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 10 of 12

Revision: 2.0

April 2022

APPENDIX C: GREEN RAIL ROUTE EAST OF ABBEY ROAD SOURCE CONTROL CALCULATIONS

Summary of Results for 100 year Return Period (+20%)

Half Drain Time : 257 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	7.211	1.011	13.5	288.8	O K
30 min Summer	7.309	1.109	15.0	327.2	O K
60 min Summer	7.398	1.198	16.4	363.8	O K
120 min Summer	7.460	1.260	17.5	390.0	O K
180 min Summer	7.471	1.271	17.6	394.8	O K
240 min Summer	7.474	1.274	17.7	396.3	O K
360 min Summer	7.472	1.272	17.7	395.5	O K
480 min Summer	7.462	1.262	17.5	391.0	O K
600 min Summer	7.446	1.246	17.2	384.4	O K
720 min Summer	7.429	1.229	16.9	376.7	O K
960 min Summer	7.382	1.182	16.2	357.1	O K
1440 min Summer	7.294	1.094	14.8	321.1	O K
2160 min Summer	7.182	0.982	13.0	277.9	O K
2880 min Summer	7.093	0.893	11.7	245.4	O K
4320 min Summer	6.946	0.746	9.5	194.8	O K
5760 min Summer	6.843	0.643	8.1	162.2	O K
7200 min Summer	6.767	0.567	7.0	139.4	O K
8640 min Summer	6.708	0.508	6.2	122.1	O K
10080 min Summer	6.661	0.461	5.6	109.1	O K
15 min Winter	7.356	1.156	15.8	346.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	184.621	0.0	26
30 min Summer	106.552	0.0	39
60 min Summer	61.496	0.0	66
120 min Summer	35.492	0.0	122
180 min Summer	25.733	0.0	164
240 min Summer	20.484	0.0	194
360 min Summer	14.851	0.0	258
480 min Summer	11.822	0.0	326
600 min Summer	9.905	0.0	396
720 min Summer	8.571	0.0	464
960 min Summer	6.770	0.0	598
1440 min Summer	4.855	0.0	864
2160 min Summer	3.482	0.0	1252
2880 min Summer	2.750	0.0	1620
4320 min Summer	1.927	0.0	2348
5760 min Summer	1.497	0.0	3104
7200 min Summer	1.231	0.0	3824
8640 min Summer	1.049	0.0	4512
10080 min Summer	0.917	0.0	5248
15 min Winter	184.621	0.0	25

Summary of Results for 100 year Return Period (+20%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
30 min Winter	7.466	1.266	17.6	392.7	O K
60 min Winter	7.566	1.366	19.2	437.6	O K
120 min Winter	7.638	1.438	20.5	471.2	O K
180 min Winter	7.653	1.453	20.7	478.5	O K
240 min Winter	7.650	1.450	20.7	477.0	O K
360 min Winter	7.641	1.441	20.5	472.8	O K
480 min Winter	7.620	1.420	20.2	462.6	O K
600 min Winter	7.592	1.392	19.7	449.7	O K
720 min Winter	7.562	1.362	19.2	435.9	O K
960 min Winter	7.493	1.293	18.0	404.7	O K
1440 min Winter	7.367	1.167	15.9	350.8	O K
2160 min Winter	7.215	1.015	13.5	290.1	O K
2880 min Winter	7.097	0.897	11.7	246.7	O K
4320 min Winter	6.915	0.715	9.1	185.0	O K
5760 min Winter	6.796	0.596	7.4	147.8	O K
7200 min Winter	6.711	0.511	6.3	123.1	O K
8640 min Winter	6.648	0.448	5.4	105.5	O K
10080 min Winter	6.599	0.399	4.8	92.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
30 min Winter	106.552	0.0	39
60 min Winter	61.496	0.0	66
120 min Winter	35.492	0.0	120
180 min Winter	25.733	0.0	174
240 min Winter	20.484	0.0	200
360 min Winter	14.851	0.0	276
480 min Winter	11.822	0.0	352
600 min Winter	9.905	0.0	426
720 min Winter	8.571	0.0	498
960 min Winter	6.770	0.0	642
1440 min Winter	4.855	0.0	914
2160 min Winter	3.482	0.0	1304
2880 min Winter	2.750	0.0	1684
4320 min Winter	1.927	0.0	2428
5760 min Winter	1.497	0.0	3168
7200 min Winter	1.231	0.0	3896
8640 min Winter	1.049	0.0	4592
10080 min Winter	0.917	0.0	5344

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 647450 264900 TM 47450 64900
C (1km)	-0.020
D1 (1km)	0.299
D2 (1km)	0.272
D3 (1km)	0.215
E (1km)	0.311
F (1km)	2.506
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.568
Cv (Winter)	0.680
Shortest Storm (mins)	15
Longest Storm (mins)	10080
Climate Change %	+20

Time Area Diagram

Total Area (ha) 1.143

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	0.381	4	8	0.381	8	12	0.381

Model Details

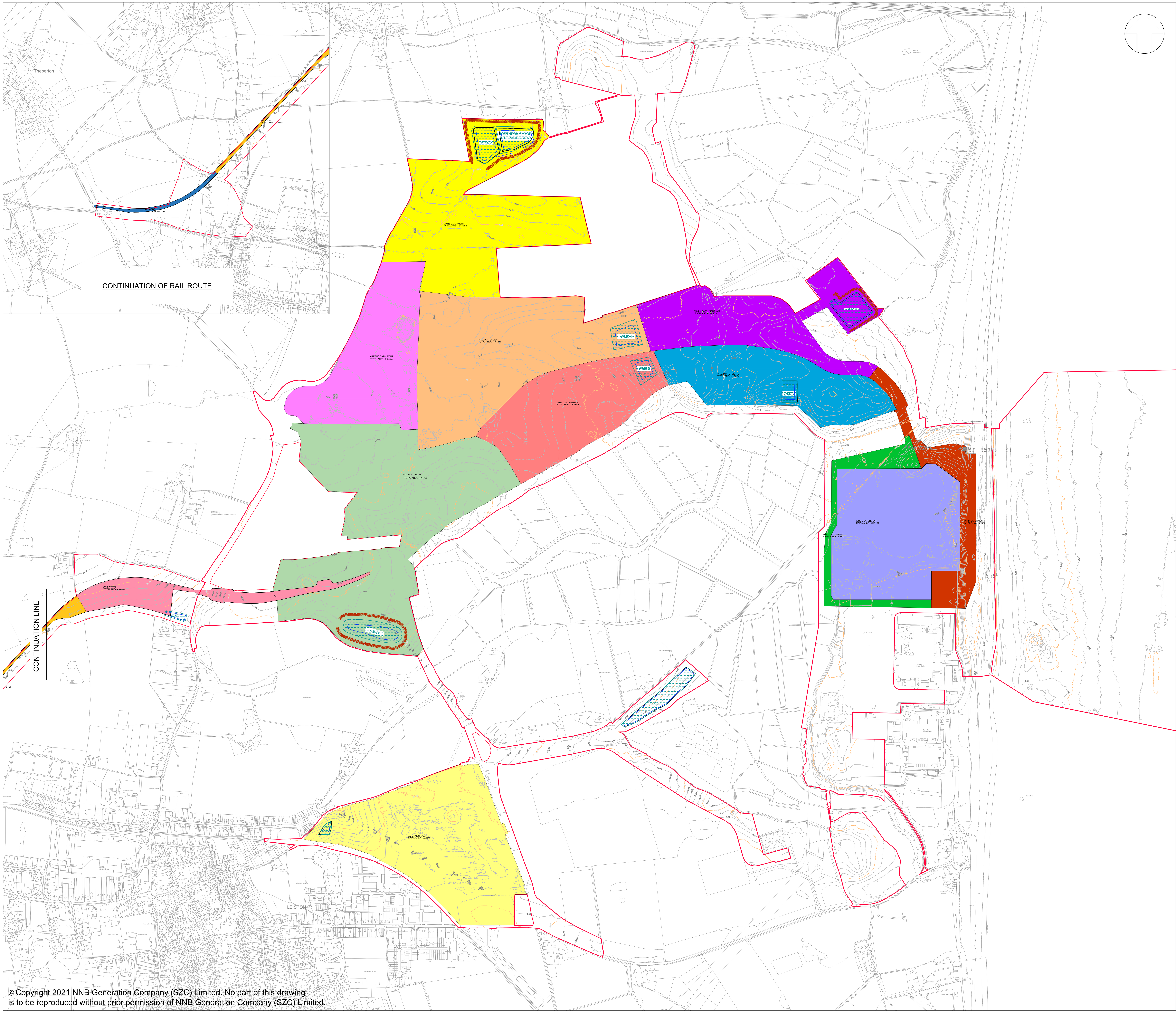
Storage is Online Cover Level (m) 8.000

Infiltration Basin Structure

Invert Level (m) 6.200 Safety Factor 1.5
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 1.00
Infiltration Coefficient Side (m/hr) 0.38160

Depth (m)	**Area (m^2)**	**Depth (m)**	**Area (m^2)**
0.000	200.0	1.500	489.2

ANNEX 2A.13: COMPARISON OF MDS BASELINE TOPOGRAPHY AND WMZ CATCHMENTS



NOTES:

KEY:

NOT PROTECTIVELY MARKED

SizewellC
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DOCUMENT:

Annex 2A.13: Comparison of MDS Baseline Topography and WMZ Catchments

DRAWING TITLE:

SIZEWELL C - TOPOGRAPHICAL PLAN WITH WMZ OVERLAY

DRAWING NO:

SZC-EW0310-ATK-XX-000-XXXXXX-DRW-CIV-000019

DATE:	DRAWN:	SCALE:	REV:
OCT2021	BM	1:5000@A0	01

SCALE BAR:

ANNEX 2A.14: TEMPORARY MARINE OUTFALL OPERATION SUMMARY

CONTENTS

FIGURES

1 INTRODUCTION

1.1.1 This document provides a summary of the proposal to discharge surface water as a precautionary principle via a temporary outfall to the sea prior to the completion of the Sizewell C (SZC) Combined Drainage Outfall (CDO). The temporary outfall will be used as a redundancy feature as a back up to the surface water proposals that follow standard sustainable drainage (SuDS) guidance. This note describes why the temporary marine outfall is required and under what circumstances it will be used.

1.2 Background

1.2.1 The Sizewell C Main Development Site (MDS) contains the Sizewell Marshes Site of Special Scientific Interest (SSSI), which is an environmentally sensitive marshland and contains a watercourse known as the Leiston Drain and the Sizewell Drain. The Sizewell Drain runs diagonally across the north-west corner of the Sizewell C Main Construction Area (MCA), before joining the Leiston Drain (shown in Figure 1-1). The watercourse heads north approximately 1.7km towards the Minsmere Sluice before discharging to the sea via a level controlling structure.

1.2.2 The Sizewell Drain needs to be realigned to pass along the western edge of the proposed MCA and connect to the Leiston Drain. Figure 1-2 shows the indicative alignment of the realigned Sizewell Drain. During construction of the MDS and prior to the completion of the CDO, management of surface water run-off and discharge is required to prevent flooding of the site and any adverse effects on the nearby ecology.

Figure 1-1- Existing Leiston and Sizewell Drains – Extract from SZC Water Framework Directive Compliance Assessment Report Part 2 Figure 2.7

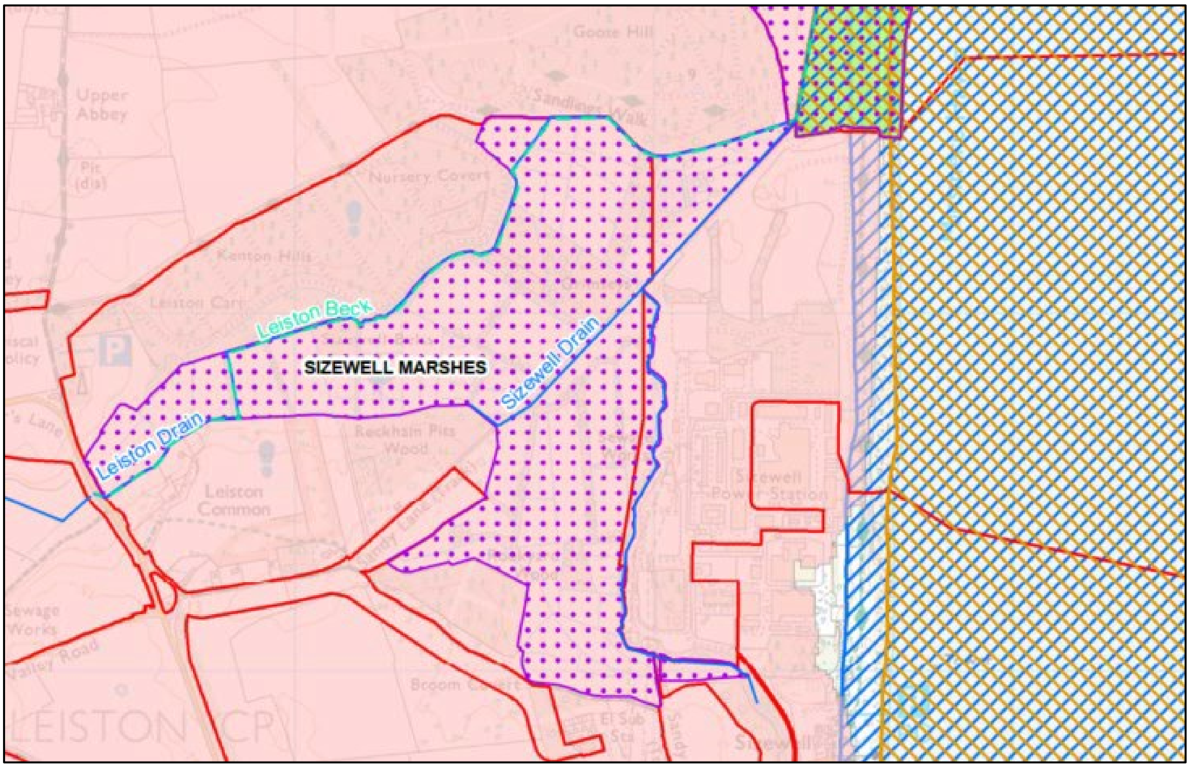
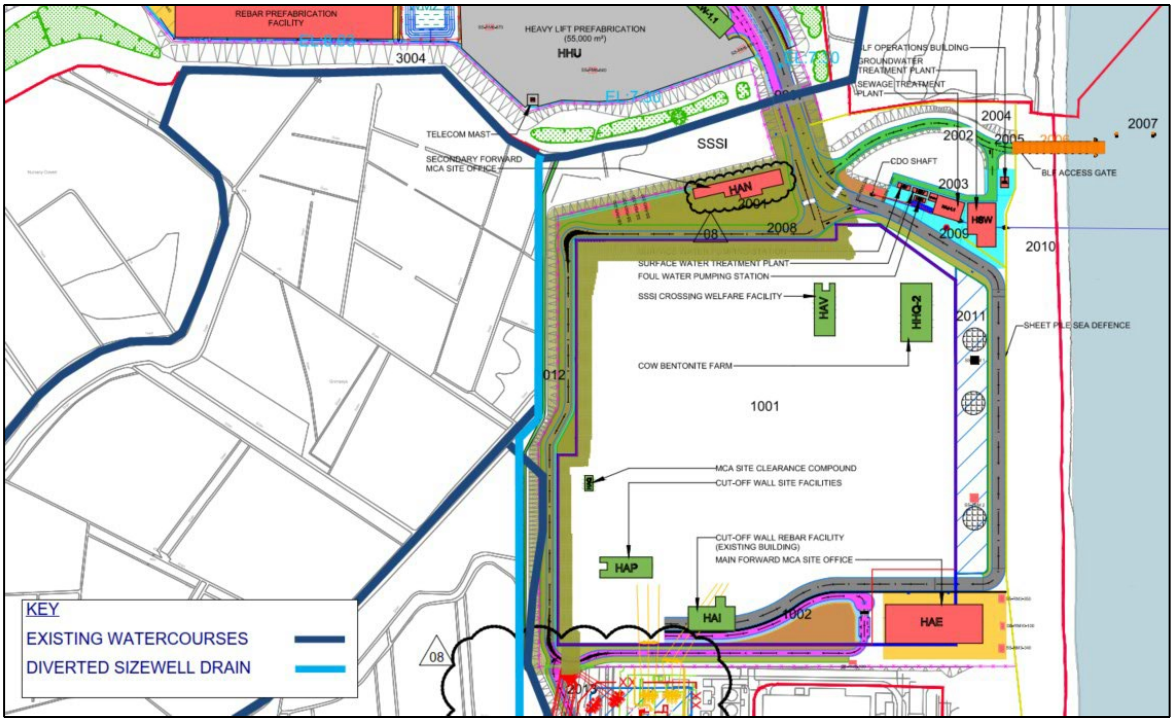


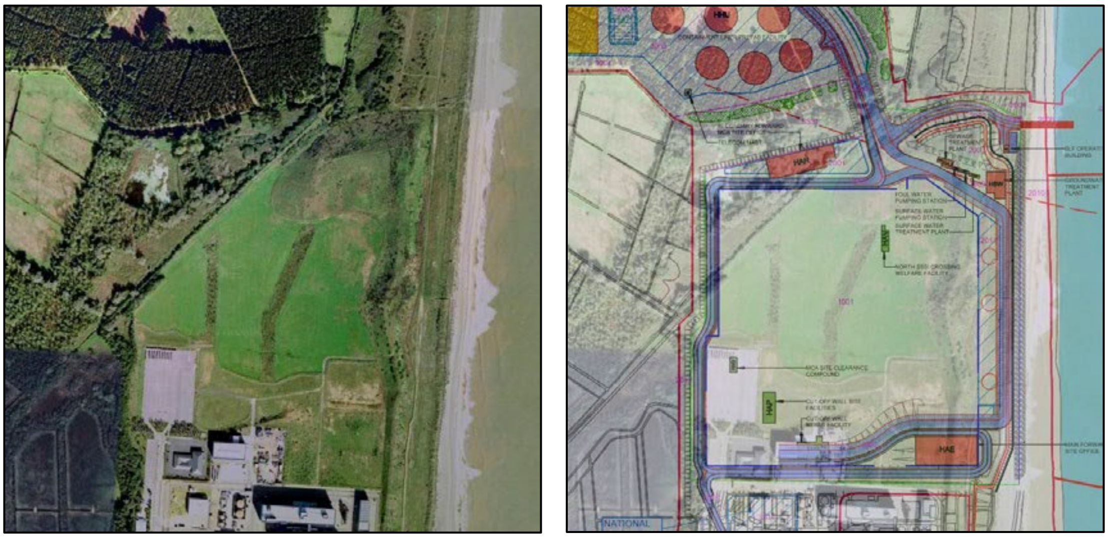
Figure 1-2 - Indicative Surface Watercourses – Existing and Diverted Sizewell Drain



1.3 Existing Site

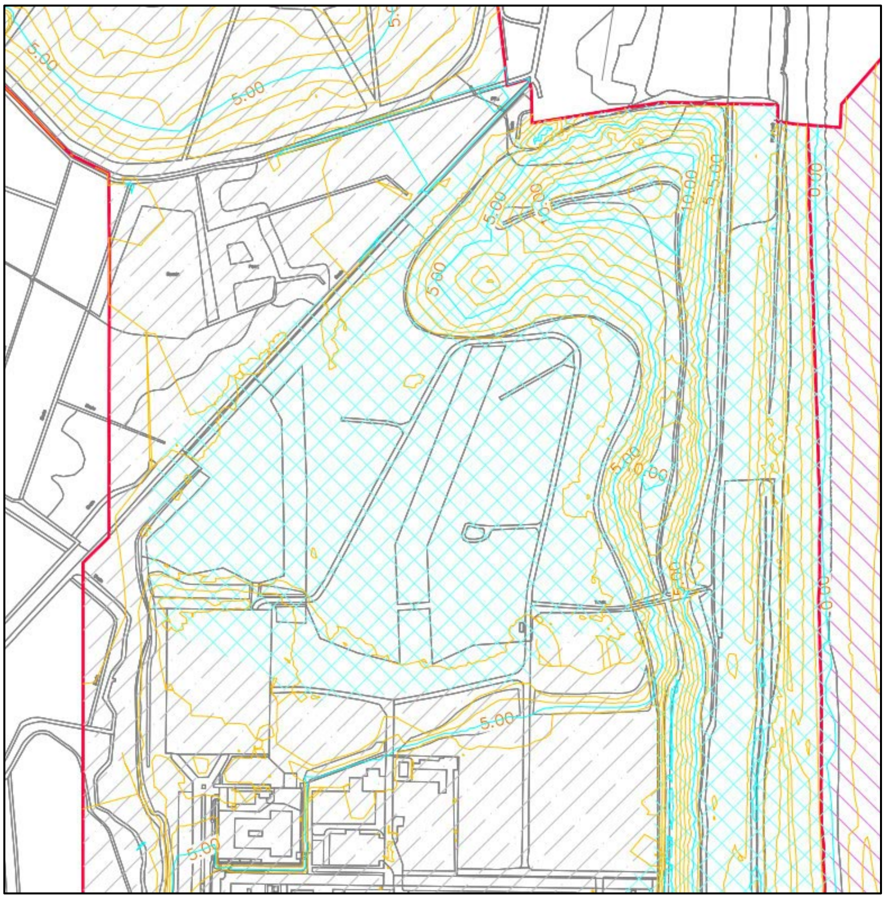
1.3.1 The existing site is largely grassland. The section of the MCA to the north-west of the Sizewell Drain makes up part of the SSSI. The area to the south and east of the Sizewell Drain includes grassland as well as some buildings and hardstanding. Figure 1-3 shows an aerial photograph of the existing site as well as a proposed construction site overlay.

Figure 1-3 - Existing Site (left) with transparent construction site overlay (right)



1.3.2 All levels given in this Technical note are designed finished levels including the existing site drains by a combination of infiltration as well as overland flow towards the Sizewell Drain. The existing site contours are shown in Figure 1-4.

Figure 1-4 - Existing Site Contours



2 SURFACE WATER MANAGEMENT

2.1 Overview

2.1.1 When construction commences on the MDS, surface water must be managed so that a storm event with a return period of 1:100 years including an allowance for 20% climate change does not leave the site. Surface water will be captured and retained on site so that it can be treated and then discharged either through infiltration or to a suitable location at pre-agreed flow rates.

2.1.2 The approximate catchment shown in Figure 2-1 needs to be allowed for in the early surface water management proposals for the MCA when earthworks commence on site.

Figure 2-1 - Approximate surface water catchment area

2.1.3 The collection of surface water across the MCA will be designed to suit the sequence of construction events. Surface water will be collected and held in temporary attenuation ponds within the MCA, before being treated using proprietary devices if required.

2.1.4 Similarly, surface water runoff within the Temporary Construction Area (TCA), north of the Sizewell Marshes SSSI, will be collected, attenuated, treated, and discharged to ground or local watercourses under normal conditions.

2.2 Surface Water Discharge

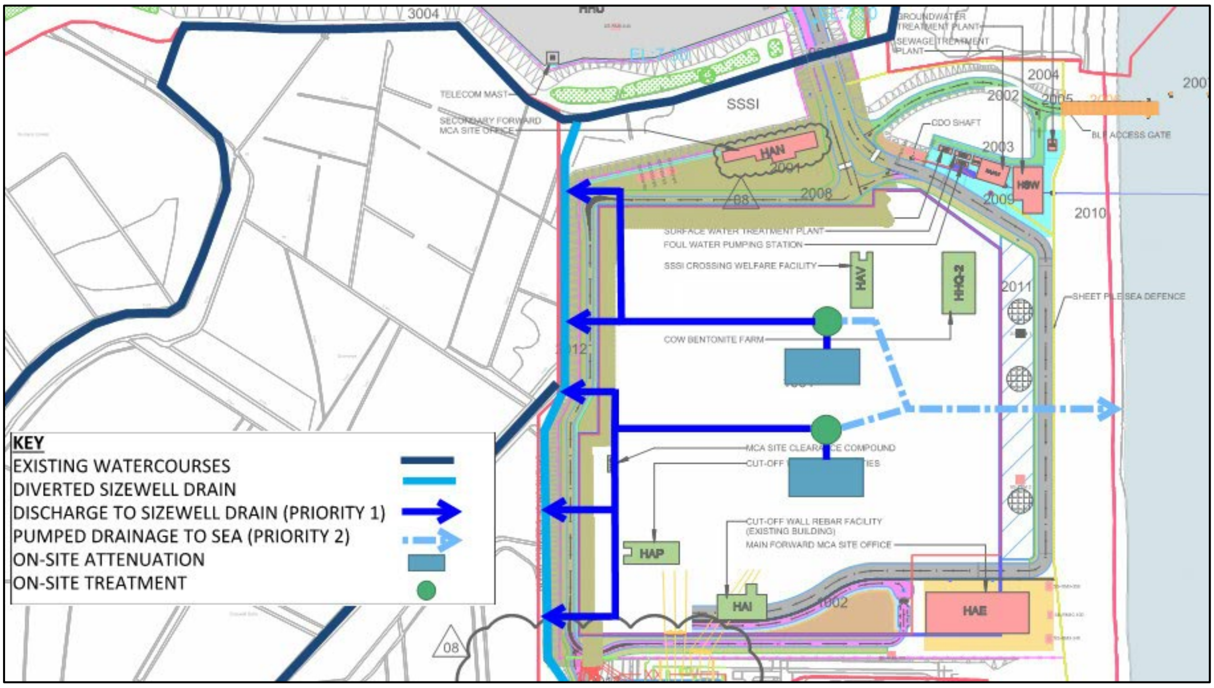
2.2.1 It is important to mimic existing conditions on the site to ensure the SSSI water levels, including ground water, are not affected by the construction works. This means surface water will discharge primarily via infiltration, using temporary infiltration ponds and other Sustainable Drainage (SuDS) features where possible.

2.2.2 Temporary infiltration ponds within the MCA will have outfalls discharging to the Sizewell Drain if infiltration alone is not sufficient to discharge surface water. The outfall locations are yet to be confirmed with the Internal Drainage Board (IDB), who are being engaged with to ensure that a discharge regime as close to existing conditions as possible is constructed. This may mean multiple discharges along the length of the existing Sizewell Drain, or on the new alignment once it has been realigned.

2.2.3 The surface water discharges to the Sizewell Drain will be restricted to greenfield runoff rates in accordance with Environment Agency (EA) guidance. The discharges will need to be permitted through a land drainage consent, with continuous monitoring to ensure flow rates do not exceed the permitted rates, and water quality meets the required treatment levels.

