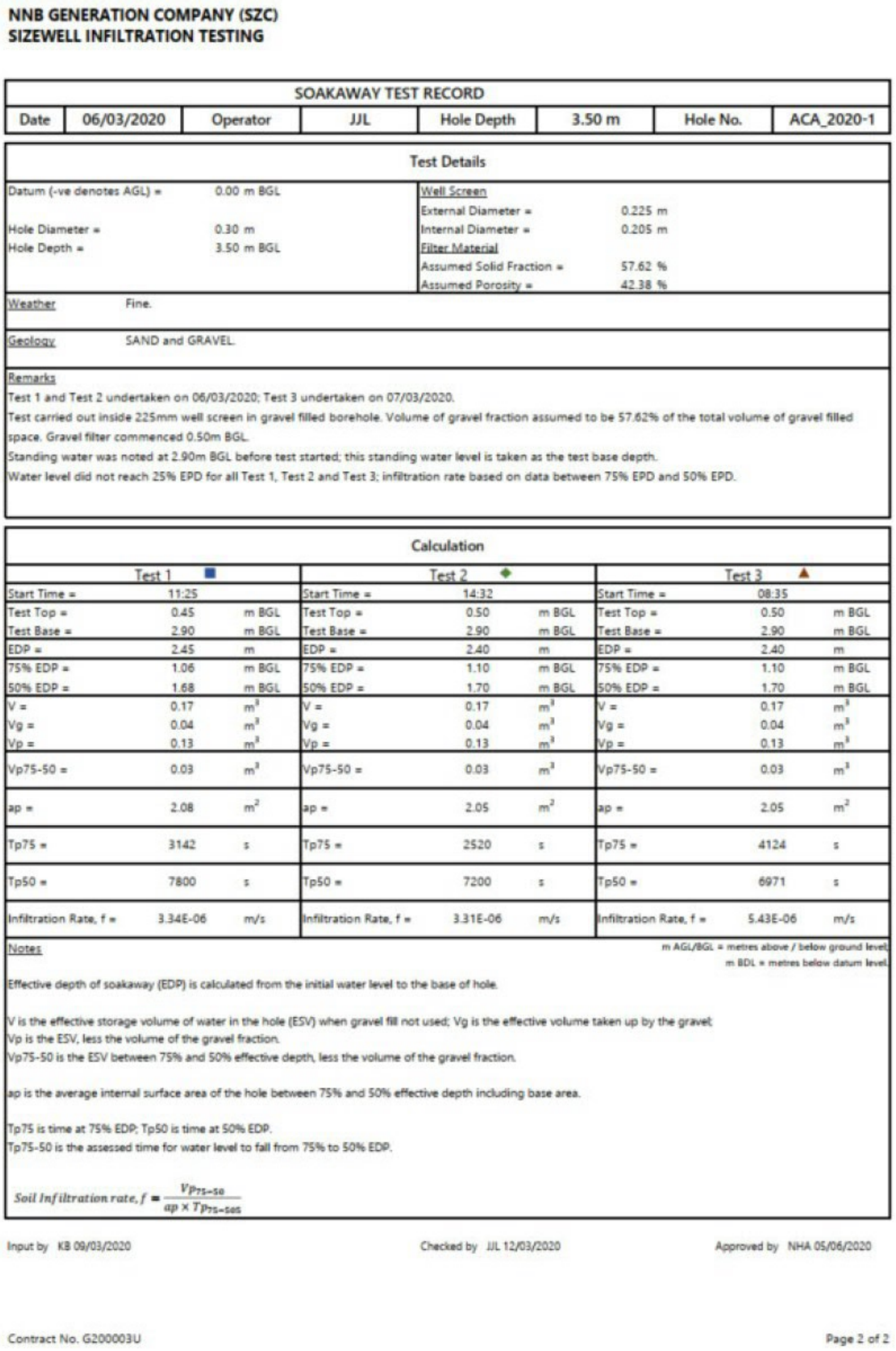

NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD

Date	06/03/2020	Operator	JJL	Hole Depth	3.50 m	Hole No.	ACA_2020-1

Test Details

Datum (-ve denotes AGL) =	0.00 m BGL	Well Screen	
		External Diameter =	0.225 m
Hole Diameter =	0.30 m	Internal Diameter =	0.205 m
Hole Depth =	3.50 m BGL	Filter Material	
		Assumed Solid Fraction =	57.62 %
		Assumed Porosity =	42.38 %

Weather Fine.

Geology SAND and GRAVEL.

Remarks
Test 1 and Test 2 undertaken on 06/03/2020; Test 3 undertaken on 07/03/2020.
Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commenced 0.50m BGL.
Standing water was noted at 2.90m BGL before test started; this standing water level is taken as the test base depth.
Water level did not reach 25% EPD for all Test 1, Test 2 and Test 3; infiltration rate based on data between 75% EPD and 50% EPD.

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	11:25		Start Time =	14:32		Start Time =	08:35	
Test Top =	0.45	m BGL	Test Top =	0.50	m BGL	Test Top =	0.50	m BGL
Test Base =	2.90	m BGL	Test Base =	2.90	m BGL	Test Base =	2.90	m BGL
EDP =	2.45	m	EDP =	2.40	m	EDP =	2.40	m
75% EDP =	1.06	m BGL	75% EDP =	1.10	m BGL	75% EDP =	1.10	m BGL
50% EDP =	1.68	m BGL	50% EDP =	1.70	m BGL	50% EDP =	1.70	m BGL
V =	0.17	m^3	V =	0.17	m^3	V =	0.17	m^3
Vg =	0.04	m^3	Vg =	0.04	m^3	Vg =	0.04	m^3
Vp =	0.13	m^3	Vp =	0.13	m^3	Vp =	0.13	m^3
Vp75-50 =	0.03	m^3	Vp75-50 =	0.03	m^3	Vp75-50 =	0.03	m^3
ap =	2.08	m^2	ap =	2.05	m^2	ap =	2.05	m^2
Tp75 =	3142	s	Tp75 =	2520	s	Tp75 =	4124	s
Tp50 =	7800	s	Tp50 =	7200	s	Tp50 =	6971	s
Infiltration Rate, f =	3.34E-06	m/s	Infiltration Rate, f =	3.31E-06	m/s	Infiltration Rate, f =	5.43E-06	m/s

Notes

m AGL/BGL = metres above / below ground level
m BDL = metres below datum level

Effective depth of soakaway (EDP) is calculated from the initial water level to the base of hole.

V is the effective storage volume of water in the hole (ESV) when gravel fill not used; Vg is the effective volume taken up by the gravel
Vp is the ESV, less the volume of the gravel fraction.
Vp75-50 is the ESV between 75% and 50% effective depth, less the volume of the gravel fraction.

ap is the average internal surface area of the hole between 75% and 50% effective depth including base area.

Tp75 is time at 75% EDP; Tp50 is time at 50% EDP.
Tp75-50 is the assessed time for water level to fall from 75% to 50% EDP.

$$Soil\ Infiltration\ rate, f = \frac{V_{P_{75-50}}}{ap \times T_{P_{75-50s}}}$$

Input by KB 09/03/2020 Checked by JJL 12/03/2020 Approved by NHA 05/06/2020

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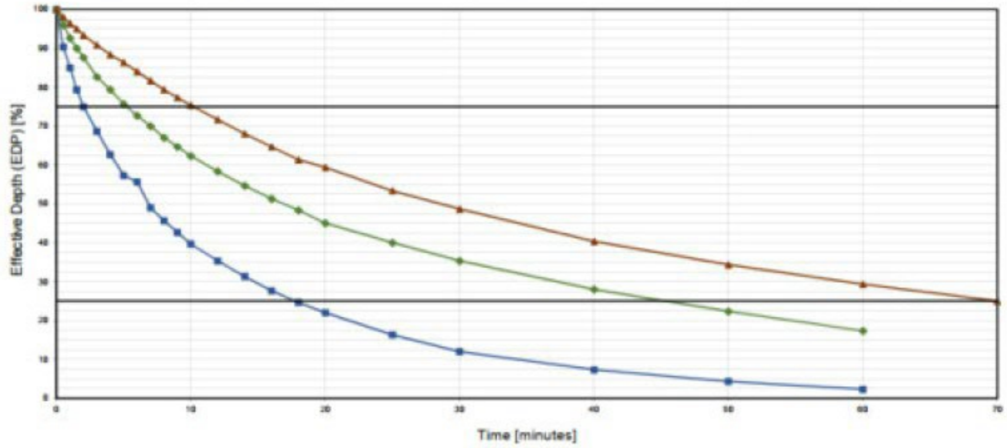
NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD

Date	07/03/2020	Operator	JJL	Hole Depth	3.50 m	Hole No.	ACA_2020-2



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	15:10		Start Time =	09:02		Start Time =	10:05	
Test Top =	0.50	m BGL	Test Top =	0.50	m BGL	Test Top =	0.50	m BGL
Test Base =	3.50	m BGL	Test Base =	3.50	m BGL	Test Base =	3.50	m BGL
75% EDP =	1.25	m BGL	75% EDP =	1.25	m BGL	75% EDP =	1.25	m BGL
25% EDP =	2.75	m BGL	25% EDP =	2.75	m BGL	25% EDP =	2.75	m BGL

Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
00:00:00		0.50	100	00:00:00		0.50	100	00:00:00		0.50	100
00:00:30		0.79	90	00:00:30		0.62	96	00:00:30		0.56	98
00:01:00		0.95	85	00:01:00		0.72	93	00:01:00		0.61	96
00:01:30		1.12	79	00:01:30		0.80	90	00:01:30		0.65	95
00:02:00		1.25	75	00:02:00		0.87	88	00:02:00		0.70	93
00:03:00		1.44	69	00:03:00		1.02	83	00:03:00		0.78	91
00:04:00		1.62	63	00:04:00		1.12	79	00:04:00		0.85	88
00:05:00		1.78	57	00:05:00		1.23	76	00:05:00		0.91	86
00:06:00		1.83	56	00:06:00		1.32	73	00:06:00		0.98	84
00:07:00		2.03	49	00:07:00		1.40	70	00:07:00		1.05	82
00:08:00		2.13	46	00:08:00		1.49	67	00:08:00		1.12	79
00:09:00		2.22	43	00:09:00		1.56	65	00:09:00		1.18	77
00:10:00		2.31	40	00:10:00		1.63	62	00:10:00		1.24	75
00:12:00		2.44	35	00:12:00		1.75	58	00:12:00		1.35	72
00:14:00		2.56	31	00:14:00		1.86	55	00:14:00		1.46	68
00:16:00		2.67	28	00:16:00		1.96	51	00:16:00		1.56	65
00:18:00		2.76	25	00:18:00		2.05	48	00:18:00		1.66	61
00:20:00		2.84	22	00:20:00		2.15	45	00:20:00		1.72	59
00:25:00		3.01	16	00:25:00		2.30	40	00:25:00		1.90	53
00:30:00		3.14	12	00:30:00		2.44	35	00:30:00		2.04	49
00:40:00		3.28	7	00:40:00		2.66	28	00:40:00		2.29	40
00:50:00		3.37	4	00:50:00		2.83	22	00:50:00		2.47	34
01:00:00		3.43	2	01:00:00		2.98	17	01:00:00		2.62	29
								01:10:00		2.75	25

Datum (-ve denotes AGL) = 0.00 m BGL
Ground Level taken as Datum.

Input by KB 09/03/2020

Checked by JJL 12/03/2020

Approved by NHA 05/06/2020

Contract No. G200003U

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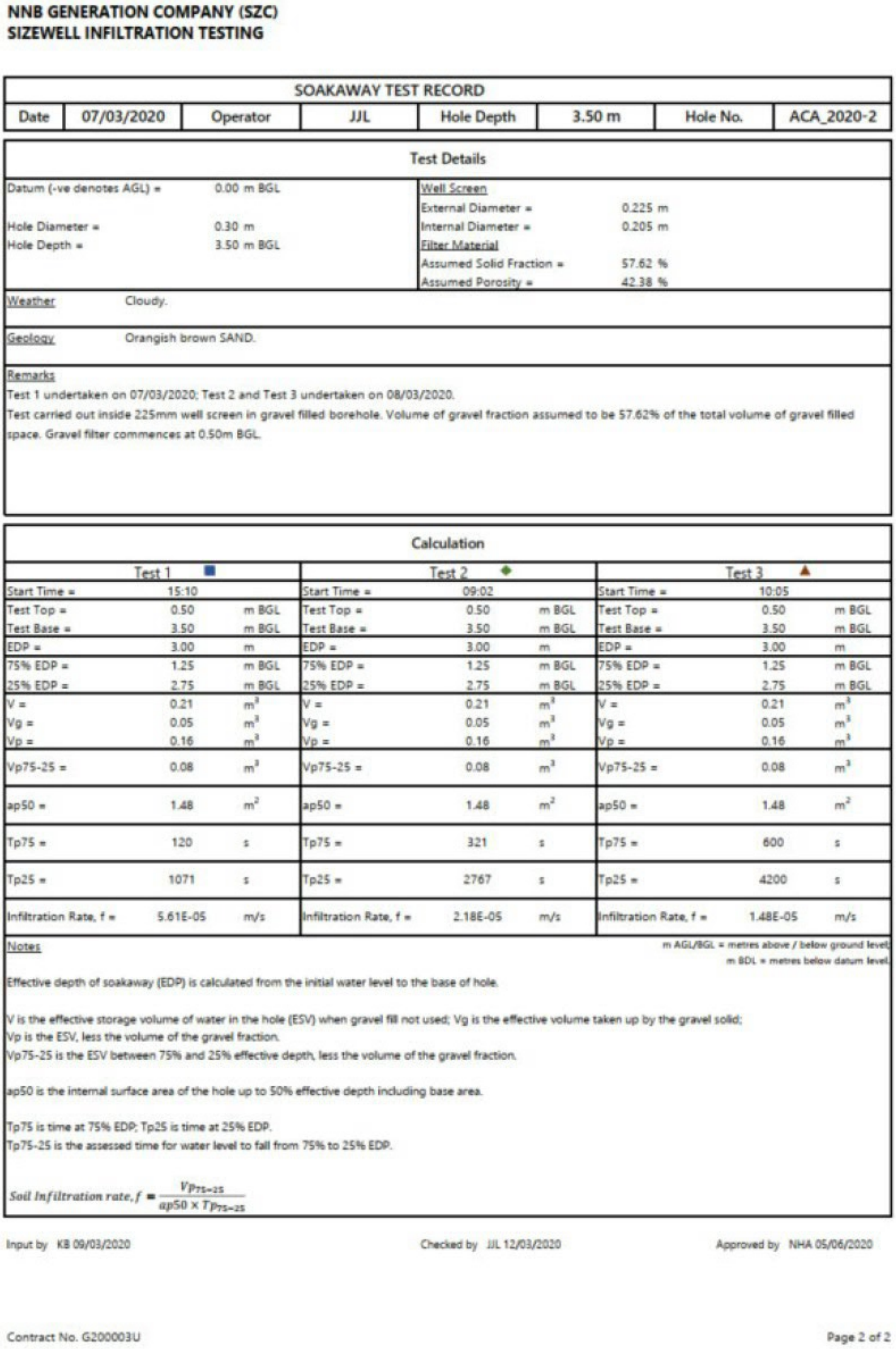

NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD

Date	07/03/2020	Operator	JJL	Hole Depth	3.50 m	Hole No.	ACA_2020-2

Test Details

Datum (-ve denotes AGL) =	0.00 m BGL	Well Screen	
		External Diameter =	0.225 m
Hole Diameter =	0.30 m	Internal Diameter =	0.205 m
Hole Depth =	3.50 m BGL	Filter Material	
		Assumed Solid Fraction =	57.62 %
		Assumed Porosity =	42.38 %

Weather Cloudy.

Geology Orangish brown SAND.

Remarks
Test 1 undertaken on 07/03/2020; Test 2 and Test 3 undertaken on 08/03/2020.
Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m BGL.

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	15:10		Start Time =	09:02		Start Time =	10:05	
Test Top =	0.50	m BGL	Test Top =	0.50	m BGL	Test Top =	0.50	m BGL
Test Base =	3.50	m BGL	Test Base =	3.50	m BGL	Test Base =	3.50	m BGL
EDP =	3.00	m	EDP =	3.00	m	EDP =	3.00	m
75% EDP =	1.25	m BGL	75% EDP =	1.25	m BGL	75% EDP =	1.25	m BGL
25% EDP =	2.75	m BGL	25% EDP =	2.75	m BGL	25% EDP =	2.75	m BGL
V =	0.21	m^3	V =	0.21	m^3	V =	0.21	m^3
Vg =	0.05	m^3	Vg =	0.05	m^3	Vg =	0.05	m^3
Vp =	0.16	m^3	Vp =	0.16	m^3	Vp =	0.16	m^3
Vp75-25 =	0.08	m^3	Vp75-25 =	0.08	m^3	Vp75-25 =	0.08	m^3
ap50 =	1.48	m^2	ap50 =	1.48	m^2	ap50 =	1.48	m^2
Tp75 =	120	s	Tp75 =	321	s	Tp75 =	600	s
Tp25 =	1071	s	Tp25 =	2767	s	Tp25 =	4200	s
Infiltration Rate, f =	5.61E-05	m/s	Infiltration Rate, f =	2.18E-05	m/s	Infiltration Rate, f =	1.48E-05	m/s

Notes

m AGL/BGL = metres above / below ground level
m BDL = metres below datum level

Effective depth of soakaway (EDP) is calculated from the initial water level to the base of hole.

V is the effective storage volume of water in the hole (ESV) when gravel fill not used; Vg is the effective volume taken up by the gravel solid;
Vp is the ESV, less the volume of the gravel fraction.
Vp75-25 is the ESV between 75% and 25% effective depth, less the volume of the gravel fraction.

ap50 is the internal surface area of the hole up to 50% effective depth including base area.

Tp75 is time at 75% EDP; Tp25 is time at 25% EDP.
Tp75-25 is the assessed time for water level to fall from 75% to 25% EDP.

$$Soil\ Infiltration\ rate, f = \frac{V_{P75-25}}{ap50 \times T_{P75-25}}$$

Input by KB 09/03/2020 Checked by JJL 12/03/2020 Approved by NHA 05/06/2020

Contract No. G200003U

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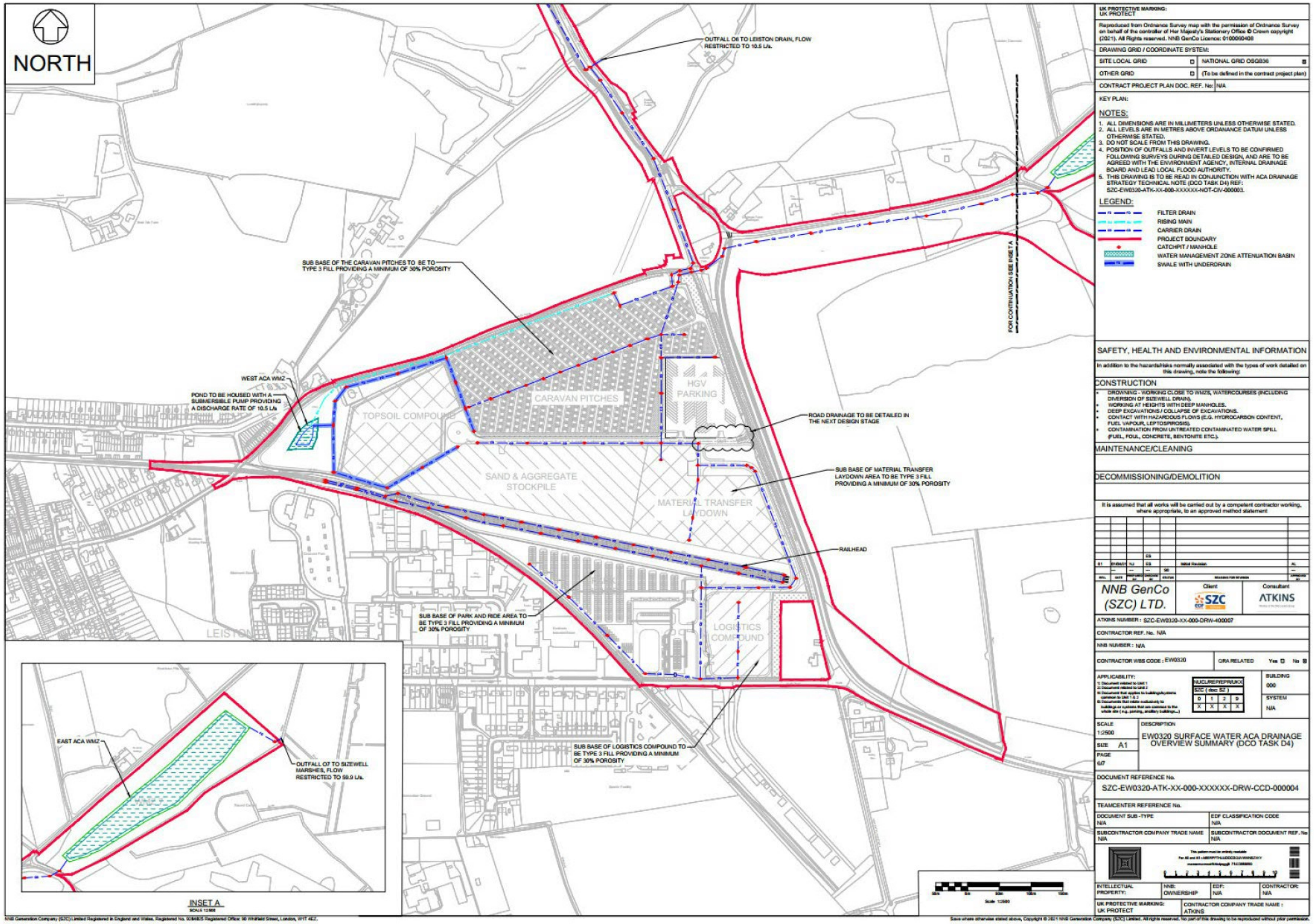
APPENDIX F:

F.1. ACA Surface Water Drainage Layout



UK PROTECTIVE MARKING:
UK PROTECT

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DRAWING GRID / COORDINATE SYSTEM:
SITE LOCAL GRID ☐ NATIONAL GRID OSGB36 ■
OTHER GRID ☐ (To be defined in the contract project plan)
CONTRACT PROJECT PLAN DOC. REF. No: N/A

KEY PLAN:

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED.
2. ALL LEVELS ARE IN METRES ABOVE ORDANANCE DATUM UNLESS OTHERWISE STATED.
3. DO NOT SCALE FROM THIS DRAWING.
4. POSITION OF OUTFALLS AND INVERT LEVELS TO BE CONFIRMED FOLLOWING SURVEYS DURING DETAILED DESIGN, AND ARE TO BE AGREED WITH THE ENVIRONMENT AGENCY, INTERNAL DRAINAGE BOARD AND LEAD LOCAL FLOOD AUTHORITY.
5. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ACA DRAINAGE STRATEGY TECHNICAL NOTE (DCO TASK D4) REF: SZC-EW0320-ATK-XX-000-XXXXXX-NOT-CIV-000003.

LEGEND:

— FD —	FILTER DRAIN
— RM —	RISING MAIN
— CD —	CARRIER DRAIN
━━	PROJECT BOUNDARY
●	CATCHPIT / MANHOLE
▨	WATER MANAGEMENT ZONE ATTENUATION BASIN
━ FD ━	SWALE WITH UNDERDRAIN

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

In addition to the hazards/risks normally associated with the types of work detailed on this drawing, note the following:

CONSTRUCTION

- DROWNING - WORKING CLOSE TO WMZS, WATERCOURSES (INCLUDING DIVERSION OF SIZEWELL DRAIN).
- WORKING AT HEIGHTS WITH DEEP MANHOLES.
- DEEP EXCAVATIONS / COLLAPSE OF EXCAVATIONS.
- CONTACT WITH HAZARDOUS FLOWS (E.G. HYDROCARBON CONTENT, FUEL VAPOUR, LEPTOSPIROSIS).
- CONTAMINATION FROM UNTREATED CONTAMINATED WATER SPILL (FUEL, FOUL, CONCRETE, BENTONITE ETC.).

MAINTENANCE/CLEANING

DECOMMISSIONING/DEMOLITION

It is assumed that all works will be carried out by a competent contractor working, where appropriate, to an approved method statement

			ES			
01	[illegible]	NJ	ES		Initial Revision	AL
	—	—	—	S0		—

NNB GenCo (SZC) LTD.
Client: SZC
Consultant: ATKINS

ATKINS NUMBER : SZC-EW0320-XX-000-DRW-400007
CONTRACTOR REF. No. N/A
NNB NUMBER : N/A
CONTRACTOR WBS CODE : EW0320
QRA RELATED Yes ☐ No ■

APPLICABILITY:

NUCL/REP/EPR/UKX			
SZC (doc: SZ)			
0	1	2	9
X	X	X	X

BUILDING 000
SYSTEM N/A

SCALE 1:2500
SIZE A1
PAGE 6/7
DESCRIPTION EW0320 SURFACE WATER ACA DRAINAGE OVERVIEW SUMMARY (DCO TASK D4)

DOCUMENT REFERENCE No.
SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000004

TEAMCENTER REFERENCE No.
DOCUMENT SUB -TYPE N/A
EDF CLASSIFICATION CODE N/A
SUBCONTRACTOR COMPANY TRADE NAME N/A
SUBCONTRACTOR DOCUMENT REF. No N/A

INTELLECTUAL PROPERTY: NNB: OWNERSHIP EDF: N/A CONTRACTOR: N/A
UK PROTECTIVE MARKING: UK PROTECT
CONTRACTOR COMPANY TRADE NAME : ATKINS

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ANNEX 2A.5: EXPLANATORY TECHNICAL NOTE

Technical Note

Project:	SZC Enabling Works Detail Design		
Subject:	**Surface Water Drainage - SCC Explanatory Note**		
Author:	DH		
Date:		Project No.:	5199744
Atkins No.:	N/A	Icepac No.:	[Not Used]
Distribution:	[Not Used]	Representing:	[Not Used]

Document history

PW Revision	Status	Purpose description	Originated	Checked	Reviewed	Authorised	Date
01	S3	P2 - Published for Costing	DH	MS	AP	KMJ	06/10/21

Client signoff

Client	
Project	SZC Enabling Works Detail Design
Project No.	5199744
Client signature / date	

1. Introduction

This document has been prepared to provide further background related to the surface water management proposals for Sizewell C (SZC) nuclear power station Enabling Works Basic Design (EWBD). This note provides responses to the technical queries raised by Suffolk County Council (SCC) and aims to provide information where available for the following sections. The items numbered below correspond to the Surface Water Drainage Action Plan:

1. Control Document – outside of scope and excluded from this technical note.
2. Infiltration figures – selection
3. Treatment Indices
4. Perimeter Swale – Space allocation
5. Basin Design for Treatment
6. Calculation of Impermeable / Permeable areas
7. Review of Hydrological Catchment
8. Basin Design (sizes)
9. Operational Infrastructure

2. Infiltration parameters – selection

The infiltration results gathered over a number of years give indications across the site of a range of infiltration values. It is recognised that tests were not all carried out according to BRE 365 and therefore may not be fully comparable to each other.

The approach in the design has always been one of caution. The infiltration value chosen for each attenuation basin was on the following basis:

1. The lowest infiltration value within the WMZ being considered.
2. Values that were technically not reliable were discounted.
3. Value was chosen from all the confirmed results.
4. Value was chosen from all the years that testing occurred.

In certain situations, Suffolk County Council (SCC) have informed us that an infiltration rate of 10 mm/hr (2.78x10-6 m/s) is used as a low operational figure. In general, the rates selected in the proposed design are below this low operational figure, with only 3 zones slightly over.

This approach gave the following figures:

WMZ 1: 8.31x10-6 m/s (2015 Structural Soils Limited, Test WMZ20). WMZ 1 – Currently the basin base level is within 1.0 m of the groundwater level and therefore no infiltration has been included within the modelling.

WMZ 2: 7.55x10-6 m/s (2017 Structural Soils Limited, Test TP-WMZ-23)

9.38x10-6 m/s not confirmed 2021 result. WMZ 2 -The value chosen for this zone equates to 27.2 mm/hr. The figure obtained in 2021 is still to be confirmed (6.64x10-6 m/s, 23.9 mm/hr) and is only marginally more conservative. We consider our value a good choice amongst the range and uncertainty.

WMZ 3: 1.34x10-6 m/s (2020 Fugro, Test WMZ3_2020-3-TP-A)

No 2021 results available. WMZ 3 – This value chosen is very low (4.8 mm/hr) and is below the SCC minimum.

WMZ 4: 7.76x10-6 m/s (2017 Structural Soils Limited, Test TP-WMZ-21)

1.90x10-5 to 1.40x10-6 m/s not confirmed 2021 results. WMZ 4 – The value chosen is significantly lower than the other values being considered. The 2021 figures have a range with one value being lower (1.4x10-6 m/s, 5.0 mm/hr). We consider our value a good choice amongst the range and uncertainty.

WMZ 5: 1.24x10-6 m/s (2017 Structural Soils Limited, Test TP-BP-4)

1.14x10-4 to 2.20x10-5 m/s not confirmed 2021 results. WMZ 5 – The value chosen is much less than other values in this area and less than the 10 mm/hr figure (1.24x10-6 m/s, 4.5 mm/hr). The 2021 figure are considerably more than previous results.

WMZ 6: 5.58x10-6 m/s (2020 Fugro, Test WMZ6_2020-2-PIT)

2.09x10-5 to 7.05x10-6 m/s not confirmed 2021 results. WMZ 6 – The value chosen is much less than other values. 5.58x10-6 m/s is 20.1 mm/hr and therefore is slightly more than the SCC low figure. All 2021 figures are higher.

ACA: No Infiltration used in design. 8.68x10-6 m/s lowest, 3.02x10-5 to 3.56x10-6 m/s not confirmed 2021 results.

Green Railway: 1.06x10-4 m/s (2014 Structural Soils Limited, Test GR11A). Abbey Road – The value chosen is the lowest value amongst the satisfactory tests carried out. Although higher than the SCC low value it is a reasonable value to use in the zone.

Campus: 3.70x10-6 m/s (2014 Structural Soils Limited, Test SA3)

2021 Results:

No 2021 results were included within the analysis for 2 reasons:

- Results were not confirmed at the time of writing.
- Results are less conservative in all relevant WMZs.

Results from 2021 campaign have not been issued formally, however draft data has been provided for some areas and the results, although to be confirmed, gave values that are less conservative than the figures chosen. The method used in the 2021 campaign comply with BRE 365.

To aid with the positioning and identification of the 2021 infiltration testing completed to date, the draft site location plans are shown in Appendix A.

3. Treatment Indices

3.1. ACA Treatment

The Simple Index Approach (SIA) was used to assess water quality management for the ACA. It was recognised that the ACA presented the largest difficulties and was the reason this assessment was carried out first. The treatment index for the SuDS features in the ACA have been reviewed and altered to Basin from Pond. The Basin index is for total suspended solids (0.5), metals (0.5) and hydrocarbons (0.6), is generally less than that of a pond (0.7, 0.7, 0.5) respectively. A summary of each area is shown below in the Table 3-1. Note that where the total mitigation index values were greater than 1, these are limited to state '>0.95' as advised by the SIA tool.

As shown, some areas within the ACA are shown to not have sufficient mitigation methods for each contaminant type. Currently the flows in some areas flow directly into the basin without upstream pre-treatment.

It is anticipated that a mixture of SuDS features and proprietary methods will be introduced during Detailed Design in the appropriate areas to address these shortfalls as noted in the ACA Drainage Strategy Technical Note DCO Task D4 (SZC-EW0320-ATK-XX-000-XXXXXX-NOT-CIV-000003).

Table 3-1 - ACA SuDS mitigation indices for discharges to surface waters

ACA area	Assigned Pollution hazard levels	SuDS features proposed	Total SuDS mitigation Index		
			TSS	Metals	Hydrocarbons
Park and Ride area	Medium	- Permeable pavement - Basin	0.95 (>0.7)	0.85 (>0.6)	>0.95 (>0.7)
Logistics compound	Medium	- Permeable Pavement - Basin	0.95 (>0.7)	0.85 (>0.6)	>0.95 (>0.7)
Railway	Medium	- Filter drains - Basin	0.65 (<0.7)*	0.65 (>0.6)*	0.7 (=0.7)*
Material Transfer Laydown	High	- Permeable Pavement - Basin	0.95 (>0.8)	0.85 (>0.8)	>0.95 (>0.9)
Sand & Aggregate Stockpile	High	- Basin	0.5 (<0.8)**	NA	NA
Topsoil compound	High	- Swale - Basin	0.75 (<0.8)	0.85 (>0.8)	0.9 (=0.9)
HGV parking	High	- Basin	0.5 (<0.8)*	0.5 (<0.8)*	0.6 (<0.9)*
Caravan Pitches	Medium	- Permeable Pavement - Basin	0.95 (>0.7)	0.85 (>0.6)	>0.95 (>0.7)

** Drainage treatment to be supplemented by proprietary non-SuDS treatment, to be discussed and agreed with LLFA.*

*** Sand & Aggregate stockpile compound to be reviewed in next design phase to investigate the use of swales or filter drains around the perimeter of this compound.*

3.2. Simplified Treatment Indices Approach

To demonstrate water quality risk management, the Simple Index Approach (SIA) outlined in Section 26.7 of CIRIA C753 The SuDS Manual can be used to characterise hazards and SuDS performance capacities by assigning simple qualitative indices. To deliver adequate treatment, the selected SuDS components should have a total pollution mitigation index (for each contaminant) type that equals or exceeds the pollution hazard index (for each contaminant type). From Table 26.2 of CIRIA C753 The SuDS Manual, the pollution hazard index for the SZC development can be assigned for different land use classifications. In general, the Main Development Site can be categorised into either 'High' or 'Medium' hazard levels as shown in the table below.

Table 3-2 - Pollution hazard indices for different land use classifications

Land use	Risk Level	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Individual property driveways, residential car parks, low traffic roads (e.g. cul de sacs, home zones and general access	Low	0.5	0.4	0.4

roads) and non-residential car parking with infrequent change (e.g. schools, offices) i.e. <300 traffic movements/day				
Commercial yard and delivery areas, non-residential car parking with frequent change (e.g. hospital, retail), all roads except low traffic roads and trunk roads/motorways.	Medium	0.7	0.6	0.7
Sites with heavy pollution (e.g. haulage yards, lorry parks, highly frequented lorry approached to industrial estates, waste sites), sites where chemicals and fuels (other than domestic fuel oil) are to be delivered, handled, stored, used or manufactured; industrial sites; trunk roads and motorways.	High	0.8	0.8	0.9

An assessment was conducted for the ACA and is presented in the ACA Drainage Strategy Technical Note DCO Task D4 (SZC-EW0320-ATK-XX-000-XXXXXX-NOT-CIV-000003). The ACA is not presented in this section.

3.2.1. Temporary Construction Area

In general, surface water runoff in the TCA will be collected and/or directed towards one or more SuDS features as shown in the table below. As outlined in the WMZ1 Surface Water Treatment Assessment Technical Note (ref. SZC-EW0320-ATK-XX-000-XXXXXX-NOT-CCD-000006), three discharge pathways are considered and are all shown to demonstrate sufficient water quality management. This approach applies to other WMZ's within the TCA.

- Pathway 1 – Filter Strip and Swale to Groundwater via infiltration trench.
- Pathway 2 – Filter Strip and Swale and Basin to Groundwater.
- Pathway 3 – Filter Strip and Swale and Basin to Watercourse.