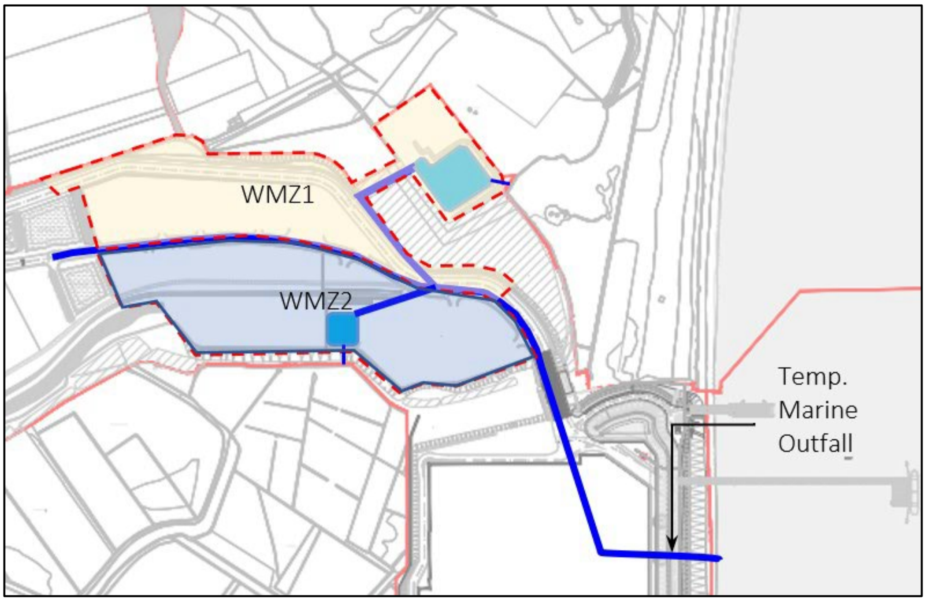
2.2.4 The Sizewell Marshes (including the Sizewell Drain) are known to flood occasionally due to either extreme rainfall events or other external factors, such as the Leiston Drain downstream being blocked or the Minsmere sluice inhibiting surface water flow to sea. In these scenarios, the Sizewell drain overtops and the low-lying areas in the SSSI become inundated with surface water. If a rainfall event occurs on the MDS while the SSSI is inundated with water, surface water runoff will be captured and attenuated in temporary infiltration ponds. However, discharging to the backed-up Sizewell Drain in these conditions is not considered suitable, even if restricted to greenfield runoff rates. In this scenario, another option for discharging surface water should be considered, and therefore a temporary marine outfall has been proposed which would discharge to sea, acting as a 'release valve'. An indication of how surface water will be discharged from the MCA is shown in Figure 2-2. The temporary marine outfall is further described in the following section.

Figure 2-2 - Schematic showing proposed discharges



2.2.5 Attenuated surface water runoff from catchments within the TCA will be discharged to the Leiston Drain at various locations if infiltration alone is not sufficient to discharge surface water. However, during the early months of site establishment of Water Management Zone (WMZ) 1 and WMZ2 when the CDO is under construction, if the site is subject to an extreme storm or inundated locally with surface water, the temporary marine outfall will be used to discharge surface water to sea. An above ground pumped network would convey surface water towards the MCA, across the SSSI and out to the sea via the temporary marine outfall, as indicated in Figure 2-3.

Figure 2-3 - Possible TCA Catchments Discharging via the Temporary Marine Outfall



2.3 Temporary Marine Outfall

2.3.1 The temporary marine outfall is proposed to be installed early in the construction programme, as a redundancy measure or a precautionary principle for discharging surface water to sea, prior to the CDO being installed. The CDO is programmed to be commissioned by 19 April 2023 as per SZC-IWS Enabling Works Construction Schedule, after which the temporary marine outfall will be removed. For a 15 month period, the temporary marine outfall would principally be used where factors external to the MDS that are out of the control of SZC result in the Sizewell Drain being unsuitable to discharge to, for example, flooding on site caused by offsite flood conditions.

2.3.2 Permitting, Operation, and Usage

2.3.3 All outfalls to the SSSI as well as the sea will be controlled through conditions imposed through the permit application procedure with the EA. This permit will be applied for in the future through the EA. The conditions from the EA may stipulate a suitable water level within the SSSI that must be reached before the temporary marine outfall can be used. Similarly, there may be a level defined by the permit conditions where the marine outfall must be switched off and discharge is returned to the SSSI for recharge of surface and groundwater. The pump may also need to be used in other exceptional events such as if water level in and around the site present a risk to health and safety.

2.3.4 Further to the above, water quality levels will be stipulated in the permitting conditions that must be met prior to discharge to sea.

2.3.5 **Proposed Arrangement**

2.3.6 The temporary marine outfall would include a pump, or series of pumps, from the MDS and discharge to a gravity pipe that would discharge to sea at mean high water springs (MHWS) level. The pressurised pipework would be installed above ground where possible however it may need to be laid below ground in places, for example, across the Suffolk Coast Path. The pipework may be restrained using hoop rings or similar, and consideration will be made to ensure the Suffolk Coastal Path remains unobstructed.

2.3.7 The proposed outfall location is shown indicatively in Figure 2-4 and Figure 2-5. It is likely the shoreline will require local erosion protection measures in the form of sandbags or other suitable protection and need to be monitored for scour issues.

Figure 2-4 - Proposed outfall location

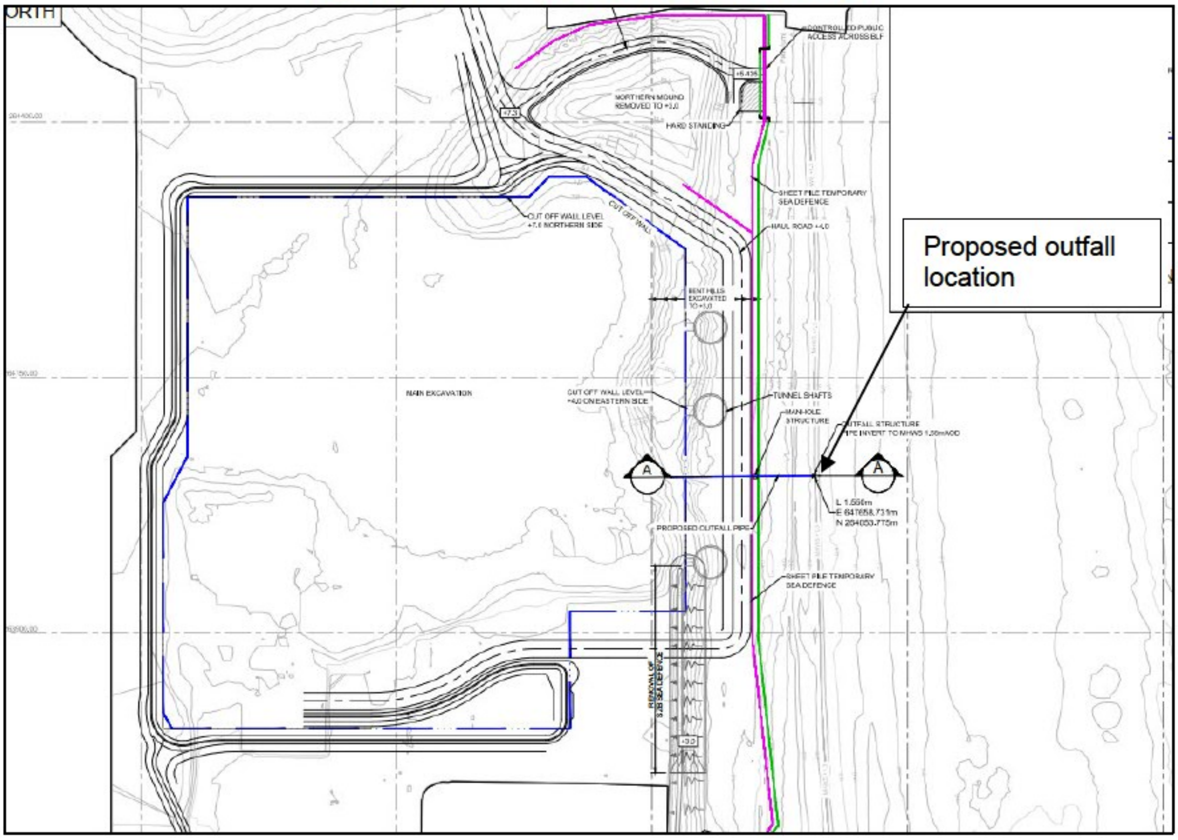
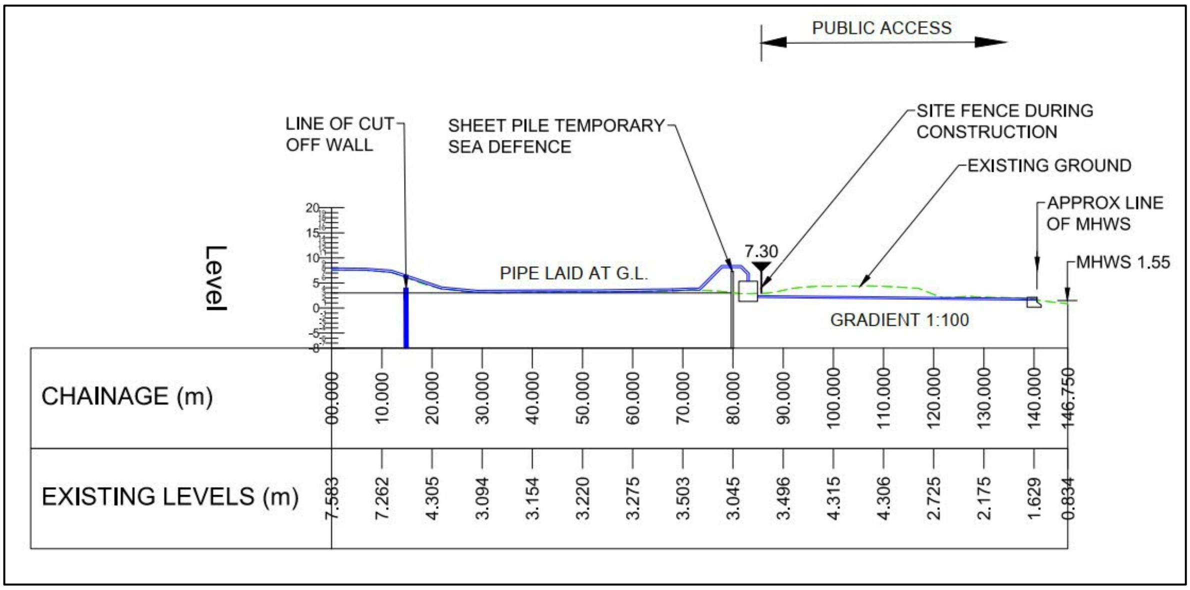


Figure 2-5 - Indicative section discharge pipe from MCA to outfall



3 SUMMARY

3.1.1 A temporary marine outfall is required prior to the installation of the CDO, to provide redundancy as a precautionary principle for discharging surface water from the MDS, if external factors mean that the Sizewell Drain is not suitable to discharge to. The outfall will be available to use for approximately 15 months, after which will be removed once the CDO is commissioned. The frequency of use depends on how these external factors coincide with rainfall events on the MDS. Further factors that may influence the use include maintenance of the Leiston Drain downstream, or surface water flooding in and around the site resulting in health and safety issues.

3.1.2 The use of the outfall would not have a significant impact on the input for surface water into the Sizewell Marshes SSSI as it would be used only when there was excess water in the SSSI. The outfall may never be used and will only be installed as a precautionary measure, to ensure that the SSSI is protected and that the construction site is still able to be operational in situations where external uncontrollable factors impact on the MDS.

3.1.3 The outfall will be controlled through conditions set by the EA under a construction water discharge activity permit.

ANNEX 2A.15: WMZ1 SURFACE WATER TREATMENT ASSESSMENT

CONTRACTOR DOCUMENT FRONT SHEET

UK PROTECT

DOCUMENT DETAILS

PROJECT		ORIGINATOR CODE		VOLUME		LOCATION		TYPE		ROLE		SEQUENTIAL NUMBER
SZC-EW0320-ATK-XX-000-XXXXXX-NOT-CCD-000006												

DOCUMENT TITLE	**WMZ1 Surface Water Treatment Assessment**	**EMPLOYER REVISION**	02

DOCUMENT STATUS	S3	**DOCUMENT PURPOSE**	S3 - Fit for Internal Review and Comment	**TOTAL PAGES (Including this page)**	15

CONTRACTOR DETAILS

CONTRACTOR NAME	Atkins Limited

ATKINS NUMBER	N/A	**CONTRACTOR REVISION**	02

ADDITIONAL INFORMATION

NNB NUMBER	N/A	**TEAMCENTER NUMBER**	

REVISION HISTORY

EMPLOYER REVISION	REVISION DATE	PREPARED BY	POSITION/TITLE	CHECKED BY	POSITION/TITLE	APPROVED BY	POSITION/TITLE
02	01/04/22	GR	Civil Engineer	MS	Water Lead	KMJ	Engineering Lead
01	01/10/21	DH	Civil Engineer	MS	Water Lead	KMJ	Engineering Lead

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CONTRACTOR DOCUMENT FRONT SHEET

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REVISION STATUS/SUMMARY OF CHANGES

Revision	Purpose	Amendment	By	Date
02	S3	P2 - Minor updates (removal of section 4 and Appendix B)	GR	01/04/22
01	S3	P2 Published for Costing	DH	01/10/21

Technical Note

Project:	SZC Enabling Works Detail Design		
Subject:	**WMZ1 Surface Water Treatment Assessment**		
Author:	GR		
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
02	S3	P2 - Minor updates (removal of section 4 and Appendix B)	GR	MS	AP	KMJ	01/04/22
01	S3	P2 Published for Costing	DH	MS	AP	KMJ	01/10/21

Client signoff

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

Glossary

C753	The SuDS Manual, published by CIRIA
CEFAS	Centre for Environment, Fisheries and Aquaculture Science
CDO	Combined Drainage Outfall
CIRIA	Construction Industry Research and Information Association
CoCP	Code of Construction Practice
CSPP	Construction Site Plot Plan
CWA	Civil Works Alliance
EDF	Electricité de France
EPR^{TM}	Trade name for reactor type proposed for SZC
HGV	Heavy Goods vehicles
HPC	Hinkley Point C
mAOD	Metres Above Ordnance Datum
MAC	Maximum Allowable Concentration
MCA	Main Construction Area
MDS	Main Development Site
NNB	Nuclear New Build (GenCo)
SIA	Simple Index Approach
SSSI	Site of Special Scientific Interest
SuDS	Sustainable Drainage Systems
SZC	Sizewell C
TBM	Tunnel Boring Machine
TCA	Temporary Construction Area
TSS	Total Suspended Solids
WMZ	Water Management Zone

1. Introduction

This technical note has been prepared to provide a summary of the surface water drainage approach in Water Management Zone 1 (WMZ 1) located within the Temporary Construction Area (TCA) of the Sizewell C Main Development Site (SZC MDS). The purpose of this document is to present a surface water treatment assessment for surface water runoff within WMZ 1 using the CIRIA C753 The SuDS Manual Simple Index Approach (SIA).

The information presented in this document is in accordance with the overarching drainage principles that are documented in the SZC Development Consent Order (DCO) Outline Drainage Strategy at Volume 2, Chapter 2, Appendix 2A of the Environmental Statement [APP-181].

This document does not provide details of treatment and discharge of water used for construction purposes (e.g. Tunnel Boring Machine (TBM) slurry treatment water).

2. WMZ 1 Catchment Overview

Water Management Zone 1 (WMZ 1) is located in the north eastern area of the proposed TCA and includes the following features and facilities:

- Haul Road
- Main Access Road
- Workshop compound
- Plant Workshop & Storage
- TBM Slurry Treatment Plant / Bentonite Farm
- Fuel Farm
- Road Sweeper Compound
- Fire & Rescue Centre
- Emergency Response Facility

The catchment encompasses sections of the site access road to the south, haul roads to the north and east, and one of the Contractor's working compounds. The catchment has a total area of 19.4 ha and is proposed to drain via combined swale and infiltration trenches with perforated pipes. Two main runs are proposed, north and south of the catchment, both running from west towards the WMZ 1 basin which is proposed to be located in the east. This is indicatively shown in Figure 2-1. It has conservatively been assumed to be 90% impermeable surface. The area of hardstanding may decrease in the future, however a more conservative value was used in the sizing the detention basin.

It is proposed to discharge surface water to the Leiston Drain tributary, east of WMZ 1 basin, at a maximum rate of 19.4 l/s (1 l/s/ha). An overflow connection is also proposed from WMZ1 basin to the Combined Drainage Outfall (CDO).

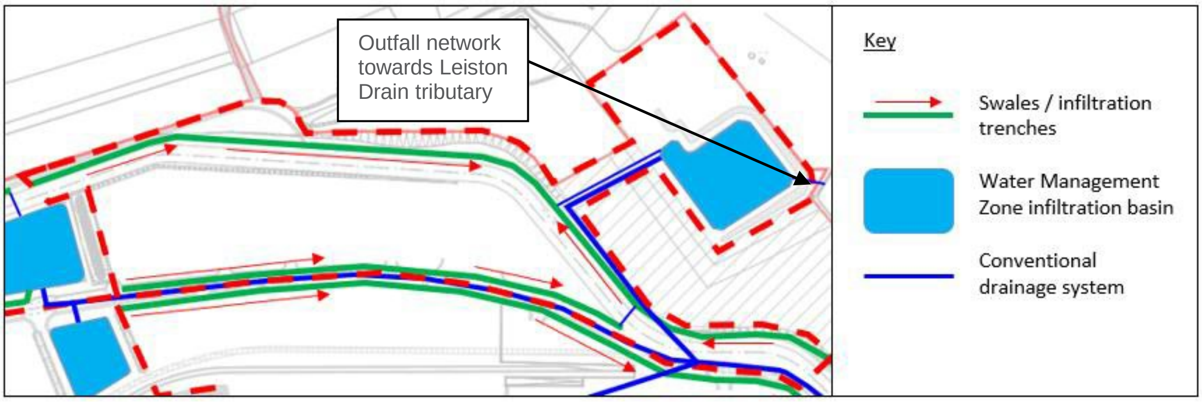


Figure 2-1 – WMZ1 surface water drainage overview

WMZ 1 is proposed to fall from west to east from approximately 11 mAOD to 2 mAOD with the contractor compound areas generally flat at approximately 8 mAOD. The WMZ1 basin is located at the low point east of the TCA and the invert is in close proximity to the groundwater table (0.3m). At this stage, the basin is proposed to be lined with an impermeable membrane and a permanent outfall is proposed from the WMZ1 basin to the nearby land drain (outfall O1). An overflow network is also proposed to discharge to the CDO in extreme circumstances via the spine network.

The groundwater contours from Winter 2018 included in the Environmental Statement show the groundwater level at the location of the WMZ 1 basin to be approximately 0.9 mAOD. Given the proximity at present, an option to raise the invert level of the basin will be considered at the next design stage, to provide a minimum 1 m separation from the groundwater level as per guidance from CIRIA C753 The SuDS Manual. This will enable the basin to discharge via infiltration, supporting the wider drainage philosophy. A treatment train including the option to infiltrate at the basin location is considered and assessed in Section 4.

3. Surface Water Treatment Design

There are several possible contaminants that need to be considered in treatment design within WMZ 1. These are largely divided into three categories:

- Sediment laden runoff
- Chemical spills (e.g. fuel farm)
- Other treatment required (e.g. Sweeper tip and TBM Slurry Treatment Plant / Bentonite Farm)

3.1. Sediment Laden Runoff

It is proposed to remove as much sediment as possible as close to its source. Sediment removal will primarily be provided through a of combination of SuDS features and conventional drainage components, which form a treatment train across the site. This concept is illustrated in Figure 3-1 below.

The following SuDS features are proposed within WMZ 1:

- Filter strips
- Swales/infiltration trenches (combined)
- Detention basin

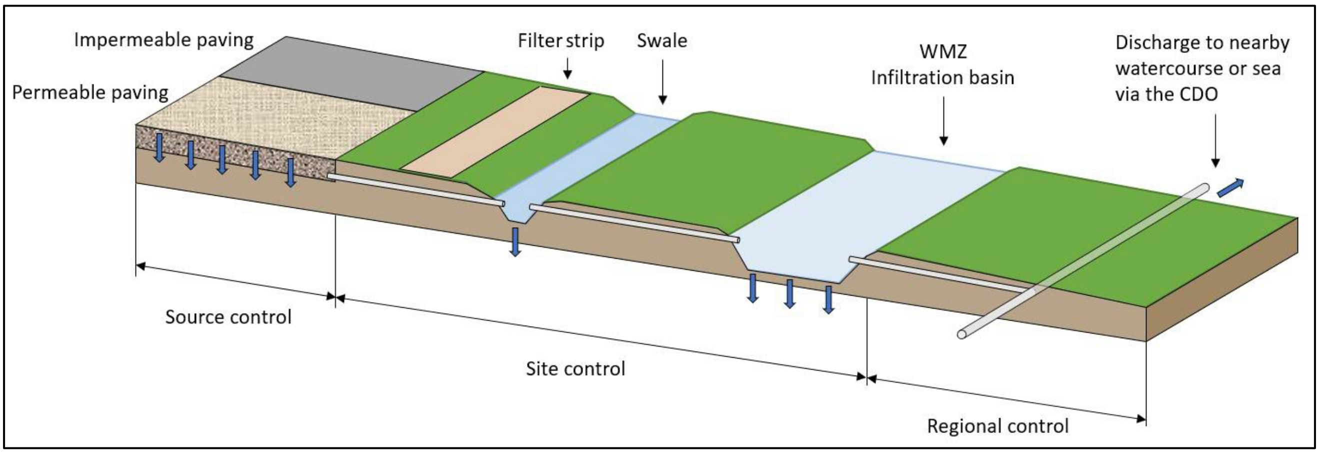


Figure 3-1 - Surface water capture, treatment, and discharge plan

3.1.1. Access Roads / Contractor Compounds

Surface water runoff will be captured to reduce silt and suspended solids through silt traps in all manholes, gullies and channels proposed within WMZ 1, and with the SuDS features listed above. The road runoff will be directed to a swale either side of the road, which will have grassed verges prior to discharge into the swale. The runoff from the contractor compounds will be directed to the swale also, via a cut-off drain and filter strip as shown in Figure 3-2. The grassed verge, cut-off drain, and filter strips will act as the primary method for silt removal. The surface water will then enter the grassed swale, which will provide secondary treatment. The water will then infiltrate through the infiltration trench into the ground. The granular material in the infiltration trench will also provide treatment.

In larger storm events, surface water will not be able to infiltrate via the infiltration trenches and will then be captured by the perforated pipe within the infiltration trench. The perforated pipe will have catchpits at changes in direction, and the catchpits will contain silt traps which can be easily maintained. The perforated pipes will convey water to the WMZ 1 basin in larger storm events, where water will be attenuated. The WMZ 1 basin will be designed to have a sediment forebay to control the spread of suspended solids and encourage further sediment removal. The design of the WMZ 1 basin will be developed during Detailed Design and will include further details of access ramps, inlet and outlet structures and maintenance regimes. Surface water will be treated and monitored prior to the outfall to ensure the concentration of total suspended solids is limited prior to entering the local watercourse. As stated above, the basin design may be modified to enable infiltration, providing further treatment, and this additional treatment potential is demonstrated in Section 4.

Further to the above, the use of road sweepers along access and haul roads can reduce the silt-build up in these areas, therefore increasing the longevity of the filter strips and swales.

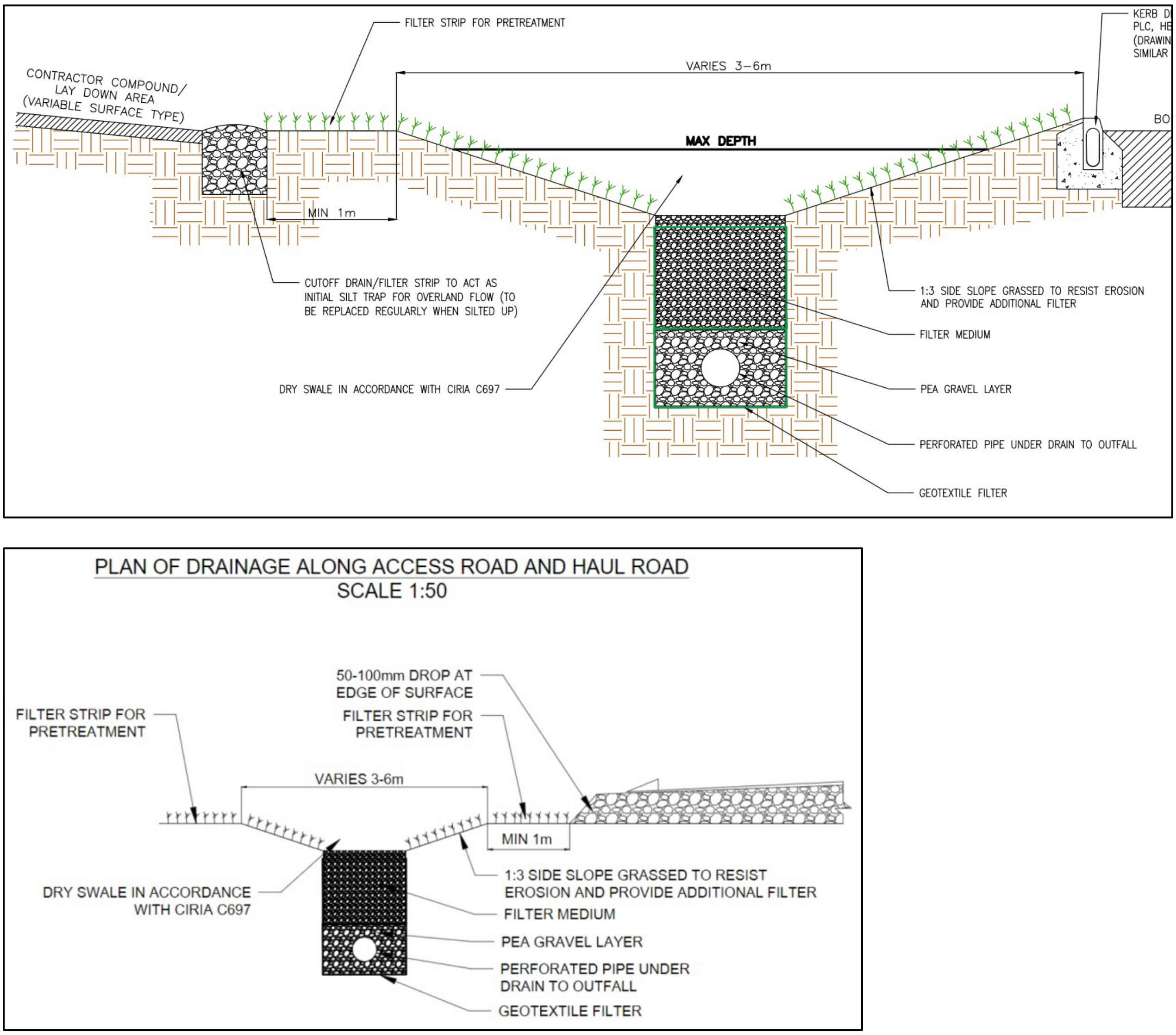


Figure 3-2 – Typical filter strip, swale and infiltration trench arrangement adjacent to Contractor Compounds and Access Roads

3.2. Oil and Chemical Spills

The permanent construction surface water drainage network has been designed with the assumption that it should not be required to treat chemical spills. Any areas considered to be at high risk of chemical contamination will be impermeable, with runoff managed by the Contractor to the Code of Construction Practice (CoCP). The Contractor will be responsible for preventing contaminated water from leaving the area and, either treated at source or cleaned up and disposed of. The use of the site over the construction period is transient, and an area once used as a contractor's compound could be used as a laydown area in future, or a truck depot, for example. The Contractor responsible for the compound area must be responsible for treatment of potential spillages. For example, if the Contractor required fuel storage on their site, they will be responsible for the installation and operation of an oil interceptor and bunding the area to ensure any spillage is captured and treated on site prior to discharge.

3.2.1. Fuel Farm Runoff

The Fuel Farm area shall have impermeable hardstanding with surface water from the forecourt draining to an oil separator. This separator will be fitted with several warning systems to prevent and detect oil spilling, overfilling tanks and vapor fumes, for example. The fuel farm area shall be bunded to allow for containment of any spillage.

3.3. Other Treatment

Other areas within the catchment which produce contaminated runoff requiring treatment are briefly outlined below.

3.3.1. TBM Slurry Treatment Plant / Bentonite Farm

The bentonite farm in the TCA is required for tunnelling, with the bentonite used to form a slurry which is then used in the tunnelling process and this slurry will need to be treated prior to discharge. This process is to be developed in Detailed Design and to be informed by the Contractor. The discharge of treated water from the slurry treatment plant will likely be conveyed to the CDO and discharged to the sea via the surface water spine network, or treated foul water network. Therefore, this area is not considered appropriate for assessment using the Simple Index Approach and not discussed further.

3.3.2. Road Sweeper Compound

Road sweepers will remove sediment from site roads to treat the sediment at source. As a result, sweeper trucks required a dedicated area to dispose of this sediment-heavy water. At Basic Design, a 900m^2 area has been allocated for the sweeper tip discharge treatment, which is a replication of the Hinkley Point C (HPC) space requirement. The treatment proposed shall include screening, coagulation, and treatment with CO_2 bubbling. Following this, it will be filter-pressed and the water discharged into the surface water network. The solid waste produced from this process will then be taken off-site.

4. Treatment Assessment

Please refer to SZC-EW0320-ATK-XX-000-XXXXXX-REP-CCD-000001 Appendix 2A.17 (Surface Water Drainage Treatment Narrative) for the latest information.

5. Maintenance

The surface water drainage design aims to minimise maintenance required on site by aiming to use gravity systems rather than pressurised systems as much as possible. However, all surface water treatment features will require an element of maintenance over time.

All surface water components are to be managed within the project by the Contractor. Regular maintenance of the surface water system will be undertaken throughout the lifecycle of the TCA. The Contractor will be required to submit a surface water operations and maintenance management plan that complies with the Code of Construction Practice (CoCP) prior to commencing construction on site.

The planned operational life of the TCA is expected to be approximately 10 years, after which it will be returned to its original greenfield condition. Filter strips, swales and detention basins will be maintained to ensure there is enough vegetation to operate as required for filtering runoff but kept cut to ensure the system is free flowing (in accordance with the CIRIA C753 The SuDS Manual). Swales and detention basins will be dredged of excess silt build up as required. The infiltration trenches may require excavation occasionally where silt build up becomes problematic.

All below ground drainage will be designed in accordance with Sewers for Adoption (7th ed.) with all allowances for access and jetting. All filter drains with internal perforated pipes will be provided with rodding eyes on the ends.

A designated maintenance management plan will be in place for the life of the development, this will be used to ensure all aspects of the drainage system are regularly maintained as regularly as deemed necessary for each drainage element. The maintenance management plan will be submitted for approval prior to construction on site.

6. Surface Water Quality

The water quality requirements adopted at HPC have been reflected and assumed to be acceptable for the SZC development. These requirements are shown in Table 6-1 below. Discussions with the Centre for Environment, Fisheries and Aquaculture Science (CEFAS) and the EA are essential to agree the water quality and monitoring requirements and inform the treatment design.

The surface water drainage and treatment design will be developed to facilitate the necessary monitoring and inspection arrangements and lead to compliance of the water quality objectives set out in the permitted activity.

Table 6-1 – Surface water quality requirements (based on HPC figures)

Criteria	Treatment Level Required at Monitoring Point	Sample Type	Notes
Visible oil or grease	No significant trace present so far as is reasonably practicable	Visual inspection	No significant trace
Suspended Solids (measured after drying at 105°C)	60mg/l (to local watercourse) 250mg/l (to sea)	Spot sample	Maximum Allowable Concentration (MAC)
pH	6 to 9	Spot sample	Minimum and maximum

7. Monitoring and Sampling

As indicated in Table 6-1, there are requirements for surface water quality and several criteria for which to measure this by. Spot samples and visual inspections will be required at specific points on the surface water network prior to discharge into the sea or watercourse. These monitoring and sampling points will be required immediately upstream of discharge points to ensure the discharged water meets the specified treatment levels. Table 7-1 outlines the location of the manhole for outfall O1. The locations and details of these sampling points will be developed in the next design phase. The outfall network will be designed to include penstock valves to isolate and control outflow in the event the discharge does not meet the quality criteria, or downstream conditions are not suitable to accept flows.

Table 7-1 – Proposed monitoring manhole location

Site	Outfall	Monitoring Manhole National Grid Reference
TCA	O1	TM 47228 64962

8. Conclusion

The SuDS Simple Index Approach has been used to assess the water quality management in the proposed surface water runoff drainage design. As per the outline drainage strategy, infiltration at source is key to the design philosophy. The SIA assessment demonstrates that the proposed SuDS features alone can provide effective capture and treatment of pollutants from surface water runoff within WMZ 1 for each discharge pathway.

There are a number of forward actions associated with further assessment of treatment of WMZ 1 that will be undertaken at the next design stage:

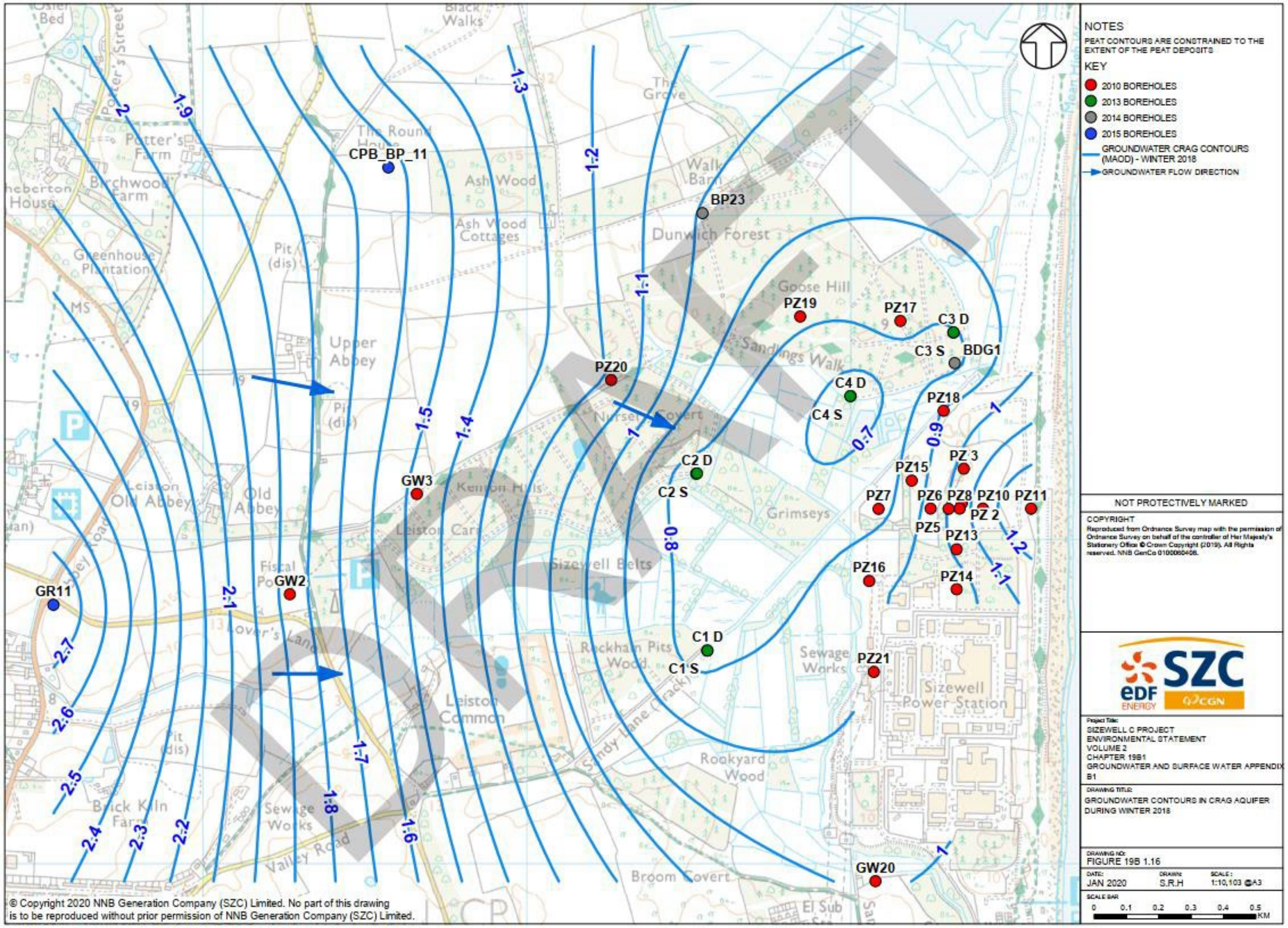
- Update to the design of WMZ 1 basin to allow for infiltration when further ground investigation campaign infiltration data is received.
- Assessment of water treatment to be completed for remaining WMZs across the MDS.

- Development of proprietary drainage methods across contractor compounds and access/haul roads, building on lessons from HPC.
- Confirm water quality requirements with CEFAS and the EA.

9. Appendices

Appendix A. Groundwater Levels

A.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
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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 2018
DRAWING NO:
FIGURE 19B 1.16
DATE:
JAN 2020
DRAWN:
S.R.H
SCALE :
1:10,103 @A3
SCALE BAR
0 0.1 0.2 0.3 0.4 0.5 KM
CPB_BP_11
BP23
PZ19
PZ17
C3 D
C3 S
BDG1
PZ20
C4 D
C4 S
PZ18
C2 D
C2 S
GW3
PZ15
PZ 3
PZ7
PZ6
PZ8
PZ10
PZ11
PZ 2
PZ5
PZ13
PZ16
PZ14
GR11
GW2
C1 D
C1 S
PZ21
GW20
DRAFT
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ANNEX 2A.16: REVIEW OF EXISTING INFILTRATION AND PERMEABILITY TEST DATA

Technical Note

Project:	SZC Enabling Works Detail Design		
Subject:	**EW0400 Review of Existing Infiltration and Permeability Test Data**		
Author:	RM		
Date:		Project No.:	5199744
Atkins No.:	SZC-EW0400-XX-000-NOT-400002	Icepac No.:	[Not Used]
Distribution:	[Not Used]	Representing:	[Not Used]

Document history

PW Revision	Status	Purpose description	Originated	Checked	Reviewed	Authorised	Date
01	S3	For Issue	RM	ES	AP	AL	26/02/21

Client signoff

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

1. Introduction

Atkins have been commissioned by NNB to undertake a high-level review of existing infiltration and permeability test data across the Sizewell C (SZC) Temporary and Ancillary Construction Areas (TCA and ACA). The scope of this review is outlined in an email from Ed Ball (Atkins) to Kiki Semertzidou (NNB) on 8 February 2021:

- Provide commentary on the validity of the existing test results; and
- Propose an appropriate test method to be utilised in the upcoming TCA and ACA ground investigation (GI) campaign.

Reliable infiltration and permeability test data is required to inform the Drainage Strategy for the Detailed Design of the Enabling Works for the construction of SZC.

The review presented in this Technical Note is based on existing information from several onshore GI campaigns across the SZC Development Area, as outlined in Section 2.

2. Summary of Existing Data

Table 2-1 below outlines the onshore GI data (including from factual reports) that has been made available to Atkins, and details which infiltration and permeability test data has been reviewed by Atkins in this Technical Note. No data from the Q3-4 2020 Enabling Works GI campaign undertaken by Fugro has been used in this technical note as final data has not yet been received.

Table 2-1 – Infiltration and permeability test data used in this technical note

Factual report title (year of issue, GI Contractor)	GI year	SZC document reference	Number and type of available infiltration/ permeability test results	Used in this Technical Note	Comments
BGS historical exploratory hole data	Various but generally pre-1980	N/A	0	N/A	-
Sizewell ‘B’ Power Station. Factual Report on 1975 Onshore Site Investigation (1976, Foundation Engineering Limited) [1]	1975	SZC-SZ0100-XX-000-REP-100002	0	N/A	-
Site Investigation (1980) for Sizewell 'B' Power Station Sizewell, Suffolk	1980	Unknown	0	N/A	-
Sizewell C Power Station Site Investigation (1995, Soil Mechanics) [2]	1994	SZC-SZ0100-XX-000-REP-100003	88No. sets of permeability tests	No	Located in and around the MCA and therefore outside the area of interest
Factual Report on Supplementary Ground Investigation at Proposed Nuclear Development at Sizewell ‘C’ (2009, SSL) [3]	2008	SZC-SZ0100-XX-000-REP-100004	2No. sets of infiltration tests	No	Located in the MCA and therefore outside the area of interest

Factual report title (year of issue, GI Contractor)	GI year	SZC document reference	Number and type of available infiltration/ permeability test results	Used in this Technical Note	Comments
			12No. sets of permeability tests		
Onshore Investigations Phase 1 for Sizewell Site. Factual Report on Ground Investigation (2011, ESG/ Soil Mechanics) [4]	2010-2011	SZC-SZ0100-XX-000-REP-100005	1No. pumping test	No	Located in the MCA and therefore outside the area of interest
Factual report on 1st Phase Ground Investigation on SZC Construction Site Area and Associated Development (2014, SSL) [5]	2014	SZC-NNBPCP-XX-000-REP-000014	6.No sets of infiltration tests 13No. sets of permeability tests	No Yes	Infiltration test results not provided in Factual Report
Factual Report on Ground Investigation for the 2015 Onshore Ground Investigation Campaign on the SZC Construction Site Area (2015, SSL) [6]	2015	SZC-SZ0100-XX-000-REP-100006	3.No sets of infiltration tests 3No. sets of permeability tests	Yes	-
Factual Report on Ground Investigation for the SZC SSSI Crossing (2016, SSL) [7]	2015	SZC-SZ0100-XX-000-REP-100007	0	N/A	-
2016 Onshore Ground Investigation Campaign. Factual Report on Ground Investigation (2017, SSL) [8]	2017	SZC-SZC030-XX-000-REP-100000	9.No sets of infiltration tests	Yes	-
Sizewell C On Shore Phase 2 Ground Investigation – 2019 Task Order 1. Factual Report on Ground Investigation (Volume 1) (2020, SSL) [9]	2019-2020	Unknown	2No. sets of permeability tests 1No. pumping test	No	Located in the MCA and therefore outside the area of interest
Report on Ground Investigation without Geotechnical Evaluation. Sizewell Infiltration Testing (2020, Fugro) [10]	2020	Unknown	23.No sets of infiltration tests	Yes	-
Report on Ground Investigation without Geotechnical Evaluation. Sizewell Infiltration Testing (Addendum) (2021, Fugro) [11]	2020	Unknown	1No. set of infiltration tests	Yes	-

3. Review of Existing Data

3.1. Review Procedure

Atkins have undertaken a review of the existing infiltration and permeability test data within the TCA and ACA as listed in Table 2-1. A 'Confidence Category' has been provided for all test sets used in this review, based on the adherence of the test sets to the applicable standard and to what extent the data can be used for Detailed Design purposes. Table 3-1 provides an explanation of the Confidence Categories used. It is noted that:

- While a test or set of tests may not fully comply with the applicable standard, they still provide meaningful data; and
- These categories are applicable to the test or set of test results for a particular location in isolation.

Table 3-1 – Explanation of Confidence Categories

Confidence Category	Explanation of Category
1	Provides useful general data as a background for design but infiltration rate/permeability cannot be used for Detailed Design purposes.
2	Data cannot be used in its current form due to inconsistencies with the relevant standards for calculation, but it would be possible to re-calculate infiltration rate. After re-calculation, results would provide reasonably reliable data (with some minor inconsistencies with the relevant standards for testing) and could be used for Detailed Design with some confidence.
3	Reasonably reliable data with some minor inconsistencies with the relevant standards for testing and calculation; can be used for Detailed Design with some confidence.
4	Reliable data resulting from testing and calculations being undertaken fully in accordance with the relevant standards; can be used for Detailed Design with confidence.

A review of existing infiltration test data within the TCA and ACA is provided in Table 3-2. This table provides a Confidence Category for each set of test results and comments on the reasoning behind that category.

A summary of existing permeability test data within the TCA and ACA is provided in Table 3-3. The results have not been reviewed in this Technical Note as the depths of test response zones are not considered relevant for the Detailed Design Drainage Strategy – they are too deep to be used in the flood storage design.

A plan showing the locations of existing infiltration test results and their assigned Confidence Category, along with the locations of proposed infiltration and permeability tests in the upcoming TCA and ACA GI campaign, is provided in Figure 3-1. It is noted that infiltration testing locations from the Q3-4 2020 Enabling Works GI campaign have not been included in Figure 3-1.

Table 3-4 summarises the number of infiltration tests from each of the historical GI campaigns reviewed in this Technical Note which have been assigned to each of the four Confidence Categories.

The current applicable standards for infiltration and permeability testing and the associated calculations are presented below:

- Infiltration tests and infiltration rate calculations should be undertaken in trial pits following BRE 365 (2016) [12], and
- Permeability tests and permeability coefficient calculations should be undertaken in boreholes following BS EN ISO 22282-2:2012 [13].

Table 3-2 – Review of infiltration test data

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
2020, (2021, Fugro)	WMZ3_2020-3-TP-A	Trial Pit	BRE 365	3	3.40 x 0.70 x 2.30; 0.30	Slightly gravelly slightly silty SAND	Gravel infill to 0.30m bgl to support unstable pit. Well screen not used.	1.34E-06, 1.32E-05, 1.16E-05	1	Test durations do not allow infiltration rate to level off or pit to fully empty. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rates for Tests 2 and 3 were not calculated correctly following BRE 365.
2020, (2020, Fugro)	ACA_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 2.90; 0.45/0.50/0.50	SAND and GRAVEL	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 (to 3.50m) before test started; this standing water level is taken as the test base depth.	3.34E-6 3.31E-6 5.43E-6	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth appear approximately correct. Infiltration rates were not calculated correctly following BRE 365 due to durations of tests. Note gravel at slightly lower depth than depth to start of test in Test 1, so gravel fraction % may require review.
2020, (2020, Fugro)	ACA_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.50	SAND	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.	5.61E-5 2.18E-5 1.48E-5	3	Testing undertaken in borehole. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	CAMPUS_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.40/0.50/0.50	Gravelly SAND	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.	7.99E-6 9.99E-6 6.14E-6	3 (Note: Category of 1 for Test 2, but of little significance as would use lowest value [Test 3] for design)	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth appear approximately correct. Infiltration rate for Test 2 was not calculated correctly following BRE 365 due to duration of test. Note gravel at slightly lower depth than depth to start of test in Test 1, so gravel fraction % may require review.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
2020, (2020, Fugro)	CAMPUS_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 2.74; 0.48/0.44/0.46	Slightly gravelly sandy CLAY	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 2.74m bgl (to 3.50m) before test started; this standing water level is taken as the test base depth.	1.17E-5 1.85E-5 1.36E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rates for all tests were not calculated correctly following BRE 365. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review.
2020, (2020, Fugro)	WMZ1_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.25; 0.25	SAND	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.	2.01E-4 1.69E-4 1.24E-4	2	Testing undertaken in borehole. Note gravel at lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	WMZ1_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.40; 0.45	SAND	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.	1.95E-4 5.73E-5 5.39E-5	3	Testing undertaken in borehole. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	WMZ1_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 1.20; 0.45	SAND with pockets of clay	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. Standing water was noted at 1.20m bgl (to 3.50m) before test started; this standing water level is	1.28E-4 1.20E-4 1.14E-4	3	Testing undertaken in borehole. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, testing and calculations in general accordance with BRE 365.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
							taken as the test base depth.			
2020, (2020, Fugro)	WMZ2_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.25; 0.60	SAND	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 0.70m bgl.	2.32E-5 1.86E-5 1.88E-5	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ2_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.52; 0.45	Slightly gravelly SAND	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.	2.88E-5 2.01E-5 1.91E-5	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ2_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.50/0.60/0.70	SAND and GRAVEL	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 0.80m bgl.	2.41E-5 1.57E-5 9.35E-6	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct. Note gravel at lower depth than depth to start of test, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ3_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.40; 0.50	SAND and GRAVEL	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of	6.07E-4 1.13E-4 5.64E-5	3	Testing undertaken in borehole. Aside from that, testing and calculations in general accordance with BRE 365.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
							the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.			
2020, (2020, Fugro)	WMZ3_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.22/0.25/0.25	Slightly gravelly SAND	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.	7.02E-5 3.85E-5 3.34E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Note gravel at lower depth than depth to start of test, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ3_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 1.62; 0.60	SAND and GRAVEL	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 0.60m bgl. Standing water was noted at 1.62m bgl (to 2.75m) before test started; this standing water level is taken as the test base depth.	1.96E-5 1.48E-5 1.80E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ4_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.50	SAND	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.	1.30E-4 5.63E-5 3.83E-5	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ4_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.56/0.45/0.20	Gravelly SAND	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	5.64E-5 3.86E-5 3.11E-5	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
							space. Gravel filter commences at 0.50m bgl.			Note gravel at lower depth than depth to start of test in Tests 2 and 3, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ4_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.46/0.45/0.10	SAND	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.	2.27E-4 1.30E-4 8.14E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Note gravel at lower depth than depth to start of test, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ5_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.56; 0.95/0.97/0.95	SAND	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 1.00m bgl.	4.23E-4 3.17E-4 2.28E-4	3	Testing undertaken in borehole. Effective depth calculated approximately correctly following BRE 365. Note gravel at slightly lower depth than depth to start of Tests 2 and 3, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ5_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 2.93; 0.60/0.40/0.40	Slightly gravelly SAND	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 0.40m bgl.	3.29E-5 2.08E-5 1.71E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ5_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.50/0.27/0.46	Slightly gravelly SAND	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.	7.22E-6 7.09E-6 6.00E-6	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct. Note gravel at lower depth than depth to start of test in Tests 2 and 3, so gravel fraction % may require review.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
										Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ6_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.40; 0.70	SAND and GRAVEL	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 0.70m bgl.	5.41E-4 1.77E-4 7.99E-5	3	Testing undertaken in borehole. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	WMZ6_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.66/0.73/0.61	Slightly gravelly SAND	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.	1.89E-5 1.59E-5 1.62E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ6_2020-2-PIT	Trial Pit	BRE 365	3	1.10 x 0.60 x 1.30; 0.54/0.57/0.54	Slightly gravelly SAND	Test carried out inside 50mm slotted pipe in gravel filled pit. Gravel infill to 0.50m bgl.	1.82E-05, 1.09E-05, 5.58E-06	1	Test durations do not allow infiltration rate to level off or pit to fully empty. In absence of longer test duration, calculations of effective depth appear correct. Infiltration rates for Tests 2 and 3 were not calculated correctly following BRE 365 due to durations of tests.
2020, (2020, Fugro)	WMZ6_2020-2-IP-A	Inspection Pit	BRE 365	3	0.40 x 0.40 x 1.30; 0.43/0.44/0.44	Slightly gravelly SAND	Test carried out inside 225mm slotted casing in gravel filled pit. Gravel infill to 0.50m bgl.	1.42E-05, 1.05E-05, 1.01E-05	3	Testing in general accordance with BRE 365, except for test pit length. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, calculations of effective depth and infiltration rates appear correct.
2017 (2017, SSL)	TP-WMZ-21	Trial Pit	BRE 365	2	1.00 x 0.60 x 2.40; 1.00/0.00	Slightly gravelly sandy CLAY with low cobble content, over slightly silty gravelly SAND (boundary at 0.50m)		<Test 1 not provided>, 7.76E-6	1	Two tests carried out at different depths, and no third test. Test durations do not allow infiltration rate to level off or hole to fully empty. Effective depth does not appear to have been calculated correctly following BRE 365.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
										Infiltration rate calculation relies on extrapolation.
2017 (2017, SSL)	TP-WMZ-22	Trial Pit	BRE 365	2	1.00 x 0.60 x 2.80/2.20; 0.82/0.00	Slightly gravelly sandy CLAY with low cobble content, over slightly silty gravelly SAND (boundary at 0.40m)		1.25E-5, 7.77E-6	1	Two tests carried out at different depths, and no third test. Test durations do not allow infiltration rate to level off or hole to fully empty. Effective depth does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations rely on extrapolation.
2017 (2017, SSL)	TP-WMZ-23	Trial Pit	BRE 365	2	1.20 x 0.60 x 2.60; 0.10/1.40	Slightly gravelly sandy CLAY with low cobble content, over gravelly SAND (boundary at 0.50m)		7.55E-6, 1.61E-5	1	Two tests carried out at different depths, and no third test. Test durations do not allow infiltration rate to level off or hole to fully empty. Note water level goes up part way through Test 2. Effective depth does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations rely on extrapolation.
2017 (2017, SSL)	TP-WMZ-24	Trial Pit	BRE 365	1	1.20 x 0.60 x 3.00; 0.35	Slightly gravelly sandy CLAY, over slightly clayey slightly gravelly SAND, over slightly gravelly sandy CLAY (boundaries at 0.40m and 1.20m)		5.68E-6	1	Test not repeated. Test duration does not allow infiltration rate to level off or hole to fully empty. Effective depth does not appear to have been calculated correctly following BRE 365. Infiltration rate calculation relies on extrapolation.
2017 (2017, SSL)	TP-WMZ-25	Trial Pit	BRE 365	1	2.10 x 0.60 x 3.00; 0.34	Slightly gravelly sandy CLAY with low cobble content, over slightly gravelly sandy CLAY, over slightly silty SAND (boundaries at 0.50m and 2.00m)		8.68E-6	1	Test not repeated. Test duration does not allow infiltration rate to level off or hole to fully empty. Effective depth does not appear to have been calculated correctly following BRE 365. Infiltration rate calculation relies on extrapolation.
2017 (2017, SSL)	TP-BP-4	Trial Pit	BRE 365	1	2.00 x 0.60 x 1.65; 0.05	Slightly gravelly sandy CLAY with low cobble content, over slightly silty gravelly		1.24E-6	1	Test not repeated. Effective depth does not appear to have been calculated correctly following BRE 365.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
						SAND (boundary at 0.50m)				Infiltration rate calculation relies on extrapolation.
2017 (2017, SSL)	TP-C-11	Trial Pit	BRE 365	2	1.00 x 0.60 x 2.70; 1.30/1.70	Slightly silty slightly gravelly SAND		7.84E-6, 8.03E-6	1	Two tests carried out at different depths, and no third test. Test durations do not allow infiltration rate to level off or hole to fully empty. Effective depth does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations rely on extrapolation.
2017 (2017, SSL)	TP-C-12	Trial Pit	BRE 365	1	1.00 x 0.60 x 2.40; 0.30	Slightly gravelly sandy CLAY, over slightly sandy slightly gravelly CLAY, over slightly silty SAND (boundaries at 0.50m and 1.20m)		1.62E-6	1	Test not repeated. Test duration does not allow infiltration rate to level off or hole to fully empty. Trial pit log states trial pit terminated at 2.40m, while infiltration test results sheet states the base of the test was at 2.50m and calculates the effective depth from 2.50m. Effective depth calculations also do not appear to follow BRE 365. Infiltration rate calculation relies on extrapolation.
2017 (2017, SSL)	TP-CPB-C-16	Trial Pit	BRE 365	1	1.00 x 0.60 x 2.60; 0.10	Slightly gravelly sandy CLAY, over gravelly very sandy CLAY (boundary at 0.40m)		4.35E-6	1	Test not repeated. Test duration does not allow infiltration rate to level off or hole to fully empty. Effective depth does not appear to have been calculated correctly following BRE 365. Infiltration rate calculation relies on extrapolation.
2015 (2015, SSL)	TP WMZ 18	Trial Pit	BRE 365	1	1.70 x 0.60 x 3.50; 2.20	Slightly clayey to clayey SAND	Pit start depth = 3.5m, pit final depth = 2.56m	1.57E-4	1	Test not repeated. Note pit collapse during infiltration test.
2015 (2015, SSL)	TP WMZ 19	Trial Pit	BRE 365	1	2.10 x 0.60 x 3.50; 2.30	SAND and GRAVEL, over slightly clayey slightly gravelly SAND (boundary at 2.40m)	Pit start depth = 3.5m, pit final depth = 2.97m	5.61E-5	1	Test not repeated. Note pit collapse during infiltration test.
2015 (2015, SSL)	TP WMZ 20	Trial Pit	BRE 365	1	2.80 x 0.60 x 1.10; 0.05	Slightly clayey gravelly SAND, over clayey SAND (boundary at 0.70m)		8.31E-6	1	Test not repeated. Test duration does not allow infiltration rate to level off or hole to fully empty. Infiltration rate calculation relies on extrapolation.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
										Effective depth does not appear to have been calculated correctly following BRE 365.