Table 3-3 - SuDS Mitigation Indices (includes mitigation indices for discharge to ground water Table 26.4 of CIRIA C753 The SuDS Manual)

Pathway	TSS	Metals	Hydrocarbons
Filter Strip + Swale (with infiltration trench)	0.85	0.9	>0.95
Filter Strip + Swale + Basin (infiltration at basin)	>0.95	>0.95	>0.95
Filter Strip + Swale + Basin (discharge to watercourse only)	0.9	0.95	>0.95

Whilst catchments differ in their proposed land use, and therefore associated risk level, a 'high' risk level has been used to demonstrate a worst-case scenario. A detailed assessment of each catchment, and their proposed land-uses (e.g. contractor compound, stockpile etc.) will be carried out at the next design stage. During Detailed Design, optimisation of proposed features will be undertaken, and additional water management features will be considered and introduced on a risk management basis where necessary.

At this stage, the WMZ 10 (Accommodation Campus area) has conservatively been assigned a 'medium' hazard risk level, however this will be reviewed during Detailed Design as this area can also be described as a 'low' risk level. Surface water runoff in WMZ 10 will generally be treated and attenuated using a porous pavement build-up. Where good infiltration potential is identified, these will be explored further at detailed design to maximise infiltration to ground. The runoff may be conveyed towards an outfall, that is consistent with the existing (non-developed) runoff, should infiltration be too low to provide an adequate solution. This runoff can be conveyed via swales to provide additional water treatment. See Section 9.1 for further information on the Campus drainage strategy.

Table 3-4 - TCA SuDS mitigation indices

Water Management Zone	Highest Hazard in Zone	Hazard Risk	Risk Indices (TSS/ Metals/ Hydrocarbons)	Discharge pathway with least treatment	Treatment Index (TSS/ Metals/ Hydrocarbons)
WMZ 1	Haul Road	High	0.8, 0.8, 0.9	1	0.85, 0.9, >0.95
WMZ 2	Haul Road	High	0.8, 0.8, 0.9	1	0.85, 0.9, >0.95
WMZ 3	Haul Road	High	0.8, 0.8, 0.9	1	0.85, 0.9, >0.95
WMZ 4	Haul Road	High	0.8, 0.8, 0.9	1	0.85, 0.9, >0.95
WMZ 5	Haul Road	High	0.8, 0.8, 0.9	1	0.85, 0.9, >0.95
WMZ 6	Haul Road	High	0.8, 0.8, 0.9	1	0.85, 0.9, >0.95
WMZ 10 – Campus	Access Road	Medium	0.7, 0.6, 0.7	Pervious Pavement only	0.7, 0.6, 0.7

3.2.2. Main Construction Area (WMZs 7, 8 and 9)

The collection of surface water across WMZs 7, 8 and 9 will be designed to suit the sequence of construction events. In the early phases, prior to the completion of the cut-off wall, surface water will be collected and held in temporary ditches/bund and sediment ponds within the MCA area, before being treated using proprietary devices, such as Siltbuster packaged treatment plant (60 mg/l suspended solids), if required. Where necessary, the packaged treatment plant will be operated to perform in line with the water quality and discharge requirements set out in the water discharge permit. The captured runoff will be discharged to the diverted Sizewell Drain, or in extreme circumstances, to the sea via the temporary marine outfall.

Upon completion of the cut-off wall, surface water within WMZ9 will be managed by constructing multiple sediment ponds at low points within the excavation, constantly evolving ahead of the main excavation areas. Water from within the ponds will infiltrate into the ground and be captured within the dewatering process and directed to the Groundwater Treatment Plant, before discharging to the sea via the Combined Drainage Outfall (CDO).

Discharge from WMZ 7 and 9 will be directly to the sea via the Combined Drainage Outfall (CDO) during construction phase, and the discharge from the plant when it becomes operational will be via the cooling water tunnel.

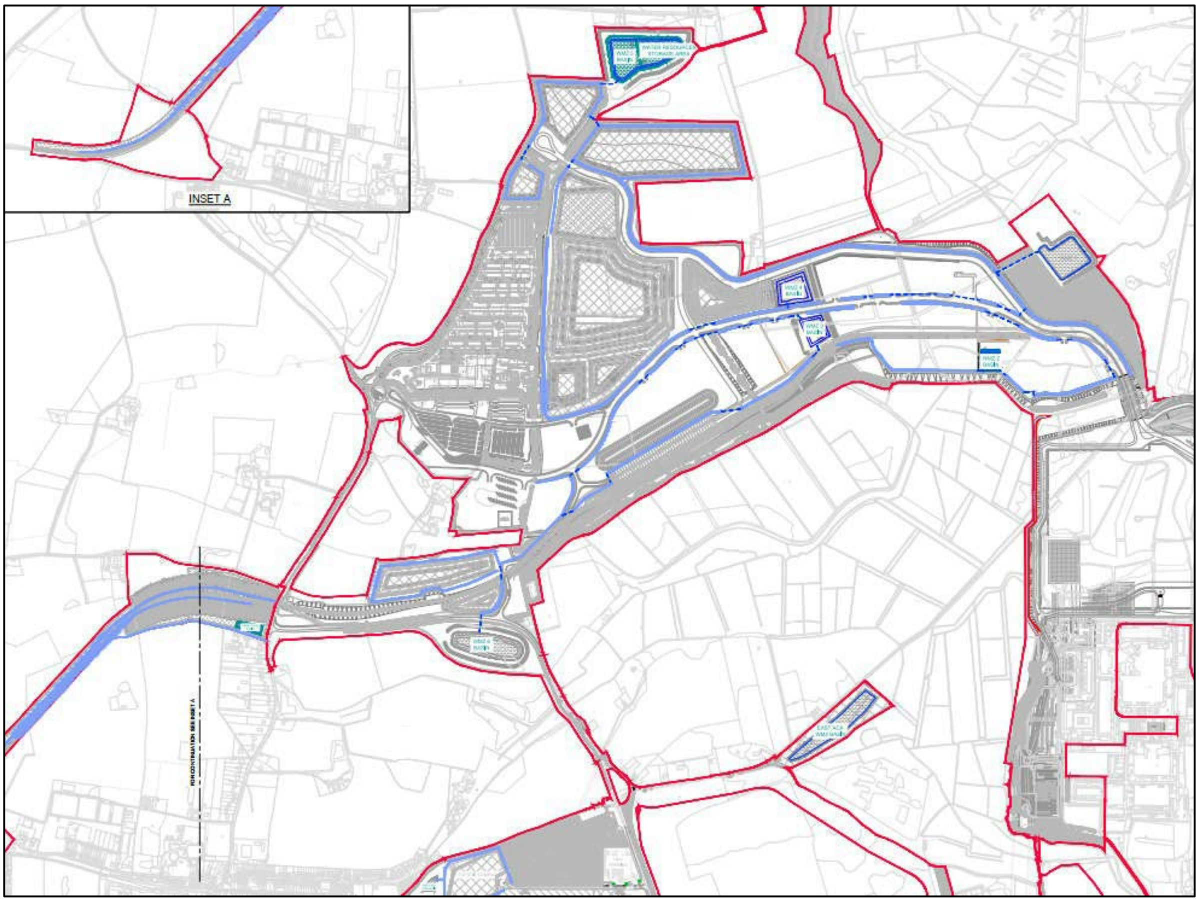
WMZ 8 is currently proposed to drain using filter drains along the verge and attenuated sub surface to restrict it to greenfield runoff rates. From the SIA, filter drains alone do not provide sufficient mitigation (0.4, 0.4, 0.4) and further work will be undertaken at the next design stage to ensure adequate water treatment is proposed. The proposals are to be developed and agreed with SCC.

Further to the above, it is proposed to remove as much sediment as possible as close to source as possible and this can be done by installing wheel washes at the MCA when trucks exit the excavation, as well as wheel washes positioned at stockpile/borrow put areas. Secondly, road sweeper operation along the access roads and haul roads is proposed, reducing the need to remove silt from the swales and filter drains. All surface water drainage proposals will be reviewed and refined in Detailed Design to ensure sufficient water treatment is provided prior to discharge to surface waters.

4. Perimeter Swale

An overview of the swale network is provided indicatively in Figure 4-1 below and in Appendix C. The swales shown on the drawing are between 4 and 6 m wide across the site. The final position and geometry of the swale network will be progressed during the next design stage and will ensure water quantity and quality benefits are realised in accordance with CIRIA C753 The SuDS Manual. This may entail dedicating a larger area for this purpose, and the provision of additional swale features across the development site.

Figure 4-1 – Indicative Swale Network Overview (ref. SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000038)



5. Basin Design for Treatment

The general guidance provided in the CIRIA C753 SuDS manual will be used assist in the design of attenuation basins for treatment. A range of these factors have been used in Hinkley Power Station, which are intended to be replicated at SZC.

The attenuation basins are to have sediment forebays upstream by dividing off areas of the basin using permeable berms. This reduces velocity of flows entering and allows sediment to build locally. These areas require regular desilting to ensure continued operation.

The quality of the water can be further improved by additional use of permeable berms that encourage a serpentine flow of water. This maximises the flow path length thereby allowing more time for sedimentation. There is an opportunity to have vegetated sides and a small permanent pool near the outlet. This option will be considered during detailed design.

The sizes of the proposed basins are large and there is an expectation that they are very unlikely to overflow to a watercourse. During normal storm events there is every reason to expect these basins to operate well, delivering the water quality required. For basins located in WMZs 1, 2, 3 & 4 there is a proposed connection to the spine network, which discharges to the CDO. This is expected to be required only in the rarest of times and allow drawdown of the basin water level.

The addition of proprietary devices, such as a Siltbuster packaged treatment plant, may be considered at detailed design to ensure the water quality requirements (60 mg/l suspended solids etc.) are adhered to.

6. Calculation of impermeable / permeable areas

The table below shows the breakdown of the type of area (roofed, paved, unpaved and soft) within each catchment and the assigned percentage impervious (PIMP) value, used to determine an overall PIMP for the catchment. The 'Design PIMP' is the value taken forward in the calculation of the required storage (Water Management Zone Summary Technical Note DCO Task D2) and is more conservative than the calculated PIMP.

WMZ	Total Catchment Area (ha)	Total Catchment Area (m^2)	Area type (m^2) and associated PIMP (%)				Overall Catchment PIMP (%)	Design PIMP (%)
			Roofing	Paved	Unpaved[1]	Soft[2]		
			100%	90%	50%	30%		
WMZ1	19.43	194300	34070	87778	72452	0	77%	90%
WMZ2	17.37	173700	61410	94247	18043	0	89%	90%
WMZ3	20.96	209600	5149	148757	55694	0	80%	90%
WMZ4	33.32	333200	0	29572	85303	205441	39%	50%
WMZ5	31.20	311952	0	11512	253282	47159	48%	50%
WMZ6	47.77	477700	17345	99984	319495	40876	58%	58%
ACA East	26.84	268410	100% PIMP Considered				100%	100%
ACA West	4.438	44380	100% PIMP Considered				100%	100%
Abbey Road	6.478	64780	50	300	64780	0	50%	50%
Campus	20.48	204800	33541	97004	74255	0	77%	80%

[1] Unpaved areas including grassed verges and landscaping to provide worst case scenario

[2] Soft areas comprise of stockpile areas only

Catchment areas, type of area and associated PIMP values may be subject to change and to be reviewed in Detailed Design.

The Design PIMP was used to calculate the Percentage Runoff (PR) and Volumetric Runoff Coefficient (Cv) for each catchment using equations 7.1 and 7.3 of Design and Analysis of Urban Storm Drainage - The Wallingford Procedure, Volume 1, September 1981.

7. Hydrological Catchment

The existing ground (surface) contours can be seen in the drawing ‘Existing Ground Surface ref. SZC-EW0000-XX-000-DRW-400008’ (Appendix B). The contours defined are at 1m (minor - yellow line) and 5m (major – cyan line) intervals. Early catchment areas were defined based on the existing levels and contour information. These catchments are approximations of where surface water would generally flow with some consideration of where runoff may be diverted/captured as a result of the initial earthworks. In places, land external to the red line boundary was included as part of the early catchment areas, as it is shown to contribute to surface water runoff within the SZC site.

The early catchments, along with early outfalls (presented in Section 8) is shown in drawing SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052. As works progress within the ACA, MCA, TCA and Railway areas, these early catchments will evolve in shape and size and become definitive catchments which have been designed in the Enabling Works Surface Water Drainage Basic Design. These catchments (late or Enabling Works) are shown in SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000053.

Drawings SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052 and SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000053 are shown in Appendix D.1 and D.2 respectively.

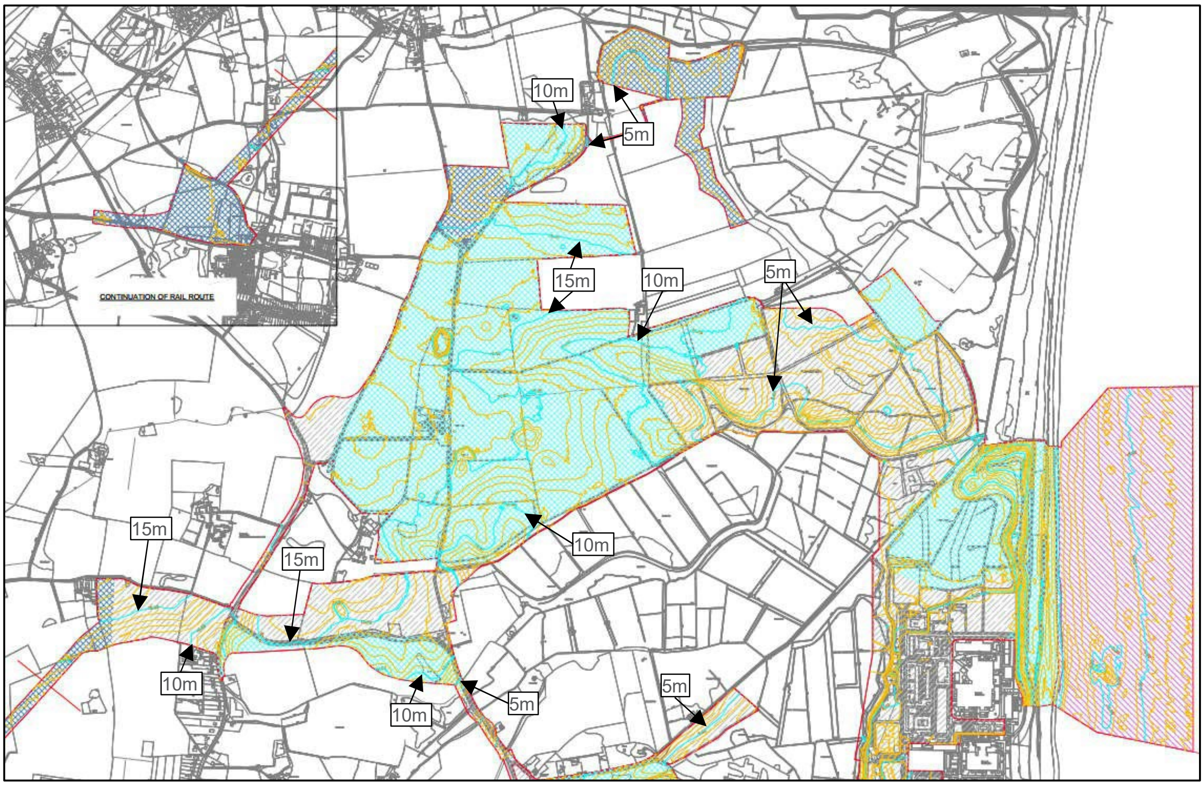


Figure 7-1 - Existing Ground Contours (SZC-EW0000-XX-000-DRW-400008)

8. Basin Design (Size)

8.1. WMZ Basin Parameters

Table 8-1 below presents the current WMZ basin dimensions and sizes that are proposed in the Main Development Site. All basins have a 1 in 3 side slope, except WMZ6 basin which has a 1 in 4 side slope. The side slopes will be no steeper than 1 in 3.

Table 8-1 - Allocated WMZ Basin Sizes

WMZ	Area at base (m²)	Base level of Basin (mAOD)	Area at freeboard level (m²)	Depth to freeboard level (m)	WMZ Basin Volume (m³) Base to Freeboard Level	Area at top of basin (m²) 300mm Freeboard	WMZ Basin Volume (m³) including freeboard
WMZ1	10579.2	1.200	12618.8	1.500	17398.5	13786.5	**21929.1**
WMZ2	3290.1	3.200	6274.5	3.700	17694.5	6554.8	**19689.8**
WMZ3	3224.3	5.000	6082.1	3.500	16286.2	6368.6	**18226.5**
WMZ4	5357	5.200	8931.8	3.500	25005.4	9279.1	**27808.6**
WMZ5	7051.6	6.000	9193.6	2.000	16245.2	9533.5	**19072.9**
WMZ6	7165.8	8.000	11287.5	2.100	19376.0	11911.5	**22892.8**
ACA East	12968.8	1.600	15431.6	1.100	15620.2	16117.6	**20360.5**
ACA West	659.6	5.700	1510.9	2.000	2170.5	1667.8	**2676.5**
Abbey Road	1268.6	6.742	1964.5	1.158	1872.0	2161	**2500.2**

8.2. Half Drain Times and Follow-on Storms

Table 8-2 below presents the input parameters, along with the basin sizes stated in Table 8-1 used in Innoyvze Source Control to determine the maximum volume and critical storm event at which this occurs for each WMZ basin for a 100-year return period (RP), and a 10-year RP. The basins have been designed with a factor of safety of 1.5 applied to the infiltration rate. This represents the recognised lower risk associated with basins used for construction purposes that are of a temporary nature. The infiltration rate is applied to side walls of the structure only and no infiltration has been applied to the base area.

Table 8-2 - Source Control Basin Design Inputs

WMZ	Basin Side Slope	WMZ catchment area (ha)	Outflow (l/s)	Water Course Outlet	Infiltration rate (m/s)	Infiltration Testing Data Set
WMZ1	1/3	19.43	19.43	Y	0	N/A
WMZ2	1/3	17.37	17.37	Y	7.55E-06	TP-WMZ-23 (2017 SSL - Test 1)
WMZ3	1/3	20.96	20.96	Y	1.34E-06	WMZ3_2020-3-TP-A (2020 Fugro – Test 1)
WMZ4	1/3	33.32	33.32	Y	7.76E-06	TP-WMZ-21 (2017 SSL Test - 2)
WMZ5	1/3	31.20	31.20	Y	1.24E-06	TP-BP-4 (2017 SSL – Test 1)
WMZ6	1/4	47.77	47.77	Y	5.58E-06	WMZ6_2020-2-PIT (2020 Fugro – Test 3)
ACA East	1/3	26.84	62.00	Y	0	N/A
ACA West	1/3	4.44	10.25	Y	0	N/A
Abbey Road	1/3	6.48	6.50	Y	1.06E-04	GR11A (Structural Soils 2014 - Test 3)

Table 8-3 below shows the maximum volume of water for a 100yr RP plus 20% climate change allowance from Source Control. The Flood Studies Report (FSR), Flood Estimation Handbook (FEH) 1999 and FEH 2013 rainfall-runoff methods were checked and for the 100yr RP, the FEH 2013 was most onerous.

Also stated in the table below is a comparison between the basin volume provided (Table 8-1) and the maximum water volume. Values for the volume drained in a 24-hour period from each WMZ basin are also provided, and are based on the proposed outflow, without infiltration. The table shows that basin volumes are adequate (except ACA East and West) to contain the 1:100+CC critical storm.

ACA East and West figures in Table 8-3 show a shortfall of approximately 900 m^3 and 2000 m^3 respectively, to contain the 1:100+CC storm represents the Source Control volume and not the detailed hydraulic model (MicroDrainage) results therefore no network volumes have been taken into consideration. This additional volume will be provided within the pipe network, swales and sub-surface attenuation that are proposed across various sub-catchments within the ACA and is further detailed in the ACA Drainage Strategy Technical Note (DCO Task D4) (ref. SZC-EW0320-ATK-XX-000-XXXXXX-NOT-CIV-000003).

Table 8-3 - Maximum Water Volume - 100yr RP +20% CC critical storm event

WMZ	Critical Storm Event	Max Storm Volume (m^3)	Max Water Depth (m)	Half Drain Time (mins)	Half Drain Time (days)	Spare Volume in basin (m^3)	Volume drained in 24hr (m^3)	Spare Volume after 24hrs (m^3)
WMZ1	2160 min Winter	15116.6	1.319	6639	4.61	6812.5	1678.8	8491.3
WMZ2	2160 min Winter	12761.1	2.916	4684	3.25	6928.7	1500.8	8429.5
WMZ3	2160 min Winter	16051.7	3.505	6796	4.72	2174.8	1810.9	3985.8
WMZ4	1440 min Winter	11433.3	1.839	2589	1.80	16375.3	2878.8	19254.1
WMZ5	1440 min Winter	11030.5	1.417	2932	2.04	8042.4	2695.7	10738.0
WMZ6	1440 min Winter	19745.4	2.147	2836	1.97	3147.4	4127.3	7274.7
ACA East	1440 min Winter	22,592.2	1.540	2906	2.02	-2,231.7	5356.8	3125.1
ACA West	1440 min Winter	3581.3	2.895	2934	2.04	-904.8	885.6	-19.2
Abbey Road	240 min Winter	1432	0.933	346	0.24	1068.2	561.6	1629.8

The SuDS manual does not require that attenuation basins should be able to receive a follow-on storm but rather that they are able to deal with a rare event such as a 1:100+CC. This has always been the basis of design.

At this stage, a simplified analysis of a subsequent storm (10yr RP) was undertaken and show a number of the basins do have additional volume to contain a follow-on storm and this volume varies from basin to basin reflecting available space on site. Table 8-4 below shows the maximum volume of water for a 10yr RP plus 20% climate change allowance from Source Control. Critical storm events for a 10yr RP varied between FEH 1999 and FEH 2013 rainfall-runoff methods as stated in the table. The purpose of this table is to approximate how each WMZ basin will manage a 100yr critical storm event, followed by a 10yr critical storm event, after 24 hours. This additional volume cannot, in all cases, contain a critical 10yr RP storm event. This is a highly improbable scenario and to achieve the volumes states would lead to an extremely conservative design. The right-hand side column shows the available volume within each basin using the peak (discrete) values only. It must be noted that, whilst it is a conservative representation, it also does not accurately represent a continuous rainfall profile. The scope of this subsequent storm analysis will be agreed with SCC and will be completed during the design development to consider continuous rainfall profiles.

Table 8-4 - Maximum Water Volume - 10yr RP +20% CC critical storm event

WMZ	Critical Storm Event	Max Storm Volume (m^3)	Spare Volume in Basin after 24hrs of 100yr RP event (m^3)	Spare Volume - 10RP Volume (m^3)
WMZ1	FEH 1999 2880 min Winter	7682	8491.3	809.3
WMZ2	FEH 1999 2160 min Winter	6577.3	8429.5	1,852.2
WMZ3	FEH 1999 2880 min Winter	8242.7	3985.8	-4,256.9
WMZ4	FEH 2013 960 min Winter	5362.5	19254.1	13,891.6
WMZ5	FEH 1999 1440 min Winter	5127.6	10738.0	5,610.4
WMZ6	FEH 1999 1440 min Winter	9432.4	7274.7	-2,157.7

ACA East	FEH 1999 1440 min Winter	10988.2	3125.1	-7863.1
ACA West	FEH 2013 720 min Winter	1630.6	-19.2	-1,649.8
Abbey Road	FEH 2013 360 min Winter	730.4	1629.8	899.4

The additional volume from WMZ3 may be interconnected with WMZ4 to alleviate any flood risk and will be considered during design development. The shortfall in WMZ6 will also be accommodated through upstream storage within the drainage network.

It must be recognised that these are extreme events and much of the surrounding area will be under water. There is no risk to habitation present next to the construction site basins as they are surrounded by open areas. In addition, there would be no risk for water quality as there are large dilution effects.

8.3. ACA West Basin Half Drain Time

The proximity of this basin means that the flood risk should be minimised, and it is therefore appropriate that the half drain time should meet the 24 hour requirement.

The source control volume for a 1:100 + 20% CC in the ACA West would require a basin volume of 3,581 m^3. For a 24-hour half drain time this equates to 4.71 l/s/ha (1,790.5 x 1000 / 24 x 3600 = 20.7 l/s for 4.44 Ha). It is anticipated that the source control volume is the worst case and that the detailed design figure, which takes into consideration other storage volumes, upstream of the basin, will reduce this pumped value. To achieve the 24-hour half drain time a pumped discharged is proposed to be set to approximately 4.71 l/s/ha, giving a maximum flow of 20.9 l/s (based upon Source Control data). This flow would discharge to Outfall O6, subject to agreement from SCC, the Internal Drainage Board and the Environment Agency. Alternatively, this additional volume can be pumped to the ACA East basin and will be considered during design development in coordination with SCC.

The pumping station arrangement would be as per Sewers for Adoption in regard to pump provision. A twin pump arrangement (duty standby) would be in place with alarms (level and failed to start). In addition to alarms the arrangement of the basin allows the water level to be easily viewed from outside and has the benefit of the proximity of staff to speedily react to them.

In the unlikely event that failure of the pumped outflow from the ACA West basin coincides with a 100yr RP storm event, a simple volume estimation is shown below. The duration of the 100yr RP storm event has been limited to 24 hours to acknowledge that a temporary solution or repair of the pumped network can be completed with 24 hours. Nonetheless, this consideration will be reviewed during the design stage and with acceptance from SCC.

WMZ	Catchment Area (ha)	PIMP (%)	Infiltration rate (m/hr)	Outflow (l/s)	Max Volume (m^3) (15-1440 min)			Storm Event (100RP + 20%CC)		
					FSR	FEH 1999	FEH 2013	FSR	FEH 1999	FEH 2013
ACA West	4.438	100	0	0	3340.5	4258	4445.4	1440 min Winter	1440 min Winter	1440 min Winter

8.4. Surface Water Outfalls – Early and Late

Greenfield runoff estimates for all areas have been calculated using the IH124 method following the online ‘greenfield runoff rate estimation’ tool hosted by HR Wallingford. The greenfield runoff rates are relatively small considering the size of the catchment areas with Q_{BAR} (peak rate of flow from a catchment for the mean annual flood - return period of approximately 1:2.3 years) generally less than 5 l/s. The Environment Agency (EA) guidance states that the limiting discharge rates for sites should be set to Q_{BAR} or 1 l/s/ha, whichever is greater, as this is an unreasonable requirement for permeable sites which results in large storage volumes (Environment Agency - Rainfall runoff management for developments ref. SC030219). This advice has been followed for each catchment.

Table 8-5 and Table 8-6 state the determined greenfield runoff rates for the early and late catchments respectively in the current design.

Table 8-5 - Greenfield Runoff Rates vs 1 l/s/ha Summary for Early Catchments

			Discharge Rate (l/s)					
Site	Catchment Name	Total Area (ha)	1 in 1 yr	1 in 30 yr	1 in 100 yr	Qbar	1 l/s/ha *	Proposed
TCA	Early Catchment 1	26.221	3.07	8.66	12.59	3.53	26.22	26.22
TCA	Early Catchment 2	19.355	2.27	6.39	9.29	2.61	19.36	19.36
TCA	Early Catchment 3a	54.478	6.34	17.86	25.95	7.29	54.48	54.48
TCA	Early Catchment 3b	29.658	3.49	9.82	14.27	4.00	29.66	29.66
TCA	Early Catchment 4	38.191	4.49	12.64	18.37	5.16	38.19	38.19
TCA	Early Catchment 5	35.216	4.14	11.66	16.94	4.75	35.22	35.22
TCA	Early Catchment 6	19.117	2.25	6.33	9.20	2.58	19.12	19.12
Rail	Early Catchment 8	14.703	1.73	4.88	7.08	1.99	14.70	14.70
Rail	Early Catchment 9	3.027	6.11	17.20	24.99	7.02	3.03	7.02
Rail	Early Catchment 10	8.163	16.47	46.39	67.40	18.93	8.16	18.93
MCA	Early MCA	38.614	4.51	12.07	18.46	5.18	38.61	38.61
ACA	Early ACA	31.278	62.86	177.03	257.23	72.25	31.28	72.25

* Rate of discharge set to 1 l/s/ha for permeable sites where the Qbar is seen to be less than 1 l/s/ha - Chapter 3.3 of EA guidance Rainfall Runoff Management for Developments.

Table 8-6 - Greenfield Runoff Rates vs 1 l/s/ha Summary for Late TCA and ACA Catchments

Site	Catchment Name	Outfall	Catchment Area (ha)	Discharge Rate (l/s)					
				1 in 1 yr	1 in 30 yr	1 in 100 yr	Qbar	1l/s/ha *	Proposed
TCA	WMZ1	O1	19.430	2.27	6.39	9.29	2.61	19.43	19.43
TCA	WMZ2	O2	17.370	2.04	5.74	8.34	2.34	17.37	17.37
TCA	WMZ3	O3	20.960	2.46	6.94	10.08	2.83	20.96	20.96
TCA	WMZ4	N/A	33.320	3.92	11.03	16.03	4.5	33.32	N/A
TCA	WMZ5	O5	31.195	3.67	10.33	15.00	4.21	31.20	31.20
TCA	WMZ6	O6	47.770	5.62	15.81	22.98	6.45	47.77	47.77
Rail	GRR West 3	O8	6.478	0.77	2.16	3.13	0.88	6.48	6.48
Rail	GRR West 2	O9	1.377	2.82	7.96	11.56	3.25	1.38	3.25
Rail	GRR West 1	O10	0.706	1.41	3.98	5.78	1.62	0.71	1.62
ACA	West ACA WMZ	O6	4.438	8.92	25.12	36.5	10.25	4.44	10.25
ACA	East ACA WMZ	O7	26.841	53.95	151.91	220.74	62.00	26.84	62.00

* Rate of discharge set to 1 l/s/ha for permeable sites where the Qbar is seen to be less than 1 l/s/ha - Chapter 3.3 of EA guidance Rainfall Runoff Management for Developments.

The outfall locations are indicative and will be progressed at the next design stage. The greenfield runoff rates and proposed discharge rates may change should catchment extents develop and are subject to agreement from SCC, the Internal Drainage Board and the Environment Agency. A summary of this information is shown in Table 8-7.

Table 8-7 - Summary of Early and Late discharges

Area	Outfall	National Grid Reference	Indicative Invert Level (mAOD)	Early		Late	
				Discharge (l/s)	Method	Discharge (l/s)	Method
MCA	EO1	TM 47659 64054	1.550	200.00	None	0.00	None
WMZ1	O1	TM 47238 64963	0.500	26.22	l/s/ha	19.43	l/s/ha
WMZ2	O2	TM 46873 64545	0.500	19.36	l/s/ha	17.37	l/s/ha
WMZ3	EO3	TM 46573 64545	0.500	54.48	l/s/ha	0.00	None
WMZ3	O3	TM46354 64123	3.300	29.66	l/s/ha	20.96	l/s/ha
WMZ4	EO4	TM 45699 63890	2.500	38.19	l/s/ha	0.00	None
WMZ5	O5	TM 46443 65809	0.764	35.22	l/s/ha	31.20	l/s/ha
WMZ6	O6	TM 45473 63483	1.422	29.96	l/s/ha	47.77	l/s/ha
ACA West	O6	TM45473 63483	1.422	10.25	QBAR	10.25	QBAR
ACA East	O7	TM46523 63487	0.450	62.00	QBAR	62.00	QBAR
Railway	O8	TM 44477 63720	6.527	14.70	l/s/ha	5.00	Proposed
Railway	O9	TM43961 63705	12.500	7.02	QBAR	5.00	Proposed
Railway	O10	TM43525 63229	20.400	18.93	QBAR	5.00	Proposed
MCA	O11	TM 47980 64340	-3.250	0.00	None	2000.00	Max Flow
MCA	O12	TM 47005 64352	0.263	6.44	Averaged l/s/ha	17.08	Proportioned l/s/ha
MCA	O13	TM 47004 64182	0.251	6.44	Averaged l/s/ha	2.03	Proportioned l/s/ha
MCA	O14	TM 47000 64094	0.292	6.44	Averaged l/s/ha	3.88	Proportioned l/s/ha
MCA	O15	TM 46979 63984	0.308	6.44	Averaged l/s/ha	2.35	Proportioned l/s/ha
MCA	O16	TM 46979 63873	0.325	6.44	Averaged l/s/ha	11.42	Proportioned l/s/ha
MCA	O17	TM 46978 63790	0.344	6.44	Averaged l/s/ha	1.85	Proportioned l/s/ha

9. Operational Infrastructure

9.1. Campus

The surface water drainage strategy for the Campus (WMZ10) relies on discharging runoff to the ground at source through infiltration, without the need to discharge to a watercourse or surface water drainage network. Rainfall runoff is proposed to be stored below ground in areas such as car parks and other paved areas located within the catchment, using pervious pavement which allow gradual infiltration.

Infiltration to the ground will occur at different rates across the site depending on the characteristics of the underlying soil. Ground investigation campaigns from 2014 to 2020 show that the rates vary with lowest recording of 3.70x10-6 m/s (2014). This worst-case rate is considered too low to provide adequate infiltration, though further investigation will be carried out to determine areas of good infiltration and these will be explored further at detailed design to maximise infiltration to ground. No runoff is proposed to be conveyed to an attenuation basin.

The existing (undeveloped) site is at a high level in comparison to adjacent TCA areas, and the ground levels fall from west to east, towards WMZ4. Should infiltration rates be too low to provide an adequate solution, the runoff may be discharged at greenfield rates to an outfall along the Leiston Drain, south-east of the WMZ10. The final outfall position will consider the existing runoff conditions and flow paths within the catchment, as well as adjacent areas. The proposed rate will be limited to the greater of 1 l/s/ha or Q_{BAR} as per the strategy proposed for other TCA areas. This will be agreed in consultation with SCC during design development.

To provide an initial estimate on the required area needed to contain sub-surface storage within pervious pavement, an attenuation structure of 500mm depth was modelled in Innovyze Source Control as an infiltration basin with a porosity of 40% to symbolise a graded granular fill. The worst-case infiltrate rate of 3.70x10-6 m/s was applied to the base area only. A permitted outflow of 20.48 l/s (equivalent 1 l/s/ha) was included in the assessment. The output shows that a 11600 m^3 of storage is sufficient to store a 100yr +20% CC storm event, which is equivalent to a footprint of 58000 m^2, which is significantly less than the available paved area within the catchment - 97004 m^2. The half drain times is approximately 744 minutes, much lower than the 24-hour requirement, therefore a subsequent storm analysis is considered necessary.

At this stage, where there are large car parking areas proposed, it is proposed that these areas use permeable surfacing. The surfacing will be robustly constructed, emulating the current drainage characteristics, whilst providing suitable treatment of an incidental oil spills. In addition, the access ways between buildings and non-heavily tracked areas with the Campus will also employ permeable surface to allow infiltration at source. Runoff from roofed areas may also conveyed to the subsurface storage where practicable, as well as storage provided in tree pits, where trees are proposed. Opportunities to provide further infiltration at source, using features such as infiltration trenches, will be explored during Detailed Design.

Following the Simple Index Approach (SIA) guidance in CIRIA C753 The SuDS Manual on water quality management, the Campus area largely falls into a low-risk hazard level. The use of porous paving alone can provide sufficient treatment and the SIA criteria will be satisfied. As the design develops and should parts of the Campus area align to a medium-risk hazard level, porous paving will still satisfy the SIA criteria. A review will be undertaken in the next design stage considering the inclusion of further SuDS features and the proposals will be discussed with SCC.

9.2. Nuclear Island

The MCA has 3 stages: Early, Construction and Operation. Each stage has a different mode of operation for the surface water.