Table 3-3 – Summary of permeability test data

GI Year (Factual Report issue year and GI Contractor)	Exploratory Hole ID	Permeability test type	Standard used, according to Factual Report	Number of tests undertaken	Hole diameter, depth interval of test section (mm, m)	Response zone geology	Test notes	Calculated permeability coefficient, *k* – Tests 1 and 2 (lowest highlighted) [m/s]
2015 (2015, SSL)	CPB BP 11	Variable head – rising head	BS5930:1999	1	250, 8.00-20.00	Gravelly SAND, over SAND, over slightly clayey slightly gravelly SAND (boundaries at 10.10m and 15.00m)	Pumping for 20mins at a rate of 27 litres per min	Not provided
2015 (2015, SSL)	CPB BP 13	Variable head – rising head	BS5930:1999	1	Unknown, 8.00-20.00	Slightly clayey locally clayey slightly gravelly locally gravelly SAND	Pumping for 20mins at a rate of 27 litres per min	Not provided
2015 (2015, SSL)	CPB BP 14	Variable head – rising head	BS5930:1999	1	Unknown, 8.00-20.00	Silty SAND, over slightly gravelly clayey SAND, over slightly clayey gravelly SAND (boundaries at 9.00m and 10.50m)	Pumping for 20mins at a rate of 27 litres per min	Not provided
2014 (2014, SSL)	C3	Falling head	Non-standard	1	150, 6.69-10.16	Sandy GRAVEL, over slightly gravelly to gravelly SAND, over SAND, over sandy CLAY (boundaries at 7.00m, 8.20m and 10.10m)	Slotted standpipe fitted with geotextile in response zone.	4.93E-6
2014 (2014, SSL)	C7	Falling head	BS5930:1999 Non-standard	1 1	150, 18.94-19.24 150, 13.76-19.76	Interlaminated silty SAND and CLAY, over slightly silty SAND, over clayey gravelly SAND (boundaries at 16.00m and 19.40m)	Casing to 18.94m bgl. Groundwater level at 12.64m bgl prior to test. Slotted standpipe fitted with geotextile in response zone. Groundwater level at 13.22m bgl prior to test.	1.33E-6 9.92E-6
2014 (2014, SSL)	C7	Rising head	BS5930:1999	1	150, 13.76-19.76	Interlaminated silty SAND and CLAY, over slightly silty SAND, over clayey gravelly SAND (boundaries at 16.00m and 19.40m)	Slotted standpipe fitted with geotextile in response zone. Groundwater level at 13.22m bgl prior to test.	Not provided
2014 (2014, SSL)	BP6	Falling head	BS5930:1999	1	200, 14.10-15.10m	Slightly gravelly SAND	Casing to 14.10m bgl. Groundwater level at 14.20m bgl prior to test. Hole collapsed to 14.10m bgl at start of test.	4.71E-6
2014 (2014, SSL)	BP6	Rising head	Non-standard	1	150, 10.10-20.17	SAND, over slightly gravelly SAND (boundary at 11.30m)	Slotted standpipe fitted with geotextile in response zone. Groundwater level at 14.54m bgl prior to test.	2.41E-6
2014 (2014, SSL)	BP7	Falling head	BS5930:1999	1	150, 10.90-11.50m	SAND	Casing to 10.90m bgl. Groundwater level at 7.60m bgl prior to test. Hole collapsed to 10.90m bgl between start and end of test.	3.78E-4

GI Year (Factual Report issue year and GI Contractor)	Exploratory Hole ID	Permeability test type	Standard used, according to Factual Report	Number of tests undertaken	Hole diameter, depth interval of test section (mm, m)	Response zone geology	Test notes	Calculated permeability coefficient, *k* – Tests 1 and 2 (lowest highlighted) [m/s]
2014 (2014, SSL)	BP7	Rising head	Non-standard	1	150, 12.00-20.36	SAND, over gravelly SAND (boundary at 15.50m)	Slotted standpipe in response zone. Groundwater level at 14.60m bgl prior to test.	8.72E-7
2014 (2014, SSL)	BP9	Falling head	BS5930:1999	1	200, 12.89-13.39	Slightly gravelly SAND, over slightly gravelly to gravelly SAND (boundary at 13.00m)	Casing to 12.89m bgl. Groundwater level at 12.50m bgl prior to test. Hole collapsed to 11.71m bgl at start of test and had collapsed further to 11.59m bgl by end of test.	8.49E-5
2014 (2014, SSL)	BP9	Rising head	Non-standard	1	150, 7.67-20.14	Gravelly to very gravelly SAND, over slightly gravelly SAND, over slightly gravelly to gravelly SAND (boundaries at 8.00m and 13.00m)	Slotted standpipe fitted with geotextile in response zone. Groundwater level at 11.80m bgl prior to test.	1.60E-5
2014 (2014, SSL)	BP12	Falling head	BS5930:1999	1	150, 14.30-14.50	Slightly gravelly SAND	Casing to 14.30m bgl. Groundwater level at 10.15m bgl prior to test. Hole collapsed to 14.38m bgl between start and end of test.	1.83E-5
2014 (2014, SSL)	BP12	Rising head	BS5930:1999	1	150, 12.00-20.00	SAND, over slightly gravelly SAND, over gravelly to very gravelly SAND (boundaries at 14.00m and 16.00m)	Slotted standpipe in response zone. Groundwater level at 10.18m bgl prior to test.	6.86E-7
2014 (2014, SSL)	BP27	Falling head	BS5930:1999 Non-standard	1 2	200, 9.89 150, 9.00-20.03	Very gravelly SAND SAND, over very gravelly SAND, over slightly gravelly SAND (boundaries at 9.70m and 12.00m)	Casing to 9.89m bgl. Groundwater level at 9.19m bgl prior to test. Slotted standpipe fitted with geotextile in response zone. Groundwater level at 10.30m bgl prior to test.	1.33E-6 2.23E-7 2.90E-7
2014 (2014, SSL)	BP27	Rising head	Non-standard	1	150, 9.00-20.03	SAND, over very gravelly SAND, over slightly gravelly SAND (boundaries at 9.70m and 12.00m)	Slotted standpipe fitted with geotextile in response zone. Groundwater level at 10.30m bgl prior to test.	1.77E-5

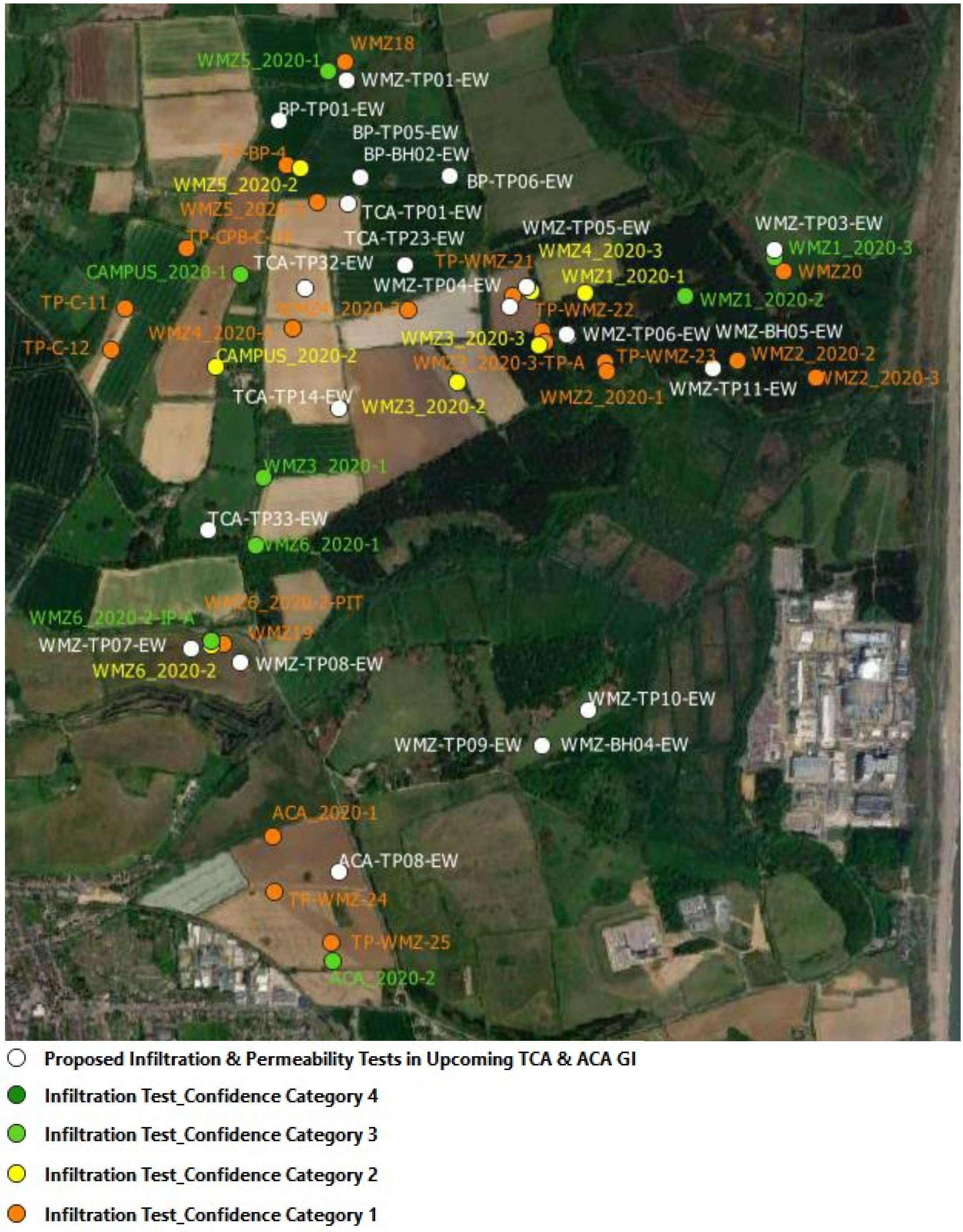

Figure 3-1 - Locations of existing infiltration and permeability test results and their assigned Confidence Category, along with the locations of proposed infiltration and permeability tests in the upcoming TCA and ACA GI campaign

Table 3-4 – Summary of the number of infiltration tests from each historical GI campaign assigned to each Confidence Category

	GI Year (Factual Report issue year and GI Contractor)			
Confidence Category	2015 (2015, SSL)	2017 (2017, SSL)	2020, (2020, Fugro)	2020, (2021, Fugro)
1	3	9	8	1
2	0	0	7	0
3	0	0	8	0
4	0	0	0	0

3.2. Test Methodology

Some common themes from the existing infiltration test methods and results arising from Atkins' review are explained below.

3.2.1. WSP report (2019) test method vs BRE 365 standard

Infiltration testing was undertaken in boreholes in the 2020 Fugro GI (main GI [10] and confirmatory hole [11]) following the method outlined in a technical note by WSP [14], which is based on BRE 365 [12]. WSP's Technical Note is specific to a soakaway test borehole which was undertaken as part of the Yoxford Junction Improvement Scheme as part of the SZC project. WSP's proposed methodology uses the BRE 365 methodology for infiltration testing and calculation of infiltration rates, but in boreholes rather than trial pits. As testing in boreholes is not covered in BRE 365, WSP made the following recommendations for the test set-up:

- Borehole drilled with as large a diameter as possible (250mm [10"] was recommended) to 3m below ground level (bgl);
- Well screen installed to the base of the borehole, with openings matched to the ground conditions, noting the well screen can be placed whilst the borehole is still cased; and
- Annulus between well screen and outside of the borehole backfilled with pea gravel.

There are potential discrepancies arising with applying the BRE 365 test method to boreholes:

- BRE 365 states "*Site testing for soil infiltration rates should give representative results for the proposed site of the soakaway. This is achieved by the following:*
    - *Excavating a soakage trial pit of sufficient size to represent a section of the soakaway.*

  BRE 365 indicates that a soakaway undertaken to less than the recommended dimensions of "*1 m to 3 m long and 0.3 m to 1 m wide*" (for example in a borehole) may not provide representative infiltration rates for the design of a soakaway.

In Atkins' view, the methodology seems reasonable and it is considered that values obtained could be used for Detailed Design However it should be noted that a borehole infiltration test could be considered as being not strictly in accordance with BRE 365, and thus not compliant. Thus if borehole tests are to be considered for use in design, some form of calibration with trial pit infiltration tests should be established.

3.2.2. Potential non-conformance with BRE 365 standard

There appear to be several non-conformances with the general testing methodologies and calculations reviewed in this Technical Note and the testing methodologies and calculations outlined in BRE 365 [12], as follows:

- BRE 365 states "*Site testing for soil infiltration rates should give representative results for the proposed site of the soakaway. This is achieved by the following:*
    - *Filling the soakage trial pit several times in quick succession while monitoring the rate of seepage. This procedure will confirm soil moisture conditions typical of the site when the soakaway becomes operative.*"

  This indicates that a single infiltration test instead of multiple tests in succession and taking the lowest value will not provide representative infiltration rates for the design of a soakaway because undertaking a single infiltration rate will over-estimate the infiltration rate. From looking at the data obtained on site where three tests were undertaken in succession, it is seen that it is likely that only undertaking a single test will be over-estimating the infiltration rate by up to 1 order of magnitude as a maximum.

- BRE 365 states "*If it is impossible to carry out a full-depth soakage test, the soil infiltration rate calculation should be based on the time for the fall of the water level from 75% to 25% of the actual maximum water depth achieved in the test. The effective area of loss from the soakage trial pit is then calculated as the internal surface area of the pit to 50% maximum depth achieved, plus the base area of the soakage trial pit*". This indicates that if an infiltration test is undertaken following BRE 365 and the pit does not fully empty, but the infiltration rate levels out before the pit reaches empty (including both if it reaches 25% full and if it doesn't reach 25% full), then the calculations should be carried out for the depth between the maximum effective storage depth and the depth that the infiltration rate becomes ~0m/s (this is the "actual maximum water depth achieved"), and not between the maximum effective storage depth and the base of the pit. The results from infiltration testing carried out in borehole CAMPUS_2020-2 in Table 3-2 are an example of where this change in depth has not been applied.
- BRE 365 does not mention if extrapolation to get a t value at 25% is applicable, but extrapolating this value would not be accurate so it is considered that the calculated infiltration rate would be un-representative.
- If a trial pit does not empty during the infiltration test and the infiltration rate does not level out (i.e. reach ~0m/s), then it is considered that the test has been terminated too early and the infiltration rate cannot be calculated. If this occurs, using 75% and 50% of the depth interval between water level at start of test and base of pit in place of 75% and 25% is not in accordance with BRE 365.

In Atkins' view, the non-conformances described above can still provide useful general data as a background for Detailed Design (i.e. a Confidence Category of "1"). If the infiltration rates may be re-calculated in accordance with BRE 365 based on the data provided, a Confidence Category of "2" has been assigned.

It is noted that it is to be expected that tests undertaken in similar areas and apparent ground conditions to give differing results, as conditions such as groundwater levels will vary throughout the year and with short-term changes in weather patterns, leading to different infiltration rates.

3.3. Literature Review

The infiltration rate is determined by soil characteristics which include ease of entry, storage capacity, and transmission rate through the soil.

Figure 3-2 provides expected permeabilities for a range of geologies, which can be used as a guide to do a high-level check of the results that are presented in Table 3-2.

k m/s	Soil type		Drainage characteristics
10 – 1	Coarse gravel, cobbles, boulders flow may become turbulent ∴ Darcy's Law may not be valid		Very good
1 – 10^{-2}	Clean gravels		
10^{-2} – 10^{-4}	Clean sands Clean sand-gravel mixtures		Good
10^{-4} – 10^{-5}		Impervious soils modified by the effects of: fissuring desiccation weathering	
10^{-5} – 10^{-6}	Very fine sands Silty sands Silts Stratified clay/silt deposits		
10^{-6} – 10^{-8}			Poor
10^{-8} – 10^{-9}			Practically Impervious
< 10^{-9}	Unweathered, unfissured, homogeneous clays (Clay content > 20%)		

Figure 3-2 - Typical values of permeability, k (adapted from Barnes, 2010 [15])

4. Recommendations for Future Works

4.1. Office-based Tasks

Based on the review of existing permeability and infiltration testing data presented in Section 3, the following recommendations are suggested to better understand the existing infiltration and permeability testing across the SZC Development Area:

- Liaise with WSP to better understand the infiltration test methodology outlined in their Technical Note [14];
- Compare empirical correlations for permeability with results from field tests, for example using the Prugh Method of estimating permeability of soils from CIRIA C750 [16] (a correlation based on laboratory particle size distribution [PSD] test results); and
- Re-calculate the infiltration rates in the 7No. holes which have been given a Confidence Category of 2 in Table 3-2, following the BRE 365 [12] methodology.

4.2. Testing Methodology for Future Investigations

Based on the review of existing permeability and infiltration testing data presented in Section 3, it is recommended that the following combination of infiltration and permeability test methods are adopted for future GI across the SZC Development Area:

- Carry out infiltration tests in trial pits to BRE 365 [12]. A gravel infill can be used if required for stability purposes, but it must be ensured that 3No. tests are carried out at each test location until the trial pit is empty or the infiltration rate has levelled off, and that calculations are also carried out to BRE 365; and
- Carry out testing in large diameter boreholes in selected locations adjacent to the trial pit tests outlined above, for correlation purposes. Testing should comprise permeability tests to BS EN ISO 22282-2:2012 [13] followed by infiltration tests following the procedure used in the Fugro Q1 2020 GI [10], understood to be in accordance with the WSP Technical Note [14], once the casing is removed from

the borehole (both tests required for correlation purposes). These tests should be carried out across the same depth intervals as the adjacent trial pit infiltration tests.

As outlined in Section 3.2.1, the methodology of the WSP Technical Note [14] adapts BRE 365 [12] for use in boreholes, despite BRE 365 not covering testing in boreholes. As many of the infiltration tests undertaken in the Fugro Q1 2020 GI [10] were carried out in boreholes, there is a requirement to assess the reliability of those test results. This will be done by comparing the results of the borehole infiltration tests with the adjacent trial pit infiltration tests.

If the BRE 365 methodology cannot be completed in any trial pits for safety reasons, there is scope to replace the trial pit testing with borehole testing in that location. Therefore, there is a requirement to assess the correlation between borehole permeability test results and trial pit infiltration test results. This will be done by undertaking both tests adjacent to each other and comparing the results, to provide a guide as to whether or not permeability tests can be used on a wider basis.

5. Summary

Atkins have undertaken a high-level review of the existing infiltration and permeability test data available from across the SZC TCA and ACA Development Area. The scope of this Technical Note is outlined in Section 1, and the data used in this Technical Note is summarised in Section 2.

36No. sets of infiltration test results from four GIs undertaken between 2015 and 2020 are reviewed in Section 3. The test results were assigned to Confidence Categories based on the adherence of the test sets to the applicable standard and to what extent the data can be used for Detailed Design purposes, as summarised in Table 3-2, Table 3-4 and illustrated in Figure 3-1.

In addition, 16No. sets of permeability test results from three GIs undertaken between 2014 and 2020 were summarised in Section 3 and Table 3-3. A review was not undertaken as the test response zones were not considered to be relevant for the Detailed Design Drainage Strategy.

Recommendations for future work, including proposals for testing methodologies to be utilised in the upcoming TCA and ACA GI campaign, are summarised in Section 4. Proposals include additional office-based review tasks and a combination of on-site testing in trial pits and boreholes.

6. References

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[11] Fugro GeoServices Limited, “Report on Ground Investigation without Geotechnical Evaluation. Sizewell Infiltration Testing (Addendum),” 2021.

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ANNEX 2A.17: SURFACE WATER DRAINAGE TREATMENT NARRATIVE

Surface Water Drainage Treatment Narrative

CONTENTS

TABLES

APPENDICES

1 INTRODUCTION

1.1.1 This short note provides an updated summary of the surface water drainage treatment proposals for the TCA in response to technical queries raised by Suffolk County Council. Whilst this note does not intend to provide detailed design of the drainage treatment features, it aims to demonstrate that suitable solutions are available with respect to space provision and selection of treatment methods.

1.1.2 The treatment of surface water at Sizewell C consists of a number of Water Management Zones (WMZs). In each zone there are proposed filter strips, swales and basins. Each zone has differing limitations which influence the ability of the design to meet The SuDS Manual requirements. This short document will present those assets (swales) that do comply and those assets where shortcomings exist (basins) but have mitigating factors.

2 SURFACE WATER TREATMENT

2.1 Space Available

2.1.1 Plans of the WMZs demonstrate that adequate space exists for both Filter Strips and Swales within the development site (**Appendix A.1** Swale Network Overview - SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000038). These are generally located along the perimeters of the WMZs and drain to the WMZ basins. The space allocated for filter strips is a width of 4m, which represents our recognition of the pollution potential of the construction site. The width of filter strips may vary depending on the size of the area being drained and the slope of the contributing area. The swales have been sized at a width of approximately 7m, having a base width of at least 2m. The swale size reflects both the hydraulic modelling requirements regarding volume and the treatment capability.

2.1.2 The gradient of the swales is low (close to 1:100) and therefore flowrates are low. There is no proposal to include check dams along the flow path at this stage, however the inclusion of check dams will be considered during Detailed Design to improve treatment benefits.

2.1.3 Cross sections of the swales have been included within the drawings to demonstrate the actual dimensions available on site (**Appendix A.2** Swale Network Cross Section - SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000010). The provisional hydraulic modelling carried out indicates that the flows generated will be controlled within the swale sizes proposed, and within the underdrain provided. The side slope of the swales is to be vegetated and constructed to a slope of 1:3 for ease of maintenance.

2.1.4 It is recognised that treatment potential of swales is dependent on the retention time and surface area of the swale. The base is to be vegetated and is wide in order to optimise the use of bioretention media forming part of the swale, thereby allowing improved treatment of water through filtration entering the underdrain. The type of vegetation, and specification of bioretention filter media and the depths required will designed in accordance with The SuDS Manual. This additional treatment can potentially resolve non-compliance of swales and basins from a treatment perspective and provide the required total index for the system using the Simple Index Approach.

2.2 Basins

2.2.1 The WMZs all have basins, which all discharge to surface waters either by pumping or by outlet. A number of the basins are able to infiltrate through their bases and therefore the discharge will be volumetrically reduced. The SuDS manual has as a number of design requirements that the basins should have in order to demonstrate compliance. These requirements include but are not limited to:

- Construction side slopes no more than 1:3,
- Ability to infiltrate through the base,
- Overall volume to contain 1:100 + CC storms,
- Half drain time of 24 hours,
- Ability to cope with a 24 hour power failure for a pumped discharge.
- Operate with a 100mm water depth for 1:1 year storms.
- Operate at a maximum of 2m depth.
- Offer treatment before discharge for suspended solids, metals and hydrocarbons.

2.2.2 The basins for Sizewell C are temporary in nature with an expected life of approximately 10 years. All the basins, except the Green Rail Route, are wholly within a fenced off construction site and therefore not open to the public. This means that although not normally considered appropriate there are some basins that are deeper than 2m but do not pose a public safety risk.

Table 2-1 - Basin characteristics in relation to SuDS Manual compliance

WMZ Basin	1	2	3	4	5	6	ACA West	ACA East	GRR
Side Slopes	Y	Y	Y	Y	Y	Y	Y	Y	Y
Base Infiltration	N	Y	Y	Y	Y	Y	N	Y	N
Volume	Y	Y	Y	Y	Y	Y	Y	Y	Y
Half Drain	N	N	N	N	N	N	N	N	Y
24 hr failure	N/A	N/A	N/A	N/A	N/A	N/A	Y	N/A	Y
1:1 yr depth	N	N	N	N	N	N	N	N	Y
2m Max depth	Y	N	N	N	Y	N	N	Y	Y

2.2.3 The basins are required in each WMZ and all comply with some parts of the SuDS manual. The most significant element of compliance is that they all offer the correct storm containment volume (1:100+CC) and those basins that have a pumped discharge are able to contain a 24 hour pump outage volume. The discharges from the basins are all limited, in most cases to the greenfield runoff rate, and therefore no basin poses any flooding risk to surrounding surface waters or residential areas.

2.2.4 The discharge flow limitation on the large basins means that the half drain times are greater than 24 hours. This does not meet the manual requirements for draining, but there is sufficient volume within the large basins to contain a follow-on storm. The additional retention time within the basins because of the slower discharge acts to improve solids sedimentation and therefore discharged water will have less suspended solids.

2.2.5 There are some basins that do not comply because of their maximum depths. WMZ space is limited and therefore the plan area has been reduced for these basins, thereby increasing the maximum depth beyond the 2m limit. The increased maximum depths of these basins means that potentially the effective treatment carried out may not be as much as a fully compliant basin. This is recognised and it has been proposed that these basins are to have dividing walls built into them so that a sequential filling of the 2 portions encourages more sedimentation within the first part of the basin. This arrangement would significantly improve the solids removal ability of the basins. A sequential filling arrangement was not used in the Hinkley Point C power station and it is thought that this sequential proposal will perform better than the serpentine approach used at Hinkley both in terms of solids removal and maintenance of the basin itself.

a) Basin Treatment Indices

2.2.6 There are treatment requirements for suspended solids, metals and hydrocarbons. A detailed analysis of WMZ1 and use of the Simple Index Method for the other WMZs showed how the treatment requirement can be met in each case. The full treatment index cannot be used for the non-compliant basins although it should be recognised that treatment within the basins is not zero and therefore some numerical value should be used to reflect this.

2.2.7 It has been mentioned that some basins have non-compliant maximum depths. Some basins are able to infiltrate and therefore a proportion of their flow is being treated by the ground. All the basins except one do not comply with the half drain time but these delayed discharges can have an overall treatment benefit. In addition it has been proposed that the deep basins are split with a dividing wall to allow preferential sedimentation in one part before passing to the next part. The long retention times within the basins means that there would be significant hydrocarbon water surface treatment as well as suspended solids removal by sedimentation.

2.2.8 As the proposed basins are not fully SuDS compliant (except the GRR basin) but do have mitigating treatment it is proposed that where basins operate as part of a sequence their treatment would still be included within the Simple Index Method but that only half the applicable value be included. (i.e. TSS = 0.25, Metals = 0.25, and Hydrocarbons = 0.3 rather than 0.5, 0.5 and 0.6). This is thought to be a fair representation of the basins' treatment value. Where the basins act as a single component (ACA East and ACA West) it is felt appropriate to include the full treatment index value. This is because other pollution control features are planned (Oil interceptors and Siltbuster proprietary systems) which do not contribute to the Simple Index Approach values and the overall treatment is under-represented.

2.3 Simple Index Approach

a) Temporary Construction Area (TCA)

2.3.1 Following the above approach, below is an overview of how the proposed drainage components can achieve the pollution hazard index using the Simple Index Approach. 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. 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 Bioretention Medium to Groundwater.
- Pathway 2 – Filter Strip and Bioretention Medium and Basin to Groundwater.
- Pathway 3 – Filter Strip and Bioretention Medium and Basin to Watercourse.

Table 2-2 – Total SuDS Mitigation Indices for each discharge pathway (Appendix B)

Pathway	TSS	Metals	Hydrocarbons
1. Filter Strip + Bioretention Medium (infiltration through bioretention component and underlain soil) to groundwater	0.8	0.8	0.9
2. Filter Strip + Bioretention Medium + Basin (infiltration to groundwater)	>0.95	>0.95	>0.95
3. Filter Strip + Bioretention Medium + Basin (discharge to watercourse only)	>0.95	>0.95	>0.95

2.3.2 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. The above shows that the combination of components can provide sufficient treatment of surface water runoff, regardless of where runoff is discharged.

2.3.3 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. This may include proprietary components such as downstream defenders to ensure surface water runoff is treated adequately, and/or as a fail-safe method of treatment to supplement primary treatment observed using natural SuDS techniques.

2.3.4 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 is proposed to 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/filter drain to provide additional water treatment.

Table 2-3 - 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 to 6	Haul Road	High	0.8, 0.8, 0.9	1	0.8, 0.8, 0.9
WMZ 10 – Campus	Access Road	Medium	0.7, 0.6, 0.7	Pervious Pavement only	0.7, 0.6, 0.7

b) Ancillary Construction Area (ACA)

2.3.5 The ACA area is divided into 2 parts: East and West. The West basin receives flows from only the Topsoil Compound whereas the East basin receives flows from all the other ACA areas.

2.3.6 The ACA West basin does not comply with the SuDS manual for depth and therefore the treatment index value used in this assessment has been halved. The results show that the flows from the Topsoil Compound are able to be adequately treated using a combination of swales containing a bioretention filter and the basin (**Table 2-4**).

2.3.7 The ACA East basin does comply with the SuDS manual in terms of maximum depth and it is thought that in this case the full index value is more appropriate. The use of other pollution control features are planned (Oil interceptors and proprietary systems such as Siltbuster packaged treatment plant) within these areas which do not contribute to the Simple Index Approach values. This approach is thought to better represent the pollution treatment available in this WMZ.

2.3.8 As shown below in Table 2-5, some ACA areas that drain to the ACA East basin do not have sufficient mitigation methods for each contaminant type and in some cases the flows are directly into the basin without upstream pre-treatment (Sand and Aggregate Stockpile, HGV Parking area).

2.3.9 Some of the specific areas have a medium pollution hazard risk and in these areas the treatment available is adequate (Park & Ride, Logistics Compound, Topsoil Compound, Caravan Pitches), except the Railway area which requires additional treatment.

2.3.10 The Material Transfer Laydown area has a high pollution risk and in this area the treatment available is also adequate.

2.3.11 The HGV parking and the Sand and Aggregate Stockpile area both have a high pollution risk and cannot be treated adequately using a basin only. It is anticipated that a mixture of SuDS features and proprietary methods will be introduced during Detailed Design in the HGV area and any other areas where it is necessary to address treatment shortfalls as noted in the ACA Drainage Strategy Technical Note DCO Task D4 (SZC-EW0320-ATK-XX-000-XXXXXX-NOT-CIV-000003). The proposals will be developed, in agreement with Suffolk County Council, to ensure adequate treatment is provided for all areas.

Table 2-4 - ACA West SuDS mitigation indices for discharges to surface waters

ACA area	Assigned Pollution hazard levels	SuDS features proposed	Total SuDS mitigation Index		
			TSS	Metals	Hydrocarbons
Topsoil compound	High	- Swale (bioretention) - Basin	>0.925 (>0.8)	>0.925 (>0.8)	>0.925 (>0.9)

Table 2-5 - ACA East 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
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)

Notes:

* 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 SUMMARY

3.1.1 The construction site has a number of WMZs and it has been shown that they can meet the SuDS manual design criteria for flood risk and treatment. Differences lie between the proposed basins and SuDS compliant basins and therefore bioretention features are proposed for the swale components. The basins are unusual in their overall size and do not completely comply with the SuDS manual, however in recognition that these basins do perform some treatment in the system we have proposed that their value is half that normally used in the assessment.

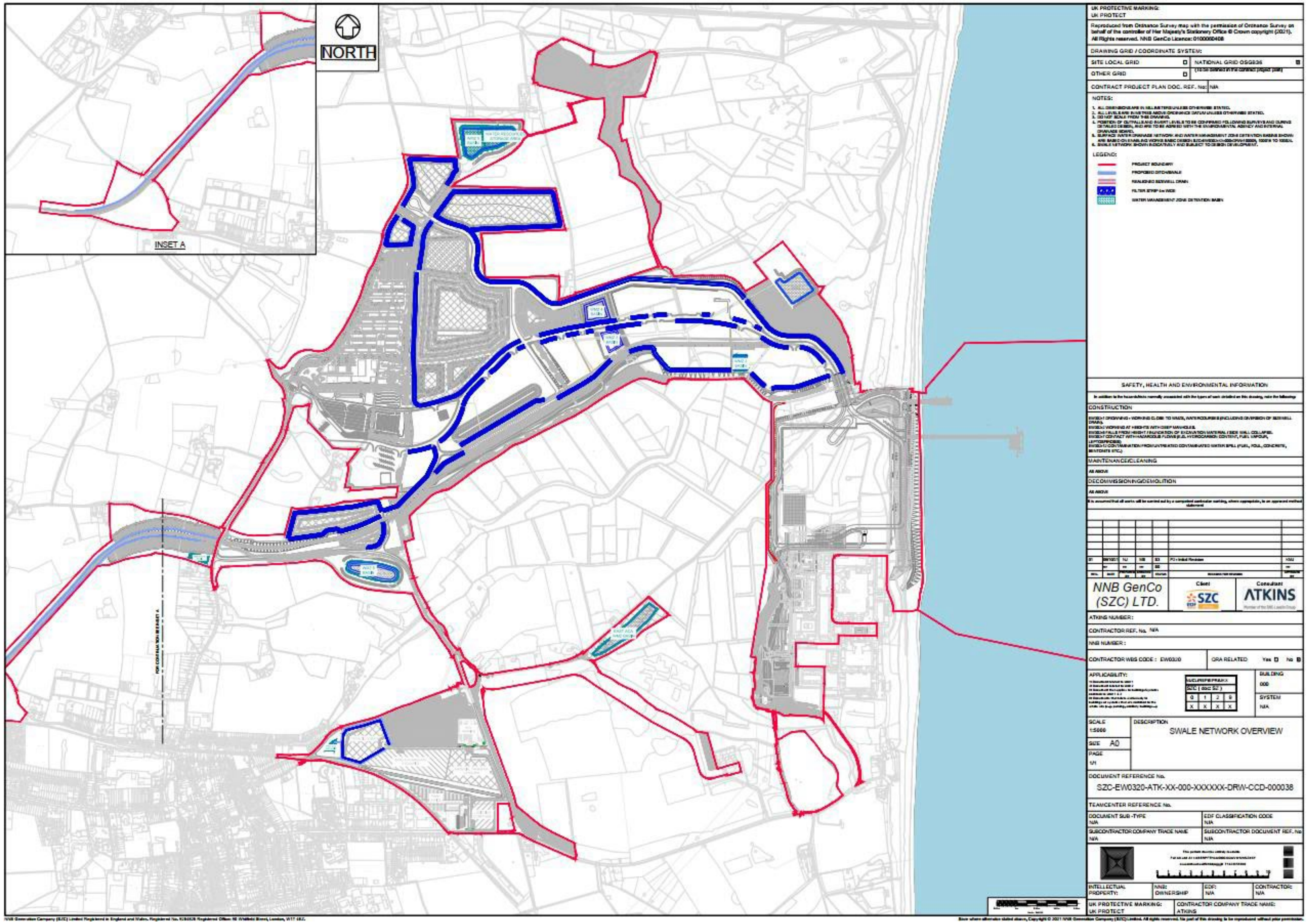
3.1.2 Areas that remain to be addressed in terms of treatment are the ACA areas: Railway, HGV Parking and Sand and Aggregate Stockpile. It is proposed that additional measures for pollution control are to be included within the detailed design.

3.1.3 It can be demonstrated that the filter strips and swales have adequate space allocated to them in drawings SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000038 and SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000010 and that the proposed swales are able to deliver their design treatment capacity.

3.1.4 It has been proposed that the basins (except the GRR basin and ACA East) are able to contribute some treatment, through retention time and sequential filling, and therefore a value of half the normal Simple Index Method be used in the overall treatment assessment. When adopting this approach, it has been shown that the proposed surface water design does meet the treatment requirement of the SuDS manual for all the WMZs proposed except for 3 limited areas in the ACA East.

APPENDIX A: DRAWINGS

A.1. Swale Network Overview - SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000038


NORTH
INSET A
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UK PROTECT
DRAWING GRID / COORDINATE SYSTEM:
SITE LOCAL GRID
NATIONAL GRID OSGB36
OTHER GRID
CONTRACT PROJECT PLAN DOC. REF. No: N/A
NOTES:
LEGEND:
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
QRA RELATED
APPLICABILITY:
BUILDING
SYSTEM
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SCALE
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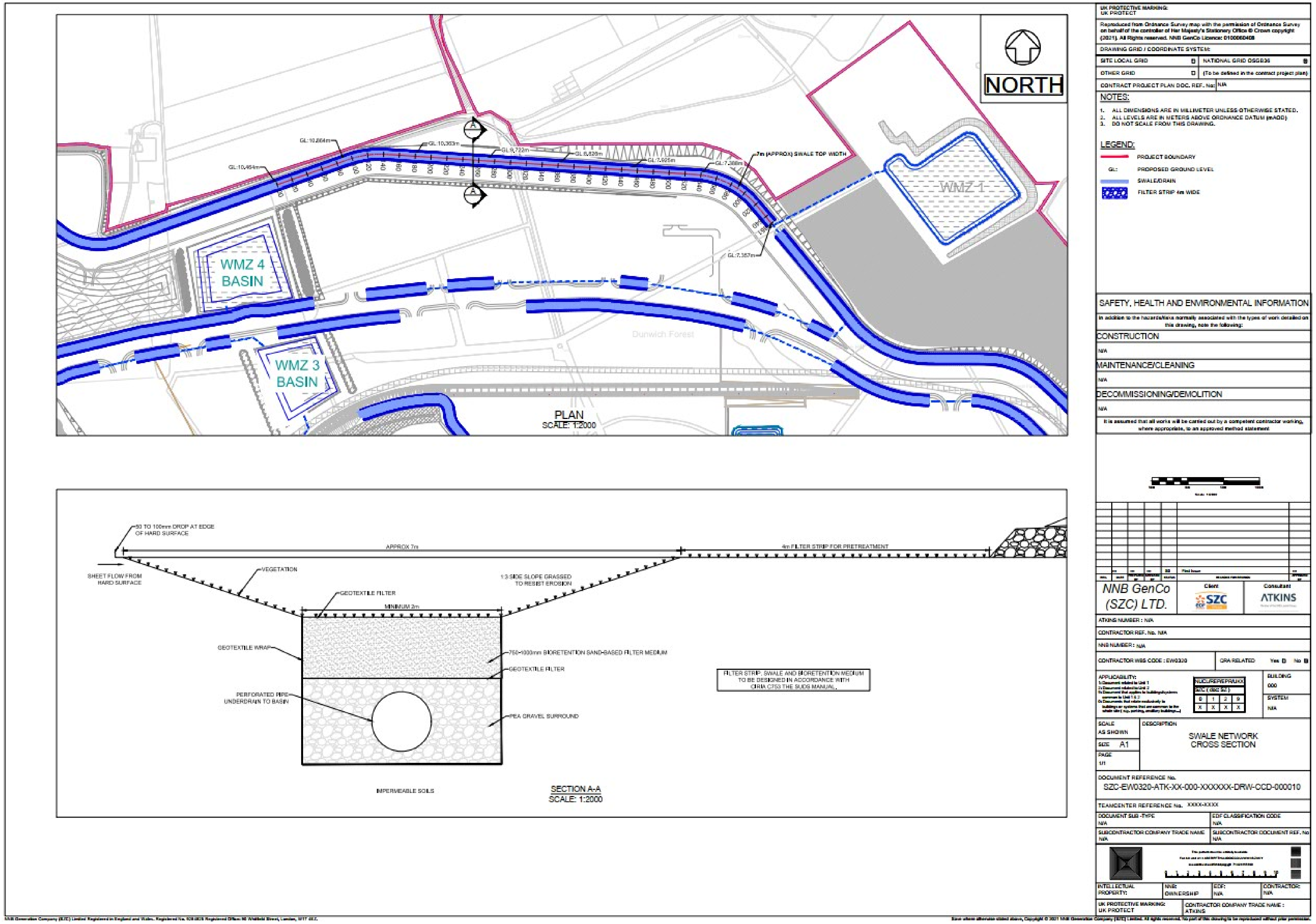
DESCRIPTION
SWALE NETWORK OVERVIEW
SIZE A0
PAGE
1/1
DOCUMENT REFERENCE No.
SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000038
TEAMCENTER REFERENCE No.
DOCUMENT SUB -TYPE
N/A
EDF CLASSIFICATION CODE
N/A
SUBCONTRACTOR COMPANY TRADE NAME
N/A
SUBCONTRACTOR DOCUMENT REF. No
N/A
INTELLECTUAL PROPERTY:
NNB OWNERSHIP
N/A
EDF:
N/A
CONTRACTOR:
N/A
UK PROTECTIVE MARKING:
UK PROTECT
CONTRACTOR COMPANY TRADE NAME:
ATKINS

A.2. Swale Network Cross Section - SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CCD-000010


NORTH
WMZ 1
WMZ 4 BASIN
WMZ 3 BASIN
7m (APPROX) SWALE TOP WIDTH
Dunwich Forest
PLAN
SCALE: 1:2000
50 TO 100mm DROP AT EDGE OF HARD SURFACE
APPROX 7m
4m FILTER STRIP FOR PRETREATMENT
SHEET FLOW FROM HARD SURFACE
VEGETATION
1:3 SIDE SLOPE GRASSED TO RESIST EROSION
GEOTEXTILE FILTER
MINIMUM 2m
GEOTEXTILE WRAP
750-1000mm BIORETENTION SAND-BASED FILTER MEDIUM
GEOTEXTILE FILTER
PERFORATED PIPE UNDERDRAIN TO BASIN
PEA GRAVEL SURROUND
FILTER STRIP, SWALE AND BIORETENTION MEDIUM TO BE DESIGNED IN ACCORDANCE WITH CIRIA C753 THE SUDS MANUAL.
IMPERMEABLE SOILS
SECTION A-A
SCALE: 1:2000
UK PROTECTIVE MARKING:
UK PROTECT
Reproduced from Ordnance Survey map with the permission of Ordnance Survey on behalf of the controller of Her Majesty's Stationery Office © Crown copyright (2021). All Rights reserved. NNB GenCo Licence: 0100060408
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 MILLIMETER UNLESS OTHERWISE STATED.
2. ALL LEVELS ARE IN METERS ABOVE ORDNANCE DATUM (mAOD)
3. DO NOT SCALE FROM THIS DRAWING.
LEGEND:
PROJECT BOUNDARY
GL: PROPOSED GROUND LEVEL
SWALE/DRAIN
FILTER STRIP 4m WIDE
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
N/A
MAINTENANCE/CLEANING
N/A
DECOMMISSIONING/DEMOLITION
N/A
It is assumed that all works will be carried out by a competent contractor working, where appropriate, to an approved method statement
NNB GenCo (SZC) LTD.
Client
Consultant
SZC
ATKINS
ATKINS NUMBER : N/A
CONTRACTOR REF. No. N/A
NNB NUMBER: N/A
CONTRACTOR WBS CODE : EW0320
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BUILDING
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SYSTEM
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AS SHOWN
SIZE A1
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1/1
DESCRIPTION
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DOCUMENT REFERENCE No.
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TEAMCENTER REFERENCE No. XXXX-XXXX
DOCUMENT SUB -TYPE
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EDF CLASSIFICATION CODE
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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 B: SIMPLE INDEX APPROACH (TCA)

B.1. Pathway 1

B.1.1. Pathway 1 - Surface water runoff passes through a filter strip before entering a swale, where water is infiltrated through bioretention medium and to the ground via infiltration.

SUMMARY TABLE	
Land Use Type	Other industrial site area
Pollution Hazard Level	High
Pollution Hazard Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.9
SuDS components proposed	
Component 1	Filter strip
Component 2	None
Component 3	None
SuDS Pollution Mitigation Indices	
TSS	0.4
Metals	0.4
Hydrocarbons	0.5
Groundwater protection type	Bioretention component underlain by 300 mm minimum depth of soils with good contamination attenuation potential
Groundwater protection Pollution Mitigation Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.8
Combined Pollution Mitigation Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.9
Acceptability of Pollution Mitigation	
TSS	Sufficient
Metals	Sufficient
Hydrocarbons	Sufficient

B.2. Pathway 2

B.2.1. Pathway 2 - Surface water runoff passes through a filter strip before entering a swale, where water is infiltrated through bioretention medium and conveyed to a WMZ basin, where water is discharged to the ground via infiltration at the basin.

SUMMARY TABLE	
Land Use Type	Other industrial site area
Pollution Hazard Level	High
Pollution Hazard Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.9
SuDS components proposed	
Component 1	Filter strip
Component 2	Bioretention system (where the system is not designed as an infiltration component)
Component 3	Basin (user defined – 0.25, 0.25, 0.3)
SuDS Pollution Mitigation Indices	
TSS	0.925
Metals	0.925
Hydrocarbons	>0.95
Groundwater protection type	300 mm minimum depth of soils with good contamination attenuation potential
Groundwater protection Pollution Mitigation Indices	
TSS	0.4
Metals	0.3
Hydrocarbons	0.3
Combined Pollution Mitigation Indices	
TSS	>0.95
Metals	>0.95
Hydrocarbons	>0.95
Acceptability of Pollution Mitigation	
TSS	Sufficient
Metals	Sufficient
Hydrocarbons	Sufficient

B.3. Pathway 3

B.3.1. Pathway 3 - Surface water runoff passes through a filter strip before entering a swale, where water is infiltrated through bioretention medium and conveyed to a WMZ basin, where water is discharged to a surface water.

SUMMARY TABLE	
Land Use Type	Other industrial site area
Pollution Hazard Level	High
Pollution Hazard Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.9
SuDS components proposed	
Component 1	Filter strip
Component 2	Bioretention system (where the system is not designed as an infiltration component)
Component 3	Basin (user defined - 0.25, 0.25, 0.3)
SuDS Pollution Mitigation Indices	
TSS	0.925
Metals	0.925
Hydrocarbons	>0.95
Groundwater protection type	None
Groundwater protection Pollution Mitigation Indices	
TSS	0
Metals	0
Hydrocarbons	0
Combined Pollution Mitigation Indices	
TSS	0.925
Metals	0.925
Hydrocarbons	>0.95
Acceptability of Pollution Mitigation	
TSS	Sufficient
Metals	Sufficient
Hydrocarbons	Sufficient

ANNEX 2A.18: PIMP VALUES EXPLANATORY NOTE

PIMP Values Explanatory Note

CONTENTS

TABLES

APPENDICES

1 INTRODUCTION

1.1.1 The SZC construction site is divided up into a number of water management zones (WMZs). These zones have a variety of surface features that vary in their permeability. This short technical note describes the permeability values chosen to suit the various surface finishes and shows how an overall value was calculated for each WMZ.

2 PERCENTAGE IMPERMEABLE (PIMP) VALUES

2.1.1 There is a variety of finishes across the proposed construction site and the PIMP values assigned have been those commonly accepted within the industry.

Table 2.1: Design PIMP for surface types

Surface Finish	PIMP Value (%)
Paved Areas (Roads, other hard surfaces)	100
Roofed Buildings	100
Unpaved (Grassed Verges, other landscaped areas)	50
Soft (Stockpiled areas)	30

2.1.2 The surface area of each type of finish within each WMZ has been estimated and assigned the relevant PIMP value. This approach can then be used within the hydraulic models to predict the flow characteristics and storage requirements needed within each WMZ.

2.2 Revised WMZ PIMP Values

2.2.1 For each WMZ a PIMP % weighted summation of all the areas contributing was derived. This overall PIMP % figure was then compared to the previously derived figures using Road surfaces as 90% (**Table 2.2** and **Table 2.3**). The results highlight that 4 out of the 10 zones have a higher overall PIMP % than previously considered.

Table 2.2: Calculated PIMP % for each WMZ

WMZ	Total Catchment Area (ha)	Total Catchment Area (m²)	Area type (m²) and associated PIMP (%)				Overall Catchment PIMP (%)
			Roofing	Paved	Unpaved[1]	Soft[2]	
			100%	100%	50%	30%	
WMZ1	19.43	194300	34070	87778	72452	0	81%
WMZ2	17.37	173700	61410	94247	18043	0	95%
WMZ3	20.96	209600	5149	148757	55694	0	87%
WMZ4	33.32	333200	0	29572	85303	205441	40%
WMZ5	31.20	311952	0	11512	253282	47159	49%
WMZ6	47.77	477700	17345	99984	319495	40876	61%
ACA East	25.22	252220	100% PIMP Considered				100%
ACA West	4.438	44380	100% PIMP Considered				100%
Abbey Road	6.478	64780	50	300	64780	0	51%
Campus	20.48	204800	46533	84012	74255	0	82%

Notes

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.

Table 2.3: Overall PIMP % comparison per WMZ

WMZ	Original Design PIMP %	Revised PIMP %	Change in PIMP %
WMZ 1	90	81	-9
WMZ 2	90	95	+5
WMZ 3	90	87	-3
WMZ 4	50	40	-10
WMZ 5	50	49	-1
WMZ 6	58	61	+3
ACA East	100	100	0
ACA West	100	100	0
Abbey Road	50	51	+1
Campus	80	82	+2

2.2.2 This approach using a PIMP value for Roads as 100% has highlighted some WMZs to be over designed and others to be under designed. There are 4 WMZs that show an increase in the overall PIMP value.

2.2.3 The PIMP value for WMZ 2 has the most significant increase (+5%), however even with this increase, the basin volume space-proofed (17694.5 m^3) in the development site is greater than that required to contain the 1:100yr storm event +20% allowance for climate change (13629.7 m^3). Estimated volumes were calculated using Innovyze Source Control and do not include upstream storage. Parameters are summarised in **Appendix A** along with Source Control calculations.

2.2.4 The WMZ 6 basin volume space-proofed (19376.0 m^3) in the main development site is slightly lower than the volume estimated using the increased PIMP value (21071.1 m^3). Excluding any upstream storage, this increased volume can be contained within the freeboard depth of the basin. The drainage network is proposed to include approximately 3000 m^3 upstream of the basin and will be sufficient to contain the increased estimated volume. Further modelling will be undertaken during Detailed Design to verify this, and the size of the basin will be refined accordingly to assure the 1:100yr +CC volume can be contained.