9.2.1. Early

Upon site establishment, and as topsoil stripping and earthworks are undertaken, the early construction site will potentially run the risk of being flooded. Surface water runoff will be retained on site by constructing temporary ditches/bunds and sediment ponds. Runoff that does not infiltrate will undergo treatment using packaged treatment plant (e.g. Siltbuster – 60 mg SS/l) if required prior to discharge to the realigned Sizewell Drain, or to the sea. Where necessary, the packaged treatment plant will be operated to perform in line with the water quality and discharge requirements set out in the water discharge permit. During this phase it is proposed to construct six outfalls along the realigned Sizewell Drain to prevent starving the Sizewell Marshes and to maintain the existing hydrological conditions. Further to this, a temporary marine outfall EO1 is available to discharge directly to sea. This runs across the beach with pedestrian protection and is proposed to allow excess surface water runoff to be discharged to the sea during construction options prior to completion of the Combined Drainage Outfall (CDO). The outfalls will be controlled through conditions set by the Environment Agency through discharge permit applications. Infiltration would still play a major role in surface water control at this stage.

9.2.2. Construction

As the site develops and on completion of the CDO, the temporary marine outfall (EO1) would no longer be required and will be removed. For the construction phase there is a series of 6 outfalls along the western edge of the MCA and when commissioned, the Combined Discharge Outfall (CDO) which outfalls to the sea.

The construction area is divided into 3 catchments, which become defined as the cut off wall is constructed:

- WMZ 7, which controls the water to the east of the main excavation. WMZ 7 is pumped to the CDO.
- WMZ 8, which includes the 6 outfalls to the west. WMZ 8 drains into the 6 outfalls to the west.
- WMZ 9, which is the main excavated area within the cut off wall (COW). WMZ 9 is pumped to the CDO.

9.2.3. Operation

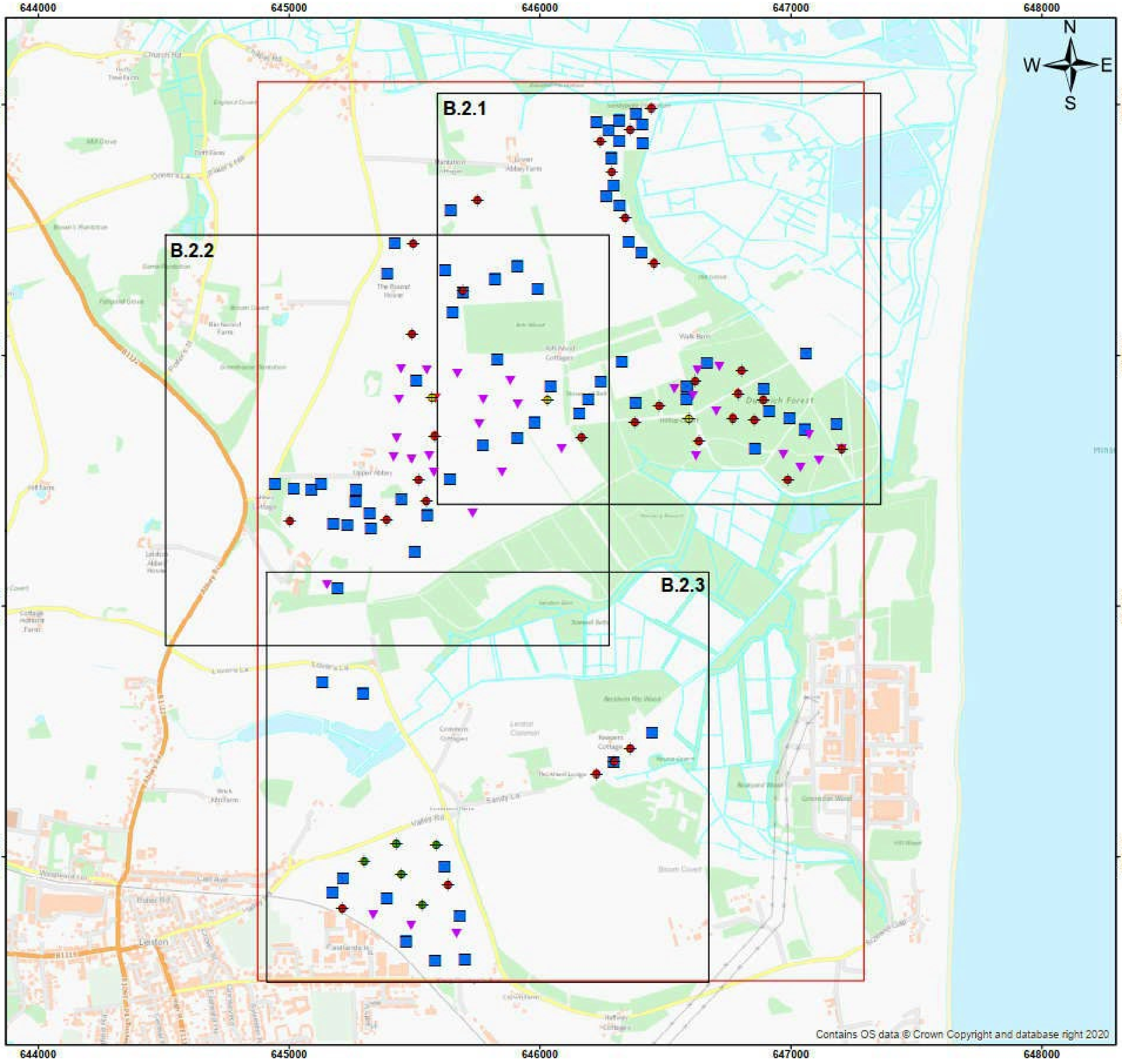
During the Operational stage the surface water is controlled in 2 ways. The western WMZ 7 is still to discharge through the 6 outfalls, whilst the remainder of the main site is to discharge to the cooling seawater outfall. The CDO would not be used.

9.2.3.1. Permanent Car Park

A permanent car park is planned within the area designated as the Temporary Construction Area. This has not been designed in detail but will comply with SuDS design philosophy and any future amendments to that design code. The design will be developed in coordination and agreement with SCC.

10. Appendices

Appendix A. 2021 GI Site Location Plan



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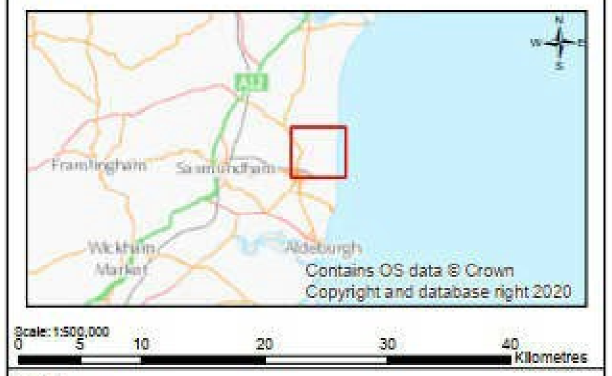

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Legend

🟥	Site Boundary
☐	Exploratory Location Plan Sheets

Location Type

●	Cable Percussion Borehole
●	Sonic Borehole
●	Dynamic Sampling Borehole
▼	CPT
▼	Inspection Pit with CPT Follow-on
■	Trial Pit

Notes
Some exploratory locations have been plotted using the setting out grid coordinates as agreed by NNB (as detailed on the exploratory location records).

Rev.	Date	Description	Initials
1	20/08/2021	Draft Issue	LCB

Scale: 1:15,000
0 150 300 600 900 1,200 Metres

Coordinate System
British National Grid

Client
NNB Generation Company (SZC) Limited
EDF-NNB, 90 Whitfield Street,
London, W1T 4EZ
Tel: 020 321 98311
Website: www.edfenergy.com

Investigation Supervisor
Atkins

Contractor
Fugro GeoServices Limited
Fugro House, Hithercroft Road, Wallingford,
Oxfordshire, OX10 9RB, United Kingdom
Registered in England No. 1284362
VAT No. GB 133 1704 69
www.fugro.com

Project Title
Sizewell C - Onshore Ground Investigation for Enabling Works - TCA/ACA/NW/SPL/BP

Drawing Title
Site Location Plan

Drawing Number
B.1.1

Drawn By	Checked By	Issued On	Project No.	Sheet Size	Rev.
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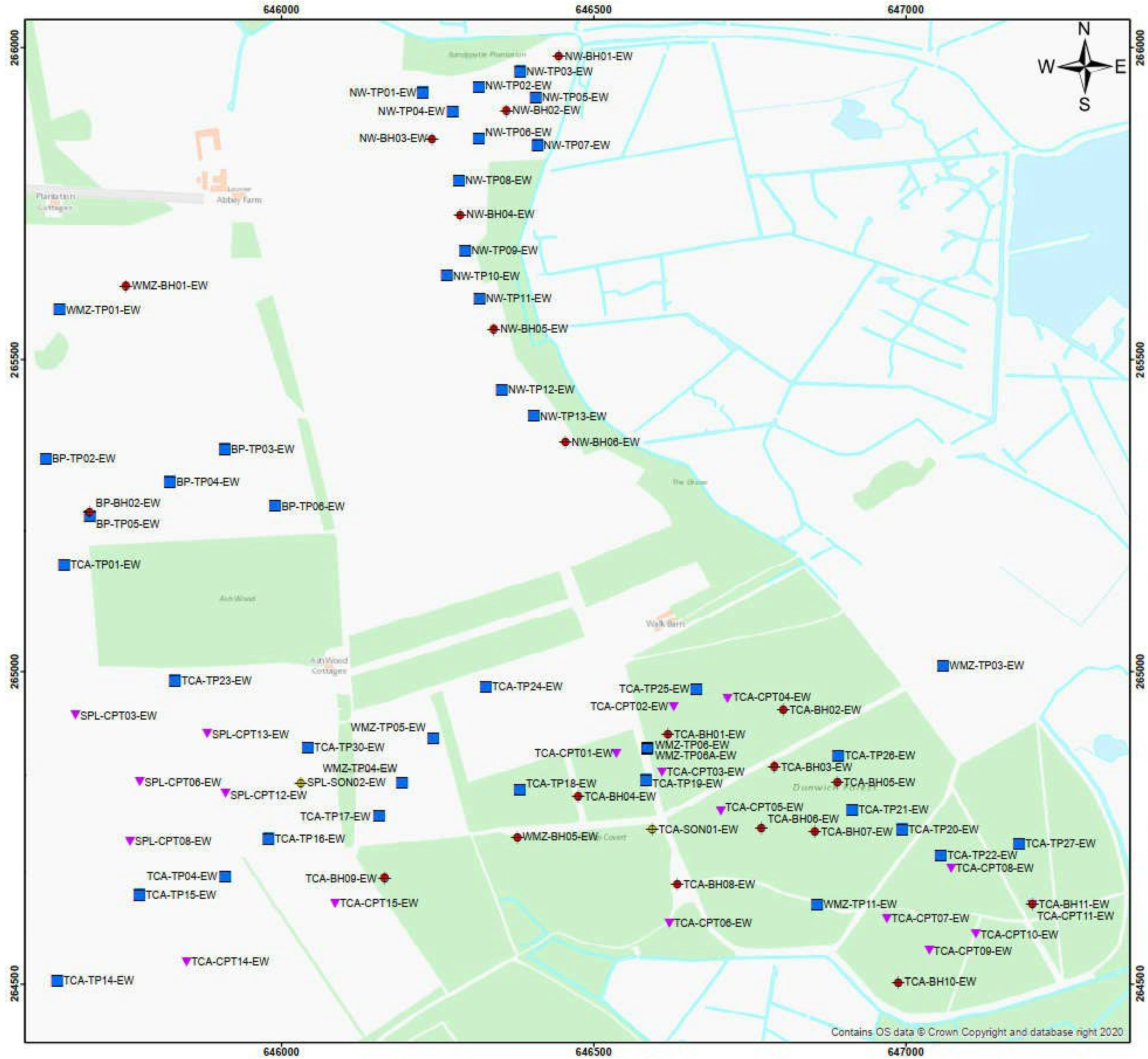

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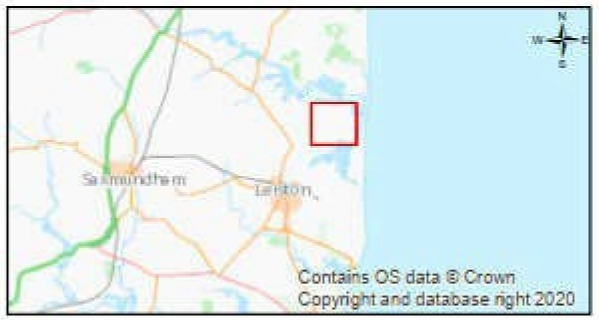

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Legend

Location Type

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- ● Sonic Borehole
- ● Dynamic Sampling Borehole
- ▼ CPT
- ▼ Inspection Pit with CPT Follow-on
- ■ Trial Pit

Notes
Some exploratory locations have been plotted using the setting out grid coordinates as agreed by NNB (as detailed on the exploratory location records).

Rev.	Date	Description	Initials
1	18/08/2021	Draft Issue	LCB

Scale: 1:6,000
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Coordinate System
British National Grid

Client
NNB Generation Company (SZC) Limited
EDF-NNB, 90 Whitfield Street,
London, W1T 4EZ
Tel: 020 321 98311
Website:www.edfenergy.com

Investigation Supervisor
Atkins

Contractor
Fugro GeoServices Limited
Fugro House, Hithercroft Road, Wallingford,
Oxfordshire, OX10 9RB, United Kingdom
Registered in England No. 1284352
VAT No. GB 133 1704 69
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Project Title

Sizewell C - Onshore Ground Investigation for Enabling Works - TCA/ACA/NW/SPL/BP

Drawing Title

Exploratory Location Plan

Drawing Number

B.2.1

Drawn By	Checked By	Issued On	Project No.	Sheet Size	Rev.
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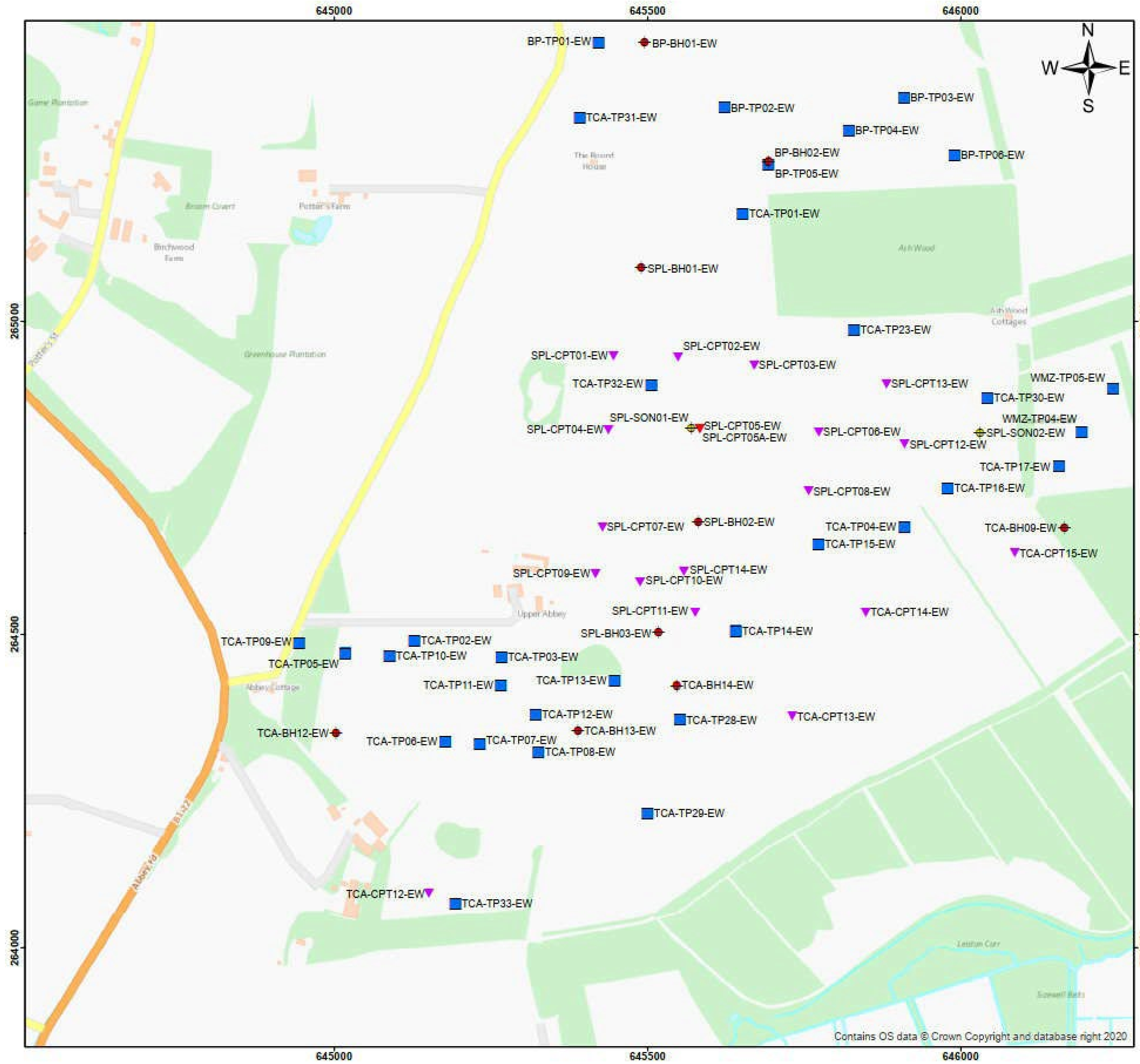

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Scale: 1:250,000
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Legend

Location Type

- ● Cable Percussion Borehole
- ● Sonic Borehole
- ● Dynamic Sampling Borehole
- ▼ CPT
- ▼ Inspection Pit with CPT Follow-on
- ■ Trial Pit

Notes

Some exploratory locations have been plotted using the setting out grid coordinates as agreed by NNB (as detailed on the exploratory location records).

Rev.	Date	Description	Initials
1	12/07/2021	Draft Issue	LCB

Scale: 1:6,000
0 50 100 200 300 400 Metres

Coordinate System

British National Grid

Client
NNB Generation Company (SZC) Limited
EDF-NNB, 90 Whitfield Street,
London, W1T 4EZ
Tel: 020 321 98311
Website: www.edfenergy.com

SZC

Investigation Supervisor
Atkins

Contractor
Fugro GeoServices Limited
Fugro House, Hithercroft Road, Wallingford,
Oxfordshire, OX10 9RB, United Kingdom
Registered in England No. 1284352
VAT No. GB 133 1704 09
www.fugro.com

FUGRO

Project Title

Sizewell C - Onshore Ground Investigation for Enabling Works - TCA/ACA/NW/SPL/BP

Drawing Title

Exploratory Location Plan

Drawing Number

B.2.2

Drawn By	Checked By	Issued On	Project No.	Sheet Size	Rev.
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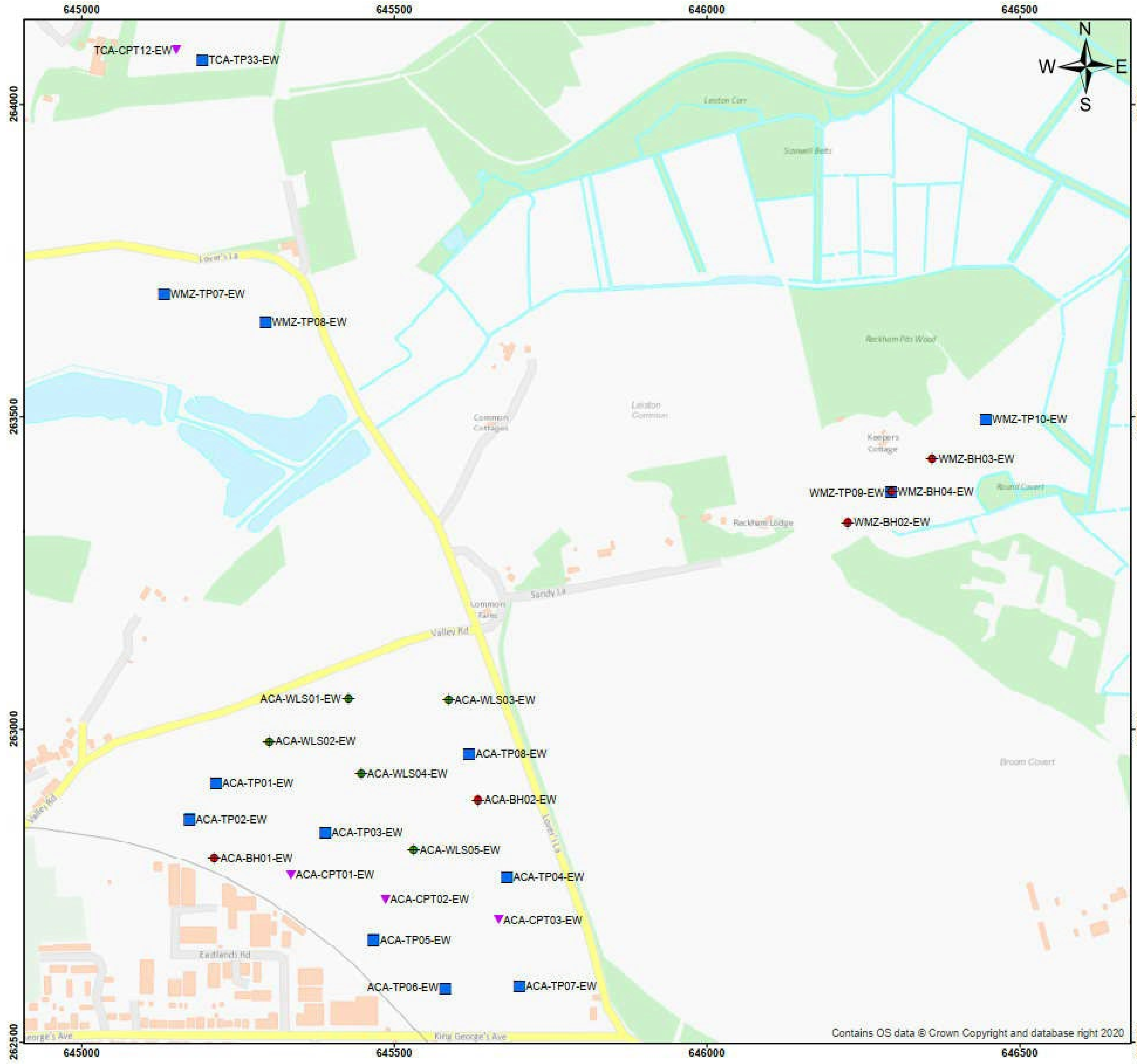

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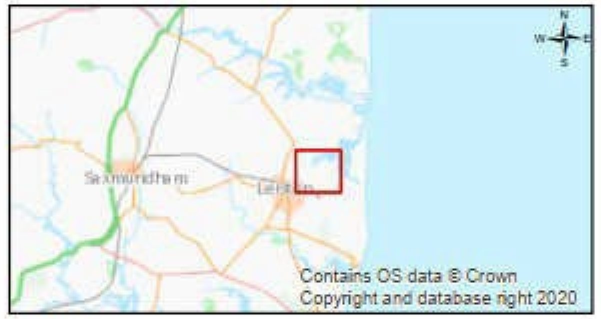
Location



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Scale: 1:250,000
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Legend

Location Type

- ⊕ Cable Percussion Borehole
- ⊕ Sonic Borehole
- ⊕ Dynamic Sampling Borehole
- ▼ CPT
- ▼ Inspection Pit with CPT Follow-on
- ■ Trial Pit

Notes
Some exploratory locations have been plotted using the setting out grid coordinates as agreed by NNB (as detailed on the exploratory location records).

Rev.	Date	Description	Initials
1	12/07/2021	Draft Issue	LCB

Scale: 1:6,000
0 50 100 200 300 400 Metres

Coordinate System
British National Grid

Client
NNB Generation Company (SZC) Limited
EDF-NNB, 90 Whitfield Street,
London, W1T 4EZ
Tel: 020 321 98311
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Investigation Supervisor
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Fugro GeoServices Limited
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Registered in England No. 1284352
VAT No. GB 133 1704 09
www.fugro.com

Project Title
Sizewell C - Onshore Ground Investigation for Enabling Works - TCA/ACA/NW/SPL/BP

Drawing Title
Exploratory Location Plan

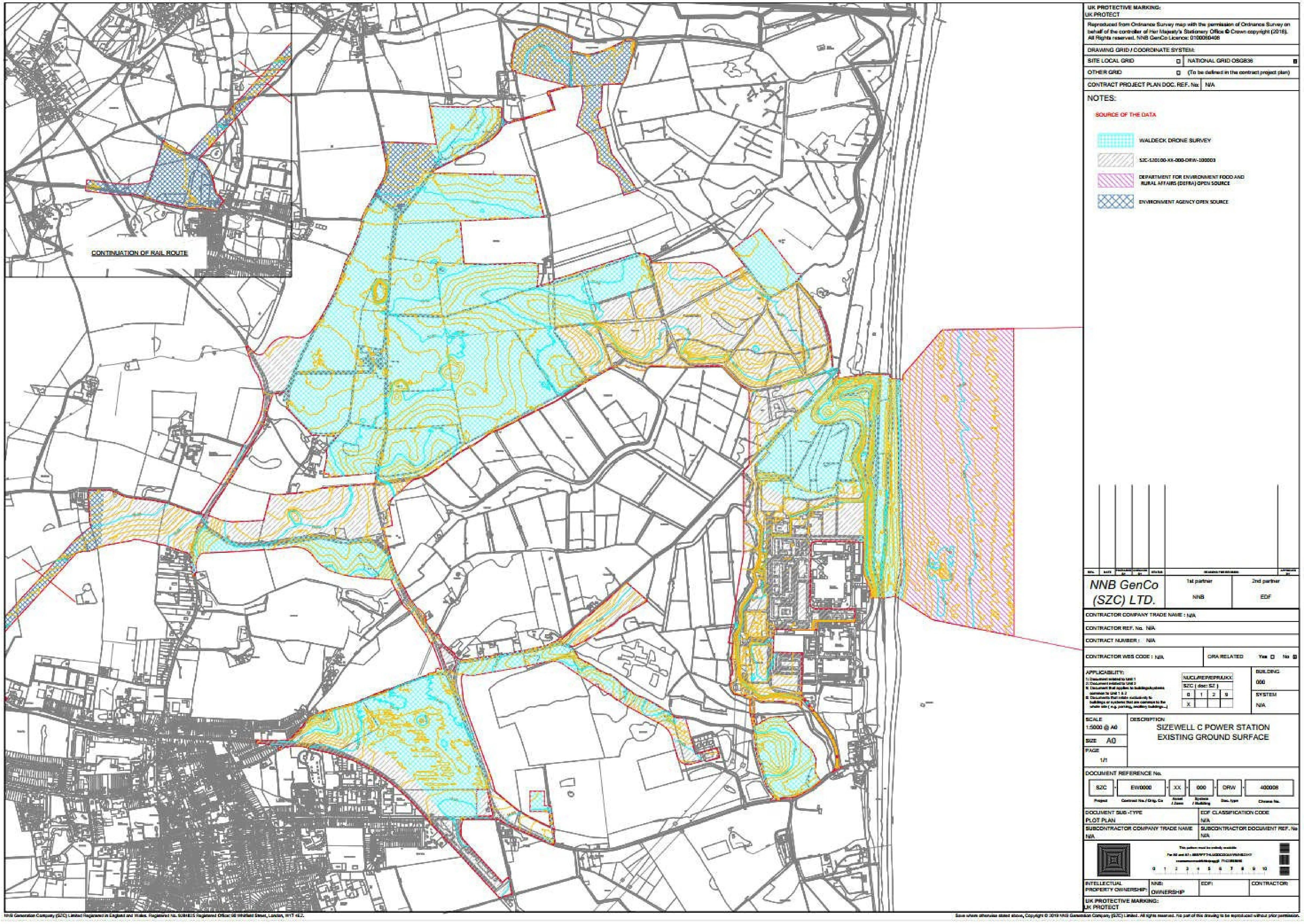
Drawing Number
B.2.3

Drawn By	Checked By	Issued On	Project No.	Sheet Size	Rev.
LCB	CAY	12/07/2021	F181377	A3	1

Appendix B. Existing Ground Surface

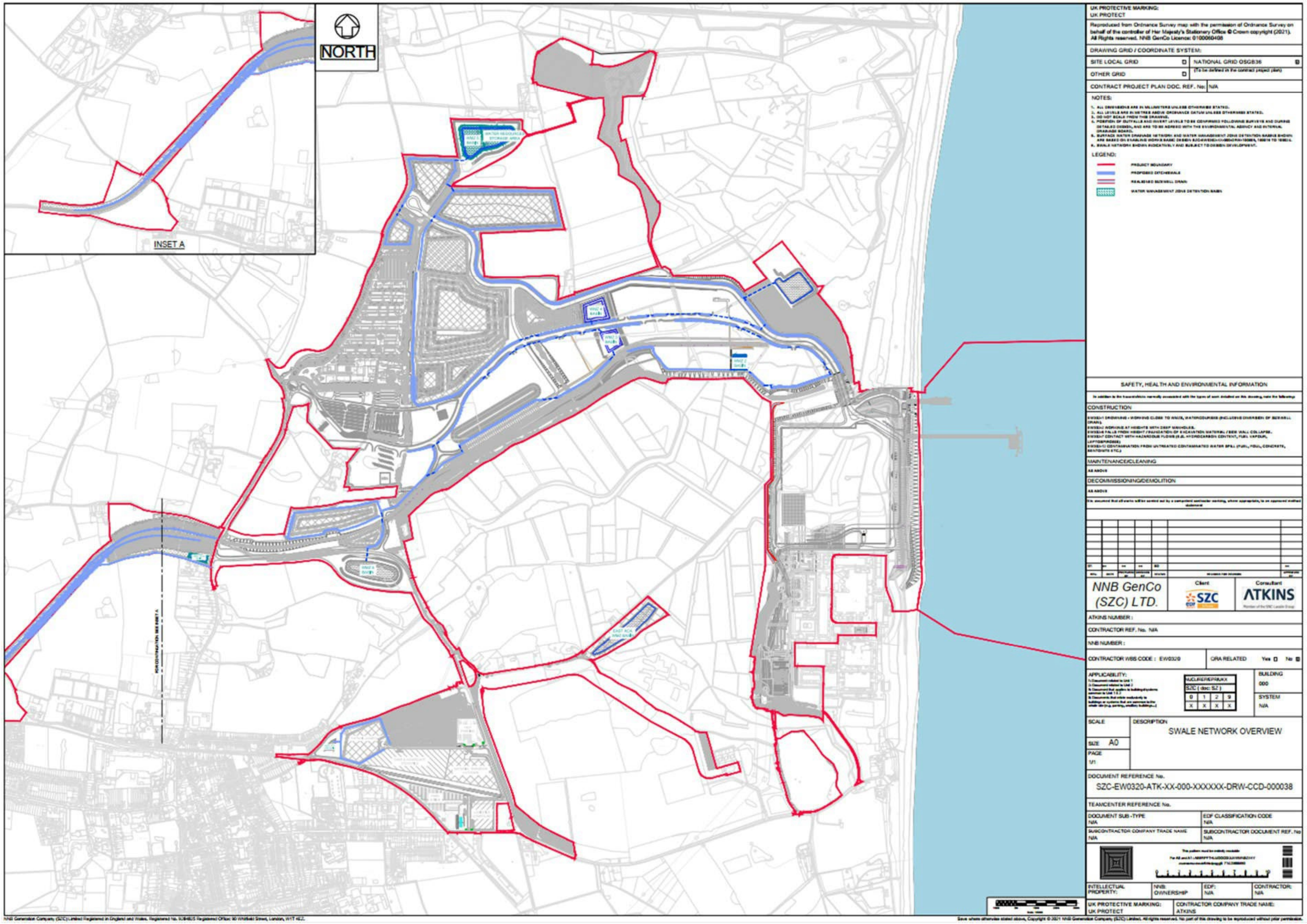
Contours defined: 1m (minor - yellow line) and 5m (major – cyan line) intervals

NOT PROTECTIVELY MARKED



NOT PROTECTIVELY MARKED

Appendix C. Swale Network Overview


NORTH
INSET A
UK PROTECTIVE MARKING:
UK PROTECT
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DRAWING GRID / COORDINATE SYSTEM:
SITE LOCAL GRID ☐
NATIONAL GRID OSGB36 ☒
OTHER GRID ☐
(To be defined in the contract project plan)
CONTRACT PROJECT PLAN DOC. REF. No: N/A
NOTES:
LEGEND:
PROJECT BOUNDARY
PROPOSED DITCH/SWALE
REALIGNED SIZEWELL DRAIN
WATER MANAGEMENT ZONE DETENTION BASIN
SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION
CONSTRUCTION
MAINTENANCE/CLEANING
DECOMMISSIONING/DEMOLITION
NNB GenCo (SZC) LTD.
Client
SZC
Consultant
ATKINS
ATKINS NUMBER :
CONTRACTOR REF. No. N/A
NNB NUMBER :
CONTRACTOR WBS CODE : EW0320
CRA RELATED Yes ☐ No ☒
APPLICABILITY:
NUCLEREREPRUKX
SZC (doc: SZ)
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X X X X
BUILDING
000
SYSTEM
N/A
SCALE
SIZE A0
PAGE
1/1
DESCRIPTION
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SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000038
TEAMCENTER REFERENCE No.
DOCUMENT SUB -TYPE
N/A
EDF CLASSIFICATION CODE
N/A
SUBCONTRACTOR COMPANY TRADE NAME
N/A
SUBCONTRACTOR DOCUMENT REF. No
N/A
INTELLECTUAL PROPERTY:
NNB: OWNERSHIP
EDF: N/A
CONTRACTOR: N/A
UK PROTECTIVE MARKING:
UK PROTECT
CONTRACTOR COMPANY TRADE NAME:
ATKINS

Appendix D. Early & Late Catchments and Outfalls

D.1. Surface Water Outfall Locations – Early - SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052

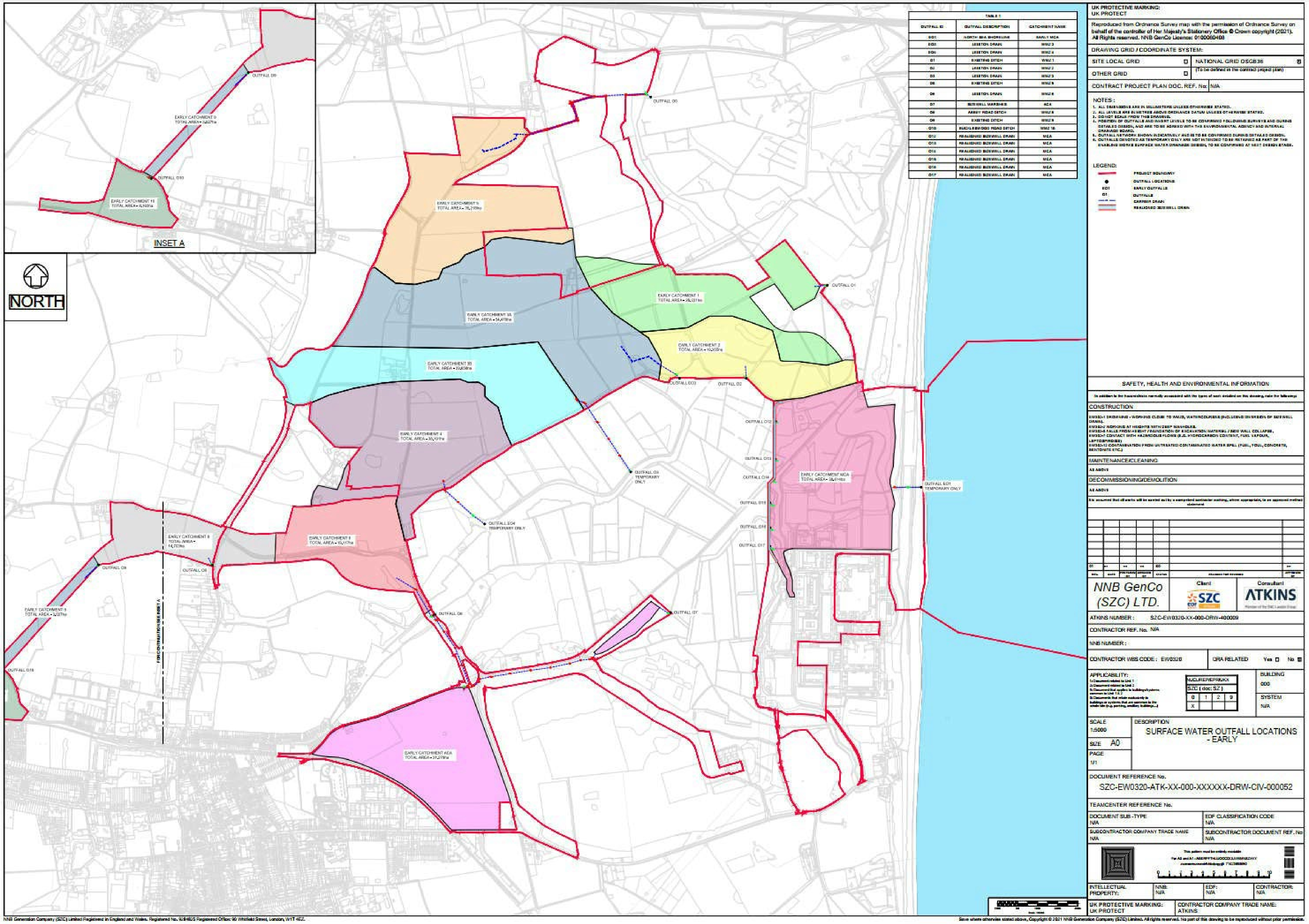

TABLE 1

OUTFALL ID	OUTFALL DESCRIPTION	CATCHMENT NAME
EO1	NORTH SEA SHORELINE	EARLY MCA
EO3	LEISTON DRAIN	WMZ 3
EO4	LEISTON DRAIN	WMZ 4
O1	EXISTING DITCH	WMZ 1
O2	LEISTON DRAIN	WMZ 2
O3	LEISTON DRAIN	WMZ 3
O5	EXISTING DITCH	WMZ 5
O6	LEISTON DRAIN	WMZ 6
O7	SIZEWELL MARSHES	ACA
O8	ABBEY ROAD DITCH	WMZ 8
O9	EXISTING DITCH	WMZ 9
O10	BUCKLESWOOD ROAD DITCH	WMZ 10
O12	REALIGNED SIZEWELL DRAIN	MCA
O13	REALIGNED SIZEWELL DRAIN	MCA
O14	REALIGNED SIZEWELL DRAIN	MCA
O15	REALIGNED SIZEWELL DRAIN	MCA
O16	REALIGNED SIZEWELL DRAIN	MCA
O17	REALIGNED SIZEWELL DRAIN	MCA

UK PROTECTIVE MARKING:
UK PROTECT

DRAWING GRID / COORDINATE SYSTEM:

SITE LOCAL GRID ☐ NATIONAL GRID OSGB36 ☒
OTHER GRID ☐ (To be defined in the contract project plan)

CONTRACT PROJECT PLAN DOC. REF. No: N/A

NOTES :
1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED.
2. ALL LEVELS ARE IN METRES ABOVE ORDNANCE DATUM UNLESS OTHERWISE STATED.
3. DO NOT SCALE FROM THIS DRAWING.
4. POSITION OF OUTFALLS AND INVERT LEVELS TO BE CONFIRMED FOLLOWING SURVEYS AND DURING DETAILED DESIGN, AND ARE TO BE AGREED WITH THE ENVIRONMENTAL AGENCY AND INTERNAL DRAINAGE BOARD.
5. OUTFALL NETWORK SHOWN INDICATIVELY AND IS TO BE CONFIRMED DURING DETAILED DESIGN.
6. OUTFALLS DENOTED AS TEMPORARY ONLY ARE NOT INTENDED TO BE RETAINED AS PART OF THE ENABLING WORKS SURFACE WATER DRAINAGE DESIGN, TO BE CONFIRMED AT NEXT DESIGN STAGE.