2.2.5 The drainage strategy and required storage for the Abbey Road Basin (Green Rail Route) and Campus areas are currently being developed. The calculated PIMP values in this assessment will be adopted unless significant changes in the catchment area definition are identified through design development. For both catchments, sufficient storage will be provided to contain the runoff for the 1:100yr +CC critical storm event.

3 SUMMARY

3.1.1 The change from 90% to 100% for the PIMP value chosen for roads has not made any significant changes to the design of the WMZs. The basin volumes space-proofed in the development site are adequate and show minor changes. The catchment areas, types of surfacing and associated PIMP values may change as the design progresses. The surface water drainage design will be developed in agreement with Suffolk County Council.

APPENDIX A: STORAGE ESTIMATE VERIFICATION

A.1. Source Control Update for WMZ 2 and WMZ 6

Table 3.1: Source Control Volume Summary for 100yr +20% CC critical storm event

WMZ	Catch-ment Area (ha)	PIMP (%)	Imperm-eable Area (ha)	Infiltrat-ion Rate (m/hr)	Out flow (l/s)	Max volume (m^3) 100yr RP	Critical Storm Event	Volume allocated in MDS (m^3)	+/- delta
WMZ2	17.37	95%	16.5015	0.0272	17.37	13629.7 (FEH 2013)	2160 min Winter (FEH 2013)	17694.5	4064.8
WMZ6	47.77	61%	29.1397	0.0201	47.77	21071.1 (FEH 2013)	1440 min Winter (FEH 2013)	22376.0	1304.9

Table 3.2: Volumetric Runoff Coefficient, Cv

WMZ	PIMP (%)	SOIL	SAAR	UCWI (winter)	PR (winter)	Cv (winter)	UCWI (summer)	PR (summer)	Cv (sum-mer)
WMZ2	95	0.15	580	122	71.321	0.751	50	65.705	0.692
WMZ6	61	0.15	581	122	43.135	0.707	50	37.519	0.615
		Ref 1	*Ref 1*	*Ref 2*	*Eq 7.3 Ref 3*	*Eq 7.21 Ref 3*	*Ref 2*	*Eq 7.3 Ref 3*	*Eq 7.21 Ref 3*

Ref 1 - UK SuDS Greenfield Estimation Tool [redacted]

Ref 2 - Figure 6.2 of Urban Drainage 3rd Edition David Butler and John W. Davies

Ref 3 - Design and Analysis of Urban Storm Drainage - The Wallingford Procedure, Volume 1, September 1981

A.2. WMZ2 Basin Source Control

Atkins (Epsom)		Page 1
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 20/01/2022 08:29 File WMZ2 FEH13 old(95).SRCX	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Half Drain Time : 4812 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	3.996	0.796	3.0	13.6	16.5	2839.6	O K
30 min Summer	4.256	1.056	4.0	13.6	16.8	3865.8	O K
60 min Summer	4.520	1.320	5.0	13.6	16.8	4958.5	O K
120 min Summer	4.829	1.629	6.3	13.6	18.1	6308.8	O K
180 min Summer	5.037	1.837	7.2	13.6	19.6	7258.1	O K
240 min Summer	5.196	1.996	7.8	13.6	20.8	8011.0	O K
360 min Summer	5.435	2.235	8.9	13.7	22.6	9176.0	O K
480 min Summer	5.604	2.404	9.6	14.2	23.8	10029.2	O K
600 min Summer	5.726	2.526	10.2	14.5	24.7	10659.8	O K
720 min Summer	5.817	2.617	10.6	14.7	25.3	11138.1	O K
960 min Summer	5.935	2.735	11.1	15.1	26.2	11770.7	O K
1440 min Summer	6.038	2.838	11.6	15.3	26.9	12330.6	O K
2160 min Summer	6.049	2.849	11.6	15.3	27.0	12393.3	O K
2880 min Summer	5.996	2.796	11.4	15.2	26.6	12102.7	O K
4320 min Summer	5.844	2.644	10.7	14.8	25.5	11279.5	O K
5760 min Summer	5.716	2.516	10.1	14.5	24.6	10607.8	O K
15 min Winter	4.059	0.859	3.2	13.6	16.6	3082.8	O K
30 min Winter	4.337	1.137	4.3	13.6	16.8	4197.4	O K
60 min Winter	4.619	1.419	5.4	13.6	16.8	5384.8	O K
120 min Winter	4.949	1.749	6.8	13.6	19.0	6853.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.080	0.0	1359.4	27
30 min Summer	68.208	0.0	1422.5	42
60 min Summer	43.872	0.0	2834.6	72
120 min Summer	28.074	0.0	2860.8	132
180 min Summer	21.656	0.0	2963.8	190
240 min Summer	18.024	0.0	3117.0	250
360 min Summer	13.906	0.0	3384.0	370
480 min Summer	11.513	0.0	3561.0	490
600 min Summer	9.884	0.0	3677.9	610
720 min Summer	8.689	0.0	3755.0	728
960 min Summer	7.018	0.0	3828.5	968
1440 min Summer	5.090	0.0	3802.7	1446
2160 min Summer	3.610	0.0	7478.3	2164
2880 min Summer	2.801	0.0	7331.2	2880
4320 min Summer	1.933	0.0	6801.6	3632
5760 min Summer	1.483	0.0	12674.8	4336
15 min Winter	100.080	0.0	1381.4	27
30 min Winter	68.208	0.0	1431.7	41
60 min Winter	43.872	0.0	2834.5	72
120 min Winter	28.074	0.0	2914.8	130

Atkins (Epsom)	Page 2
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW	
Date 20/01/2022 08:29 File WMZ2 FEH13 old(95).SRCX	Designed by HIRA5452 Checked by
Innovyze	Source Control 2020.1.3

Summary of Results for 100 year Return Period (+20%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
180 min Winter	5.171	1.971	7.7	13.6	20.6	7887.6	O K
240 min Winter	5.340	2.140	8.5	13.6	21.9	8708.6	O K
360 min Winter	5.595	2.395	9.6	14.1	23.7	9982.1	O K
480 min Winter	5.775	2.575	10.4	14.6	25.0	10917.8	O K
600 min Winter	5.906	2.706	11.0	15.0	26.0	11612.3	O K
720 min Winter	6.003	2.803	11.4	15.2	26.7	12141.1	O K
960 min Winter	6.131	2.931	12.0	15.6	27.6	12847.6	O K
1440 min Winter	6.246	3.046	12.6	15.8	28.4	13496.1	O K
2160 min Winter	6.270	3.070	12.7	15.9	28.6	13629.7	O K
2880 min Winter	6.227	3.027	12.5	15.8	28.3	13385.2	O K
4320 min Winter	6.070	2.870	11.7	15.4	27.1	12508.2	O K
5760 min Winter	5.927	2.727	11.1	15.0	26.1	11726.4	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
180 min Winter	21.656	0.0	3089.2	188
240 min Winter	18.024	0.0	3283.5	248
360 min Winter	13.906	0.0	3562.2	364
480 min Winter	11.513	0.0	3745.7	482
600 min Winter	9.884	0.0	3866.0	600
720 min Winter	8.689	0.0	3944.3	718
960 min Winter	7.019	0.0	4016.0	952
1440 min Winter	5.090	0.0	3977.7	1418
2160 min Winter	3.610	0.0	7871.0	2104
2880 min Winter	2.801	0.0	7701.3	2772
4320 min Winter	1.932	0.0	7117.0	4020
5760 min Winter	1.483	0.0	13295.6	4512

Atkins (Epsom)		Page 3
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 20/01/2022 08:29 File WMZ2 FEH13 old(95).SRCX	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 647450 264900 TM 47450 64900
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.692
Cv (Winter)	0.751
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+20

Time Area Diagram

Total Area (ha) 16.501

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	5.500	4	8	5.500	8	12	5.501

Atkins (Epsom)		Page 4
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 20/01/2022 08:29 File WMZ2 FEH13 old(95).SRCX	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Model Details

Storage is Online Cover Level (m) 7.200

Infiltration Basin Structure

Invert Level (m) 3.200 Safety Factor 1.5
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 1.00
Infiltration Coefficient Side (m/hr) 0.02720

Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	3290.1	3.700	6274.5	4.000	6554.8

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0150-1740-3700-1740
Design Head (m)	3.700
Design Flow (l/s)	17.4
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	150
Invert Level (m)	3.200
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1500

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	3.700	17.4
Flush-Flo™	0.647	13.6
Kick-Flo®	1.334	10.7
Mean Flow over Head Range	-	13.4

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	5.4	1.200	12.0	3.000	15.7	7.000	23.6
0.200	11.0	1.400	11.0	3.500	16.9	7.500	24.4
0.300	12.4	1.600	11.7	4.000	18.0	8.000	25.2
0.400	13.1	1.800	12.3	4.500	19.1	8.500	25.9
0.500	13.5	2.000	13.0	5.000	20.1	9.000	26.6
0.600	13.6	2.200	13.6	5.500	21.0	9.500	27.3
0.800	13.5	2.400	14.1	6.000	21.9		
1.000	13.1	2.600	14.7	6.500	22.8		

A.3. WMZ6 Basin Source Control

Atkins (Epsom)		Page 1
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		
Date 20/01/2022 08:27 File WMZ6 FEH13 (61) actual....	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Half Drain Time : 3000 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	8.576	0.576	3.9	47.7	51.6	4426.1	O K
30 min Summer	8.767	0.767	5.3	47.7	52.6	6022.5	O K
60 min Summer	8.959	0.959	6.7	47.7	52.9	7707.5	O K
120 min Summer	9.185	1.185	8.4	47.7	52.9	9771.3	O K
180 min Summer	9.336	1.336	9.5	47.7	52.9	11214.5	O K
240 min Summer	9.452	1.452	10.4	47.7	52.9	12351.2	O K
360 min Summer	9.623	1.623	11.7	47.7	53.9	14070.8	O K
480 min Summer	9.740	1.740	12.7	47.7	56.3	15292.1	O K
600 min Summer	9.822	1.822	13.3	47.7	57.9	16162.6	O K
720 min Summer	9.881	1.881	13.8	47.7	59.1	16792.8	O K
960 min Summer	9.950	1.950	14.3	47.7	60.5	17547.6	O K
1440 min Summer	9.989	1.989	14.7	47.7	61.2	17967.9	O K
2160 min Summer	9.940	1.940	14.3	47.7	60.3	17438.4	O K
15 min Winter	8.657	0.657	4.5	47.7	52.2	5094.8	O K
30 min Winter	8.872	0.872	6.0	47.7	52.9	6934.6	O K
60 min Winter	9.089	1.089	7.6	47.7	52.9	8880.3	O K
120 min Winter	9.343	1.343	9.5	47.7	52.9	11274.3	O K
180 min Winter	9.512	1.512	10.9	47.7	52.9	12948.1	O K
240 min Winter	9.641	1.641	11.9	47.7	54.3	14259.8	O K
360 min Winter	9.832	1.832	13.4	47.7	58.1	16264.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.080	0.0	3468.4	31
30 min Summer	68.208	0.0	4254.6	45
60 min Summer	43.872	0.0	7118.3	74
120 min Summer	28.074	0.0	8555.6	134
180 min Summer	21.656	0.0	8779.4	194
240 min Summer	18.024	0.0	8724.1	254
360 min Summer	13.906	0.0	8662.6	372
480 min Summer	11.513	0.0	8664.7	492
600 min Summer	9.884	0.0	8690.2	610
720 min Summer	8.689	0.0	8725.2	730
960 min Summer	7.018	0.0	8800.7	968
1440 min Summer	5.090	0.0	8783.2	1444
2160 min Summer	3.610	0.0	17336.2	2104
15 min Winter	100.080	0.0	3882.0	30
30 min Winter	68.208	0.0	4391.1	45
60 min Winter	43.872	0.0	8008.4	74
120 min Winter	28.074	0.0	8822.4	132
180 min Winter	21.656	0.0	8777.5	190
240 min Winter	18.024	0.0	8766.3	250
360 min Winter	13.906	0.0	8844.4	366

Atkins (Epsom)		Page 2
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 20/01/2022 08:27 File WMZ6 FEH13 (61) actual....	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control Σ (l/s)	Max Outflow (l/s)	Max Volume (m³)	Status
480 min Winter	9.964	1.964	14.5	47.7	60.7	17702.6	O K
600 min Winter	10.058	2.058	15.2	47.7	62.5	18739.2	O K
720 min Winter	10.125	2.125	15.8	48.1	63.8	19499.7	Flood Risk
960 min Winter	10.208	2.208	16.4	49.0	65.4	20440.4	Flood Risk
1440 min Winter	10.262	2.262	16.8	49.5	66.4	21071.1	Flood Risk
2160 min Winter	10.229	2.229	16.6	49.2	65.8	20681.8	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
480 min Winter	11.513	0.0	8989.1	484
600 min Winter	9.884	0.0	9167.9	600
720 min Winter	8.689	0.0	9345.2	716
960 min Winter	7.019	0.0	9520.9	950
1440 min Winter	5.090	0.0	9475.9	1408
2160 min Winter	3.610	0.0	18231.8	2076

Atkins (Epsom)		Page 3
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 20/01/2022 08:27 File WMZ6 FEH13 (61) actual....	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 647450 264900 TM 47450 64900
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.615
Cv (Winter)	0.707
Shortest Storm (mins)	15
Longest Storm (mins)	2160
Climate Change %	+20

Time Area Diagram

Total Area (ha) 29.139

Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)	Time (mins) From:	To:	Area (ha)
0	4	7.285	4	8	7.285	8	12	7.285	12	16	7.284

Atkins (Epsom)		Page 4
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 20/01/2022 08:27 File WMZ6 FEH13 (61) actual....	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Model Details

Storage is Online Cover Level (m) 10.400

Infiltration Basin Structure

Invert Level (m) 8.000 Safety Factor 1.5
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 1.00
Infiltration Coefficient Side (m/hr) 0.02010

Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	7165.8	2.100	11287.5	2.400	11911.5

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0274-4780-2100-4780
Design Head (m)	2.100
Design Flow (l/s)	47.8
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	274
Invert Level (m)	8.000
Minimum Outlet Pipe Diameter (mm)	300
Suggested Manhole Diameter (mm)	2100

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	2.100	47.8
Flush-Flo™	0.621	47.7
Kick-Flo®	1.362	38.8
Mean Flow over Head Range	-	41.2

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	8.6	1.200	43.3	3.000	56.8	7.000	85.6
0.200	28.2	1.400	39.3	3.500	61.2	7.500	88.6
0.300	43.8	1.600	41.9	4.000	65.3	8.000	91.4
0.400	46.2	1.800	44.4	4.500	69.1	8.500	94.1
0.500	47.3	2.000	46.7	5.000	72.7	9.000	96.8
0.600	47.7	2.200	48.9	5.500	76.2	9.500	99.4
0.800	47.2	2.400	51.0	6.000	79.5		
1.000	45.9	2.600	53.0	6.500	82.6		

ANNEX 2A.19: CAMPUS OUTLINE DRAINAGE STRATEGY TECHNICAL NOTE

Campus Outline Drainage Strategy – Technical Note

CONTENTS

TABLES

PLATES

APPENDICES

1 INTRODUCTION

1.1.1 This technical note provides an overview of the surface water drainage strategy for Water Management Zone 10 (WMZ10) of the Sizewell C (SZC) development, also referred to as the Accommodation Campus area.

1.1.2 This document is not intended to provide detailed design of WMZ10, but instead begin discussions on the drainage strategy and design development in collaboration with Suffolk County Council (SCC).

2 OUTLINE DRAINAGE STRATEGY

2.1.1 The Campus Accommodation area is approximately 20ha and is in the western end of Temporary Construction Area (TCA). The Campus is designated for accommodation. The surface water drainage strategy for WMZ10 relies on discharge to a watercourse or surface water drainage network. Discharging runoff to the ground at source through infiltration is not considered feasible at this stage due to the low infiltration rates measured in recent ground investigations. This is discussed in more detail in the following section.

2.1.2 Runoff is proposed to be stored below ground in areas such as car parks and other paved areas located within the catchment. The majority of parking is provided within a two-storey car park at the northern end of the site. This is likely to be of modular, steel framed construction and will feature a flat roof above the top deck.

2.1.3 WMZ10 will be split into two sub-catchments. One sub-catchment will capture runoff from the car park roof, whilst the other sub-catchment will capture runoff from all other areas including access ways between buildings and non-heavily tracked areas. The sub-catchment strategies are discussed in more detail from Section 2.5 onwards.

2.2 Infiltration Rates

2.2.1 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 2021 show that the rates vary with a lowest recording of 8.56 x10-7 m/s (Test ACC-TP01-EW Fugro October 2021 Draft). This worst-case rate is less than 2.78 x10-6 m/s – 10 mm/hr, therefore discharge via infiltration from the Campus is not feasible.

2.2.2 Raw test results are included in **Table 2-1** below and **Appendix A**. These show that out of the five campaigns carried out since 2014, only one infiltration rate measurement is BRE 365 compliant. This result is taken from the 2021 Draft Factual Report, the final report of which is due to be published in early February 2021.

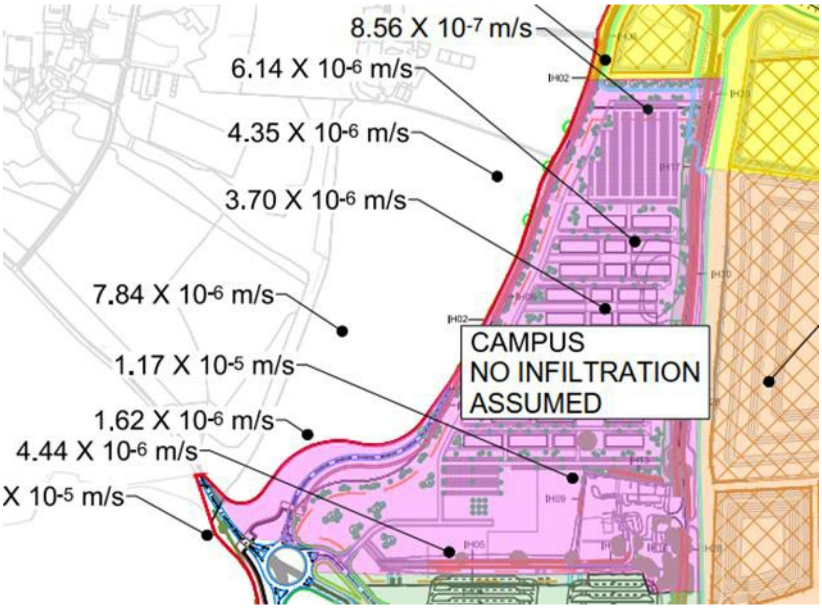
Table 2.1: Infiltration Test Summary

Campaign	BRE 365 Compliant	Reference	Tests Carried Out	Infiltration Rate(s) (m/s)	Notes
2021 (Fugro Draft Factual Report)	Yes, these tests were carried out with the approach set out in BRE Digest 365.	ACC-TP01-EW	3	8.56E-07 N/A N/A	Water level variations cannot be discerned around 25% EDP for Test 1 and 75% EDP for Test 3.
		ACC-TP17-EW	3	N/A N/A N/A	Water level did not reach 25% EDP for Test 1 and 2; water level variations cannot be discerned around 75% and 25% EDP for Test 3.
2020 (Fugro)	In general accordance – not fully compliant	CAMPUS_2020-1	3	7.99E-06 9.99E-06 6.14E-06	Lowest value: 6.14E-06
		CAMPUS_2020-2	3	1.17E-05 1.85E-05 1.36E-05	Lowest value: 1.17E-05
2017 (Structural Soils Ltd)	In general accordance – not fully compliant	TP-C-11	1	7.84E-06	
		TP-C-12	1	1.62E-06	
		TP-CPB-C-16	1	4.35E-06	
2015 (Structural Soils Ltd)	No infiltration tests carried out within Campus site area.				
	In general accordance	SA2	3	4.68E-06 4.44E-06	Lowest value: 4.44E-06

Campaign	BRE 365 Compliant	Reference	Tests Carried Out	Infiltration Rate(s) (m/s)	Notes
2014 (Structural Soils Ltd)	– not fully compliant			4.76E-06	
		SA3	3	6.22E-06 3.70E-06 4.63E-06	Lowest value: 3.70E-06
		SA4	3	8.47E-06 5.66E-05 2.88E-05	Lowest value: 8.47E-06

2.2.3 The infiltration rates taken for test locations inside the Campus area are shown in **Plate 2-1**.

Plate 2.1: Infiltration rates recorded in WMZ10



2.3 Allowable Discharge

2.3.1 Given infiltration rates are too low, the runoff is proposed to be discharged at greenfield rates to the Leiston Drain. The conveyed runoff from WMZ10 will either join the discharge downstream of a WMZ basin which has an outfall or will be managed by a complex control to ensure the existing WMZ functions as intended. The final outfall position will consider the existing runoff conditions and flow paths described in Section 2.4. Based on the existing catchment definition, it is proposed that this water would join the WMZ4 outfall. Alternatively, this area could be discharged towards

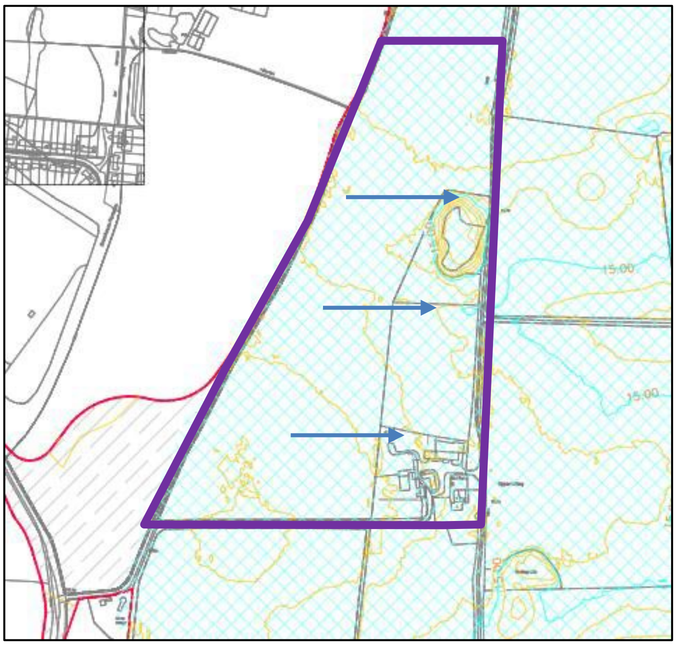
WMZ6 (south of the Campus), which discharges to the Leiston Drain at Lover's Lane. The final outfall position is to be reviewed and developed in consultation with Suffolk County Council (SCC).

2.3.2 In coordination with the discharge strategy agreed for other TCA areas, the proposed rate for WMZ10 will be limited to 1 l/s/ha (20.48 l/s), as the QBAR (peak rate of flow from a catchment for the mean annual flood - return period of approximately 1:2.3 years) for WMZ10 is estimated as 2.76 l/s using the IH124 method (see **Appendix B** for greenfield runoff rate calculation). This proposed rate had previously been accepted for outline design by SCC.

2.4 Runoff in the existing condition

2.4.1 WMZ10 is at a high level and varies from 15 to 19 mAOD. **Plate 2-2** below shows the existing contours and indicates the catchment falls from west to east, towards WMZ4. The groundwater contours from Winter 2018 included in the Environmental Statement showed the groundwater levels are approximately between 1.8 and 2.2 mAOD (see **Appendix C**), a considerable depth below the proposed ground levels.

Plate 2.2: Indicative WMZ10 catchment overlaid on existing surface level contours



2.5 Catchment Design Parameters

2.5.1 This section presents the catchment parameters used in the input to determine approximate storage volumes for WMZ10.

2.5.2 As shown in **Table 2-2** below, an average Percentage Impervious (PIMP) of 82% is calculated for WMZ10 by acknowledging individual areas and associated PIMP factors within the catchment. This estimate was taken forward to determine the Percentage Runoff (PR) and Volumetric Runoff Coefficient (Cv).

Table 2.2: WMZ10 PIMP Estimation

Total Catchment Area (m^2)	Area (m^2) and assigned PIMP (%)			Overall PIMP	PIMP used in storage estimate
	Roofed	Paved	Unpaved (e.g. verges, soft landscaping)		
	100%	100%	50%		
204800	46533	84012	74255	82%	82%

2.5.3 A more accurate value for the Percentage Runoff (PR) and Volumetric Runoff Coefficient, Cv, for WMZ10 was calculated using equations 7.3 and 7.21 of Design and Analysis of Urban Storm Drainage - The Wallingford Procedure, Volume 1, September 1981. **Table 2-3** below shows the PR and Cv values for summer and winter profiles.

Table 2.3: WMZ10 Percentage Runoff and Volumetric Runoff Coefficient Cv Calculation

Catchment	PIMP (%)	SOIL index	SAAR (mm)	UCWI (winter)	PR (winter)	Cv (winter)	UCWI (summer)	PR (summer)	Cv (summer)
WMZ10	82	0.15	581	122	60.544	0.738	50	54.928	0.670

2.5.4 As previously mentioned, WMZ10 is split into two sub-catchments: the car park roof sub-catchment and the rest of the site hereafter referred to as the pervious pavement sub-catchment. The sub-catchment design parameters are detailed in the following sections.

a) Car Park Roof

2.5.5 The decked car park is proposed at the northern end of the Campus site, shown in **Plate 2.3**.

Plate 2.3: Decked Car Park Area

2.5.6 As shown in **Table 2-4** below, a Percentage Impervious (PIMP) of 100% is calculated for the car park roof and this was taken forward to determine the Percentage Runoff (PR) and Volumetric Runoff Coefficient (Cv).

Table 2.4: Car Park Roof PIMP Estimation

Total Catchment Area (m^2)	Area (m^2) and assigned PIMP (%)	Overall PIMP	PIMP used in storage estimate
	Roofed		
	100%		
13000	13000	100%	100%

2.5.7 A more accurate value for the Percentage Runoff (PR) and Volumetric Runoff Coefficient, Cv, for the car park roof was calculated using equations 7.3 and 7.21 of Design and Analysis of Urban Storm Drainage - The Wallingford Procedure, Volume 1, September 1981. **Table 2-3** below shows the PR and Cv values for summer and winter profiles.