LEGEND:

🟥	PROJECT BOUNDARY
●	OUTFALL LOCATIONS
EO1	EARLY OUTFALLS
O1	OUTFALLS
	CARRIER DRAIN
	REALIGNED SIZEWELL DRAIN

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

CONSTRUCTION

MAINTENANCE/CLEANING

AS ABOVE

DECOMMISSIONING/DEMOLITION

AS ABOVE

NNB GenCo (SZC) LTD.

Client: SZC

Consultant: ATKINS

ATKINS NUMBER : SZC-EW0320-XX-000-DRW-400009

CONTRACTOR REF. No. N/A

NNB NUMBER :

CONTRACTOR WBS CODE : EW0320

QRA RELATED Yes ☐ No ☒

APPLICABILITY:

NUCLIRE/EPR/PRIMA
SZC (doc: SZ)

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X			

BUILDING
000

SYSTEM
N/A

SCALE
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SIZE A0

PAGE
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SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052

TEAMCENTER REFERENCE No.

DOCUMENT SUB -TYPE
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EDF CLASSIFICATION CODE
N/A

SUBCONTRACTOR COMPANY TRADE NAME
N/A

SUBCONTRACTOR DOCUMENT REF. No
N/A

INTELLECTUAL PROPERTY: NNB: N/A EDF: N/A CONTRACTOR: N/A

UK PROTECTIVE MARKING: UK PROTECT

CONTRACTOR COMPANY TRADE NAME: ATKINS

D.2. Surface Water Outfall Locations – Late - SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000053

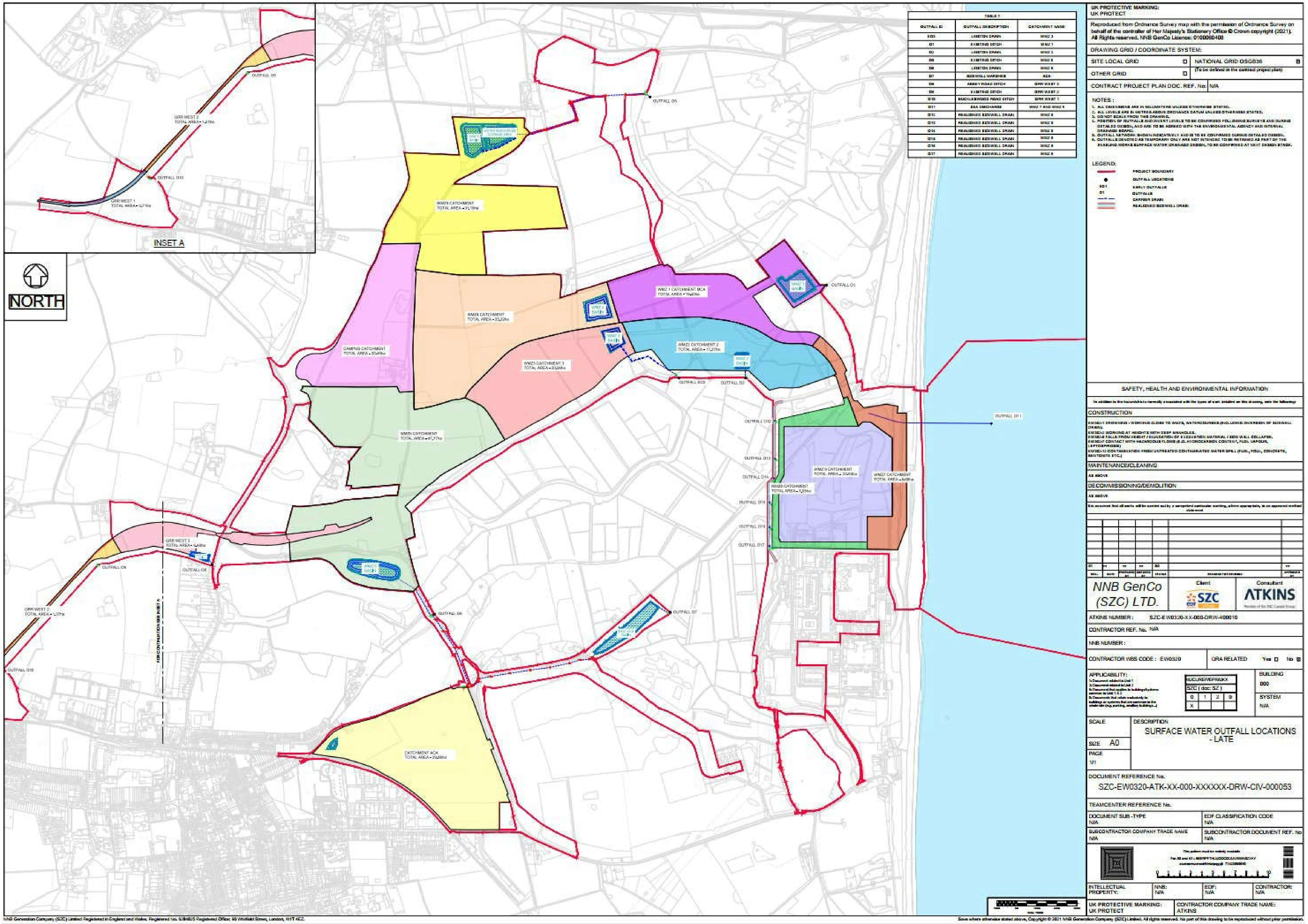

TABLE 1		
OUTFALL ID	OUTFALL DESCRIPTION	CATCHMENT NAME
EO1	LEISTON DRAIN	WMZ 3
O1	EXISTING DITCH	WMZ 1
O2	LEISTON DRAIN	WMZ 3
O5	EXISTING DITCH	WMZ 6
O6	LEISTON DRAIN	WMZ 6
O7	SIZEWELL MARSHES	ACA
O8	ABBEY ROAD DITCH	GRR WEST 3
O9	EXISTING DITCH	GRR WEST 2
O10	BUCKLESWOOD ROAD DITCH	GRR WEST 1
O11	SEA DISCHARGE	WMZ 7 AND WMZ 9
O12	REALIGNED SIZEWELL DRAIN	WMZ 8
O13	REALIGNED SIZEWELL DRAIN	WMZ 8
O14	REALIGNED SIZEWELL DRAIN	WMZ 8
O15	REALIGNED SIZEWELL DRAIN	WMZ 8
O16	REALIGNED SIZEWELL DRAIN	WMZ 8
O17	REALIGNED SIZEWELL DRAIN	WMZ 8

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DRAWING GRID / COORDINATE SYSTEM:

SITE LOCAL GRID ☐ NATIONAL GRID OSGB36 ☒
OTHER GRID ☐ (To be defined in the contract project plan)

CONTRACT PROJECT PLAN DOC. REF. No: N/A

NOTES :
1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED.
2. ALL LEVELS ARE IN METRES ABOVE ORDNANCE DATUM UNLESS OTHERWISE STATED.
3. DO NOT SCALE FROM THIS DRAWING.
4. POSITION OF OUTFALLS AND INVERT LEVELS TO BE CONFIRMED FOLLOWING SURVEYS AND DURING DETAILED DESIGN, AND ARE TO BE AGREED WITH THE ENVIRONMENTAL AGENCY AND INTERNAL DRAINAGE BOARD.
5. OUTFALL NETWORK SHOWN INDICATIVELY AND IS TO BE CONFIRMED DURING DETAILED DESIGN.
6. OUTFALLS DENOTED AS TEMPORARY ONLY ARE NOT INTENDED TO BE RETAINED AS PART OF THE ENABLING WORKS SURFACE WATER DRAINAGE DESIGN, TO BE CONFIRMED AT NEXT DESIGN STAGE.

LEGEND:

🟥	PROJECT BOUNDARY
●	OUTFALL LOCATIONS
EO1	EARLY OUTFALLS
O1	OUTFALLS
—	CARRIER DRAIN
🟥	REALIGNED SIZEWELL DRAIN

SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION

CONSTRUCTION

MAINTENANCE/CLEANING

AS ABOVE

DECOMMISSIONING/DEMOLITION

AS ABOVE

NNB GenCo (SZC) LTD.

Client
Consultant
ATKINS

ATKINS NUMBER : SZC-EW0320-XX-000-DRW-400010

CONTRACTOR REF. No. N/A

NNB NUMBER :

CONTRACTOR WBS CODE : EW0320 QRA RELATED Yes ☐ No ☒

APPLICABILITY:

NUCL/REP/EPR/AUX			
SZC (doc: SZ)			
0	1	2	9
X			

BUILDING
000

SYSTEM
N/A

SCALE

SIZE A0

PAGE
1/1

DESCRIPTION
SURFACE WATER OUTFALL LOCATIONS - LATE

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ANNEX 2A.6: NORTHERN PARK AND RIDE DRAINAGE DESIGN NOTE

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1 INTRODUCTION

1.1.1 NNB Generation Company (SZC) Limited (SZC Co.) submitted an application for a Development Consent Order (DCO) to the Planning Inspectorate under the Planning Act 2008 for the Sizewell C Project (referred to as the ‘Application’) in May 2020. The Application was accepted for examination in June 2020.

1.1.2 The northern park and ride development was originally submitted to the Planning Inspectorate (PINS) as part of the Application to build and operate a new nuclear power station to the north of Sizewell B.

1.1.3 SZC Co. has undertaken work to validate and develop the design of the northern park and ride that was originally submitted as part of the Application. This document forms one of a series of design validation and evolution documents being provided to the Examining Authority in support of the **Outline Drainage Strategy** [REP2-033].

1.1.4 The northern park and ride forms one of the Associated Developments (AD) which are required to mitigate traffic impacts arising from the main development site. The northern park and ride would be located alongside the A12 at Darsham. Its function would be to provide a transport hub from which construction workforce are driven to site by coach, thus reducing the construction traffic needing to access the main development site. Full details of its facilities are contained in **Volume 3 Northern Park and Ride Chapter 2 Description of the Northern Park and Ride** [APP-350] and are described in summary below.

1.1.5 The site would consist of workforce parking, welfare, security and amenity buildings. The workforce parking includes car parking spaces, accessible spaces, minibus/van spaces, pick up and motorcycle spaces.

1.1.6 The site access road and A12 roundabout would be designed to Suffolk County Council’s (SCC) adoptable standards.

1.1.7 The northern park and ride site would generate surface water runoff from paved areas and roofs which would require to be removed, treated as necessary and disposed.

1.1.8 The site entrance and access from the A12 would generate highway runoff which would require to be removed, treated as necessary and disposed.

1.1.9 The northern park and ride welfare facilities would generate foul water flows which would require to be removed, treated as necessary and disposed.

1.1.10 The northern park and ride facility and its associated access and A12 road changes would remain in place and use during construction of the power station. Once construction is complete the site would be closed and decommissioned. It would then return to current agricultural use.

1.1.11 It is intended that the proposed access roundabout would be removed and the A12 would be returned to its current alignment.

2 PURPOSE

2.1.1 The **Outline Drainage Strategy** [REP2-033] identified at concept level the proposed drainage approach required for:

- The effective removal of highway and surface water runoff from the proposed northern park and ride, A12 roundabout and site access road, together with its treatment and disposal
- The effective removal of foul water generated by the workforce from the proposed northern park and ride.

2.1.2 The proposed drainage infrastructure was described in the concept drainage design submitted as part of the Application. This concept design was based on data and information available at that time. The design was supported by the submission of the **Northern Park and Ride Flood Risk Assessment** (FRA) [APP-115].

2.1.3 This concept drainage strategy was developed in consultation with drainage regulators and local authorities, including SCC and the Environment Agency (EA). The observations/requirements of drainage regulators were incorporated in the strategy.

2.1.4 The purpose of this technical note is to provide details of data which validates the Outline Drainage Strategy, a description of how the proposed concept drainage infrastructure is developing and evolving and to demonstrate that it continues to provide for the effective and satisfactory drainage of the northern park and ride and its associated external road modification, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution. This technical note is updated at revision 03 to include for new infiltration data that has become available, provide additional information and responses to points raised by SCC following their review during the DCO Examination Stage.

2.1.5 This technical note is updated at revision 04 to address comments raised by SCC following their review of revision 03. These are shown in Appendix G

2.1.6 It is intended that this updated drainage strategy and resultant drainage infrastructure will remain in accordance with the with the **Outline Drainage Strategy** [REP2-033] submitted to the Examining Authority. It is further intended that following consultation with the Lead Local Flood Authority, it will be submitted to and approved by East Suffolk Council.

3 DESCRIPTION OF DCO DRAINAGE CONCEPT DESIGN

3.1.1 The northern park and ride concept drainage at DCO stage was developed by SZC Co. Proposals were developed for both the northern park and ride development site and associated modification of existing public highway required in order to provide access to and from the site.

3.1.2 Subject to achievable infiltration rates all surface water generated within the northern park and ride red line boundary would be contained within the site and discharged to ground. If necessary, excess runoff which couldn’t infiltrate would be discharged to a local watercourse, located within the red line boundary, at pro rata greenfield rates.

3.1.3 External roads modified to access the site would discharge to swales and filter drains where they infiltrate to ground.

3.1.4 Traditional drainage with surface outlets, gullies, combined kerb drains (CKDs) etc would be provided at the A12 roundabout and discharge into the filter drains.

3.1.5 A final infiltration basin was proposed at the limit of the roundabout northern arm. This would collect and infiltrate runoff which is not removed by the swales and filter drains.

3.1.6 Although the presence of a public foul water sewer was identified located running along the A12, given its shallow depth it was considered that a gravity connection would not be possible. Accordingly, at that stage whilst retaining the theoretical option of discharging the site generated foul water to public sewer, the proposed infrastructure would be a local private foul water network discharging into a package sewage treatment plant. The treated effluent would discharge to ground by infiltration.

3.1.7 If the flow generation is too low or intermittent to be treated to the required standard or infiltration does not work, then a sealed tank (cess tank) would be provided with effluent being collected and removed by tanker for offsite treatment.

3.1.8 A single remote security cabin at the site entrance would drain to a septic tank with infiltration to ground. If infiltration rates are inadequate the septic tank would in effect become a cess tank.

3.1.9 The internal site layout showing the position of proposed swales, with potential outfall to watercourse and the sewage treatment plant is shown in **Plates 1** and **2** which are an extract from Application drawing **”Chapter 2 Description of the Northern Park and Ride** Figure 2.4” [APP-351].

Plate 1: Northern park and ride internal layout showing concept drainage infrastructure to the south

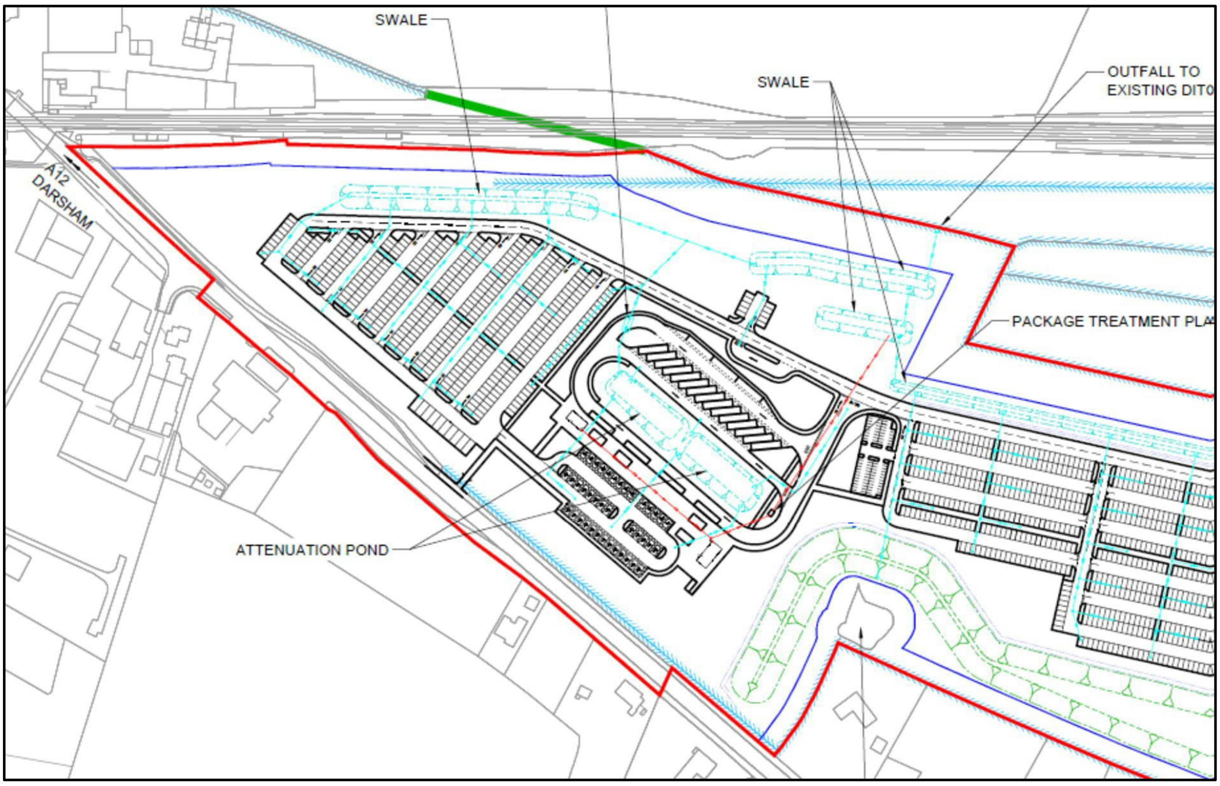
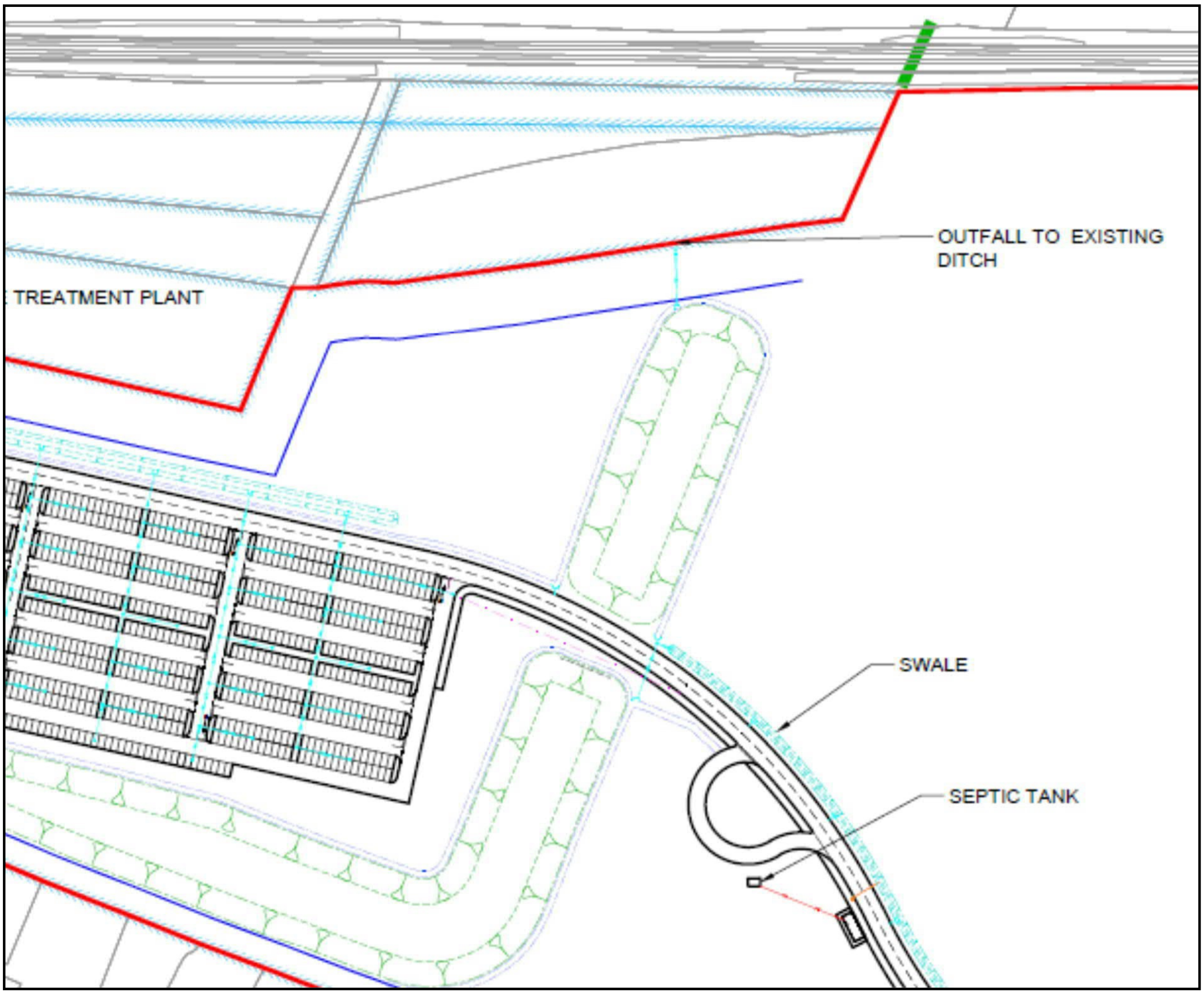
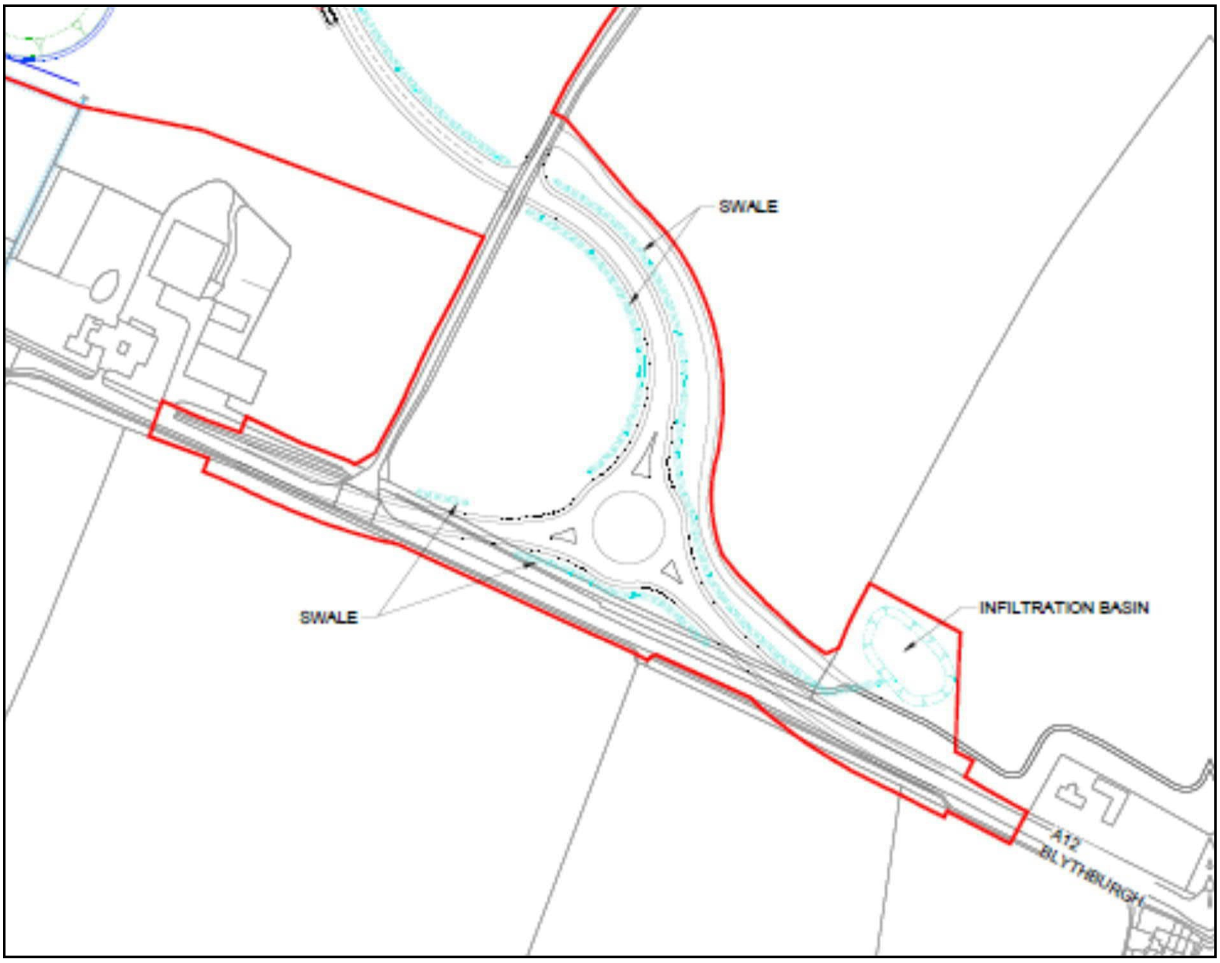


Plate 2: Northern park and ride internal layout showing concept drainage infrastructure to the north



3.1.10 The external site layout showing the road modifications with swales and infiltration basin is shown in **Plate 3**.

Plate 3: Northern park and ride external roads layout showing concept drainage infrastructure

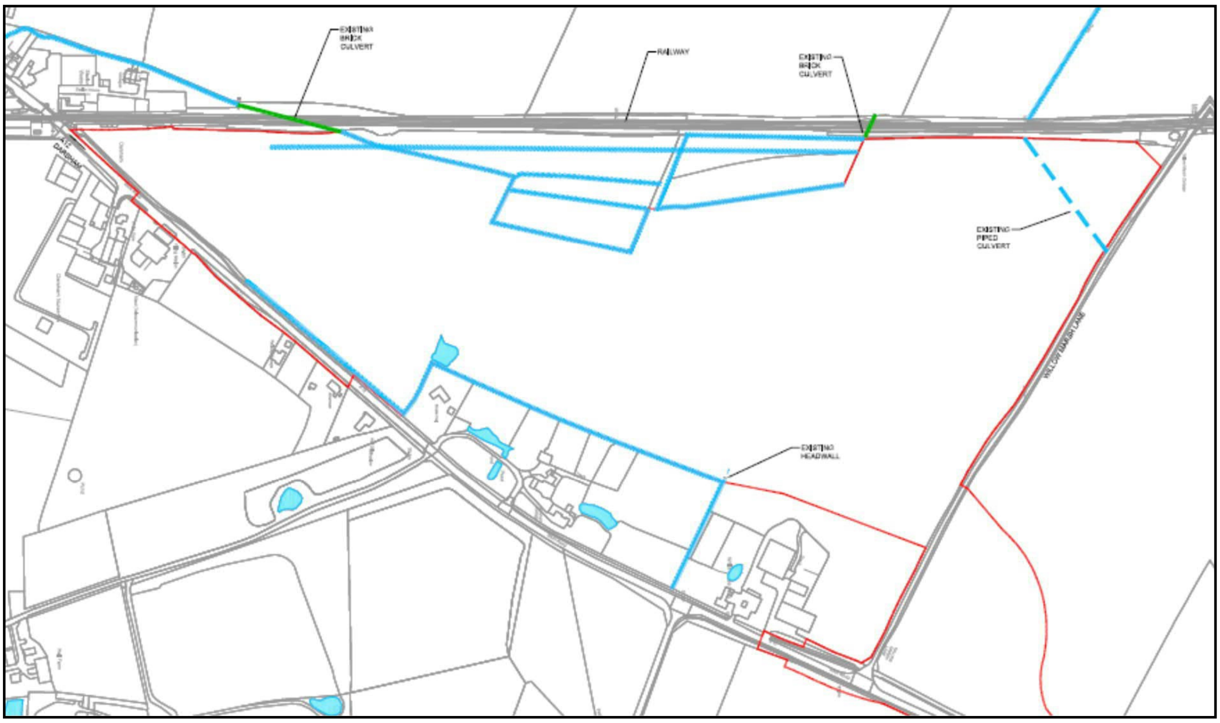


4 EXISTING SITE AND ADJACENT HIGHWAY DRAINAGE ARRANGEMENTS

4.1.1 Subsequent to development of the initial concept drainage strategy some site investigation had been undertaken both within and adjacent to the red line boundary. Elements of existing drainage infrastructure were identified but their function and condition are not fully understood.

4.1.2 Locations of drainage infrastructure are shown in **Plate 4** and are described below.

Plate 4: Northern park and ride existing drainage infrastructure



4.1.3 The extent of highway inspected is the A12 from the southern boundary of the site to Willow Marsh Lane and along Willow Marsh Lane alongside the northern site boundary. The A12 highway that continues to the north past the junction with Willow Marsh Lane and the location of the future roundabout was excluded from investigation.

4.1.4 It has been established that the northbound carriageway of the A12 has formal highway drainage with gulley outlets. These appear to discharge into a ditch located within the red line boundary and behind the highway boundary hedge. This ditch runs north and deviates west to run along the rear boundary of the properties Moat Hall, Darsham Cottage and White House Farm which front the road.

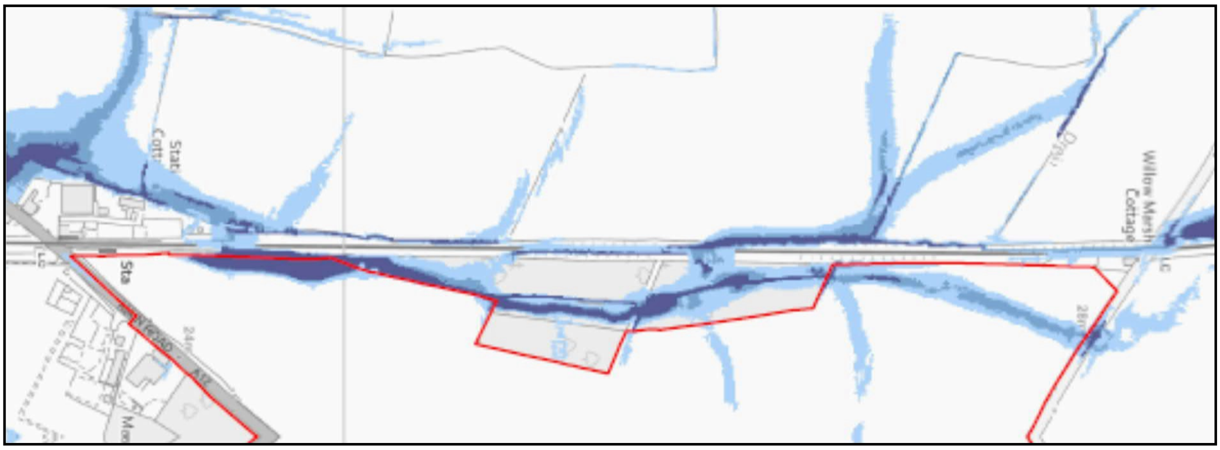
4.1.5 The ditch terminates in a small pond at the rear of White House Farm. The pond drains to an outfall pipe which appears to run in a westly direction and is assumed to cross the site to discharge into one of the ditches in the Little Nursery wood area.

4.1.6 Local ditches exist on either side of Willow Marsh Lane and run to the west before discharging into a culvert which cuts across the corner within the site before appearing to discharge into a watercourse at the railway boundary.

4.1.7 There are a series of ditches and watercourse that run mostly between the red line boundary and the railway and these run south towards Darsham station before passing under the railway to the west in a culvert.

4.1.8 As shown in **Plate 5**, the Environment Agency Surface Water Flood Map predicts that there is a medium to high risk of flooding of the site from these ditches and watercourses, within the site adjacent to the western boundary.

Plate 5: northern park and ride Western Boundary Surface Water Flood Risk Locations



4.1.9 No detailed site inspection of the A12 to the north of Willow Marsh Lane was initially undertaken. However, based on remote inspection of the A12 using Google Streetview there was no clear sign of obvious highway drainage infrastructure.

4.1.10 Following the issue of report revision 1, a detailed site inspection was undertaken on 4 August 2021 and the existing drainage arrangements have been confirmed.