Table 2.5: Car Park Roof Percentage Runoff and Volumetric Runoff Coefficient Cv Calculation

Catchment	PIMP (%)	SOIL index	SAAR (mm)	UCWI (winter)	PR (winter)	Cv (winter)	UCWI (summer)	PR (summer)	Cv (summer)
Car Park Roof	100	0.15	581	122	75.466	0.755	50	69.85	0.699

b) Pervious Pavement

2.5.8 As shown in **Table 2-6** below, an average Percentage Impervious (PIMP) of 76% is calculated for the pervious pavement sub-catchment by acknowledging individual areas and associated PIMP factors within the catchment. This estimate was taken forward to determine the Percentage Runoff (PR) and Volumetric Runoff Coefficient (Cv).

Table 2.6: Pervious Pavement PIMP Estimation

Total Catchment Area (m^2)	Area (m^2) and assigned PIMP (%)			Overall PIMP	PIMP used in storage estimate
	Roofed	Paved	Unpaved (e.g. verges, soft landscaping)		
	100%	100%	50%		
191808	33541	84012	74255	76%	76%

2.5.9 A more accurate value for the Percentage Runoff (PR) and Volumetric Runoff Coefficient, Cv, for WMZ10 was calculated using equations 7.3 and 7.21 of Design and Analysis of Urban Storm Drainage - The Wallingford Procedure, Volume 1, September 1981. **Table 2-7** below shows the PR and Cv values for summer and winter profiles.

Table 2.7: Pervious Pavement Percentage Runoff and Volumetric Runoff Coefficient Cv Calculation

Catchment	PIMP (%)	SOIL index	SAAR (mm)	UCWI (winter)	PR (winter)	Cv (winter)	UCWI (summer)	PR (summer)	Cv (summer)
WMZ10	76	0.15	581	122	55.57	0.731	50	49.954	0.657

c) Rainfall Parameters

2.5.10 In addition to the above, the following input parameters were used to determine the critical storm volume for a 100-year return period for storm durations up to 7 days, included a 20% allowance for climate change.

Table 2.8: Input parameters for Innovyze Source Control storage volumes

	Parameter	Notes
Rainfall-Runoff method	Flood Studies Report (FSR), Flood Estimation Handbook (FEH) 1999 and 2013	Sensitivity check using FEH 1999 and 2013
Return Period (years)	100	As per DCO Outline Drainage Strategy [1]
Storm duration (minutes)	15 – 10080	As per DCO Outline Drainage Strategy [1]
Climate Change (%)	20	As per DCO Outline Drainage Strategy [1] and EA guidance [2]

[1] Environmental Statement – 6.3 Volume 2 Main Development Site, Chapter 2 Description of the Permanent Development, Appendix 2A Outline Drainage Strategy (EN010012-001802-SZC_Bk6_ES_V2_Ch2_Appx2A)

[2] Environment Agency – Flood risk assessment: climate change allowances - Table 2: peak rainfall intensity allowance in small catchments (less than 5 km^2) or urban drainage catchments (based on a 1961 to 1990 baseline)

2.5.11 All three rainfall-runoff methods were used to undertake sensitivity checks on the design volumes. Using FSR, Sizewell, Suffolk was used as the location with M5-60 and 'r' ratio of 18.2 mm and 0.4 taken respectively. For FEH 1999, the catchment descriptors shown in were inputted.

Table 2.9: FEH 1999 rainfall parameters

FEH Site	C (1km)	D1 (1km)	D2 (1km)	D3 (1km)	E (1km)	F (1km)
GB 647450 264900	-0.02	0.299	0.272	0.215	0.311	2.506

2.6 Storage Estimate

2.6.1 Attenuation of runoff can be provided by several drainage features. Runoff from the sub-catchments will be managed in two ways, the car park roof will utilise geocellular storage whilst permeable paving will be used for the rest of the site. The former will capture runoff from the car park roof only whilst the latter will capture runoff from the majority of the site, including the accommodation blocks and roads, as described in more detail below.

2.6.2 The proposed discharge rate for Campus site is 20.48 l/s. At this stage, to assist with the estimated storage volume required, a proportional discharge rate for each sub-catchment has been considered to ensure the 1 l/s/ha limit is not exceeded. The discharge rate for the car park roof and the pervious pavement sub-catchments are 1.3 and 19.2, respectively. **Table 2-10** below presents the discharge rates for the two sub-catchments described above.

Table 2.10: Sub-catchment flow rates based on 1 l/s/ha

Sub-catchment	PIMP (%)	Area (ha)	Discharge Rate (l/s)
Car Park Roof	100	1.3	1.3
Pervious Pavement	76	19.2	19.2

2.6.3 It is worth noting that although infiltration is not considered viable at this stage, opportunities to provide natural SuDS, using features such as infiltration trenches, rain gardens and/or swales, will be explored during Detailed Design following more detailed ground investigations.

a) Car Park Roof

2.6.4 Attenuation for the decked car park was modelled as a geocellular structure of 500mm depth and with a porosity of 95% in Innovyze Source control. As above, no infiltration was considered in this assessment and a permitted outflow of 1.3l/s was used.

Table 2.11: Decked Car Park Source Control Storage Estimates

Innovyze Source Control Summary					
Rainfall	**Inputted Storage Volume (m^3)**	**Critical Volume (100yr RP) (m^3)**	**Critical Storm Event (100yr RP)**	**Max Water Height in storage (mm)**	**Half Drain Time (mins)**
FSR	878.8	867.6	4320 winter storm	494	5668
FEH 1999	1140.0	1127.7	2880 winter storm	495	7401
FEH 2013	1163.8	1150.5	2880 winter storm	494	7555

2.6.5 As shown in **Table 2-11** the FEH 2013 rainfall-runoff method provided more conservative values in comparison to FEH 1999 and FSR. Results for the FEH 2013 Source Control are shown in **Appendix D**. The output shows that a 1163.8 m^3 of storage is sufficient to store a 100yr +20% CC storm event, which is equivalent to a footprint of 2450 m^2. The plan area of the decked car park is approximately 13000 m^2, therefore it is assumed this attenuation structure can be easily accommodated below the ground level car park. As with the permeable paving, the half drain times are much larger 24-hours, however as the system is not infiltrating it is assumed the 24-hour half drain time rule does not apply.

b) Pervious Pavement

2.6.6 For simplicity, and to provide an initial estimate on the required area necessary to contain sub-surface storage within pervious pavement, an attenuation structure of 600mm depth was modelled in Innovyze Source Control as a basin with a porosity of 30% to symbolise a graded granular sub-base.

2.6.7 As described above, the worst-case measured infiltrate rate of 8.56 x10-7 m/s is taken as the assumed infiltration rate for the Campus at this stage and it is therefore assumed no infiltration is possible in the Campus area. A permitted outflow of 19.18 l/s (equivalent 1 l/s/ha) was included in the assessment.

Table 2.12: Pervious Pavement Source Control Storage Estimates

Innovyze Source Control Summary					
Rainfall	**Inputted Storage Volume (m^3)**	**Critical Volume (100yr RP) (m^3)**	**Critical Storm Event (100yr RP)**	**Max Water Height in storage (mm)**	**Half Drain Time (mins)**
FSR	9180	9085.3	2880 winter storm	594	3956
FEH 1999	12060	11910.1	2880 winter storm	593	5213
FEH 2013	12240	12229.1	2160 winter storm	599	5339

2.6.8 As shown in **Table 2-12** the FEH 2013 rainfall-runoff method provided more conservative values in comparison to FEH 1999 and FSR. Results for the FEH 2013 Source Control are shown in **Appendix E**. The output shows that a 12240 m^3 of storage is sufficient to store a 100yr +20% CC storm event, which is equivalent to a footprint of 68000 m^2, which is less than the available paved area within the catchment - 84000 m^2. The half drain times are much larger 24-hours, however as the system is not infiltrating it is assumed the 24-hour half drain time rule does not apply.

2.6.9 At this stage, where there are areas of hardstanding proposed (except for the car park roof, discussed below), it is proposed that these areas use Type C permeable surfacing (no infiltration to subgrade). The surfacing will be robustly constructed, emulating the current drainage characteristics, whilst providing suitable treatment of any incidental oil spills. Access ways between buildings and non-heavily tracked areas with the Campus will also employ a Type C permeable surface. 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. Consideration will also be given to blue/green roof infrastructure as well as rainwater harvesting.

2.7 Pollution and Treatment

2.7.1 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, as all roofed areas present low risk and roads are most aligned to low traffic residential parking. The use of porous paving alone for the permeable paving sub-catchment can provide sufficient treatment and the SIA criteria will be satisfied for these areas. As the design develops, further consideration will be given and should parts of the Campus area align to a medium-risk hazard level, porous paving will still satisfy the SIA criteria.

2.7.2 At this stage, the proposed decked car park is proposed to drain through an oil separator to mitigate the risk of hydrocarbon contamination. The drainage design will comply with the initiatives and best practice guidance for pollution prevention for multi-storey car parks. As the car park will be sheltered, the car park surfaces will not drain rainfall, therefore, the oil separator proposed will be a Class 2 discharge to the foul network. Alternatively, and subject to agreement from SCC, flows from the car park may be conveyed to the surface water network with the use of a Class 1 oil separator.

2.7.3 As described in the previous section, a review will be undertaken in the next design stage considering the inclusion of further SuDS features to maximise treatment and prevent adverse effects on the existing hydrology.

3 CONCLUSION

3.1.1 This note outlines the basic surface water drainage strategy for the Campus area. The worst-case infiltration test record has indicated poor infiltration potential within WMZ10, therefore infiltration has not been considered a feasible method of discharge at this stage. An estimate of the required storage has been undertaken with the inclusion of a permitted discharge of 1 l/s/ha. The calculations show that there is sufficient space for attenuation for the Campus, comprising of storage within the sub-base of paved areas, as well as geocellular storage beneath the decked car park. A more detailed analysis will be conducted during design development to determine the actual depth required as well as positioning of pervious systems, geocellular storage and other drainage features, in conjunction with a review of the infiltration potential. Pollution control can be managed by the pervious pavement and through the use of catchpits and oil separators for the car park surfaces.

3.1.2 The Campus surface water drainage design, including the location of the outfall, needs to be developed further, in conjunction with external stakeholders such as East Suffolk Council, SCC, Natural England, the EA and East Suffolk Internal Drainage Board.

APPENDIX A: INFILTRATION TESTING RECORDS

A.1. 2021 Campaign (Draft)

A.1.1. Infiltration testing in 2021 was carried out by Fugro and presented in a Ground Investigation report issued on 16th December 2021. Relevant soakaway test results are extracted below.

NNB Generation Company (SZC) Limited

F.3.1 Soakaway Test Results

Title	Reference
Soakaway Test Results (Trial Pits)	Referenced by Location ID

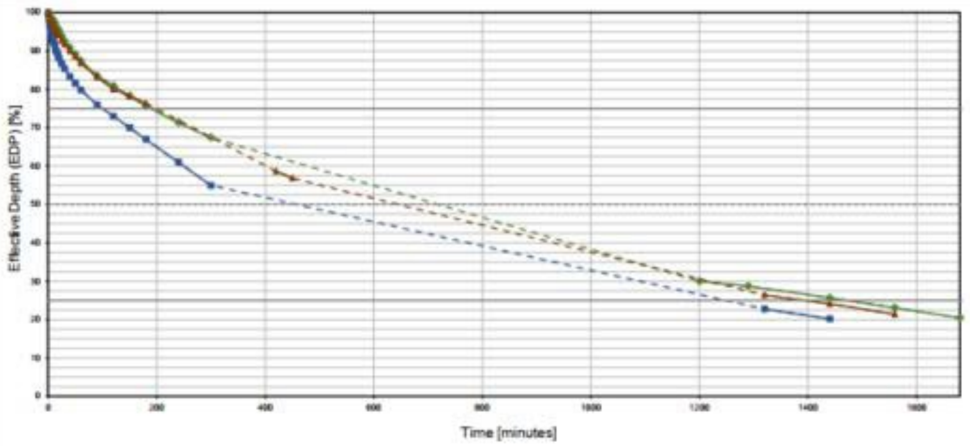
F181383-GIR 01 | Sizewell C – Onshore Ground Investigation – Accommodation Area
Appendix F | F.3.1 | Contents

FUGRO

NNB GENERATION COMPANY (SZC) LIMITED
SIZEWELL C – ONSHORE GROUND INVESTIGATION – ACCOMMODATION AREA

FUGRO

SOAKAWAY TEST RECORD							Method BRE DG 365 (2016)
Date	24/10/2021	Operator	TC	Pit Depth	2.50 m	Hole No.	ACC-TP01-EW



Test 1 ■				Test 2 ◆				Test 3 ▲			
Start Time =		11:02		Start Time =		12:36		Start Time =		08:57	
Test Top =		0.17	m BGL	Test Top =		0.20	m BGL	Test Top =		0.30	m BGL
Test Base =		2.50	m BGL	Test Base =		2.50	m BGL	Test Base =		2.50	m BGL
75% EDP =		0.75	m BGL	75% EDP =		0.78	m BGL	75% EDP =		0.85	m BGL
50% EDP =		1.34	m BGL	50% EDP =		1.35	m BGL	50% EDP =		1.40	m BGL
25% EDP =		1.92	m BGL	25% EDP =		1.93	m BGL	25% EDP =		1.95	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	0.67	0.17	100	00:00:00	0.70	0.20	100	00:00:00	0.80	0.30	100
00:01:30	0.72	0.22	98	00:01:30	0.73	0.23	99	00:01:30	0.81	0.31	100
00:03:00	0.75	0.25	97	00:05:00	0.73	0.23	99	00:05:00	0.84	0.34	98
00:05:00	0.78	0.28	95	00:08:00	0.74	0.24	98	00:06:00	0.85	0.35	98
00:07:00	0.82	0.32	94	00:10:00	0.75	0.25	98	00:08:00	0.86	0.36	97
00:08:00	0.83	0.33	93	00:12:00	0.76	0.26	97	00:09:00	0.87	0.37	97
00:09:00	0.84	0.34	93	00:14:00	0.78	0.28	97	00:10:00	0.88	0.38	96
00:10:00	0.85	0.35	92	00:16:00	0.79	0.29	96	00:12:00	0.89	0.39	96
00:12:00	0.87	0.37	91	00:18:00	0.80	0.30	96	00:14:00	0.90	0.40	95
00:14:00	0.90	0.40	90	00:20:00	0.81	0.31	95	00:16:00	0.91	0.41	95
00:16:00	0.91	0.41	90	00:25:00	0.84	0.34	94	00:18:00	0.93	0.43	94
00:18:00	0.93	0.43	89	00:30:00	0.87	0.37	93	00:20:00	0.93	0.43	94
00:20:00	0.95	0.45	88	00:40:00	0.91	0.41	91	00:25:00	0.96	0.46	93
00:25:00	0.98	0.48	87	00:50:00	0.95	0.45	89	00:30:00	0.98	0.48	92
00:30:00	1.01	0.51	85	01:00:00	0.99	0.49	87	00:40:00	1.02	0.52	90
00:40:00	1.06	0.56	83	01:30:00	1.08	0.58	83	00:50:00	1.05	0.55	89
00:50:00	1.10	0.60	82	02:00:00	1.14	0.64	81	01:00:00	1.09	0.59	87
01:00:00	1.14	0.64	80	02:30:00	1.20	0.70	78	01:30:00	1.17	0.67	83
01:30:00	1.23	0.73	76	03:00:00	1.26	0.76	76	02:00:00	1.24	0.74	80
02:00:00	1.30	0.80	73	04:00:00	1.36	0.86	71	02:30:00	1.28	0.78	78
02:30:00	1.37	0.87	70	05:00:00	1.45	0.95	67	03:00:00	1.32	0.82	76
03:00:00	1.44	0.94	67	20:00:00	2.31	1.81	30	07:00:00	1.71	1.21	59
04:00:00	1.58	1.08	61	21:30:00	2.34	1.84	29	07:30:00	1.75	1.25	57
05:00:00	1.72	1.22	55	24:00:00	2.41	1.91	26	22:00:00	2.42	1.92	26
22:00:00	2.47	1.97	23	26:00:00	2.47	1.97	23	24:00:00	2.47	1.97	24
24:00:00	2.53	2.03	20	28:00:00	2.53	2.03	20	26:00:00	2.53	2.03	21

Datum (-ve denotes AGL) = -0.50 m BGL.
Water depth measurements were taken from the Datum Level chosen on field, then converted to referencing Ground Level for calculations.

Input by RC 03/11/2021 Checked by CAY 22/11/2021 Approved by SAF 16/12/2021

Contract No. F181383 Page 1 of 2

NNB GENERATION COMPANY (SZC) LIMITED
SIZEWELL C – ONSHORE GROUND INVESTIGATION – ACCOMMODATION AREA

FUGRO

SOAKAWAY TEST RECORD							Method BRE DG 365 (2016)
Date	24/10/2021	Operator	TC	Pit Depth	2.50 m	Hole No.	ACC-TP01-EW

Test Details

Datum (-ve denotes AGL) =	-0.50 m BGL	Well Screen Well screen not used		
Pit Length =	2.80 m	Filter Material		
Pit Width =	0.50 m	Assumed Solid Fraction =	63.00 %	
Pit Depth =	2.50 m BGL	Assumed Porosity =	37.00 %	
Weather	Cold, dry, light wind, damp.			
Geology	SAND			

Remarks
Test 1 undertaken on 24/10/2021 and 25/10/2021; Test 2 undertaken on 25/10/2021 and 26/10/2021; Test 3 undertaken on 27/10/2021 and 28/10/2021. Readings taken during site hours only. Volume of gravel fraction assumed to be 63.00% of the total volume of gravel filled space. Water level variations cannot be discerned around 25% EDP for Test 1 and around 75% EDP for Test 3; infiltration rates cannot be given.

Test carried out in gravel filled pit; gravel filled up to 0.30m BGL.
Water depth measurements taken from top of measuring pipe that stuck up 0.50m AGL.

Calculation

Test 1			Test 2			Test 3		
Start Time =	11:02		Start Time =	12:36		Start Time =	08:57	
Test Top =	0.17	m BGL	Test Top =	0.20	m BGL	Test Top =	0.30	m BGL
Test Base =	2.50	m BGL	Test Base =	2.50	m BGL	Test Base =	2.50	m BGL
EDP =	2.33	m	EDP =	2.30	m	EDP =	2.20	m
75% EDP =	0.75	m BGL	75% EDP =	0.78	m BGL	75% EDP =	0.85	m BGL
50% EDP =	1.34	m BGL	50% EDP =	1.35	m BGL	50% EDP =	1.40	m BGL
25% EDP =	1.92	m BGL	25% EDP =	1.93	m BGL	25% EDP =	1.95	m BGL
V =	3.26	m^3	V =	3.22	m^3	V =	3.08	m^3
Vg =	2.06	m^3	Vg =	2.03	m^3	Vg =	1.94	m^3
Vp =	1.21	m^3	Vp =	1.19	m^3	Vp =	1.14	m^3
Vp75-25 =	0.60	m^3	Vp75-25 =	0.60	m^3	Vp75-25 =	0.57	m^3
ap =	9.09	m^2	ap =	8.99	m^2	ap =	8.66	m^2
Tp75 =	6000	s	Tp75 =	11400	s	Tp75 =		s
Tp25 =		s	Tp25 =	88800	s	Tp25 =	83400	s
Infiltration Rate, f =		m/s	Infiltration Rate, f =	8.56E-07	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 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; Vp75-50 is used when 25% EDP was not reached.

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; Tp75-50 is used when 25% EDP was not reached.

$$Soil\ Infiltration\ rate, f = \frac{Vp_{75-25}}{ap \times Tp_{75-25}} \quad \text{or} \quad Soil\ Infiltration\ rate, f = \frac{Vp_{75-50}}{ap \times Tp_{75-50}}$$

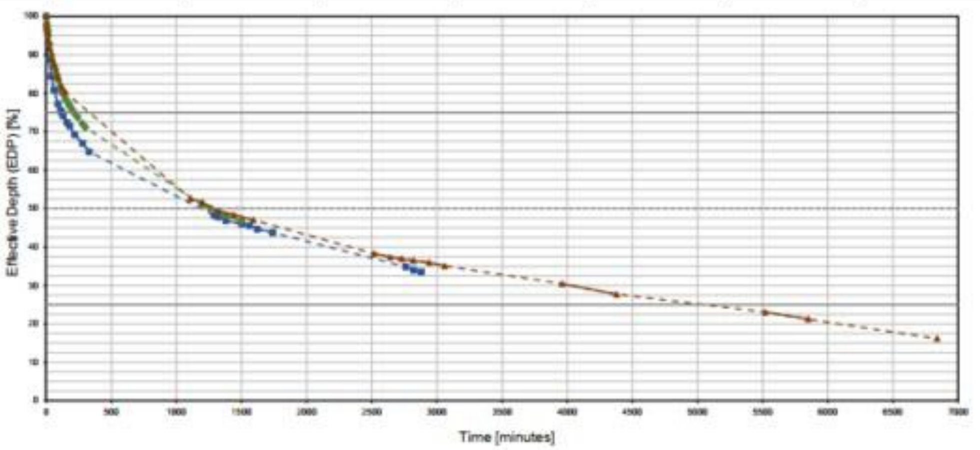
Input by RC 03/11/2021 | Checked by CAY 22/11/2021 | Approved by SAF 16/12/2021

Contract No. F181383 | Page 2 of 2

NNB GENERATION COMPANY (SZC) LIMITED
SIZEWELL C – ONSHORE GROUND INVESTIGATION – ACCOMMODATION AREA

FUGRO

SOAKAWAY TEST RECORD							Method BRE DG 365 (2016)
Date	23/10/2021	Operator	LR	Pit Depth	2.50 m	Hole No.	ACC-TP17-EW



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	10:53		Start Time =	12:08		Start Time =	14:08	
Test Top =	0.26	m BGL	Test Top =	0.32	m BGL	Test Top =	0.33	m BGL
Test Base =	2.50	m BGL	Test Base =	2.50	m BGL	Test Base =	2.50	m BGL
75% EDP =	0.82	m BGL	75% EDP =	0.87	m BGL	75% EDP =	0.87	m BGL
50% EDP =	1.38	m BGL	50% EDP =	1.41	m BGL	50% EDP =	1.42	m BGL
25% EDP =	1.94	m BGL	25% EDP =	1.96	m BGL	25% EDP =	1.96	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	0.76	0.26	100	00:00:00	0.82	0.32	100	00:00:00	0.83	0.33	100
00:05:00	0.81	0.31	98	00:05:00	0.87	0.37	98	00:05:00	0.89	0.39	97
00:10:00	0.85	0.35	96	00:10:00	0.90	0.40	96	00:10:00	0.92	0.42	96
00:16:00	0.92	0.42	93	00:16:00	0.93	0.43	95	00:20:00	0.97	0.47	94
00:25:00	1.01	0.51	89	00:25:00	0.98	0.48	93	00:30:00	1.01	0.51	92
00:40:00	1.11	0.61	84	00:40:00	1.04	0.54	90	00:50:00	1.07	0.57	89
01:00:00	1.19	0.69	81	01:00:00	1.10	0.60	87	01:00:00	1.10	0.60	88
01:30:00	1.27	0.77	77	01:10:00	1.12	0.62	86	01:30:00	1.17	0.67	84
01:50:00	1.31	0.81	75	01:20:00	1.15	0.65	85	02:00:00	1.23	0.73	82
02:00:00	1.32	0.82	75	01:30:00	1.18	0.68	83	02:30:00	1.26	0.76	80
02:10:00	1.34	0.84	74	01:50:00	1.22	0.72	82	18:30:00	1.86	1.36	53
02:40:00	1.38	0.88	72	02:00:00	1.25	0.75	80	20:00:00	1.88	1.38	52
03:00:00	1.40	0.90	71	02:10:00	1.26	0.76	80	22:00:00	1.93	1.43	49
03:40:00	1.45	0.95	69	02:30:00	1.29	0.79	78	24:00:00	1.95	1.45	48
04:40:00	1.50	1.00	67	02:50:00	1.31	0.81	78	26:30:00	1.98	1.48	47
05:30:00	1.55	1.05	65	03:00:00	1.33	0.83	77	42:00:00	2.17	1.67	38
21:30:00	1.92	1.42	48	03:20:00	1.35	0.85	76	44:00:00	2.19	1.69	37
22:00:00	1.93	1.43	48	03:40:00	1.37	0.87	75	45:30:00	2.20	1.70	37
23:00:00	1.95	1.45	47	04:00:00	1.39	0.89	74	47:00:00	2.21	1.71	36
25:00:00	1.97	1.47	46	04:40:00	1.43	0.93	72	49:00:00	2.22	1.72	36
26:00:00	1.98	1.48	46	05:00:00	1.45	0.95	71	51:00:00	2.24	1.74	35
27:00:00	2.00	1.50	45	20:00:00	1.89	1.39	51	66:00:00	2.34	1.84	30
29:00:00	2.02	1.52	44	21:00:00	1.91	1.41	50	73:00:00	2.40	1.90	28
46:00:00	2.22	1.72	35	22:00:00	1.93	1.43	49	92:00:00	2.50	2.00	23
47:00:00	2.24	1.74	34	23:00:00	1.95	1.45	48	97:30:00	2.54	2.04	21
48:00:00	2.25	1.75	33	25:00:00	1.98	1.48	47	114:00:00	2.65	2.15	16

Datum (-ve denotes AGL) = -0.50 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 02/11/2021 | Checked by CAY 09/11/2021 | Approved by SAF 16/12/2021

Contract No. F181383 | Page 1 of 2

NNB GENERATION COMPANY (SZC) LIMITED
SIZEWELL C – ONSHORE GROUND INVESTIGATION – ACCOMMODATION AREA

FUGRO

SOAKAWAY TEST RECORD							Method BRE DG 365 (2016)
Date	23/10/2021	Operator	LR	Pit Depth	2.50 m	Hole No.	ACC-TP17-EW

Test Details

Datum (-ve denotes AGL) =	-0.50 m BGL	Well Screen Well screen not used		
Pit Length =	2.60 m	Filter Material		
Pit Width =	0.50 m	Assumed Solid Fraction =	63.00 %	
Pit Depth =	2.50 m BGL	Assumed Porosity =	37.00 %	

Weather: Cold, dry, light wind, dry ground; very heavy rain noted on 31/10/2021 morning (data discarded).

Geology: SAND

Remarks

Test 1 undertaken from 23/10/2021 to 25/10/2021; Test 2 undertaken on 25/10/2021 and 26/10/2021; Test 3 undertaken from 26/10/2021 to 01/11/2021. Test 1 and Test 2 terminated early as agreed with client representative. Test 3 readings beyond elapsed time 114 hr are discarded (affected by very heavy rain). Readings taken during site hours only (Test 3 readings beyond elapsed time 66hr were taken by client representative). Selective data are presented; see field records for full set of data. Volume of gravel fraction assumed to be 63.00% of the total volume of gravel filled space. Water level did not reach 25% EDP for Test 1 and Test 2; water level variations cannot be discerned around 75% and 25% EDP for Test 3; infiltration rates cannot be given.