4.1.11 The length of both northbound and southbound carriageway between the junction with Willow Marsh Lane and the tie in point for the roundabout diversion is drained by highway gullies. These discharge into ditches located behind the highway boundary hedge.

4.1.12 The ditch to the west of the A12 which collects highway runoff from the northbound carriageway discharges to the north outfalling into a large rectangular culvert which passes under the road and discharges into a watercourse.

4.1.13 It was also noted that a field ditch is located next to a boundary hedge which is at 90^0 to the road. This ditch is culverted beneath the farmers access track and discharges into the roadside ditch. These ditches are clearly shown in the **Appendix F** topographic survey.

4.1.14 The ditch to the east of the A12 which collects highway runoff from the southbound carriageway also discharges north and into the same watercourse as the culvert. This ditch is outside the extent of the topographic survey.

4.1.15 The Environment Agency Surface Water Flood Map shows predicted flooding of the land to the west of the A12 and across the A12. The extent is shown in **Plate 6**.

Plate 6: A12 predicted surface water flood risk locations at roundabout northern tie in

4.1.16 The site visit and topographic survey confirm that the land to the west of the A12 is at a lower level such that the A12 forms a barrier. The surface water flood map predicts that overland flow from fields to the west builds up to road level and overflows across the road and then follows the field boundary on the east of the A12 before discharging into a watercourse located within

150 m of the A12. However, the flood map, based on topography, does not allow for the culvert and hence flooding of the road is unlikely to occur.

5 REVISED DRAINAGE DESIGN STRATEGY INPUT DATA

5.1.1 The concept design which was included in the original DCO drainage design has been developed based on the DCO drainage design strategy but modified to take account of data which has become available since the Application.

5.1.2 The new data which informs the design development is listed below:

- Ground Investigation and infiltration testing undertaken in May 2020
- Ground Investigation and infiltration testing undertaken in July 2021
- Site visit and inspection of northern park and ride extent

6 GROUND INVESTIGATION AND INFILTRATION TESTING RESULTS

6.1.1 In order to validate the Drainage Strategy of infiltration to ground three trial pits were excavated within the site at locations shown in **Plate 7**.

Plate 7: Northern park and ride site infiltration test trial holes locations

6.1.2 The nature of the strata was confirmed to be Lowestoft Formation which is a stiff but slightly gravelly clay. A single BRE365 (Ref. 1) infiltration test was carried out at each location. Since there was no discernible drop in water in the trial pit over 24 hours, second and third tests were not undertaken.

6.1.3 Subsequent to the first revision of this report, further infiltration testing to BRE 365 has been undertaken at 2 locations. Trial pit DTP219A delivered a similar result with no discernible drop in water over 3 hours. Trial pit

DTP220 provided the following positive infiltration rate results, test 1, 7.74 x 10^{-5}m/s, test 2, 6.66 x 10^{-5} m/s, test 3 5.47 x 10^{-5}m/s.

6.1.4 Insitu permeability testing results were obtained at DBH101 and 102 and are stated to be 2.10 x 10^{-5} m/s and 2.37 x 10^{-6}m/s. This form of testing is not accepted by SCC and has to be treated with caution.

6.1.5 Full details of infiltration testing are provided in **Appendix A**.

6.1.6 Although there is a single BRE 365 positive result and the insitu permeability results, the overall results clearly demonstrate that infiltration is not viable and therefore surface water runoff from the development site must be disposed to the available watercourse to the west of the site, within the red line boundary.

6.1.7 Highway runoff from the A12 roundabout area must be disposed to watercourse via the existing culvert.

7 UPDATED SURFACE WATER CONCEPT DRAINAGE DESIGN

7.1.1 The surface water arrangements for removal remain as broadly as described in **Volume 3 Northern Park and Ride Chapter 2 Description of the Northern Park and Ride** [APP-350] but are modified to take account of the infiltration test results obtained in May 2020, July 2021 and the site inspection. Surface water runoff will be attenuated and discharge to watercourse at a controlled rate, with no allowance for infiltration. Runoff from roofs would be drained via downpipes and gullies, as appropriate to underground carrier drains and discharge into attenuation basins and swales.

7.1.2 Runoff from the internal roads and the bus/HGV standing areas with impermeable surface would be drained via surface outlets, gullies, linear channels and drains etc. These would discharge into underground carrier drains which would convey the runoff to the same attenuation basins and swales.

7.1.3 Bypass separators would be installed downstream of the bus/HGV standing areas in order to remove hydrocarbon and silt contaminants which would improve the water quality of discharge to the attenuation basins and swales.

7.1.4 The extensive car parking areas would have a permeable surface allowing runoff to permeate into and be temporarily stored in the sub-base. This would assist with attenuating peak flow rate, provide some storage and initial treatment of the runoff. The sub-base would allow flow to drain into the carrier drains.

7.1.5 The underground carrier drains would discharge all surface water into a series of cascading attenuation basins and swales which would provide suitable final treatment in accordance with CIRIA C753 The SuDS Manual (Ref. 2). They would also provide attenuation storage for all runoff required in order that discharge to watercourse from the site is limited to the equivalent greenfield runoff.

7.1.6 Initial calculations for the required total attenuation storage volume are shown in **Table 1**. These assumed a controlled discharge rate to the watercourse at a 1 in 100 year return period greenfield runoff rate.

Table 1: Northern park and ride flow control rates and storage volumes

Parameters	Values
Estimated Qbar rate	39.75 l/sec
Proposed Discharge Rate; Greenfield 1 in 100 +40 %cc	141.5 l/sec
Proposed Attenuation Storage Volume 1 in 100 +40 %cc	4,253 m^3

7.1.7 Upon review it is noted that a discharge rate based on 1 in 100 year return period greenfield runoff rate would not be compliant with SCC policy which is based on permitting a discharge rate from new development to watercourse set at Qbar or 2 l/s/Ha.

7.1.8 Hydraulic modelling calculations have been undertaken to determine a required attenuation storage volume if the discharge rate is limited to a Qbar rate of 39.75 l/s. The calculations have been updated and revised taking into account SCC requirement to use FEH rainfall data and allow for climate change. The full calculations including the requested greenfield runoff data are shown in **Appendix C**.

7.1.9 There are two basin calculations, one being for the small basin located in the car parking area and the other for the larger basin to the west. The larger basin calculation indicates insufficient storage volume with resultant flooding but this shortfall is made up by the storage provided in the smaller basin.

7.1.10 The storage included in the model is 13,425 m^3 with a basin depth of 1.8 m. The model predicted volume is 13,435 m^3 with a maximum depth of water of 1.74 m. This is substantially more than the original assumed concept design volume due to the restriction to Qbar. The storage provided beneath the permeable paving is not accounted for in the volume.

7.1.11 It is acknowledged that the maximum depth of water at 1.74 m exceeds that permitted in SCC guidance. However, the attenuation basins are located in a secure private site to which the public have no access and the basins are removed when the Park and Ride facility closes.

7.1.12 As shown in a copy of the site layout plan in **Appendix B** the plan area of required for the attenuation basin of approximately 9.330 m2 represents a small proportion of the site and would be accommodated within the Order Limits, enabling the appropriate discharge rate to be met. Even if SCC decline to agree to a basin with maximum depth of water exceeding 1.0 m there is sufficient space for a larger basin.

7.1.13 The layout drawing shown in **Appendix B** continues to show an infiltration basin within the developed area and swales between the developed area and the watercourse to the west. The infiltration basin would become an attenuation basin. It is intended that the additional required storage would include these features but more swales and basins would be required. The plan areas shown are for illustrative purposes only and do not represent the fixed or final position of the attenuation storage positions.

7.1.14 The concept design assumed a free outfall to the watercourse within the western area of the site and no increased flood risk from the watercourse, but this would require to be confirmed.

7.1.15 **Plate 5** shows the Environment Agency surface water flood map and indicates the area adjacent to the watercourse to be at risk of flooding due to a 1 in 30 year return period event. As a result, it cannot be assumed that there would be a free outfall. The site topography survey shown in **Appendix F** shows a steep fall of level towards the watercourse but does not include watercourse levels. The depth of the watercourse is not determined.

7.1.16 The proposed position of the additional attenuation facilities located next to the main access road will ensure that there is no risk of flooding from the watercourses and the flow control devices will have a free outfall.

7.1.17 Whilst designed as attenuation basins, there is no necessity for their sides to be lined thus if infiltration is possible, it will occur but is not allowed for in storage volume calculations.

8 UPDATED SURFACE WATER POLLUTION MITIGATION STRATEGY

8.1.1 In addition to the provision of drainage infrastructure for the removal of surface water runoff and avoidance of unacceptable flood risk, it is also necessary to ensure that the runoff is disposed in a way that avoids pollution of the receiving water, whether watercourse or aquifer/groundwater.

8.1.2 An assessment of the ability of the proposed drainage infrastructure to mitigate pollution risk to an acceptable level has been undertaken using the CIRIA C753 SuDS Manual Simplified Index Approach methodology. A sample calculation has been shared with SCC who have confirmed acceptance of this approach.

8.1.3 Details of the calculations and results are shown in **Appendix E.** They demonstrate that there is sufficient treatment provided to mitigate pollution to an acceptable level. These results were shared with SCC.

8.1.4 Following review, SCC have confirmed that whilst in respect of the A12 roundabout MSIA calculations have been provided they would require a separate HEWRAT assessment but are satisfied that this can be undertaken as part of detailed design.

9 UPDATED FOUL WATER CONCEPT DRAINAGE DESIGN STRATEGY – PARK AND RIDE

9.1.1 The foul water drainage strategy remains unchanged with foul water flows collected by an underground gravity pipe drainage network and discharged into a package sewage treatment plant. However, whilst previously the treated effluent would discharge to ground via infiltration through a drainfield network, the infiltration test results demonstrate that this is not feasible. Therefore, the treated effluent would need to discharge to the watercourse via the surface drainage network.

9.1.2 The implications of a change to discharge the sewage treatment plant flows to the watercourse is that the package treatment plant may be required by the EA to deliver an enhanced treatment to achieve higher quality of treated effluent. Alternatively, instead of or in addition to an enhanced treatment within the sewage treatment plant, an additional treatment train infrastructure could be considered during preliminary design, for example reed beds could be installed downstream.

9.1.3 Given that that foul water flow rates generated would be low and intermittent with a range of flow it may make the delivery of a consistent treated effluent to meet the requirements of the required environmental permit more challenging. If a suitable package plant and associated treatment infrastructure cannot be developed during preliminary design or consent to a discharge of treated effluent to watercourse cannot be agreed, the alternative would be to collect the foul water sewage in an underground sealed cess tank from which it can be collected and regularly removed by tanker for treatment offsite.

9.1.4 The remote security cabin arrangement of discharge into a septic tank would remain. Solids would be collected in the tank and removed by tanker for treatment offsite. Liquid effluent would discharge to ground via a drainfield network. The drainfield typically consists of an arrangement of trenches containing perforated pipes and porous material (often gravel) covered by a layer of soil to prevent animals (and surface runoff) from reaching the wastewater distributed within those trenches.

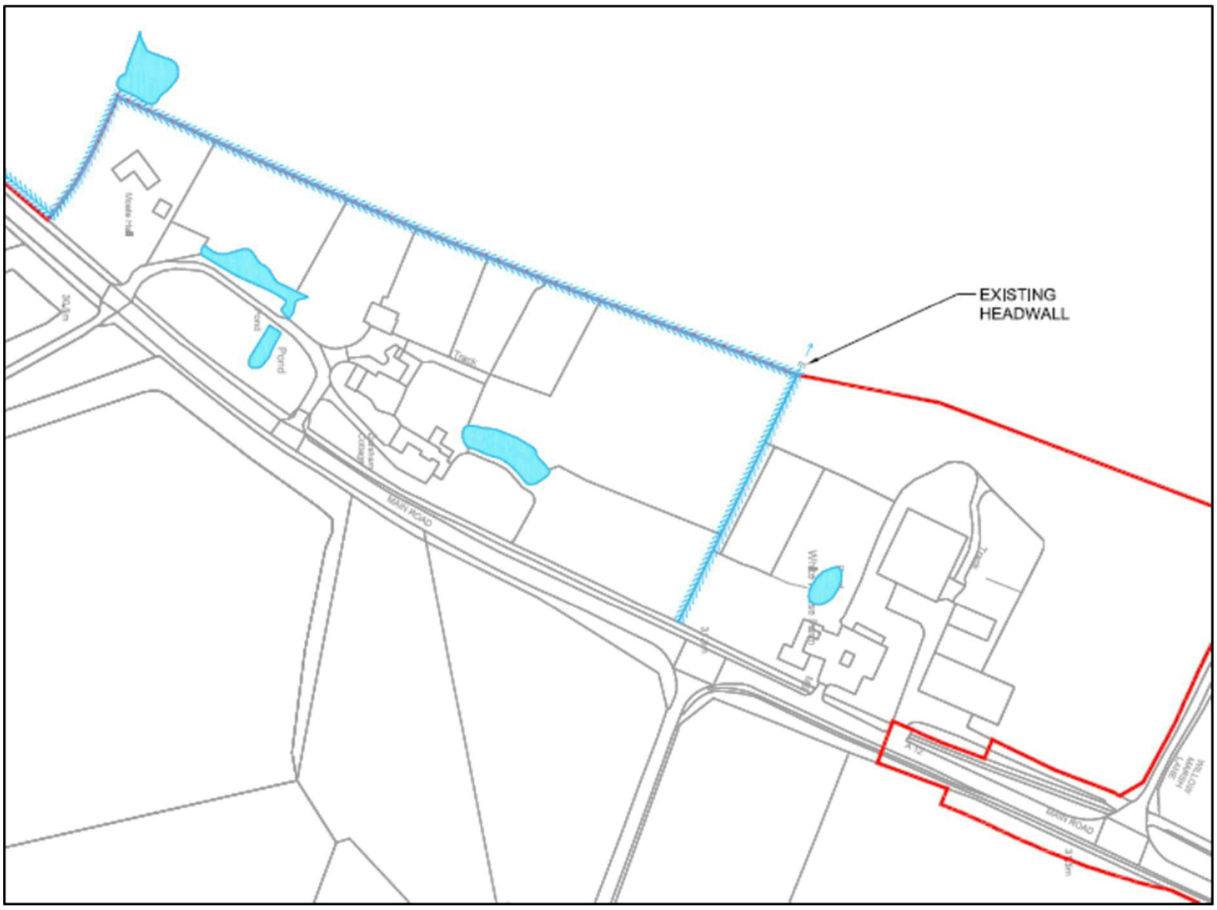
9.1.5 During design development should it be determined that the infiltration rate is insufficient for the provision of a drainfield and therefore creating a flood risk, it would be necessary to collect wastewater and sewage in a cesspit from which it can be collected and regularly be removed by tanker for treatment offsite or at the site treatment plant, if that option is pursued.

10 PROTECTION OF EXISTING DRAINAGE

10.1.1 As noted in Section 4 there is an existing ditch network within the site boundary and this provides an outfall for runoff from the A12 highway and also it is believed an outfall for the properties to the west of the A12. The site layout would be modified to ensure that this arrangement remains in place and removal of runoff is not impeded. The 3 m high bund which is provided to minimise impact on the local properties would be moved into the site by such distance as is required in order to provide access to and maintain the existing ditch along the eastern site boundary.

10.1.2 The existing pond outfall ditch runs along behind the properties and terminates at an existing headwall as shown in Plate 8.

Plate 8: Northern park and ride existing drainage and outfall headwall



10.1.3 The headwall outfall drain appears to run west and across the site where it is assumed there is discharge to the watercourse. This outfall drain is within the part of the site which is undeveloped and should remain as grassland. As a result, the drain should be able to remain in place and used as at present. However, it would be crossed by the site access road so its location, depth and structural condition will need to be confirmed and, if necessary, the outfall drain would be replaced.

10.1.4 The existing ditches which run alongside Willow Marsh Lane would be retained and discharge to an existing retained culvert that passes through the northwestern part of the site. The existing ditches would be culverted where they cross the northern park and ride access road.

10.1.5 The existing section of A12 to the north of the Willow Marsh Lane junction will be left in place between the tie in points for the roundabout. The gullies whjch drain the road will be left in place and drain the carriageway.

10.1.6 The existing ditch on the west side of the road will be culverted beneath the roundabout tie in roads. The length of culvert will be kept to a minimum required for highway safety and at least of 450 mm diameter.

11 UPDATED SURFACE WATER CONCEPT DRAINAGE DESIGN STRATEGY – A12 ROUNDABOUT AND MAIN SITE ACCESS ROAD

11.1.1 The surface water drainage strategy for the highway drainage subject to adoption by SCC remains unchanged being infiltration to ground to the extent that this is achievable. Within the proposed A12 roundabout highway, runoff would be collected by surface water outlets, gullies and CKDs into carrier drains which would discharge to swales located adjacent to the 3 arms of the roundabout. The three arms of the roundabout would drain “over the edge” to swales. The swales would have an underlying filter drain which may partially infiltrate to ground before discharging to the proposed infiltration basin adjacent the roundabout. Dependent on topography and the bed level, the site access road arm may in part discharge to the existing adjacent ditches along Willow Marsh Lane.

11.1.2 The swales would have a continuous fall to the infiltration basin. The required size of the basin would be determined at preliminary design stage by hydraulic modelling using infiltration results of future testing at this location.

11.1.3 As part of the ground investigation undertaken in 2021 a trial pit DTP218 was excavated within the footprint of the proposed infiltration basin. This confirmed that the strata consists of impermeable Lowestoft till clays, consistent with the main site. As a result, no infiltration testing has been undertaken.

11.1.4 On the basis that infiltration is not viable, the infiltration basin will change to an attenuation basin with a positive outfall to the culvert which passes under the A12 and along the field boundary to the existing watercourse located within 150 m.

11.1.5 Hydraulic calculations have been undertaken to establish the required attenuation basin storage volume and are shown in **Appendix D**. These include the greenfield runoff calculations and take account of the topographic survey which is now available.

11.1.6 In summary, based on Qbar calculated as being 4.6 l/s and assuming a basin with 1 in 3 side slopes and a maximum permitted storage depth of 1.0 m during a 1 in 100 year return period rainfall event plus 30% climate change, the storage volume required would be 1,579 m3. This will require a surface footprint of approximately 2,260 m2 which is greater than the land available at the location allocated.

11.1.7 In order to resolve this shortfall, it is proposed that the basin be designed with flow released to the watercourse at 16.2 l/s which represents the 1 in 100 year greenfield runoff which maintains the current situation. Whilst SCC normal requirement is that discharge is limited to Qbar or 2/l/s/Ha, its SuDS Guidance Appendix A does permit discharge at the 1 in 100 year rate subject to detail and long term provision of storage.

11.1.8 If this is not acceptable additional storage volume could be provided upstream of the basin.

11.1.9 The required footprint for the basin is shown in **Appendix B.** The additional available areas are also identified.

11.1.10 SCC have provided comments in which concern is expressed at the projected shortfall in storage and risk of flooding across the A12 which would be unacceptable. However, by reference to the topography data in Appendix F it can be seen that the A12 is approximately 0.8 m higher than the land in which the attenuation basin is located. In addition, as noted in section 4 .1.12 the area is drained via a rectangular culvert.

11.1.11 In Appendix B, SZC has identified an addition area in red where additional storage could be provided in proximity to the proposed attenuation basin. It would also be possible to provide attenuation facilities with controlled outflow rates further upstream in proximity to the A12 roundabout.

11.1.12 It is considered that at this stage there is sufficient evidence to demonstrate that sufficient attenuation storage volume can be achieved thus validating the updated drainage strategy. The design of attenuation storage can be developed at detailed design stage.

12 SUMMARY AND CONCLUSION

12.1.1 The purpose of this technical note is to validate the **Outline Drainage Strategy** [REP2-033] and subsequent **Drainage Strategy** (submitted at Deadline 7) for the northern park and ride. It describes how the concept design has needed to evolve as a result of provision of new information and design development.

12.1.2 The drainage design for both the internal northern park and ride facility and A12 roundabout modification and site access road have been developed to a level of detail to provide sufficient evidence of an achievable drainage strategy that is compliant with national planning and environmental regulatory requirements.

12.1.3 Subject to the acceptance of the drainage design strategy principles contained in this updated report, which are intended to address SCC review comments, the drainage designs will be developed to preliminary design stage.

12.1.4 The northern park and ride facility drainage design will be based on CIRIA C753, SuDS Manual, Design and Construction Guidance for Foul and Surface Water Sewers (formerly Sewers for Adoption) (Ref. 3), and PPG4 Treatment and Disposal of Sewage where no Foul Water Sewer is Available (Ref. 4).

12.1.5 The adoptable highway drainage design would be based on Design Manual for Roads and Bridges (DMRB) (Ref. 5), Manual of Contract Documents for Highway Works (MCHW) (Ref. 6) and SCC specific guidance (Refs. 7 and 8).

12.1.6 As preliminary design progresses SZC Co. will liaise with SCC and the EA through design review meetings to achieve acceptance of the drainage infrastructure and to enable compliance with regulatory requirements and environmental permits.

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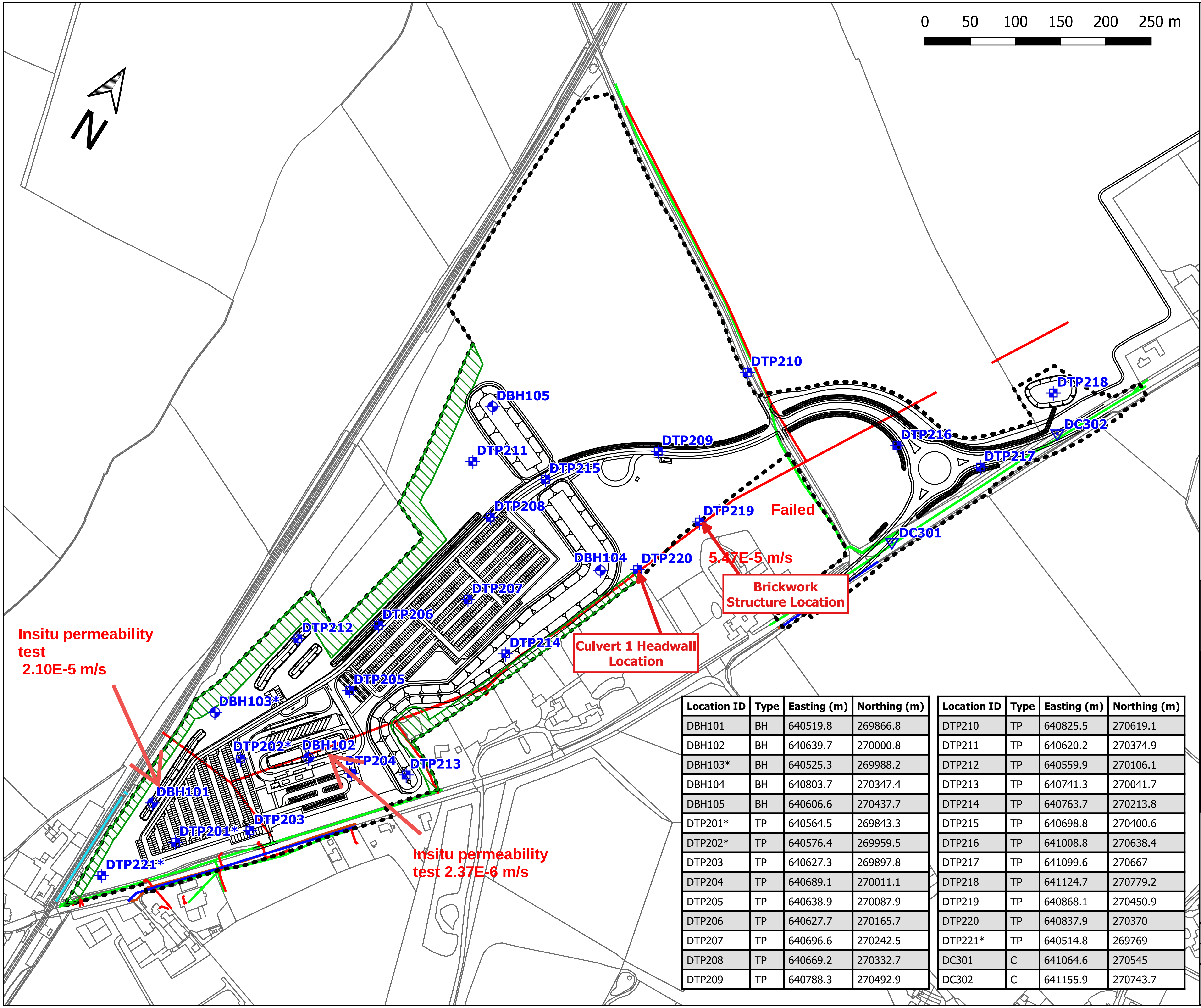
[redacted]

7. Design Guide, Suffolk County Council, 2000, https://www.suffolk.gov.uk/planning-waste-and-environment/planning-and-development-advice/suffolk-design-guide-for-residential-areas/

8. Sustainable Drainage Systems (SuDS) a Local Design Guide Appendix A to the Suffolk Flood Risk Management Strategy, Suffolk County Council, May 2018 [redacted]

APPENDIX A: INFILTRATION TEST DATA AND RESULTS



Location ID	Type	Easting (m)	Northing (m)
DBH101	BH	640519.8	269866.8
DBH102	BH	640639.7	270000.8
DBH103*	BH	640525.3	269988.2
DBH104	BH	640803.7	270347.4
DBH105	BH	640606.6	270437.7
DTP201*	TP	640564.5	269843.3
DTP202*	TP	640576.4	269959.5
DTP203	TP	640627.3	269897.8
DTP204	TP	640689.1	270011.1
DTP205	TP	640638.9	270087.9
DTP206	TP	640627.7	270165.7
DTP207	TP	640696.6	270242.5
DTP208	TP	640669.2	270332.7
DTP209	TP	640788.3	270492.9

Location ID	Type	Easting (m)	Northing (m)
DTP210	TP	640825.5	270619.1
DTP211	TP	640620.2	270374.9
DTP212	TP	640559.9	270106.1
DTP213	TP	640741.3	270041.7
DTP214	TP	640763.7	270213.8
DTP215	TP	640698.8	270400.6
DTP216	TP	641008.8	270638.4
DTP217	TP	641099.6	270667
DTP218	TP	641124.7	270779.2
DTP219	TP	640868.1	270450.9
DTP220	TP	640837.9	270370
DTP221*	TP	640514.8	269769
DC301	C	641064.6	270545
DC302	C	641155.9	270743.7

NOTES

1. All Dimensions in metres unless otherwise noted.

2. All coordinates are in metres relative to ordnance survey national grid (OS GB 36).

3. Locations marked with an astrix may be cancelled or amended during investigation depending on results from nearby exploratory holes.

4. Utilities provided by Atkins on drawing no. 96057, dated 10/06/2021. Drawing notes: "Information on buried services is provided for information only and is based upon records available at the time of issue. Accuracy of information cannot be guaranteed and must be verified prior to undertaking any works."

5. No works may be undertaken in the ecological buffer zones indicated on this drawing.

6. DBH102, DTP213 and DTP214 to be located minimum safe distance from overhead powerlines in accordance with requirements of the utility provider UK Power Networks. DTP219 and DTP220, may be required to be hand dug depending on position of overhead line in relation to the exploratory hole.

LEGEND

- ⋯ Site Boundary
- ▨ Ecological Buffer Zone

Proposed GI Locations

- ⊕ Proposed Borehole
- ⊞ Proposed Trial Pit
- ▽ Proposed Pavement Core

Utilities (Atkins, 10/06/2021)

- — Anglian Water Sewerage
- — BT Openreach Telecoms
- — Essex and Suffolk Water
- — Network Rail
- — UK Power Networks

FOR CONSTRUCTION

PROJECT

Sizewell C
Associated Developments

edf

TITLE

Northern Park and Ride Site
Ground Investigation Layout

Royal HaskoningDHV
Enhancing Society Together

DRAWN MGD	CHECKED CH	APPROVED PJ
DATE 24/06/2021	SCALE AT A3 1:4000	REF -

DRAWING No.	Revision
PC1834-RHD-GI-ZZ-DR-G-0001	C1

GEOSPHERE ENVIRONMENTAL

Our Ref: 4029,SK,Ltr03,JD/18.05.20/V1
Your Ref: 4029,SK

Royal Haskoning DHV
9TH Floor Manchester One
Portland Street
Manchester
M1 3LF

Date: 18 May 2020

For the attention of Mr Philip John

By Email: [redacted]

Dear Mr John,

INFILTRATION TESTING AT LAND AT DARSHAM, IP17 3PL

1. Introduction

This letter report has been prepared on behalf of Royal Haskoning DHV.

The primary objective of this ground investigation was to assess the infiltration potential of the natural soils beneath the site.

This was achieved by:

- Excavating three machine-dug trial pits across the site at pre-determined areas;
- Undertaking soakage testing in line with BRE Digest 365 guidance; and
- Undertaking infiltration calculations to allow for an assessment of the suitability of soakaways or infiltration techniques for the future development of the site.

It was understood that the proposed development will comprise an area of hardstanding to provide a temporary 'park and ride' service to facility the construction element of the 'Sizewell C' project.

A Site Location Plan, Drawing ref.4029,SK/004/Rev0, is presented at the end of this letter report in Appendix 4.

The purpose of this letter report is to provide factual data only.

2. Site Works

2.1 Methodology

This ground investigation was carried out on the basis of the practices set out in BRE Digest 365, 'Soakaway Design'. 2016, which requires, in summary, a total of three infiltration tests to be undertaken in succession over a 24-hour period or tests to be undertaken on consecutive days.

The exploratory holes were positioned through liaison with the client and other EDF stakeholders to provide a representative, site wide spread, whilst mitigating against ecological and archaeological interests within the area.

2.2 Scope

Site works were carried out between the 6 and 7 May 2020, and comprised the following:

- Excavation of three machine excavated trial pits, (SK01 to SK03), to depths of 1.85mbgl;
- Undertaking infiltration testing in line with BRE Digest 365 guidance; and
- Undertaking infiltration calculations to allow for an assessment of the suitability of soakaways for the future development of the site.

An Exploratory Hole Location Plan, Drawing ref.4029,SK/005/Rev0, is presented in Appendix 4.

2.3 Ground Conditions Encountered

The sequence of the strata encountered during the investigation generally adheres to the geology, as cited by the British Geological Survey viewer data [redacted]l; the site is underlain by Lowestoft Formation Diamicton. (A narrow band of Head Deposits is denoted along the western site boundary, interpreted to be associated with the natural ditch-line along that boundary.)

The sequence and indicative thickness of strata are summarised in Table 1 below, with the Exploratory Hole Logs provided in Appendix 2:

Table 1 - Ground Conditions

Strata	Depth Encountered (mbgl)		Strata Thickness (m)	Location and Composition
	From	To		
Topsoil.	0.00	0.10	0.10	All exploratory holes: Organic clay / topsoil: brown slightly sandy and gravelly clay; gravel is flint/chert and chalk.
Lowestoft Formation (Diamicton).	0.10	1.85	Proved to 1.85 only.	All exploratory holes: Stiff to very stiff multicoloured* gravelly and slightly cobbly CLAY (*varying from predominantly light brown and brown to grey).

2.4 Groundwater

No groundwater was encountered in any of the exploratory holes during the intrusive investigation.

2.5 Infiltration Testing Results

Soil infiltration testing was undertaken in accordance with BRE 365, 2016. The results are summarised in Table 2 overleaf and are provided in full in Appendix 3, presented at the end of this letter report:

Summary of Soil Infiltration Results				
Location	**Test 1 (m/s)**	**Test 2 (m/s)**	**Test 3 (m/s)**	**Notes**
SK01	N/A	-	-	Negligible infiltration achieved in 24 hours.
SK02	N/A	-	-	Negligible infiltration achieved in 24 hours.
SK03	N/A	-	-	Negligible infiltration achieved in 24 hours.

All three test locations showed negligible infiltration within a 24 hour period and as such the second and third tests were unable to be completed on the same or consecutive days as per BRE365.

We trust the above is clear and acceptable. If you have any questions, please do not hesitate to contact us.

Yours sincerely,

[redacted]

Jim Dawson
Associate Director
Geosphere Environmental Ltd
jim@geosphere-environmental.co.uk

Enclosures:
Appendix 1 – Report Limitations and Conditions
Appendix 2 – Exploratory Hole Logs
Appendix 3 – Infiltration Testing Results
Appendix 4 – Drawings

APPENDICES

APPENDIX 1 – REPORT LIMITATIONS AND CONDITIONS

This report refers, within the limitations stated, to the condition of the site at the time of the inspections. No warranty is given as to the possibility of future changes in the condition of the site.

This report has been prepared for the sole use of the Client for the purposes described and no extended duty of care to any third party is implied or offered. Third parties using any information contained within this report do so at their own risk.

This report is prepared and written for the use stated herein; it should not be used for any other purposes without reference to Geosphere Environmental Limited. The report has been prepared in relation to the proposed end-use, should another end-use be intended, a further re-assessment may be required. It is likely that over time practises will improve and the relevant guidance and legislation be amended or superseded, which may necessitate a re-assessment of the site.

The accuracy of any map extracts cannot be guaranteed. It is possible that different conditions existed onsite, between and subsequent to the various map surveys appended.

Whilst the report may express an opinion on possible configurations of strata between or beyond exploratory holes discussed or on the possible presence of features based upon visual, verbal or published evidence, this is for guidance only and no liability can be accepted for its accuracy.

APPENDIX 2 – EXPLORATORY HOLE LOGS

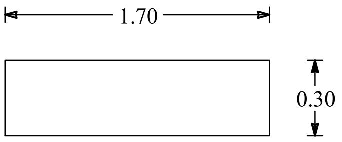
Trial Pit Logs
(SK01 to SK03)

Geosphere Environmental Ltd
Brightwell Barns, Ipswich Road
Brightwell, Suffolk, IP10 0BJ
Telephone: 01603 298076

TRIAL PIT LOG

Project Darsham, IP17		Client EDF c/o RHDHV		TRIAL PIT No **SK01**
Job No 4029,SK	Date 06-05-20 06-05-20	Ground Level (m)	Coordinates () 640522, 269928	
Fieldwork By GEL		Logged By JD		Sheet 1 of 1

Depth	DESCRIPTION	Legend	Depth	No	Remarks/Tests
0.00-0.10	Brown slightly gravelly slightly sandy fine to coarse ORGANIC CLAY. Sand is fine to coarse. Gravel is angular to rounded fine to coarse flint. TOPSOIL				
0.10-1.00	Stiff becoming very stiff multicoloured, light brown, mottled grey and orange brown, speckled white, gravelly CLAY. Gravel is angular to rounded fine to coarse flint and chalk 0.50 Becoming brown				
1.00-1.85	Stiff to very stiff multicoloured, brown, light brown, light grey, black, mottled white and speckled white gravelly CLAY with low cobble content of subangular flint, chalk and igneous rock (basalt?). Gravel is angular to rounded, fine to coarse flint and chalk.				
1.85	END OF EXPLORATORY HOLE AT 1.85m BGL. DRY AND OPEN ON COMPLETION				



Shoring/Support: Gravel backfilled
Stability: Stable

All dimensions in metres Scale 1:20.8333333333333	Method Trial Pit/trench	Plant Used MECHANICAL EXCAVATOR	Checked By GF

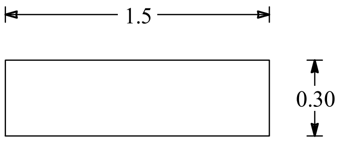
GEL AGS TP BETA 4029,SK DARSHAM.GPJ GINT STD AGS 3_1.GDT 12/5/20

Geosphere Environmental Ltd
Brightwell Barns, Ipswich Road
Brightwell, Suffolk, IP10 0BJ
Telephone: 01603 298076

TRIAL PIT LOG

Project Darsham, IP17		Client EDF c/o RHDHV		TRIAL PIT No **SK02**
Job No 4029,SK	Date 06-05-20 06-05-20	Ground Level (m)	Coordinates () 640556, 270005	
Fieldwork By GEL		Logged By JD		Sheet 1 of 1

Depth	DESCRIPTION	Legend	Depth	No	Remarks/Tests
0.00-0.10	Brown slightly gravelly slightly sandy fine to coarse ORGANIC CLAY. Sand is fine to coarse. Gravel is angular to rounded fine to coarse flint.				
0.10-0.30	TOPSOIL				
0.30-1.85	Stiff becoming very stiff multicoloured, light brown, mottled grey and orange brown, speckled white, gravelly CLAY. Gravel is angular to rounded fine to coarse flint and chalk. Stiff to very stiff grey locally mottled brown speckled white gravelly CLAY. Gravel is angular to rounded, fine to coarse flint and chalk.				
1.85	END OF EXPLORATORY HOLE AT 1.85m BGL. DRY AND OPEN ON COMPLETION				



Shoring/Support: Gravel backfilled
Stability: Stable

All dimensions in metres Scale 1:20.8333333333333	Method Trial Pit/trench	Plant Used MECHANICAL EXCAVATOR	Checked By GF

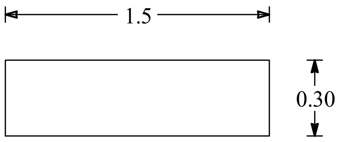
GEL AGS TP BETA 4029,SK DARSHAM.GPJ GINT STD AGS 3_1.GDT 12/5/20

Geosphere Environmental Ltd
Brightwell Barns, Ipswich Road
Brightwell, Suffolk, IP10 0BJ
Telephone: 01603 298076

TRIAL PIT LOG

Project Darsham, IP17	Client EDF c/o RHDHV		TRIAL PIT No **SK03**
Job No 4029,SK	Date 06-05-20 06-05-20	Ground Level (m)	Coordinates () 640793, 270437
Fieldwork By GEL	Logged By JD		Sheet 1 of 1

Depth	DESCRIPTION	Legend	Depth	No	Remarks/Tests
0.00-0.10 0.10-0.25	Brown slightly gravelly slightly sandy fine to coarse ORGANIC CLAY. Sand is fine to coarse. Gravel is angular to rounded fine to coarse flint. TOPSOIL				
0.25-0.40	Firm brown slightly gravelly sandy SILT. Sand is fine to coarse. Gravel is angular fine and medium flint.				
0.40-0.80	Firm orange brown slightly gravelly CLAY. Gravel is angular fine and medium flint.				
	Firm multicoloured, light orangish grey, light brown speckled and mottled, slightly grey and white and light brown slightly gravelly CLAY. Gravel is rounded fine and medium chalk and angular medium flint.				
0.80-1.85	Stiff light grey mottled brown, speckled white slightly gravelly CLAY with low cobble of rounded chalk, mudstone and igneous rock. Gravel is angular to rounded fine to coarse chalk and flint.				
1.85	END OF EXPLORATORY HOLE AT 1.85m BGL. DRY AND OPEN ON COMPLETION				



Shoring/Support: Gravel backfilled
Stability: Stable

All dimensions in metres Scale 1:20.8333333333333	Method Trial Pit/trench	Plant Used MECHANICAL EXCAVATOR	Checked By GF

APPENDIX 3 – INFILTRATION TEST RESULTS

(SK01 to SK03)

TRIAL PIT INFILTRATION TEST - BRE DIGEST 365

GEO GEOSPHERE ENVIRONMENTAL

Project Number: 4029,SK

Date: 18/05/2020

Project Name: Darsham, IP17

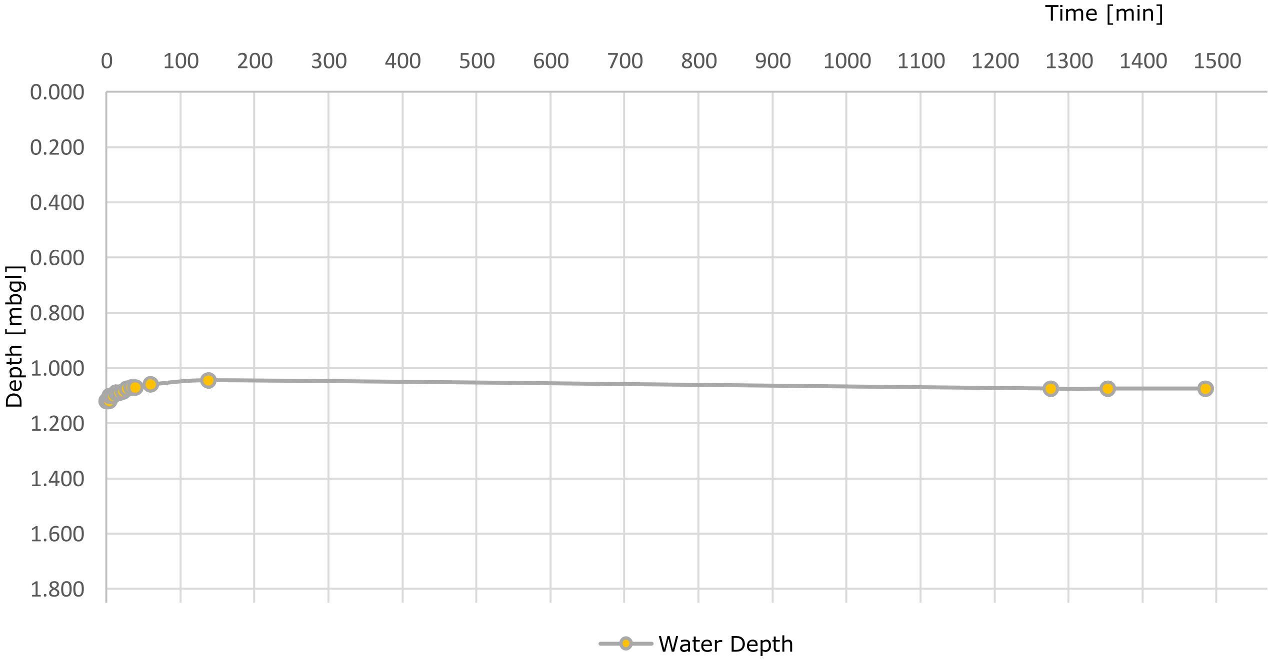
Time [min]	Depth to Water [mbgl]
0	1.120
1	1.120
3	1.120
3	1.120
4	1.120
5	1.100
9	1.100
13	1.090
18	1.090
23	1.085
28	1.075
34	1.070
39	1.070
60	1.060
138	1.045
1276	1.075
1353	1.075
1485	1.075

Pit Size [m]		
Length	Width	Depth
1.7	0.3	1.85

It was not possible to undertake full-depth soakaway test.

Trial Pit:	SK01
Run:	1 of 1
Test Date:	6-7 May 2020
Groundwater Encountered:	n/a

Soakage Rate



mbgl - (metres below ground level)

Calculated by: GF

Checked by: JD

TRIAL PIT INFILTRATION TEST - BRE DIGEST 365

GEOSPHERE ENVIRONMENTAL

Project Number: 4029,SK

Date: 18/05/2020

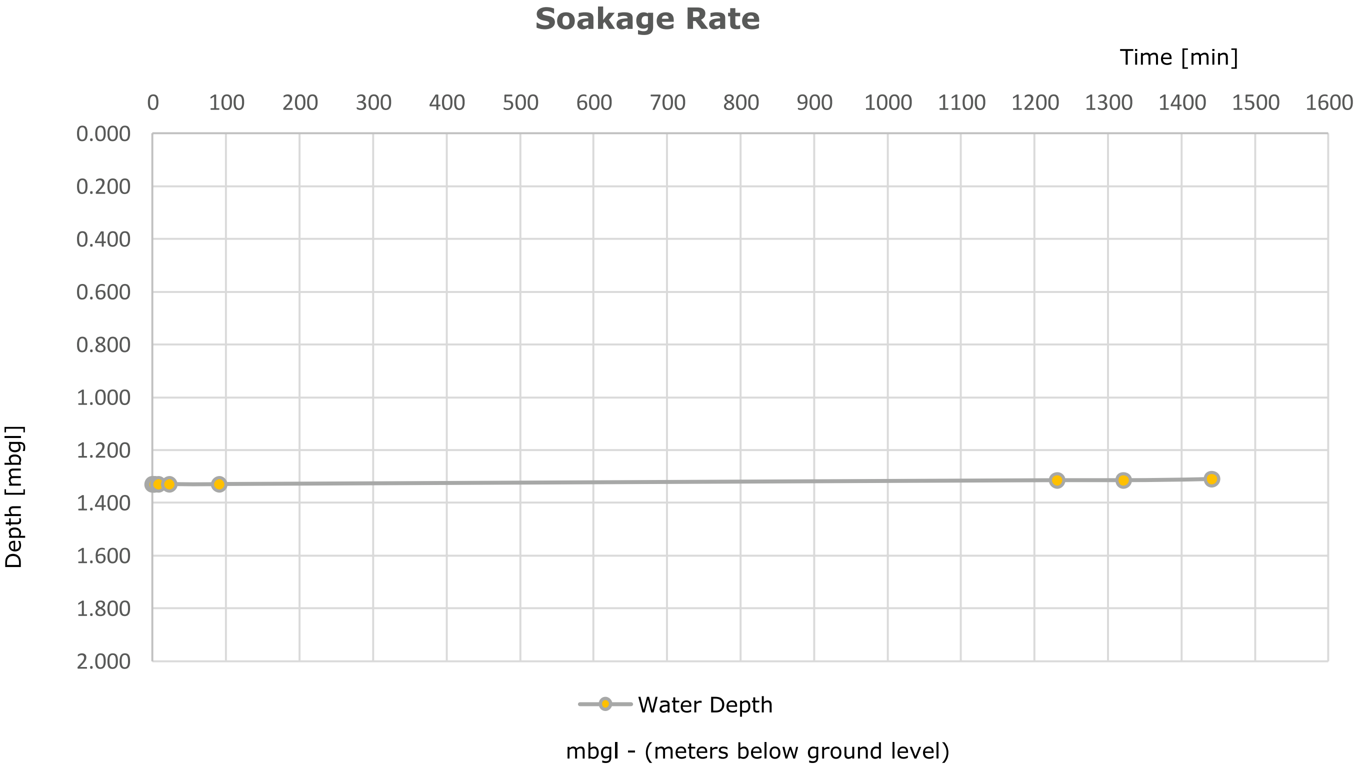
Project Name: Darsham, IP17

Time [min]	Depth to Water [mbgl]
0	1.330
1	1.330
2	1.330
3	1.330
8	1.330
23	1.330
91	1.330
1231	1.315
1321	1.315
1441	1.310

Pit Size [m]		
Length	**Width**	**Depth**
1.5	0.3	1.85

It was not possible to undertake full-depth soakaway test.

Trial Pit:	SK02
Run:	1 of 1
Test Date:	6-7 May 2020
Groundwater Encountered:	n/a



Calculated by: GF Checked by: JD

TRIAL PIT INFILTRATION TEST - BRE DIGEST 365

Project Number: 4029,SK

Date: 18/05/2020

Project Name: Darsham, IP17

Time [min]	Depth to Water [mbgl]
0	1.325
1	1.325
2	1.325
4	1.325
9	1.325
14	1.325
1177	1.330
1295	1.330
1442	1.330

Pit Size [m]		
Length	Width	Depth
1.5	0.3	1.85

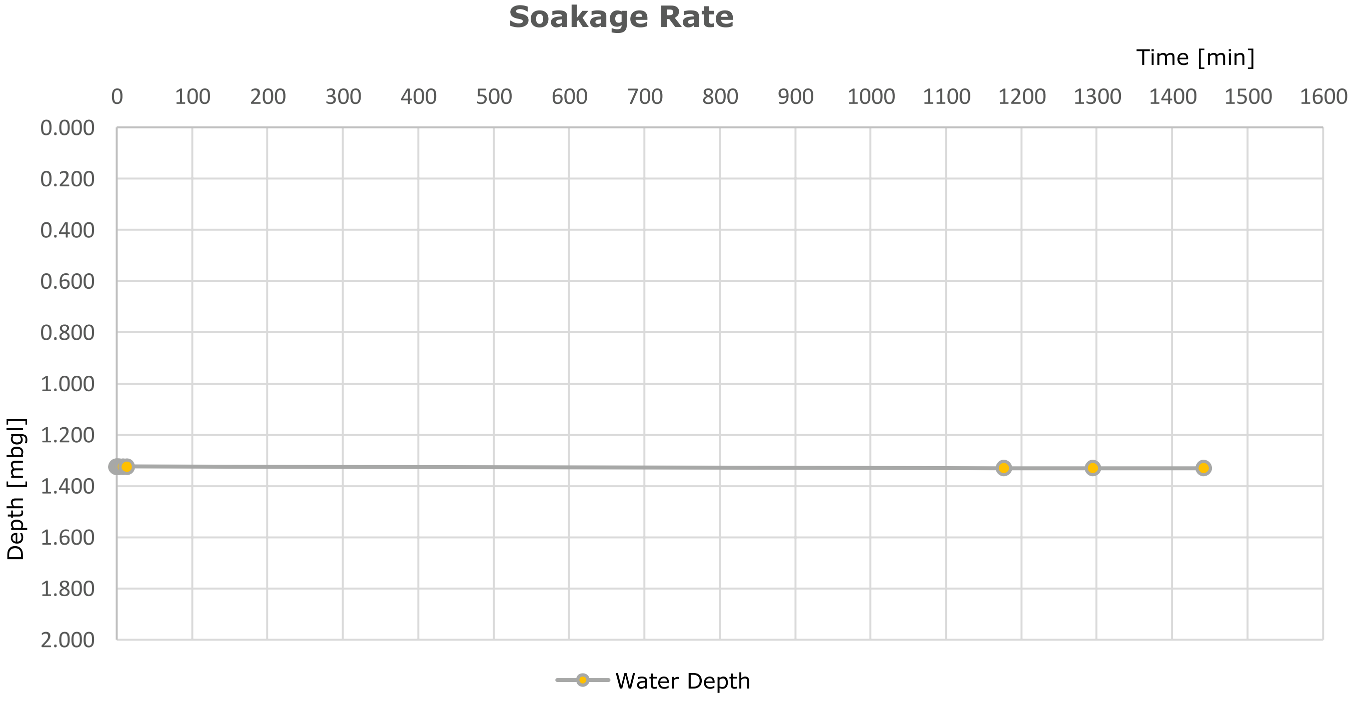
It was not possible to undertake full-depth soakaway test.

Trial Pit: SK03

Run: 1 of 1

Test Date: 6-7 May 2020

Groundwater Encountered: n/a



mbgl - (meters below ground level)

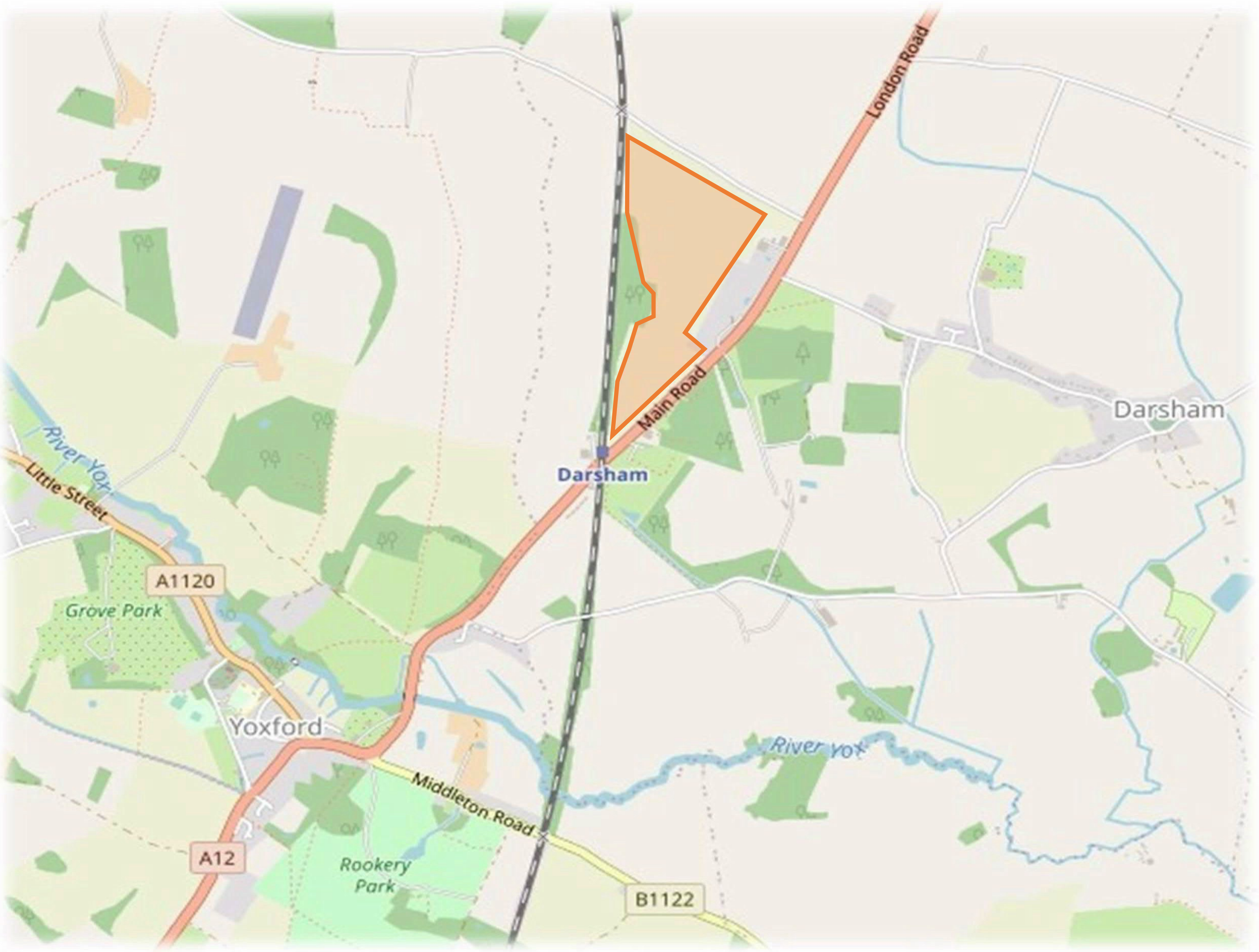
Calculated by: GF

Checked by: JD

APPENDIX 4 – DRAWINGS

Site Location Plan - Drawing ref. 4029,SK/004/Rev0

Exploratory Hole Location Plan – Drawing ref. 4029,SK/005/Rev0



GEOSPHERE ENVIRONMENTAL

LEGEND

🟥	Approximate site boundary

SOURCE

© OpenStreetMap contributors

PROJECT

Land at Darsham, IP17 3PL

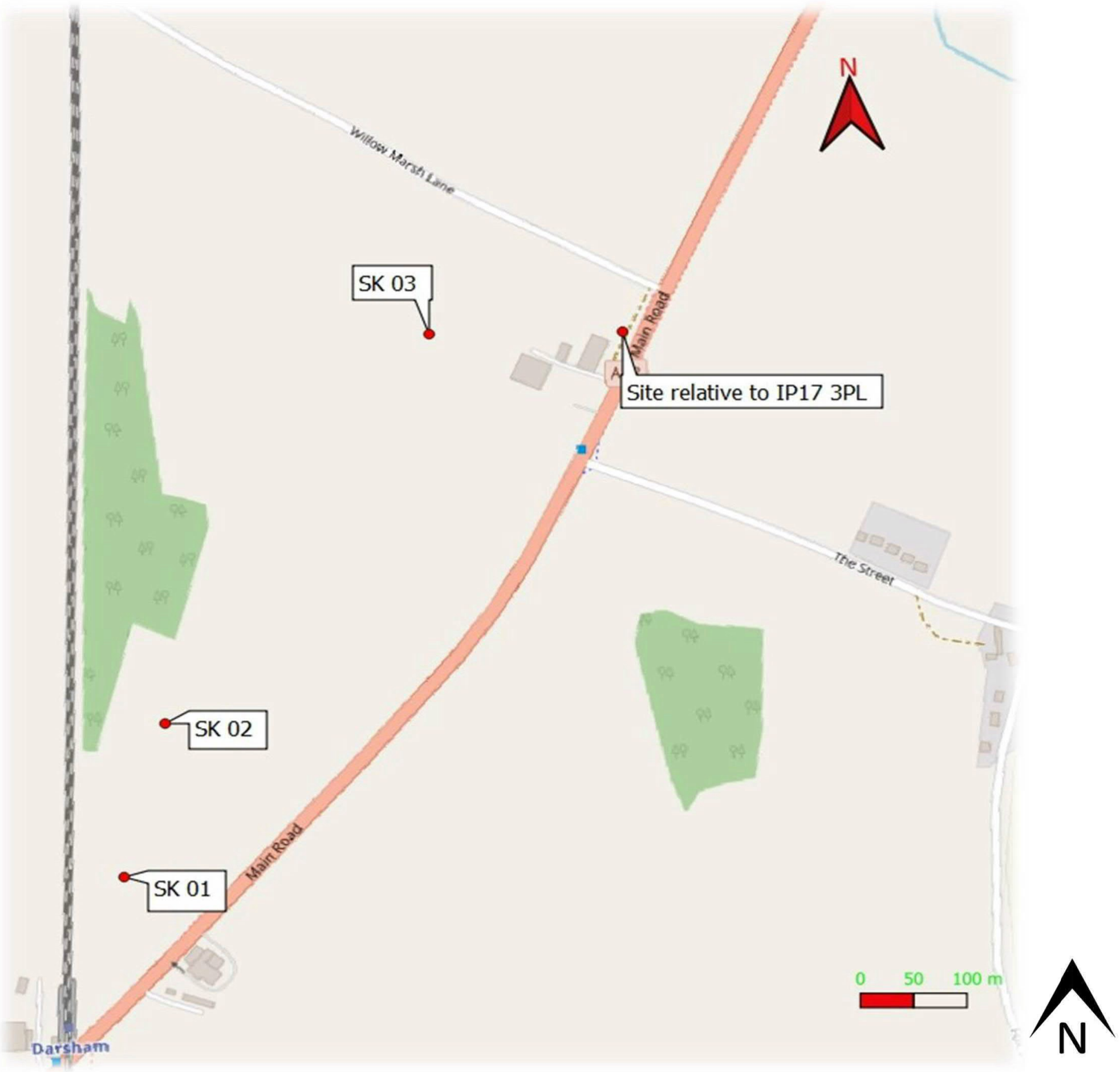
TITLE

Site Location Plan

DRAWING NUMBER

4029,SK/004/Rev0

SCALE	**DATE**
As marked	18/05/2020
DRAWN BY	**CHECKED BY**
JD	JT



GEOSPHERE ENVIRONMENTAL

LEGEND

● Infiltration pit locations (approximate)

SOURCE

© OpenStreetMap contributors

PROJECT

Land at Darsham, IP17 3PL

TITLE

Exploratory Hole Location Plan

DRAWING NUMBER

4029,SK/005/Rev0

SCALE	**DATE**
As marked	18/05/2020
DRAWN BY	**CHECKED BY**
JD	JT

GEO

GEOSPHERE ENVIRONMENTAL

Ec Ecology.

Fr Flood Risk.

Ge Geotechnical.

En Environmental.

Kw Knotweed.

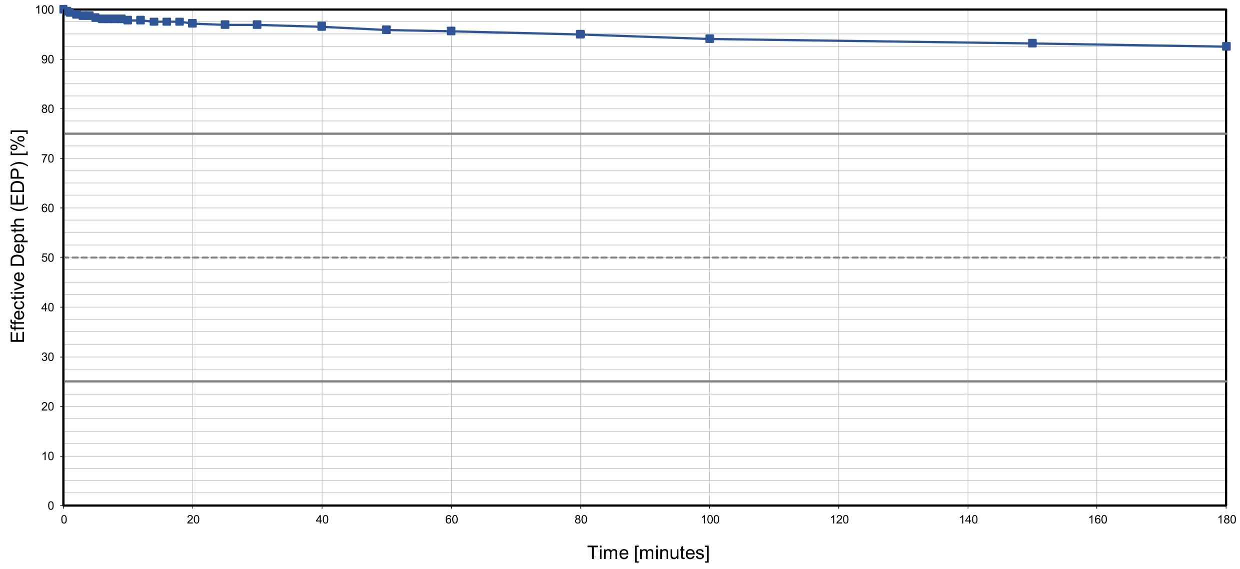
GEOSPHERE ENVIRONMENTAL LTD
Brightwell Barns, Ipswich Road, Brightwell, Suffolk, IP10 0BJ
T: 01603 298076 | 01473 353519 | **E**: info@geosphere-environmental.co.uk | **W**: geosphere-environmental.co.uk

NNB GENERATION COMPANY (SZC) LIMITED
Sizewell C Associated Developments: Nothern and Southern Park and Ride, and Freight Management Facility Sites

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	7/15/2021	Operator	JL	Pit Depth	3.50 m	Location ID	DTP219A



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	11:00		Start Time =			Start Time =		
Test Top =	0.26	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	3.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
75% EDP =	1.07	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	1.88	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	2.69	m BGL	25% EDP =		m BGL	25% EDP =		m BGL

Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
0:00:00		0.26	100								
0:00:30		0.27	100								
0:01:00		0.28	99								
0:02:00		0.29	99								
0:03:00		0.30	99								
0:04:00		0.30	99								
0:05:00		0.31	98								
0:06:00		0.32	98								
0:07:00		0.32	98								
0:08:00		0.32	98								
0:09:00		0.32	98								
0:10:00		0.33	98								
0:12:00		0.33	98								
0:14:00		0.34	98								
0:16:00		0.34	98								
0:18:00		0.34	98								
0:20:00		0.35	97								
0:25:00		0.36	97								
0:30:00		0.36	97								
0:40:00		0.37	97								
0:50:00		0.39	96								
1:00:00		0.40	96								
1:20:00		0.42	95								
1:40:00		0.45	94								
2:30:00		0.48	93								
3:00:00		0.50	93								

Datum (-ve denotes AGL) = 0.00 m BGL.
Ground Level taken as Datum.

Input by AH 01/09/2021

Checked by JD 03/11/2021

Approved by SAF 04/11/2021

Contract No. F187026

NNB GENERATION COMPANY (SZC) LIMITED
Sizewell C Associated Developments: Nothern and Southern Park and Ride, and Freight Management Facility Sites

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	7/15/2021	Operator	JL	Pit Depth	3.50 m	Location ID	DTP219A

Test Details

Datum (-ve denotes AGL) = 0.00 m BGL

Pit Length = 3.90 m
Pit Width = 0.80 m
Pit Depth = 3.50 m BGL

Well Screen
Well screen not used

Filter Material
Assumed Solid Fraction = 57.13 %
Assumed Porosity = 42.87 %

Weather: Dry, pleasant

Geology: CLAY over SAND

Remarks
Test 1 stopped at 3 hours in accordance with the specification and on instruction by the Investigation Supervisor. Test 2 and 3 were cancelled by the Investigation Supervisor.

Water level did not reach 75 % or 25 % effective depth, unable to calculate the infiltration rate.

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	11:00		Start Time =			Start Time =		
Test Top =	0.26	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	3.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
EDP =	3.24	m	EDP =		m	EDP =		m
75% EDP =	1.07	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	1.88	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	2.69	m BGL	25% EDP =		m BGL	25% EDP =		m BGL
V =	10.11	m^3	V =		m^3	V =		m^3
Vg =	0.89	m^3	Vg =		m^3	Vg =		m^3
Vp =	9.22	m^3	Vp =		m^3	Vp =		m^3
Vp75-25 =	4.61	m^3	Vp75-25 =		m^3	Vp75-25 =		m^3
ap =	18.35	m^2	ap =		m^2	ap =		m^2
Tp75 =		s	Tp75 =	480	s	Tp75 =	630	s
Tp25 =		s	Tp25 =	3150	s	Tp25 =	3600	s
Infiltration Rate, f =		m/s	Infiltration Rate, f =		m/s	Infiltration Rate, f =		m/s

Notes

Pit sides are assumed to be vertical; dimensions at mid-depth of pit used in general.

m AGL/BGL = metres above / below ground level;
m BDL = metres below datum level.

Effective depth of soakaway (EDP) is calculated from the initial water level to the base of the pit.

V is the effective storage volume of water in the hole (ESV) when gravel fill not used; Vg is the effective volume taken up by the gravel solid;
Vp is the ESV, less the volume of the gravel fraction.
Vp75-25 is the ESV between 75% and 25% effective depth, less the volume of the gravel fraction.

ap is the internal surface area of the pit including base area during the test.

Tp75 is time at 75% EDP; Tp50 is the time at 50% EDP; Tp25 is time at 25% EDP.
Tp75-25 is the assessed time for water level to fall from 75% to 25% EDP.

$$Soil\ Infiltration\ rate, f = \frac{Vp_{75-25}}{ap \times Tp_{75-25}}$$

Input by AH 01/09/2021

Checked by JD 03/11/2021

Approved by

Approved by SAF 04/11/2021

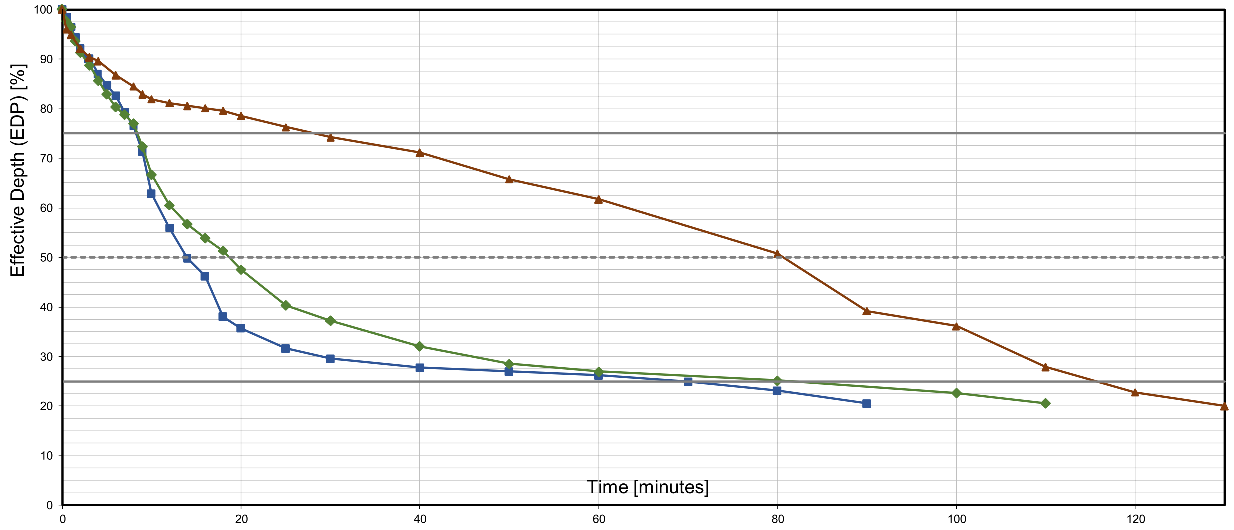
Contract No. F187026

NNB GENERATION COMPANY (SZC) LIMITED
Sizewell C Associated Developments: Nothern and Southern Park and Ride, and Freight Management Facility sites

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	7/16/2021	Operator	AD	Pit Depth	4.20 m	Location ID	DTP220



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	13:05		Start Time =	9:52		Start Time =	11:53	
Test Top =	0.30	m BGL	Test Top =	0.30	m BGL	Test Top =	0.29	m BGL
Test Base =	4.20	m BGL	Test Base =	4.20	m BGL	Test Base =	4.20	m BGL
75% EDP =	1.28	m BGL	75% EDP =	1.28	m BGL	75% EDP =	1.27	m BGL
50% EDP =	2.25	m BGL	50% EDP =	2.25	m BGL	50% EDP =	2.25	m BGL
25% EDP =	3.23	m BGL	25% EDP =	3.23	m BGL	25% EDP =	3.22	m BGL

Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
0:00:00	1.22	0.30	100	0:00:00	1.22	0.30	100	0:00:00	1.21	0.29	100
0:00:30	1.28	0.36	98	0:00:30	1.31	0.39	98	0:00:30	1.37	0.45	96
0:01:00	1.36	0.44	96	0:01:00	1.36	0.44	96	0:01:00	1.41	0.49	95
0:01:30	1.44	0.52	94	0:01:30	1.47	0.55	94	0:02:00	1.52	0.60	92
0:02:00	1.53	0.61	92	0:02:00	1.56	0.64	91	0:03:00	1.59	0.67	90
0:03:00	1.61	0.69	90	0:03:00	1.66	0.74	89	0:04:00	1.62	0.70	90
0:04:00	1.73	0.81	87	0:04:00	1.78	0.86	86	0:06:00	1.73	0.81	87
0:05:00	1.82	0.90	85	0:05:00	1.89	0.97	83	0:08:00	1.82	0.90	84
0:06:00	1.90	0.98	83	0:06:00	1.99	1.07	80	0:09:00	1.88	0.96	83
0:07:00	2.03	1.11	79	0:07:00	2.05	1.13	79	0:10:00	1.92	1.00	82
0:08:00	2.14	1.22	76	0:08:00	2.12	1.20	77	0:12:00	1.95	1.03	81
0:09:00	2.34	1.42	71	0:09:00	2.30	1.38	72	0:14:00	1.97	1.05	81
0:10:00	2.67	1.75	63	0:10:00	2.52	1.60	67	0:16:00	1.99	1.07	80
0:12:00	2.94	2.02	56	0:12:00	2.76	1.84	61	0:18:00	2.01	1.09	80
0:14:00	3.18	2.26	50	0:14:00	2.91	1.99	57	0:20:00	2.05	1.13	79
0:16:00	3.32	2.40	46	0:16:00	3.02	2.10	54	0:25:00	2.14	1.22	76
0:18:00	3.64	2.72	38	0:18:00	3.12	2.20	51	0:30:00	2.22	1.30	74
0:20:00	3.73	2.81	36	0:20:00	3.27	2.35	47	0:40:00	2.34	1.42	71
0:25:00	3.89	2.97	32	0:25:00	3.55	2.63	40	0:50:00	2.55	1.63	66
0:30:00	3.97	3.05	29	0:30:00	3.67	2.75	37	1:00:00	2.71	1.79	62
0:40:00	4.04	3.12	28	0:40:00	3.87	2.95	32	1:20:00	3.14	2.22	51
0:50:00	4.07	3.15	27	0:50:00	4.01	3.09	28	1:30:00	3.59	2.67	39
1:00:00	4.10	3.18	26	1:00:00	4.07	3.15	27	1:40:00	3.71	2.79	36
1:10:00	4.15	3.23	25	1:20:00	4.14	3.22	25	1:50:00	4.03	3.11	28
1:20:00	4.22	3.30	23	1:40:00	4.24	3.32	23	2:00:00	4.23	3.31	23
1:30:00	4.32	3.40	21	1:50:00 AM	4.32	3.40	21	2:10:00 AM	4.34	3.42	20

Datum (-ve denotes AGL) = -0.92 m BGL.

Water depth measurements were taken from the Datum Level chosen on field, then converted to referencing Ground Level for calculations.

Input by AH 01/09/2021

Checked by JD 03/11/2021

Approved by SAF 04/11/2021

Contract No. F187026

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	7/16/2021	Operator	AD	Pit Depth	4.20 m	Location ID	DTP220

Test Details

Datum (-ve denotes AGL) =	-0.92 m BGL	Well Screen	
		Well screen not used	
Pit Length =	4.10 m	Filter Material	
Pit Width =	0.80 m	Filter not used	
Pit Depth =	4.20 m BGL		

Weather: Cold, dry, wet ground

Geology: CLAY/SAND

Remarks

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	13:05		Start Time =	9:52		Start Time =	11:53	
Test Top =	0.30	m BGL	Test Top =	0.30	m BGL	Test Top =	0.29	m BGL
Test Base =	4.20	m BGL	Test Base =	4.20	m BGL	Test Base =	4.20	m BGL
EDP =	3.90	m	EDP =	3.90	m	EDP =	3.91	m
75% EDP =	1.28	m BGL	75% EDP =	1.28	m BGL	75% EDP =	1.27	m BGL
50% EDP =	2.25	m BGL	50% EDP =	2.25	m BGL	50% EDP =	2.25	m BGL
25% EDP =	3.23	m BGL	25% EDP =	3.23	m BGL	25% EDP =	3.22	m BGL
V =	12.79	m^3	V =	12.79	m^3	V =	12.82	m^3
Vg =		m^3	Vg =		m^3	Vg =		m^3
Vp =		m^3	Vp =		m^3	Vp =		m^3
Vp75-25 =	6.40	m^3	Vp75-25 =	6.40	m^3	Vp75-25 =	6.41	m^3
ap =	22.39	m^2	ap =	22.39	m^2	ap =	22.44	m^2
Tp75 =	510	s	Tp75 =	510	s	Tp75 =	1680	s
Tp25 =	4200	s	Tp25 =	4800	s	Tp25 =	6900	s
Infiltration Rate, f =	7.74E-05	m/s	Infiltration Rate, f =	6.66E-05	m/s	Infiltration Rate, f =	5.47E-05	m/s

Notes

Pit sides are assumed to be vertical; dimensions at mid-depth of pit used in general.

m AGL/BGL = metres above / below ground level;
m BDL = metres below datum level.

Effective depth of soakaway (EDP) is calculated from the initial water level to the base of the pit.

V is the effective storage volume of water in the hole (ESV) when gravel fill not used; Vg is the effective volume taken up by the gravel solid;
Vp is the ESV, less the volume of the gravel fraction.
Vp75-25 is the ESV between 75% and 25% effective depth, less the volume of the gravel fraction.

ap is the internal surface area of the pit including base area during the test.

Tp75 is time at 75% EDP; Tp50 is the time at 50% EDP; Tp25 is time at 25% EDP.
Tp75-25 is the assessed time for water level to fall from 75% to 25% EDP.

$$Soil\ Infiltration\ rate, f = \frac{Vp_{75-2}}{ap \times Tp_{75-25}}$$

Input by AH 01/09/2021

Checked by JD 03/11/2021

Approved by SAF 04/11/2021

Contract Name	Sizewell C Associated Developments, Northern and Southern Park and Ride and Freight Management Facility Sites			Location ID **DTP218**
Client	NNB Generation Company (SZC) Limited			
Fugro Reference	F187026			
Coordinates (m)	E641124.66 N270779.18	Ground Elevation (m Datum)	27.45	Sheet 1 of 1
Hole Type	Trial Pit / Trench			Status: Final

Sampling and In Situ Testing

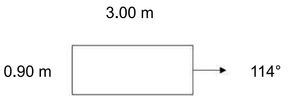
Depth (m)	Type	No.	Test Results
0.00 - 0.30	LB	1	
0.05 - 0.15	D	2	
0.15 - 0.30	ES	3	
0.15	PID		< 0.1 ppm
0.30 - 1.00	LB	4	
0.45 - 0.60	D	5	
0.85 - 1.00	ES	6	
0.85	PID		< 0.1 ppm
1.00 - 2.00	LB	7	
1.00	HVane		23 kPa (*11 kPa*)
1.00	HVane		28 kPa (*13 kPa*)
1.00	HVane		28 kPa (*16 kPa*)
1.20 - 1.40	D	8	
1.55 - 1.80	ES	9	
1.55	PID		< 0.1 ppm
2.50 - 3.50	LB	10	
2.80 - 3.00	D	11	
3.10 - 3.45	ES	12	
3.10	PID		< 0.1 ppm
3.80 - 4.50	LB	13	
3.90 - 4.10	D	14	

Strata Details

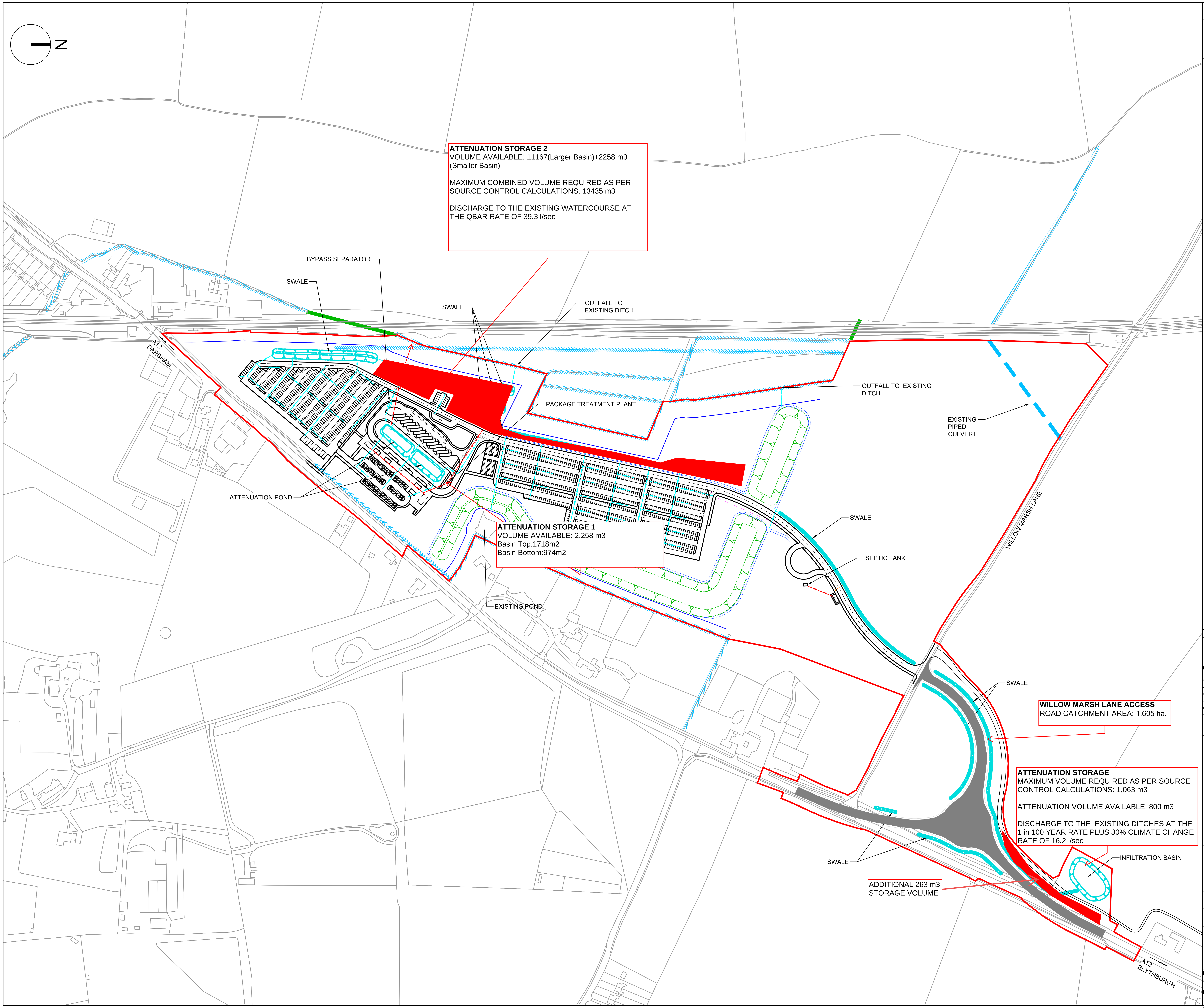
Strata Descriptions	Depth (Thickness) (m)	Level (m Datum)
TOPSOIL. Firm dark brown slightly gravelly CLAY with frequent rootlets (<10mm x 200mm) and low cobble content. Gravel is subangular and subrounded fine to coarse of flint. Cobbles (<100mm x 120mm x 150mm) are subangular of flint. [TOPSOIL] [CLAY]	(0.30) 0.30	27.15
Firm orangish brown slightly gravelly slightly sandy CLAY with low cobble content. Sand is fine to coarse. Gravel is subangular and subrounded fine to coarse of chalk and flint. Cobbles (<80mm x 100mm x 100mm) are subangular and subrounded of chalk and flint. [LOWESTOFT FORMATION] [CLAY]	(0.70) 1.00	26.45
Firm light grey slightly gravelly CLAY with pockets (<300mm x 300mm x 400mm) of orange sand and medium cobble content. Sand is fine to coarse. Gravel is subangular and subrounded fine to coarse of chalk. Cobbles (<75mm x 100mm x 150mm) are subrounded and rounded of chalk. [LOWESTOFT FORMATION] [CLAY]	(1.50) 2.50	24.95
Stiff dark grey slightly sandy slightly gravelly CLAY with medium cobble content. Sand is fine to coarse. Gravel is subangular and subrounded fine to coarse of chalk. Cobbles (<80mm x 120mm x 150mm) are subrounded and rounded of chalk. [LOWESTOFT FORMATION] [CLAY]	(2.00) 4.50	22.95
End of Trial Pit / Trench at 4.50 m		

Groundwater: Water Strike — ; Backfill / Installation

Notes	Pit Stability	Plan
- Abbreviations and results data defined on 'Notes on Exploratory Position Records'	Stable	



APPENDIX B: LAYOUT PLAN SHOWING ATTENUATION STORAGE LOCATIONS



UK PROTECTIVE MARKING:
Not Protectively Marked

NOTES:

1. Do not scale from this drawing. All dimensions are in metres unless noted otherwise.

KEY

🟥	DEVELOPMENT SITE BOUNDARY
—	BUFFER ZONE
▭	SWALE / INFILTRATION BASIN (INDICATIVE)
▨	LANDSCAPE BUND
—	PROPOSED SURFACE WATER DRAINAGE (INDICATIVE)
—	PROPOSED FOUL WATER DRAINAGE (INDICATIVE)
—	PROPOSED DITCH (INDICATIVE)
▨	EXISTING DITCH / WATER COURSE
🟩	EXISTING BRICK CULVERT
▭	PROPOSED BYPASS SEPARATOR (INDICATIVE)
— —	EXISTING PIPED CULVERT

NOT FOR CONSTRUCTION

NOT FOR APPROVAL

REV.	DATE	PREPARED BY	CHECKED BY	STATUS	REASONS FOR REVISION	APPROVED BY
H	12.06.20	AB	KA	S3	MINOR AMENDMENTS FOLLOWING INFILTRATION TESTS	PJ
G	21.02.20	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING HSF REVIEW	PJ
F	10.01.20	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING HSF REVIEW	PJ
E	12.11.19	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING QUOD REVIEW	PJ
D	31.10.19	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING TEAM REVIEW	PJ
C	24.09.19	NKS	KA	S3	UPDATED WSP DESIGN ADDED.	PJ
B	24.07.19	NKS	KA	S3	MINOR AMENDMENTS	PJ
A	09.07.19	NKS	KA	S3	FIRST ISSUE	PJ

NNB GenCo EDF ENERGY	1st partner EDF ENERGY	2nd partner NNB GENCO

CONTRACTOR COMPANY TRADE NAME : ROYAL HASKONINGDHV

CONTRACTOR REF. No. PB7869

CONTRACT NUMBER : SZ0204

CONTRACTOR WBS CODE : N/A

QRA RELATED Yes ☐ No ☒

APPLICABILITY:
1: Document related to Unit 1
2: Document related to Unit 2
9: Document that applies to buildings/systems common to Unit 1 & 2
0: Documents that relate exclusively to buildings or systems that are common to the whole site (e.g. parking, ancillary buildings...)

NUCL/REP/EPR/UKX							
HPC (doc: HK)				SZC (doc: SZ)			
0	1	2	9	0	1	2	9

BUILDING
000

SYSTEM
000

SCALE
1:2000

SIZE A1

PAGE
1/1

DESCRIPTION

SIZEWELL C
NORTHERN PARK AND RIDE SITE
DRAINAGE LAYOUT

DOCUMENT REFERENCE No.

Project	Contract No. / Orig. Co	Asset / Zone	System / Building	Doc. type	Chrono No.
SZC	SZ0204	FP	000	DRW	100045

DOCUMENT SUB -TYPE
N/A

EDF CLASSIFICATION CODE
N/A

SUBCONTRACTOR COMPANY TRADE NAME
N/A

SUBCONTRACTOR DOCUMENT REF. No
N/A

This pattern must be entirely readable
For A0 and A1 : ABERPFTHLIJOOCGQUVWMNSZXKY
zsaeocmuvnwxirfkhbdpqgyjlt 71423855690
20.24 m
0 1 2 3 4 5 6 7 8 9 10

INTELLECTUAL PROPERTY:
O-Property of Owner

NNB:

EDF:
NNB GenCo © 2018

UK PROTECTIVE MARKING:
Not Protectively Marked

EDF ACCESSIBILITY:
INTERNE ☐ **RESTREINT** ☐ **CONFIDENTIEL** ☐

APPENDIX C: MAIN DEVELOPMENT ATTENUATION STORAGE CALCULATIONS

ICP SUDS Mean Annual Flood

```
                         Input

Return Period (years)       2 SAAR (mm)    600          Urban     0.000
           Area (ha)   13.850       Soil 0.400 Region Number Region 5

                      Results    l/s

                    QBAR Rural   39.3
                    QBAR Urban   39.3

                      Q2 years   35.2

                       Q1 year   34.2
                     Q30 years   94.5
                    Q100 years  140.1
```

Summary of Results for 100 year Return Period (+30%)

Storm Event	**Max Level (m)**	**Max Depth (m)**	**Max Control (l/s)**	**Max Volume (m³)**	**Status**
15 min Summer	19.150	0.550	39.1	3247.1	O K
30 min Summer	19.312	0.712	39.1	4261.5	O K
60 min Summer	19.473	0.873	39.1	5294.0	O K
120 min Summer	19.661	1.061	39.1	6534.1	O K
180 min Summer	19.791	1.191	39.1	7416.6	O K
240 min Summer	19.892	1.292	39.1	8106.8	O K
360 min Summer	20.041	1.441	39.1	9152.0	Flood Risk
480 min Summer	20.145	1.545	39.1	9893.3	Flood Risk
600 min Summer	20.217	1.617	39.5	10414.5	Flood Risk
720 min Summer	20.267	1.667	40.0	10783.4	Flood Risk
960 min Summer	20.324	1.724	40.6	11197.9	FLOOD
1440 min Summer	20.342	1.742	40.8	11331.9	FLOOD
2160 min Summer	20.271	1.671	40.1	10809.2	Flood Risk
2880 min Summer	20.184	1.584	39.1	10178.6	Flood Risk
4320 min Summer	20.026	1.426	39.1	9044.5	Flood Risk
5760 min Summer	19.897	1.297	39.1	8145.4	O K
15 min Winter	19.213	0.613	39.1	3643.2	O K
30 min Winter	19.393	0.793	39.1	4782.6	O K
60 min Winter	19.572	0.972	39.1	5947.6	O K
120 min Winter	19.781	1.181	39.1	7348.1	O K
180 min Winter	19.926	1.326	39.1	8345.8	O K

Storm Event	**Rain (mm/hr)**	**Flooded Volume (m³)**	**Discharge Volume (m³)**	**Time-Peak (mins)**
15 min Summer	127.140	0.0	3008.8	30
30 min Summer	83.590	0.0	3309.4	45
60 min Summer	52.260	0.0	5426.1	74
120 min Summer	32.611	0.0	6289.5	134
180 min Summer	24.934	0.0	6281.9	194
240 min Summer	20.651	0.0	6189.0	254
360 min Summer	15.843	0.0	6092.5	372
480 min Summer	13.081	0.0	6076.4	490
600 min Summer	11.214	0.0	6100.7	610
720 min Summer	9.847	0.0	6150.0	728
960 min Summer	7.943	174.1	6260.0	968
1440 min Summer	5.750	308.1	6289.2	1444
2160 min Summer	4.072	0.0	12324.7	2068
2880 min Summer	3.160	0.0	11955.5	2392
4320 min Summer	2.186	0.0	11036.1	3120
5760 min Summer	1.683	0.0	16787.6	3928
15 min Winter	127.140	0.0	3196.2	30
30 min Winter	83.590	0.0	3312.6	45
60 min Winter	52.260	0.0	5953.0	74
120 min Winter	32.611	0.0	6314.3	132
180 min Winter	24.934	0.0	6196.7	190

Summary of Results for 100 year Return Period (+30%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	20.038	1.438	39.1	9131.3	Flood Risk
360 min Winter	20.205	1.605	39.3	10330.0	Flood Risk
480 min Winter	20.323	1.723	40.6	11189.4	FLOOD
600 min Winter	20.406	1.806	41.5	11801.8	FLOOD
720 min Winter	20.466	1.866	42.1	12243.6	FLOOD
960 min Winter	20.536	1.936	42.8	12763.6	FLOOD
1440 min Winter	20.572	1.972	43.2	13022.8	FLOOD
2160 min Winter	20.512	1.912	42.6	12587.3	FLOOD
2880 min Winter	20.413	1.813	41.6	11855.8	FLOOD
4320 min Winter	20.223	1.623	39.5	10458.0	Flood Risk
5760 min Winter	20.060	1.460	39.1	9289.0	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	20.651	0.0	6133.4	248
360 min Winter	15.843	0.0	6125.7	366
480 min Winter	13.081	165.6	6210.1	482
600 min Winter	11.214	778.0	6347.0	600
720 min Winter	9.848	1219.8	6455.5	716
960 min Winter	7.943	1739.9	6568.4	948
1440 min Winter	5.750	1999.0	6576.4	1406
2160 min Winter	4.072	1563.5	12589.6	2068
2880 min Winter	3.160	832.0	12299.6	2684
4320 min Winter	2.186	0.0	11616.0	3332
5760 min Winter	1.683	0.0	18801.1	4264

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 640286 267538 TM 40286 67538
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+30

Time Area Diagram

Total Area (ha) 13.852

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	3.463	4	8	3.463	8	12	3.463	12	16	3.463

Model Details

Storage is Online Cover Level (m) 20.300

Tank or Pond Structure

Invert Level (m) 18.600

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	5646.0	1.700	7361.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0257-3930-1700-3930
Design Head (m)	1.700
Design Flow (l/s)	39.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	257
Invert Level (m)	18.500
Minimum Outlet Pipe Diameter (mm)	300
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.700	39.3	Kick-Flo®	1.148	32.5
Flush-Flo™	0.527	39.1	Mean Flow over Head Range	-	33.6

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	8.3	1.200	33.2	3.000	51.7	7.000	77.9
0.200	26.3	1.400	35.8	3.500	55.6	7.500	80.6
0.300	37.1	1.600	38.2	4.000	59.4	8.000	83.1
0.400	38.6	1.800	40.4	4.500	62.9	8.500	85.6
0.500	39.1	2.000	42.5	5.000	66.1	9.000	88.0
0.600	39.0	2.200	44.5	5.500	69.3	9.500	90.4
0.800	38.1	2.400	46.4	6.000	72.3		
1.000	36.2	2.600	48.2	6.500	75.1		

Summary of Results for 100 year Return Period (+30%)

Storm Event	**Max Level (m)**	**Max Depth (m)**	**Max Control (l/s)**	**Max Volume (m³)**	**Status**
15 min Summer	20.891	2.291	46.3	3236.6	FLOOD
30 min Summer	21.474	2.874	51.4	4236.9	FLOOD
60 min Summer	22.058	3.458	56.1	5240.9	FLOOD
120 min Summer	22.739	4.139	61.1	6410.6	FLOOD
180 min Summer	23.210	4.610	64.3	7220.5	FLOOD
240 min Summer	23.570	4.970	66.6	7839.0	FLOOD
360 min Summer	24.093	5.493	69.8	8736.9	FLOOD
480 min Summer	24.437	5.837	71.9	9328.5	FLOOD
600 min Summer	24.653	6.053	73.2	9699.1	FLOOD
720 min Summer	24.781	6.181	73.9	9918.5	FLOOD
960 min Summer	24.852	6.252	74.3	10040.2	FLOOD
1440 min Summer	24.711	6.111	73.5	9798.6	FLOOD
2160 min Summer	24.313	5.713	71.2	9115.3	FLOOD
2880 min Summer	23.906	5.306	68.7	8416.0	FLOOD
4320 min Summer	23.226	4.626	64.4	7248.1	FLOOD
5760 min Summer	22.709	4.109	60.8	6358.6	FLOOD
15 min Winter	21.121	2.521	48.4	3631.3	FLOOD
30 min Winter	21.776	3.176	53.9	4755.6	FLOOD
60 min Winter	22.435	3.835	58.9	5889.3	FLOOD
120 min Winter	23.211	4.611	64.3	7220.9	FLOOD
180 min Winter	23.751	5.151	67.7	8149.2	FLOOD

Storm Event	**Rain (mm/hr)**	**Flooded Volume (m³)**	**Discharge Volume (m³)**	**Time-Peak (mins)**
15 min Summer	127.140	1016.0	3272.3	30
30 min Summer	83.590	2016.3	3570.1	45
60 min Summer	52.260	3020.3	5428.8	74
120 min Summer	32.611	4190.0	6775.1	132
180 min Summer	24.934	4999.9	7771.1	192
240 min Summer	20.651	5618.3	8311.4	250
360 min Summer	15.843	6516.3	8830.8	368
480 min Summer	13.081	7107.9	9297.0	486
600 min Summer	11.214	7478.4	9615.4	604
720 min Summer	9.847	7697.9	9832.0	724
960 min Summer	7.943	7819.6	10060.7	960
1440 min Summer	5.750	7578.0	10078.3	1178
2160 min Summer	4.072	6894.7	15229.8	1556
2880 min Summer	3.160	6195.4	15755.9	1964
4320 min Summer	2.186	5027.4	15407.1	2776
5760 min Summer	1.683	4138.0	16785.7	3584
15 min Winter	127.140	1410.7	3431.4	30
30 min Winter	83.590	2535.0	3731.9	44
60 min Winter	52.260	3668.7	6080.6	74
120 min Winter	32.611	5000.3	7588.4	130
180 min Winter	24.934	5928.5	8399.7	188

Summary of Results for 100 year Return Period (+30%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	24.166	5.566	70.3	8862.5	FLOOD
360 min Winter	24.776	6.176	73.9	9910.1	FLOOD
480 min Winter	25.186	6.586	76.2	10614.5	FLOOD
600 min Winter	25.452	6.852	77.6	11071.8	FLOOD
720 min Winter	25.620	7.020	78.5	11360.8	FLOOD
960 min Winter	25.753	7.153	79.3	13435.3	FLOOD
1440 min Winter	25.604	7.004	78.5	11333.0	FLOOD
2160 min Winter	25.154	6.554	76.0	10560.5	FLOOD
2880 min Winter	24.650	6.050	73.1	9694.4	FLOOD
4320 min Winter	23.730	5.130	67.6	8113.8	FLOOD
5760 min Winter	23.026	4.426	63.0	6904.2	FLOOD

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	20.651	6641.9	8771.6	246
360 min Winter	15.843	7689.5	9467.9	362
480 min Winter	13.081	8393.9	9956.8	478
600 min Winter	11.214	8851.2	10283.0	592
720 min Winter	9.848	9140.2	10503.2	706
960 min Winter	7.943	11367.7	10726.1	926
1440 min Winter	5.750	9112.4	10704.6	1332
2160 min Winter	4.072	8339.9	17056.1	1652
2880 min Winter	3.160	7473.8	17585.2	2112
4320 min Winter	2.186	5893.1	16671.5	2996
5760 min Winter	1.683	4683.6	18800.3	3864

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 640286 267538 TM 40286 67538
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+30

Time Area Diagram

Total Area (ha) 13.852

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	3.463	4	8	3.463	8	12	3.463	12	16	3.463

Model Details

Storage is Online Cover Level (m) 20.300

Tank or Pond Structure

Invert Level (m) 18.600

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	934.0	1.700	1718.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0257-3930-1700-3930
Design Head (m)	1.700
Design Flow (l/s)	39.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	257
Invert Level (m)	18.500
Minimum Outlet Pipe Diameter (mm)	300
Suggested Manhole Diameter (mm)	1800

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.700	39.3	Kick-Flo®	1.148	32.5
Flush-Flo™	0.527	39.1	Mean Flow over Head Range	-	33.6

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	8.3	1.200	33.2	3.000	51.7	7.000	77.9
0.200	26.3	1.400	35.8	3.500	55.6	7.500	80.6
0.300	37.1	1.600	38.2	4.000	59.4	8.000	83.1
0.400	38.6	1.800	40.4	4.500	62.9	8.500	85.6
0.500	39.1	2.000	42.5	5.000	66.1	9.000	88.0
0.600	39.0	2.200	44.5	5.500	69.3	9.500	90.4
0.800	38.1	2.400	46.4	6.000	72.3		
1.000	36.2	2.600	48.2	6.500	75.1		

APPENDIX D: ACCESS ROUNDABOUT ATTENUATION STORAGE CALCULATIONS

ICP SUDS Mean Annual Flood

```
                               Input

Return Period (years)      2  SAAR (mm)    600          Urban     0.000
            Area (ha)  1.605       Soil  0.400  Region Number  Region 5

                           Results    l/s

                          QBAR Rural   4.6
                          QBAR Urban   4.6

                            Q2 years   4.1

                             Q1 year   4.0
                           Q30 years  11.0
                          Q100 years  16.2
```

Summary of Results for 100 year Return Period (+30%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	26.336	0.436	16.2	366.5	O K
30 min Summer	26.442	0.542	16.2	477.9	O K
60 min Summer	26.537	0.637	16.2	584.6	O K
120 min Summer	26.634	0.734	16.2	702.0	Flood Risk
180 min Summer	26.692	0.792	16.2	775.0	Flood Risk
240 min Summer	26.729	0.829	16.2	824.7	Flood Risk
360 min Summer	26.773	0.873	16.2	884.2	Flood Risk
480 min Summer	26.792	0.892	16.2	909.8	Flood Risk
600 min Summer	26.799	0.899	16.2	919.1	Flood Risk
720 min Summer	26.798	0.898	16.2	919.0	Flood Risk
960 min Summer	26.785	0.885	16.2	899.8	Flood Risk
1440 min Summer	26.733	0.833	16.2	830.1	Flood Risk
2160 min Summer	26.635	0.735	16.2	703.1	Flood Risk
2880 min Summer	26.521	0.621	16.2	565.8	O K
4320 min Summer	26.324	0.424	16.2	355.0	O K
5760 min Summer	26.189	0.289	16.2	227.3	O K
15 min Winter	26.380	0.480	16.2	412.1	O K
30 min Winter	26.497	0.597	16.2	538.0	O K
60 min Winter	26.601	0.701	16.2	660.4	Flood Risk
120 min Winter	26.706	0.806	16.2	794.2	Flood Risk
180 min Winter	26.770	0.870	16.2	879.6	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	127.140	0.0	373.7	26
30 min Summer	83.590	0.0	493.1	40
60 min Summer	52.260	0.0	624.7	68
120 min Summer	32.611	0.0	780.3	128
180 min Summer	24.934	0.0	895.2	186
240 min Summer	20.651	0.0	988.8	244
360 min Summer	15.843	0.0	1138.1	362
480 min Summer	13.081	0.0	1253.0	456
600 min Summer	11.214	0.0	1342.6	512
720 min Summer	9.847	0.0	1414.6	576
960 min Summer	7.943	0.0	1520.7	704
1440 min Summer	5.750	0.0	1649.1	980
2160 min Summer	4.072	0.0	1761.5	1392
2880 min Summer	3.160	0.0	1822.2	1764
4320 min Summer	2.186	0.0	1888.2	2468
5760 min Summer	1.683	0.0	1943.3	3120
15 min Winter	127.140	0.0	419.2	26
30 min Winter	83.590	0.0	552.8	40
60 min Winter	52.260	0.0	700.0	68
120 min Winter	32.611	0.0	874.2	126
180 min Winter	24.934	0.0	1003.0	182

Summary of Results for 100 year Return Period (+30%)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m^3)	Status
240 min Winter	26.813	0.913	16.2	939.4	Flood Risk
360 min Winter	26.866	0.966	16.2	1015.0	Flood Risk
480 min Winter	26.891	0.991	16.2	1052.2	Flood Risk
600 min Winter	26.898	0.998	16.2	1063.3	Flood Risk
720 min Winter	26.895	0.995	16.2	1058.2	Flood Risk
960 min Winter	26.878	0.978	16.2	1033.2	Flood Risk
1440 min Winter	26.811	0.911	16.2	936.8	Flood Risk
2160 min Winter	26.678	0.778	16.2	756.9	Flood Risk
2880 min Winter	26.511	0.611	16.2	554.7	O K
4320 min Winter	26.229	0.329	16.2	263.6	O K
5760 min Winter	26.091	0.191	15.5	143.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m^3)	Discharge Volume (m^3)	Time-Peak (mins)
240 min Winter	20.651	0.0	1107.7	240
360 min Winter	15.843	0.0	1274.9	354
480 min Winter	13.081	0.0	1403.5	464
600 min Winter	11.214	0.0	1503.7	568
720 min Winter	9.848	0.0	1584.3	658
960 min Winter	7.943	0.0	1702.9	750
1440 min Winter	5.750	0.0	1845.9	1058
2160 min Winter	4.072	0.0	1973.1	1512
2880 min Winter	3.160	0.0	2041.2	1904
4320 min Winter	2.186	0.0	2115.7	2520
5760 min Winter	1.683	0.0	2176.7	3008

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 640286 267538 TM 40286 67538
Data Type	Point
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.750
Cv (Winter)	0.840
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+30

Time Area Diagram

Total Area (ha) 1.605

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.535	4	8	0.535	8	12	0.535

Model Details

Storage is Online Cover Level (m) 26.900

Tank or Pond Structure

Invert Level (m) 25.900

Depth (m)	**Area (m²)**	**Depth (m)**	**Area (m²)**
0.000	690.0	1.000	1492.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0181-1620-1000-1620
Design Head (m)	1.000
Design Flow (l/s)	16.2
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	181
Invert Level (m)	25.900
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	**Head (m)**	**Flow (l/s)**
Design Point (Calculated)	1.000	16.2
Flush-Flo™	0.324	16.2
Kick-Flo®	0.706	13.7
Mean Flow over Head Range	-	13.7

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

Depth (m)	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**
0.100	6.4	1.200	17.7	3.000	27.4	7.000	41.2
0.200	15.6	1.400	19.0	3.500	29.5	7.500	42.6
0.300	16.2	1.600	20.3	4.000	31.4	8.000	43.9
0.400	16.1	1.800	21.4	4.500	33.3	8.500	45.2
0.500	15.8	2.000	22.5	5.000	35.0	9.000	46.5
0.600	15.2	2.200	23.6	5.500	36.6	9.500	47.7
0.800	14.6	2.400	24.6	6.000	38.2		
1.000	16.2	2.600	25.5	6.500	39.7		

APPENDIX E: POLLUTION MITIGATION MEASURES ASSESSMENT

Introduction

The purpose of this technical note is to provide an assessment to demonstrate that the proposed drainage infrastructure for the NP&R will provide treatment train facilities to mitigate unacceptable risk of pollution to the water environment. The CIRIA C753 SuDS Manual Simplified Index Approach has been applied as an appropriate tool.

Proposed Drainage Strategy

Following infiltration testing it is confirmed that removal of surface water runoff and disposal by infiltration to ground is not viable. Therefore, it is necessary to remove the surface water runoff by discharge to existing local watercourse. The flow rate for discharge to watercourse is set at QBar greenfield runoff which is 39.7 l/s for the main site and the 1 in 100 year greenfield runoff rate of 16.2 l/s for the separate A12 Access Roundabout catchment to the north. Attenuation of flow is required in order to limit discharge to these rates.

Proposed Drainage Infrastructure

The proposed drainage infrastructure is described in the Environmental Statement submitted as part of DCO submission. Its subsequent development and the current proposals are described in the more recent Northern Park and Ride Drainage Strategy issued with the intention that it would be acceptable to regulators such that it may be included in the statement of common ground at DCO Examination Stage.

In summary, for the main site, runoff from roofs will be drained via downpipes and gullies, as appropriate to underground carrier drains and discharge into attenuation basins and swales.

Runoff from the internal roads and the bus/HGV standing areas with impermeable surface will be drained via surface outlets, gullies, linear channels and drains etc. These will discharge into same underground carrier drains.

Bypass interceptors will be installed downstream of the bus/HGV standing areas in order to remove hydrocarbon and silt contaminants which will improve the water quality of discharge to the attenuation basins and swales.

The extensive car parking areas will have a permeable surface allowing runoff to permeate into and be temporarily stored in the sub-base. This will assist with attenuating peak flow rate, provide some storage and initial treatment of the runoff. The sub-base will allow flow to drain into the carrier drains.

The underground carrier drains will discharge all surface water into a series of cascading attenuation basins and swales which will provide suitable final treatment. These will also provide attenuation storage for all runoff, required in order that discharge to watercourse from the site is limited to the equivalent greenfield runoff.

Unpaved areas will drain directly by infiltration to ground.

In summary, for the A12 roundabout

Highway, runoff will be collected by surface water outlets, gullies and CKDs into carrier drains which will discharge to swales located adjacent to the 3 arms of the roundabout.

The three arms of the roundabout will drain “Over the Edge” to swales.

The swales will have an underlying filter drain.

The swale/filter drains will discharge into an attenuation basin required in order that discharge to watercourse from the A12 roundabout and its three arms is limited to the equivalent greenfield runoff.

Simplified Index Approach (SIA) Assessment

The SIA methodology considers the relative potential pollution risk based on land use and assigns a level of risk. Based on the risk it then assigns indices for 3 pollutants, these being Total Suspended Solids, Metals and Hydrocarbons.

This is shown in Table 26.2, reproduced from the CIRIA SuDs Manual and reproduced below.

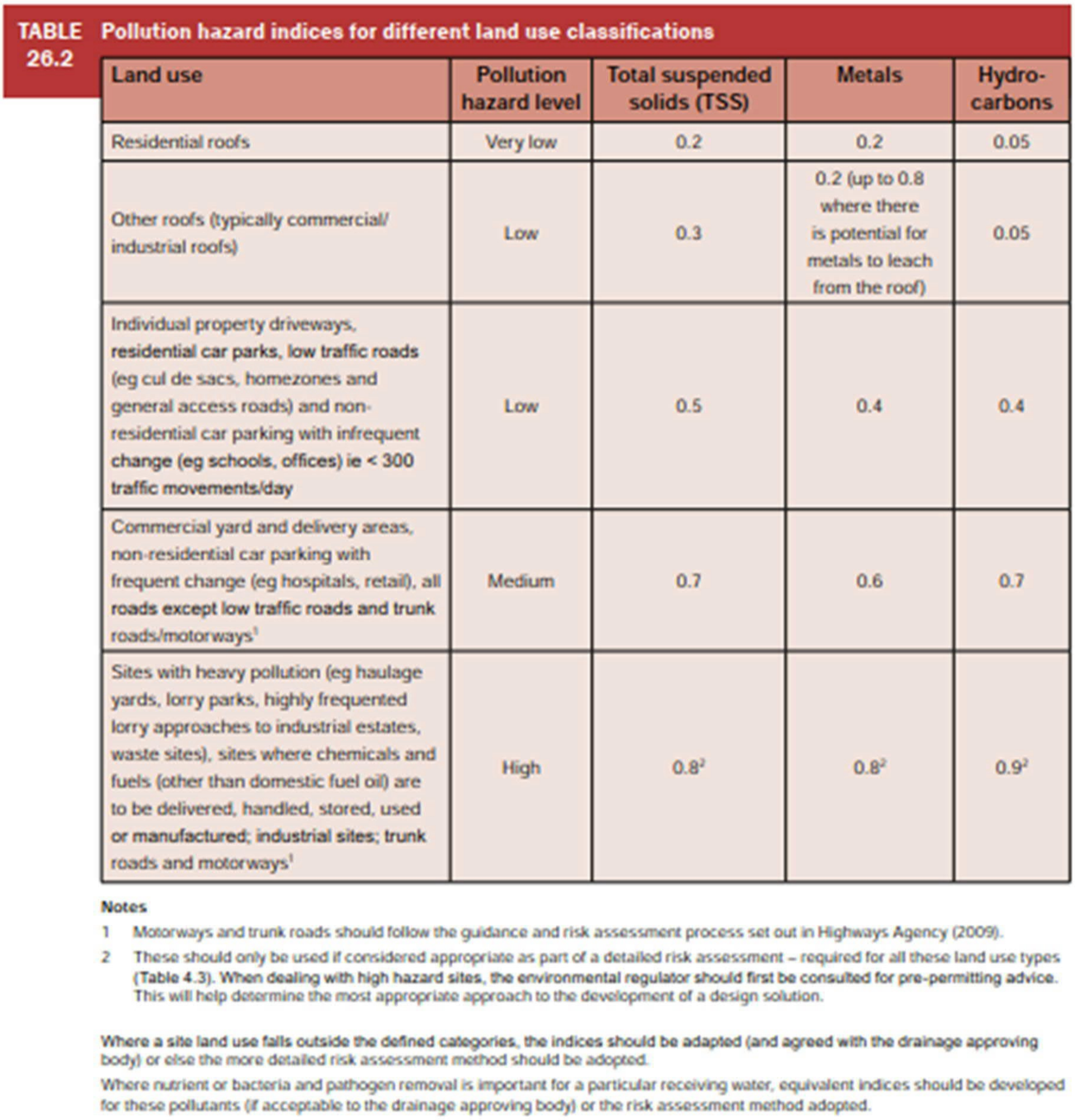


TABLE 26.2 **Pollution hazard indices for different land use classifications**

Land use	Pollution hazard level	Total suspended solids (TSS)	Metals	Hydro-carbons
Residential roofs	Very low	0.2	0.2	0.05
Other roofs (typically commercial/ industrial roofs)	Low	0.3	0.2 (up to 0.8 where there is potential for metals to leach from the roof)	0.05
Individual property driveways, residential car parks, low traffic roads (eg cul de sacs, homezones and general access roads) and non-residential car parking with infrequent change (eg schools, offices) ie < 300 traffic movements/day	Low	0.5	0.4	0.4
Commercial yard and delivery areas, non-residential car parking with frequent change (eg hospitals, retail), all roads except low traffic roads and trunk roads/motorways[1]	Medium	0.7	0.6	0.7
Sites with heavy pollution (eg haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemicals and fuels (other than domestic fuel oil) are to be delivered, handled, stored, used or manufactured; industrial sites; trunk roads and motorways[1]	High	0.8[2]	0.8[2]	0.9[2]

Notes

1 Motorways and trunk roads should follow the guidance and risk assessment process set out in Highways Agency (2009).

2 These should only be used if considered appropriate as part of a detailed risk assessment – required for all these land use types (Table 4.3). When dealing with high hazard sites, the environmental regulator should first be consulted for pre-permitting advice. This will help determine the most appropriate approach to the development of a design solution.

Where a site land use falls outside the defined categories, the indices should be adapted (and agreed with the drainage approving body) or else the more detailed risk assessment method should be adopted.

Where nutrient or bacteria and pathogen removal is important for a particular receiving water, equivalent indices should be developed for these pollutants (if acceptable to the drainage approving body) or the risk assessment method adopted.

Once the level of risk has been selected, the indices for the pollutants are confirmed. Appropriate pollution control measures are selected. These are shown in Table 26.3 below.

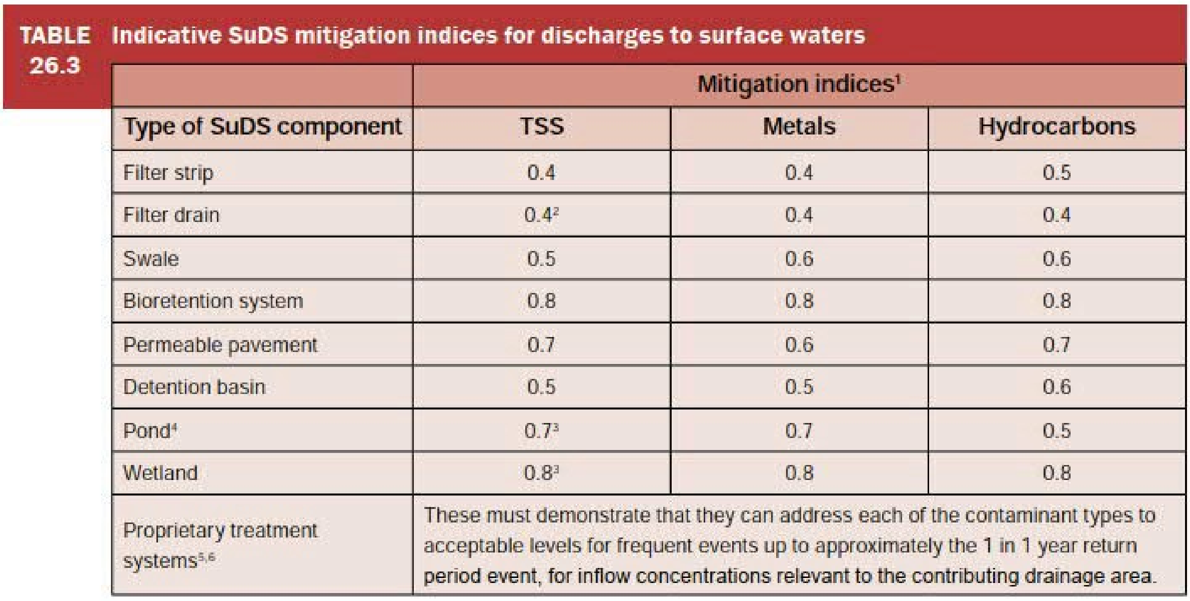


TABLE 26.3 Indicative SuDS mitigation indices for discharges to surface waters

	Mitigation indices[1]		
Type of SuDS component	**TSS**	**Metals**	**Hydrocarbons**
Filter strip	0.4	0.4	0.5
Filter drain	0.4[2]	0.4	0.4
Swale	0.5	0.6	0.6
Bioretention system	0.8	0.8	0.8
Permeable pavement	0.7	0.6	0.7
Detention basin	0.5	0.5	0.6
Pond[4]	0.7[3]	0.7	0.5
Wetland	0.8[3]	0.8	0.8
Proprietary treatment systems[5,6]	These must demonstrate that they can address each of the contaminant types to acceptable levels for frequent events up to approximately the 1 in 1 year return period event, for inflow concentrations relevant to the contributing drainage area.		

Each measure is assigned an indice. If only one measure is used. then the indice for that measure is applied. Providing the Table 26.3 indices for each pollutant are equal or greater than those stated in Table 26.2 then the measure is considered to provide appropriate mitigation. If the value is less, then additional treatment measures are required. However, for each additional measure the mitigation indices values are divided by two.

It should be noted that Indices are not provided for Proprietary Treatment Systems. These be obtained from the manufacturer/supplier.

Application of SIA to NP&R

Based on Land Use descriptions it is considered that NP&R has a medium pollution hazard level.

Pollution Hazard Level	Total Suspended Solids	Metals	Hydrocarbons
Medium	0.7	0.6	0.7

For the main site, the proposed drainage infrastructure which removes the surface water runoff and can mitigate pollutants consists in order use of the following

Gullies and linear channels
Catchpit manholes
Bypass Separators
Permeable Pavement
Attenuation Basins
Swales

Regulators will often decline to recognise the use of gullies and catchpit manholes on the basis that whilst they will settle out solids and hold back liquids, everything can be remobilised during follow on more intense rainfall events. Therefore, no contribution to mitigation indices has been considered for NP&R.

Based on available information and consultation with supplier, mitigation indices for Bypass Separators have been obtained as below. Indices for the surface infrastructure are taken from Table 26.3 and reproduced below

Infrastructure	Total Suspended Solids	Metals	Hydrocarbons
Bypass Separator	0.4	0.4	0.8
Permeable Paving	0.7	0.6	0.7
Attenuation Basin	0.5	0.5	0.6
Swale	0.5	0.6	0.6

Applying these values to the DCO design would give a total mitigation indices result as shown below for the impermeable roads and parking areas

Pollution Hazard Level	Total Suspended Solids	Metals	Hydrocarbons
Medium	0.7	0.6	0.7
Mitigation	0.9	0.95	>0.95

Applying these values to the DCO design would give a total mitigation indices result as shown below for the permeable parking areas

Pollution Hazard Level	Total Suspended Solids	Metals	Hydrocarbons
Medium	0.7	0.6	0.7
Mitigation	>0.95	>0.95	>0.95

This demonstrates that the DCO drainage design for the main site does provide sufficient mitigation.

For the offsite A12 roundabout and its spurs the proposed drainage infrastructure which removes the surface water runoff and can mitigate pollutants consists in order use of the following

Gullies and linear channels
Catchpit manholes
Attenuation Basins
Swales

Regulators will often decline to recognise the use of gullies and catchpit manholes on the basis that whilst they will settle out solids and hold back liquids, everything can be remobilised during follow on more intense rainfall events. Therefore, no contribution to mitigation indices has been considered for NP&R.

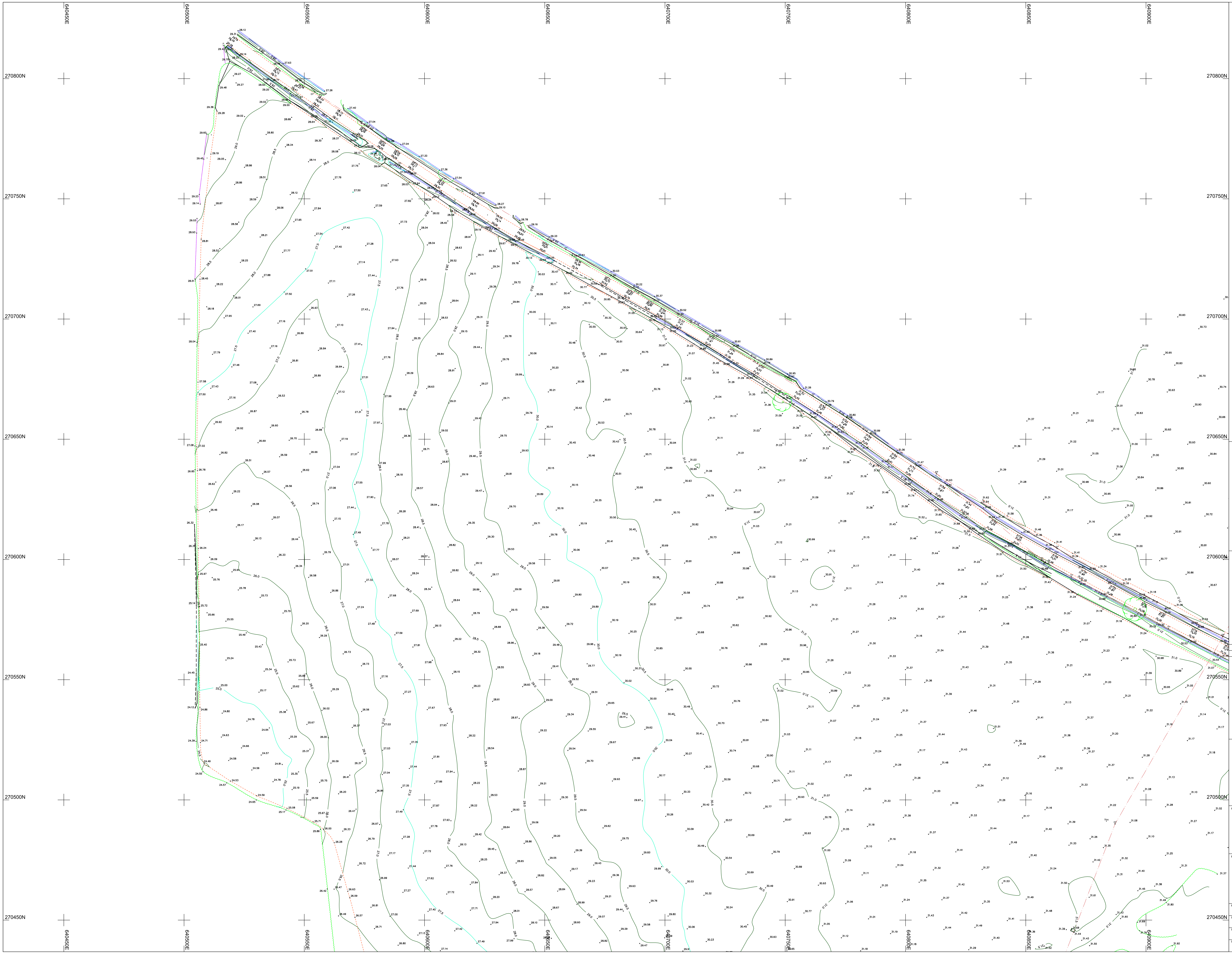
Applying the swale and attenuation basin values to the DCO design would give a total mitigation indices result as shown below

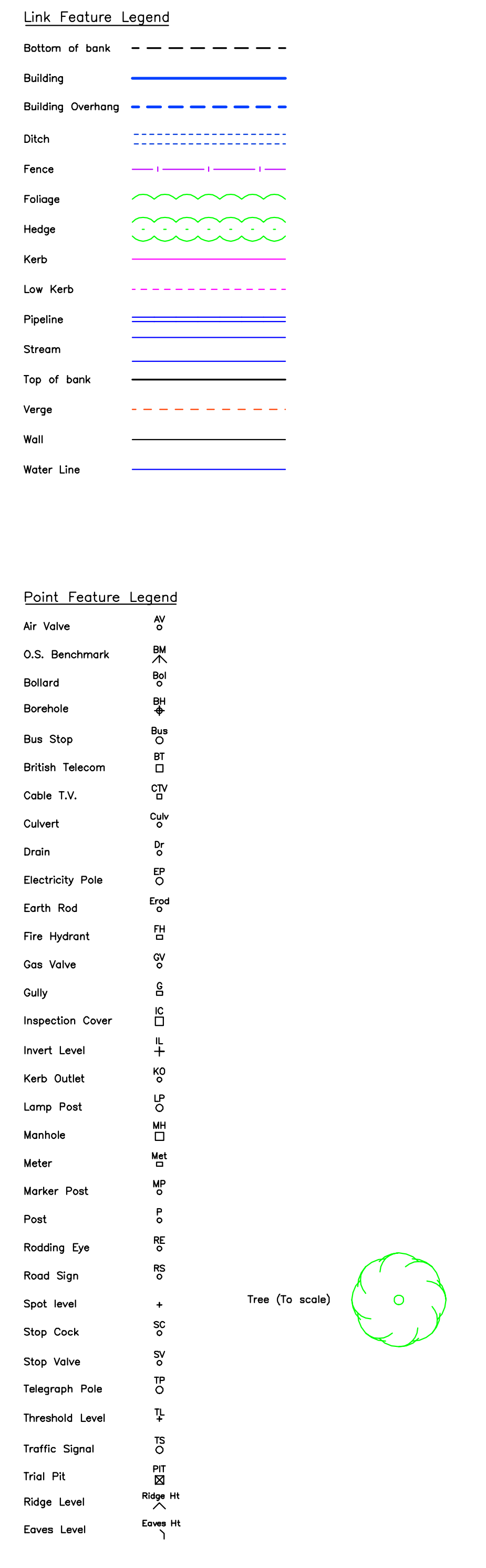
Pollution Hazard Level	Total Suspended Solids	Metals	Hydrocarbons
Medium	0.7	0.6	0.7
Mitigation	0.75	0.8	0.9

Conclusion

The SIA calculations demonstrate that for both the main site and the external A12 Roundabout, the sum of mitigation indices exceeds the Land Use Pollution Hazard indices. This demonstrates that the proposed treatment train infrastructure is sufficient to mitigate pollution risk to a low level such that no additional measures are required.

APPENDIX F: EXISTING TOPOGRAPHY





SURVEY IS ROTATED TO GRID NORTH.

Notes:

Horizontal Control points are relative to the NATIONAL GRID.

All levels are relative to ORDNANCE DATUM.

RTK corrections were obtained using the Trimble VRS Active Network.

CONTROL STATION COORDINATES

STN	EASTING	NORTHING	LEVEL
1	641125.267	270660.322	29.056
2	641045.690	270517.585	31.223
3	641189.808	270796.113	27.835
4	640578.741	269811.108	24.330
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6	640710.303	269963.040	28.712

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CHARTERED SURVEYORS
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Ballot Road
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KA12 OHW
E-mail: mail@aspectsurveys.com
Web: www.aspectsurveys.com

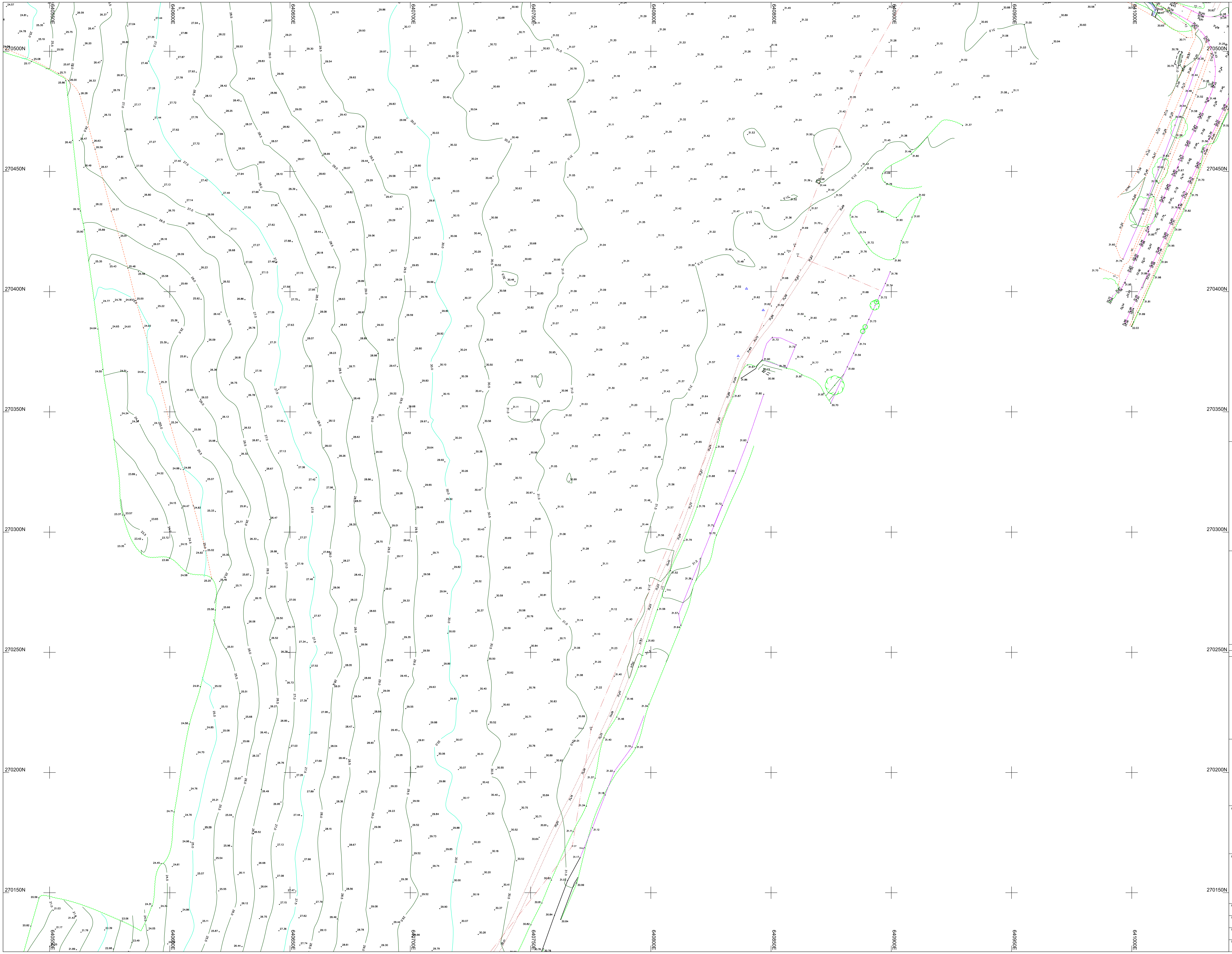
Client :

FUGRO GEOSERVICES LTD
ARMSTRONG HOUSE
UNIT 43, NUMBER ONE INDUSTRIAL ESTATE
MEDOMSLEY ROAD
CONSETT
DH8 6TW

Project Title :

TOPOGRAPHIC SURVEY
NORTHERN PARK AND RIDE SITE
SIZEWELL C POWER STATION
SUFFOLK

Project No: A7769	Scale : 1:500
Surveyed date : 13th July 2021	Issued date : 28th July 2021
Surveyed by : AJS	Checked by: RM
Sheet No. : 2 of 4	Plot Scale: 1:1 @ A0



Link Feature Legend

Bottom of bank	— — —
Building	━━━
Building Overhang	━ ━ ━
Ditch	- - - -
Fence	—‖—‖—
Foliage	︵︵︵
Hedge	︵︵︵
Kerb	———
Low Kerb	- - - -
Pipeline	═══
Stream	═══
Top of bank	———
Verge	- - - -
Wall	———
Water Line	———

Point Feature Legend

Air Valve	AV
O.S. Benchmark	BM
Bollard	Bol
Borehole	BH
Bus Stop	Bus
British Telecom	BT
Cable T.V.	CTV
Culvert	Culv
Drain	Dr
Electricity Pole	EP
Earth Rod	Erod
Fire Hydrant	FH
Gas Valve	GV
Gully	G
Inspection Cover	IC
Invert Level	IL
Kerb Outlet	KO
Lamp Post	LP
Manhole	MH
Meter	Met
Marker Post	MP
Post	P
Rodding Eye	RE
Road Sign	RS
Spot level	+
Stop Cock	SC
Stop Valve	SV
Telegraph Pole	TP
Threshold Level	TL
Traffic Signal	TS
Trial Pit	PIT
Ridge Level	Ridge Ht
Eaves Level	Eaves Ht

Tree (To scale)

SURVEY IS ROTATED TO GRID NORTH.

Notes:

Horizontal Control points are relative to the NATIONAL GRID.

All levels are relative to ORDNANCE DATUM.

RTK corrections were obtained using the Trimble VRS Active Network.

CONTROL STATION COORDINATES

STN	EASTING	NORTHING	LEVEL
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SUFFOLK

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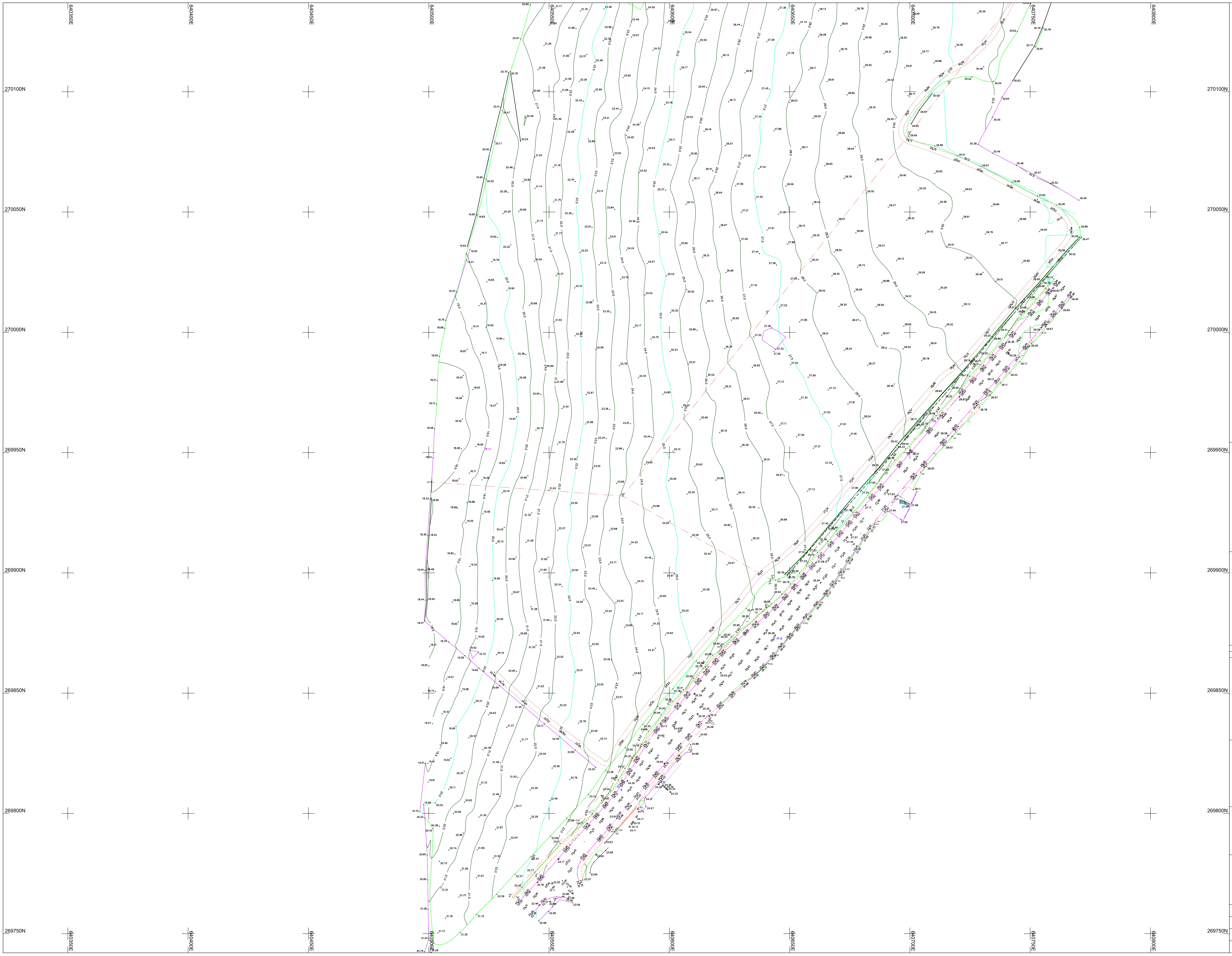


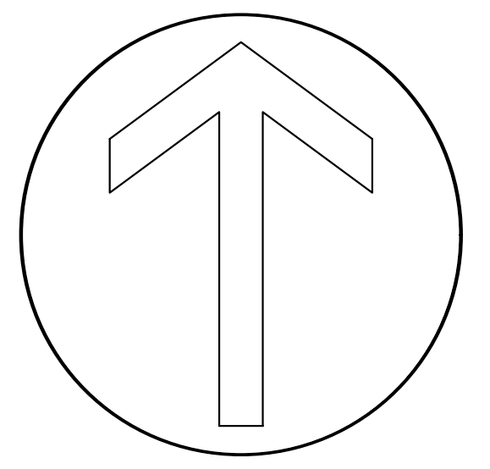
Link Feature Legend

- Bottom of bank
- Building
- Building Overhang
- Ditch
- Fence
- Foliage
- Hedge
- Kerb
- Low Kerb
- Pipeline
- Stream
- Top of bank
- Verge
- Wall
- Water Line

Point Feature Legend

Air Valve	AV
O.S. Benchmark	BM
Bollard	Bol
Borehole	BH
Bus Stop	Bus
British Telecom	BT
Cable T.V.	CTV
Culvert	Culv
Drain	Dr
Electricity Pole	EP
Earth Rod	Erod
Fire Hydrant	FH
Gas Valve	GV
Gully	G
Inspection Cover	IC
Invert Level	IL
Kerb Outlet	KO
Lamp Post	LP
Manhole	MH
Meter	Met
Marker Post	MP
Post	P
Rodding Eye	RE
Road Sign	RS
Spot level	+
Stop Cock	SC
Stop Valve	SV
Telegraph Pole	TP
Threshold Level	TL
Traffic Signal	TS
Trial Pit	PIT ☒
Ridge Level	Ridge Ht
Eaves Level	Eaves Ht

Tree (To scale)



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Surveyed by : AJS	Checked by: RM
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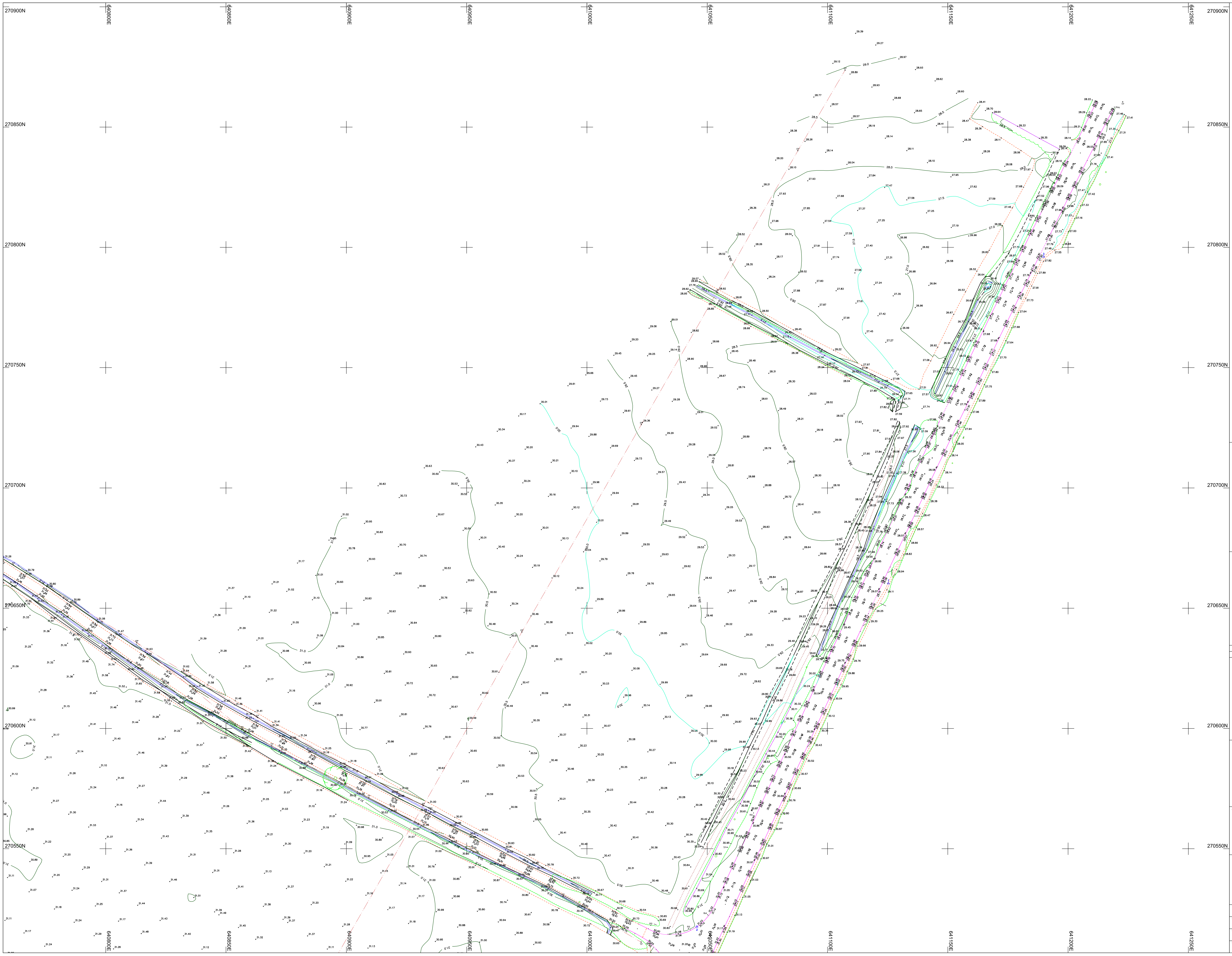

Link Feature Legend

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- Building
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Threshold Level	TL
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Trial Pit	PIT
Ridge Level	Ridge Ht
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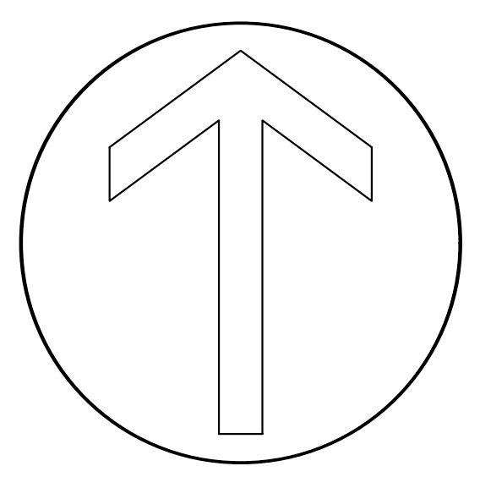
Tree (To scale)



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APPENDIX G: RECORD OF SCC COMMENTS AND SZC ACTIONS

SCC Comments on Rev 01	SZC Response
7.1.10 – Basin depth and maximum water depth would leave freeboard <300mm, but I note you have additional space available	
10.1.6 – Please note that length of culverting should be minimised through good design	
11.1.6 & 11.1.7 – Provide greenfield runoff calcs to support stated rates	Greenfield calcs and basin dimensions to be clarified. Storage areas to be clarified.
11.1.7 – Whilst SCC guidance does permit discharge at 1:100, we prefer Qbar. If you want to use 1:100, you need to implement the Long-Term Storage method to manage additional runoff volume. Not quite as simple as simply matching 1:100 rate.	
July 2021 testing – I note the test which achieved infiltration was at significant depth so wouldn't be accepted anyway. Happy to proceed on the basis the site has no infiltration	
Appendix B – Main Site – OK, especially given no storage in permeable surfacing has been accounted for	
A12 – At 16.2l/s discharge, you need 1,063m3 storage but have only demonstrated 800m3. As per earlier comment, your discharge rate would be less than 16.2l/s using LTS so your attenuation requirement will be larger than stated. Whilst I appreciate the area marked red could be available for storage, I can't estimate how much storage this would provide. Current design would result in flooding to the A12 in excess of 200m3 which we would regard as	

significant - @Steve Merry FYI	
Appendix E - 5l/s discharge rate for A12 should be amended based on above	
Tables refer to a high pollution hazard level when medium has been selected	
A12 will need HEWRAT assessment but I don't think this will be a problem given the proposed stages of treatment so content to leave that for detailed design	