Test carried out in gravel filled pit; gravel filled up to 0.30m BGL.
About 900L water was added for Test 1 (2 min 20 sec); about 700L water was added for Test 2 (5 min 20 sec); about 550L water was added for Test 3 (4 min 40 sec). Water depth measurements taken from top of measuring pipe that stuck up 0.50m AGL.

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	10:53		Start Time =	12:08		Start Time =	14:08	
Test Top =	0.26	m BGL	Test Top =	0.32	m BGL	Test Top =	0.33	m BGL
Test Base =	2.50	m BGL	Test Base =	2.50	m BGL	Test Base =	2.50	m BGL
EDP =	2.24	m	EDP =	2.18	m	EDP =	2.17	m
75% EDP =	0.82	m BGL	75% EDP =	0.87	m BGL	75% EDP =	0.87	m BGL
50% EDP =	1.38	m BGL	50% EDP =	1.41	m BGL	50% EDP =	1.42	m BGL
25% EDP =	1.94	m BGL	25% EDP =	1.96	m BGL	25% EDP =	1.96	m BGL
V =	2.91	m^3	V =	2.83	m^3	V =	2.82	m^3
Vg =	1.80	m^3	Vg =	1.80	m^3	Vg =	1.80	m^3
Vp =	1.11	m^3	Vp =	1.03	m^3	Vp =	1.02	m^3
Vp75-25 =	0.56	m^3	Vp75-25 =	0.52	m^3	Vp75-25 =	0.51	m^3
ap =	8.24	m^2	ap =	8.06	m^2	ap =	8.03	m^2
Tp75 =	6900	s	Tp75 =	13200	s	Tp75 =		s
Tp25 =		s	Tp25 =		s	Tp25 =		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 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; Vp75-50 is used when 25% EDP was not reached.

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; Tp75-50 is used when 25% EDP was not reached.

$$Soil\ Infiltration\ rate,\ f = \frac{V_{P75-25}}{ap \times Tp_{75-25}} \quad \text{or} \quad Soil\ Infiltration\ rate,\ f = \frac{V_{P75-50}}{ap \times Tp_{75-50}}$$

Input by AH 02/11/2021 | Checked by CAY 09/11/2021 | Approved by SAF 16/12/2021

Contract No. F181383

Page 2 of 2

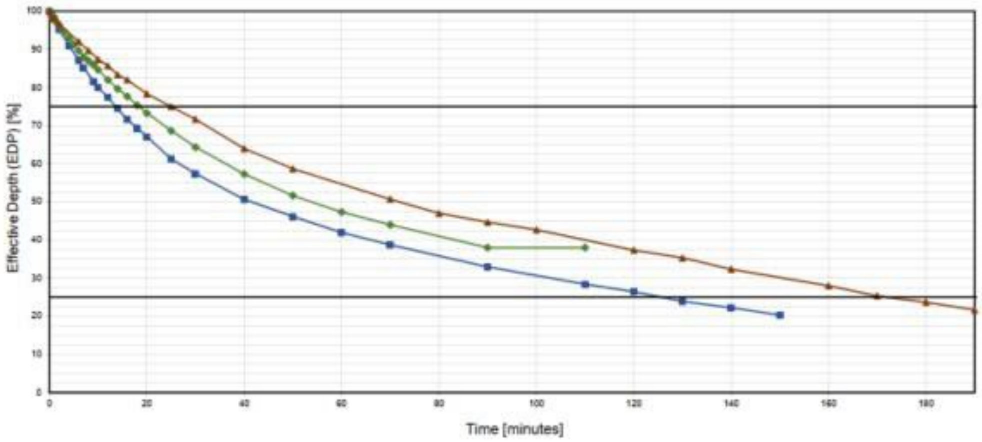
A.2. 2020 Campaign

A.2.1. Infiltration testing in 2020 was carried out by Fugro and presented in a Ground Investigation report issued on 5th June 2020. Relevant soakaway test results are extracted below.

NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD

Date	16/02/2020	Operator	JET	Hole Depth	3.50 m	Hole No.	CAMPUS_2020-1



Test 1 ■				Test 2 ◆				Test 3 ▲			
Start Time =	09:00			Start Time =	11:40			Start Time =	13:10		
Test Top =	0.40	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 =	1.18	m BGL		75% EDP =	1.25	m BGL		75% EDP =	1.25	m BGL	
25% EDP =	2.73	m BGL		50% EDP =	2.00	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	1.10	0.40	100	00:00:00	1.20	0.50	100	00:00:00	1.20	0.50	100
00:00:30	1.12	0.42	99	00:00:30	1.23	0.53	99	00:00:30	1.25	0.55	98
00:01:00	1.16	0.46	98	00:01:00	1.26	0.56	98	00:01:00	1.26	0.56	98
00:02:00	1.25	0.55	95	00:01:30	1.29	0.59	97	00:02:00	1.30	0.60	97
00:04:00	1.38	0.68	91	00:02:00	1.31	0.61	96	00:06:00	1.44	0.74	92
00:06:00	1.50	0.80	87	00:03:00	1.36	0.66	95	00:08:00	1.51	0.81	90
00:07:00	1.56	0.86	85	00:04:00	1.41	0.71	93	00:10:00	1.58	0.88	87
00:09:00	1.67	0.97	82	00:05:00	1.46	0.76	91	00:12:00	1.63	0.93	86
00:10:00	1.72	1.02	80	00:06:00	1.51	0.81	90	00:14:00	1.70	1.00	83
00:12:00	1.80	1.10	77	00:07:00	1.56	0.86	88	00:16:00	1.74	1.04	82
00:14:00	1.89	1.19	75	00:08:00	1.59	0.89	87	00:20:00	1.85	1.15	78
00:16:00	1.98	1.28	72	00:09:00	1.62	0.92	86	00:25:00	1.95	1.25	75
00:18:00	2.05	1.35	69	00:10:00	1.66	0.96	85	00:30:00	2.05	1.35	72
00:20:00	2.12	1.42	67	00:12:00	1.74	1.04	82	00:40:00	2.28	1.58	64
00:25:00	2.30	1.60	61	00:14:00	1.81	1.11	80	00:50:00	2.44	1.74	59
00:30:00	2.42	1.72	57	00:16:00	1.87	1.17	78	01:10:00	2.68	1.98	51
00:40:00	2.63	1.93	51	00:18:00	1.94	1.24	75	01:20:00	2.79	2.09	47
00:50:00	2.77	2.07	46	00:20:00	2.00	1.30	73	01:30:00	2.86	2.16	45
01:00:00	2.90	2.20	42	00:25:00	2.14	1.44	69	01:40:00	2.92	2.22	43
01:10:00	3.00	2.30	39	00:30:00	2.27	1.57	64	02:00:00	3.08	2.38	37
01:30:00	3.18	2.48	33	00:40:00	2.48	1.78	57	02:10:00	3.14	2.44	35
01:50:00	3.32	2.62	28	00:50:00	2.65	1.95	52	02:20:00	3.23	2.53	32
02:00:00	3.38	2.68	26	01:00:00	2.78	2.08	47	02:40:00	3.36	2.66	28
02:10:00	3.46	2.76	24	01:10:00	2.88	2.18	44	02:50:00	3.44	2.74	25
02:20:00	3.51	2.81	22	01:30:00	3.06	2.36	38	03:00:00	3.49	2.79	24
02:30:00	3.57	2.87	20	01:50:00	3.06	2.36	38	03:10:00	3.55	2.85	22

Datum (-ve denotes AGL) = -0.70 m BGL

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

Input by JJL 26/02/2020 | Checked by CAY 28/02/2020 | Approved by NHA 05/06/2020

Contract No. G200003U

Page 1 of 2

NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD							
Date	16/02/2020	Operator	JET	Hole Depth	3.50 m	Hole No.	CAMPUS_2020-1

Test Details

Datum (-ve denotes AGL) =	-0.70 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: Storm Dennis.

Geology: Yellowish brown gravelly SAND.

Remarks
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 at 0.50m BGL.
Water depth measurements were taken from top of pipe 0.70m AGL.

Calculation

Test 1			Test 2			Test 3		
Start Time =	09:00		Start Time =	11:40		Start Time =	13:10	
Test Top =	0.40	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 =	3.10	m	EDP =	3.00	m	EDP =	3.00	m
75% EDP =	1.18	m BGL	75% EDP =	1.25	m BGL	75% EDP =	1.25	m BGL
25% EDP =	2.73	m BGL	50% EDP =	2.00	m BGL	25% EDP =	2.75	m BGL
V =	0.22	m^3	V =	0.21	m^3	V =	0.21	m^3
Vg =	0.06	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	Vp75-50 =	0.04	m^3	Vp75-25 =	0.08	m^3
ap50 =	1.53	m^2	ap =	1.84	m^2	ap50 =	1.48	m^2
Tp75 =	840	s	Tp75 =	1080	s	Tp75 =	1500	s
Tp25 =	7536	s	Tp50 =	3240	s	Tp25 =	10200	s
Infiltration Rate, f =	7.99E-06	m/s	Infiltration Rate, f =	9.99E-06	m/s	Infiltration Rate, f =	6.14E-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 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.

$$Soil\ Infiltration\ rate, f = \frac{Vp_{75-50}}{ap \times Tp_{75-50}}$$

Input by JJL 26/02/2020 | Checked by CAY 28/02/2020 | Approved by NHA 05/06/2020

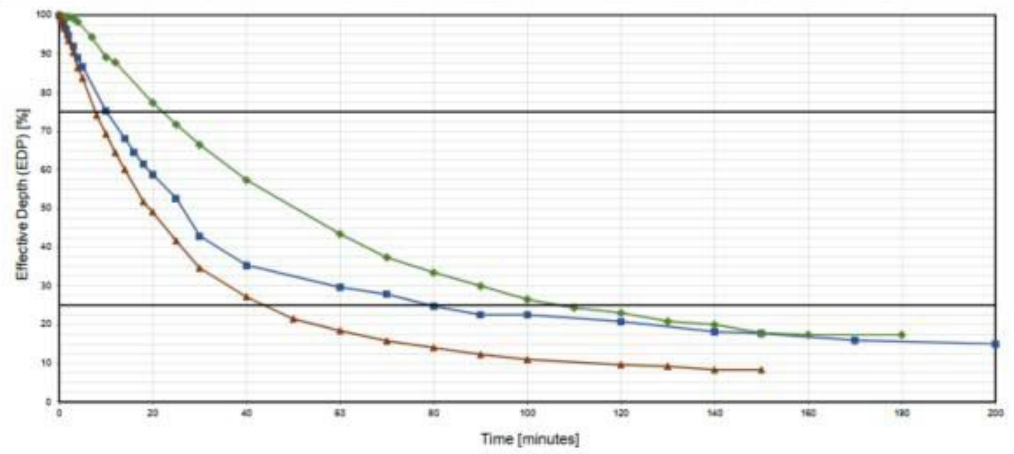
Contract No. G200003U

Page 2 of 2

NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD

Date	15/02/2020	Operator	JJL	Hole Depth	3.50 m	Hole No.	CAMPUS_2020-2



Test 1 ■				Test 2 ◆				Test 3 ▲			
Start Time =	08:59			Start Time =	13:02			Start Time =	08:55		
Test Top =	0.48		m BGL	Test Top =	0.44		m BGL	Test Top =	0.46		m BGL
Test Base =	2.74		m BGL	Test Base =	2.74		m BGL	Test Base =	2.74		m BGL
75% EDP =	1.05		m BGL	75% EDP =	1.02		m BGL	75% EDP =	1.03		m BGL
50% EDP =	1.61		m BGL	50% EDP =	1.59		m BGL	50% EDP =	1.60		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		0.48	100	00:00:00		0.44	100	00:00:00		0.46	100
00:00:30		0.50	99	00:00:30		0.45	100	00:00:30		0.49	99
00:01:00		0.53	98	00:01:00		0.45	100	00:01:00		0.53	97
00:01:30		0.56	96	00:01:30		0.45	100	00:02:00		0.61	93
00:02:00		0.60	95	00:02:00		0.45	100	00:03:00		0.68	90
00:03:00		0.66	92	00:03:00		0.46	99	00:04:00		0.77	86
00:04:00		0.73	89	00:04:00		0.48	98	00:05:00		0.83	84
00:05:00		0.78	87	00:07:00		0.57	94	00:08:00		1.05	74
00:10:00		1.04	75	00:10:00		0.69	89	00:10:00		1.16	69
00:14:00		1.20	68	00:12:00		0.72	88	00:12:00		1.27	64
00:16:00		1.28	65	00:20:00		0.96	77	00:14:00		1.37	60
00:18:00		1.35	62	00:25:00		1.09	72	00:18:00		1.56	52
00:20:00		1.41	59	00:30:00		1.21	67	00:20:00		1.62	49
00:25:00		1.55	53	00:40:00		1.42	57	00:25:00		1.79	42
00:30:00		1.77	43	01:00:00		1.74	43	00:30:00		1.95	35
00:40:00		1.94	35	01:10:00		1.88	37	00:40:00		2.12	27
01:00:00		2.07	30	01:20:00		1.97	33	00:50:00		2.25	21
01:10:00		2.11	28	01:30:00		2.05	30	01:00:00		2.32	18
01:20:00		2.18	25	01:40:00		2.13	27	01:10:00		2.38	16
01:30:00		2.23	23	01:50:00		2.18	24	01:20:00		2.42	14
01:40:00		2.23	23	02:00:00		2.21	23	01:30:00		2.46	12
02:00:00		2.27	21	02:10:00		2.26	21	01:40:00		2.49	11
02:20:00		2.33	18	02:20:00		2.28	20	02:00:00		2.52	10
02:30:00		2.34	18	02:30:00		2.33	18	02:10:00		2.53	9
02:50:00		2.38	16	02:40:00		2.34	17	02:20:00		2.55	8
03:20:00		2.40	15	03:00:00		2.34	17	02:30:00		2.55	8

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

Input by JJL 26/02/2020 — Checked by CAY 28/02/2020 — Approved by NHA 05/06/2020

Contract No. G200003U

Page 1 of 2

NNB GENERATION COMPANY (SZC)
SIZEWELL INFILTRATION TESTING

SOAKAWAY TEST RECORD							
Date	15/02/2020	Operator	JJL	Hole Depth	3.50 m	Hole No.	CAMPUS_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: Wet and windy; Storm Dennis.

Geology: Orangish brown slightly gravelly sandy CLAY.

Remarks
Test 1 and Test 2 undertaken on 15/02/2020; Test 3 undertaken on 16/02/2020.
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 at 0.50m BGL.
Ground water was noted at 3.42m BGL and at 2.74m BGL before Test 1 and Test 3, respectively. Standing water level taken as 2.74m

Calculation

Test 1			Test 2			Test 3		
Start Time =	08:59		Start Time =	13:02		Start Time =	08:55	
Test Top =	0.48	m BGL	Test Top =	0.44	m BGL	Test Top =	0.46	m BGL
Test Base =	2.74	m BGL	Test Base =	2.74	m BGL	Test Base =	2.74	m BGL
EDP =	2.26	m	EDP =	2.30	m	EDP =	2.28	m
75% EDP =	1.05	m BGL	75% EDP =	1.02	m BGL	75% EDP =	1.03	m BGL
50% EDP =	1.61	m BGL	50% EDP =	1.59	m BGL	50% EDP =	1.60	m BGL
V =	0.16	m^3	V =	0.16	m^3	V =	0.16	m^3
Vg =	0.04	m^3	Vg =	0.04	m^3	Vg =	0.04	m^3
Vp =	0.12	m^3	Vp =	0.12	m^3	Vp =	0.12	m^3
Vp75-50 =	0.03	m^3	Vp75-50 =	0.03	m^3	Vp75-50 =	0.03	m^3
ap =	2.57	m^2	ap =	2.56	m^2	ap =	2.56	m^2
Tp75 =	600	s	Tp75 =	1008	s	Tp75 =	294	s
Tp50 =	1590	s	Tp50 =	1650	s	Tp50 =	1160	s
Infiltration Rate, f =	1.17E-05	m/s	Infiltration Rate, f =	1.85E-05	m/s	Infiltration Rate, f =	1.36E-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.
Vp75-50 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 during the test 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.

$$Soil\ Infiltration\ rate, f = \frac{Vp_{75-50}}{ap \times Tp_{75-50}}$$

Input by JJL 26/02/2020 | Checked by CAY 28/02/2020 | Approved by NHA 05/06/2020

Contract No. G200003U | Page 2 of 2

A.3. 2017 Campaign

A.3.1. Infiltration testing in 2017 was carried out by Structural Soils Limited and presented in a Ground Investigation report issued in July 2017. Relevant soakaway test results are extracted below.

FULL SCALE SOAKAWAY TEST

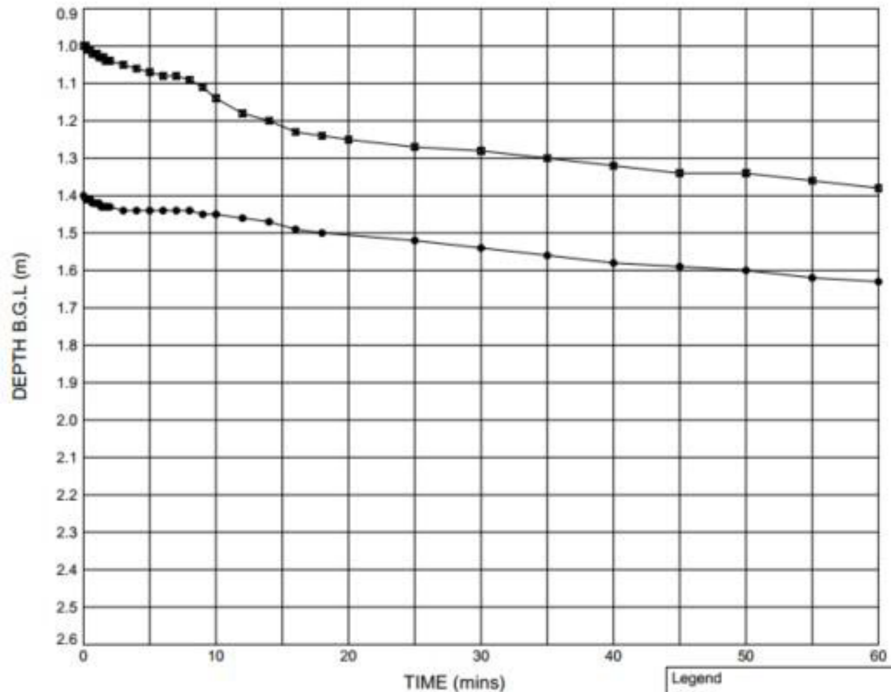
Non-standard test

Soakaway Test - Position ID : **TP-C-11**

Ground Level (m): **15.97**

National Grid Co-ordinates: **E:644915.3 N:264821.6**

PLOT OF DEPTH OF WATER BELOW GROUND LEVEL AGAINST TIME



		Test 1	Test 2	
Pit start depth:	=	*2.70*	**2.70**	m
Pit final depth:	=	*2.70*	**2.70**	m
Effective depth, De	=	*1.30*	**1.70**	m
Effective storage volume, V_{p75-25}	=	*0.4680*	**0.5100**	m^3
Surface area, a_{p50}	=	*3.0600*	**3.3200**	m^2
Time, t_{p75-25}	=	*19500*	**19125**	secs
Infiltration rate, f	=	*7.84×10^{-6}*	**8.03×10^{-6}**	m/s

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

Legend

- ● Test 1 (10.3.17)
- ■ Test 2 (10.3.17)

Plan (Not to scale)

1.00

0.60

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**	

FULL SCALE SOAKAWAY TEST

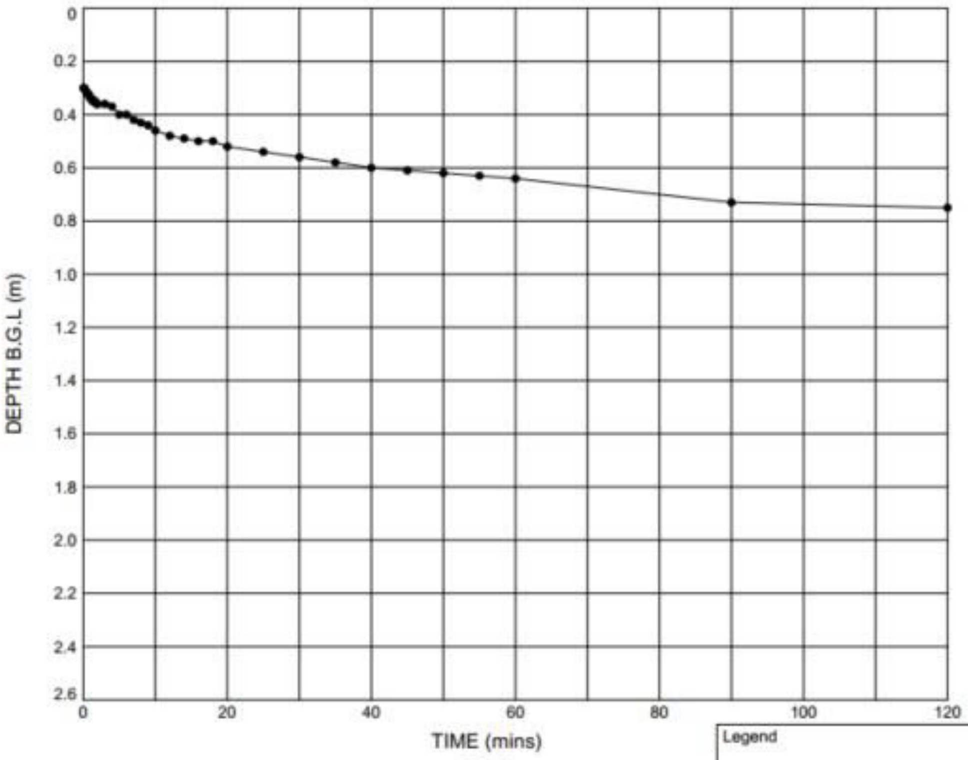
Non-standard test

Soakaway Test - Position ID : **TP-C-12**

Ground Level (m): **17.19**

National Grid Co-ordinates: **E:644870.4 N:264687.2**

PLOT OF DEPTH OF WATER BELOW GROUND LEVEL AGAINST TIME



Test 1

Pit start depth:	=	**2.50**	m
Pit final depth:	=	**2.50**	m
Effective depth, De	=	**2.20**	m
Effective storage volume, V_{p75-25}	=	**0.6600**	m^3
Surface area, a_{p50}	=	**4.1200**	m^2
Time, t_{p75-25}	=	**99000**	secs
Infiltration rate, f	=	**1.62×10^{-6}**	m/s

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

Legend

● Test 1 (10.3.17)

Plan (Not to scale)

1.00

0.60

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 l - TP SOAKAWAY - 2 - FINAL REPORT - A4P | 764156 - SIZEWELL SECTION 53 LAND.GPJ - v8_06 | 14/6/17 - 10:01 | AC7 |

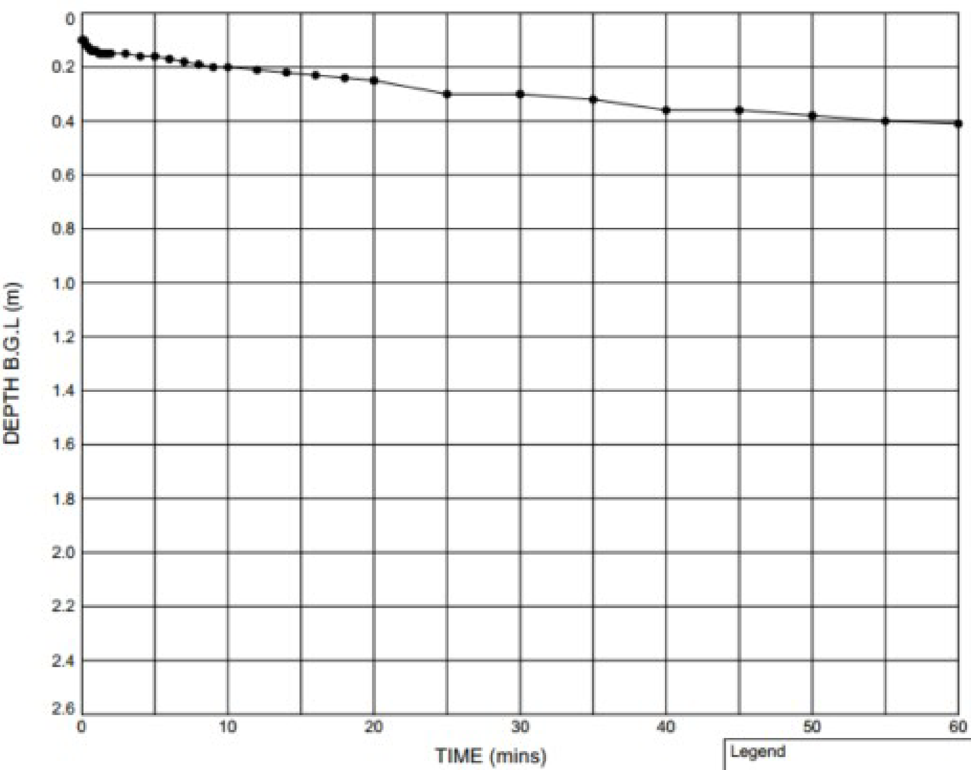
FULL SCALE SOAKAWAY TEST

Non-standard test

Soakaway Test - Position ID : **TP-CPB-C-16**

Ground Level (m): **16.31** National Grid Co-ordinates: **E:645116.8 N:265022.8**

PLOT OF DEPTH OF WATER BELOW GROUND LEVEL AGAINST TIME



Test 1

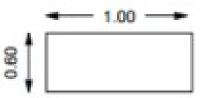
Pit start depth:	=	**2.60**	m
Pit final depth:	=	**2.60**	m
Effective depth, De	=	**2.50**	m
Effective storage volume, $V_{p75\text{-}25}$	=	**0.7500**	m^3
Surface area, a_{p50}	=	**4.6000**	m^2
Time, $t_{p75\text{-}25}$	=	**37500**	secs
Infiltration rate, f	=	**4.35×10^{-6}**	m/s

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

Legend

● Test 1 (9.3.17)

Plan (Not to scale)



No Bearing Taken

STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	Compiled By: [redacted]	Date: 14/6/17	Checked By: [redacted]	Date: 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 - A&P | 764156 - SIZEWELL SECTION 53 LAND.GPJ - v8_06 | 14/6/17 - 10:02 | AC7 |

A.4. 2014 Campaign

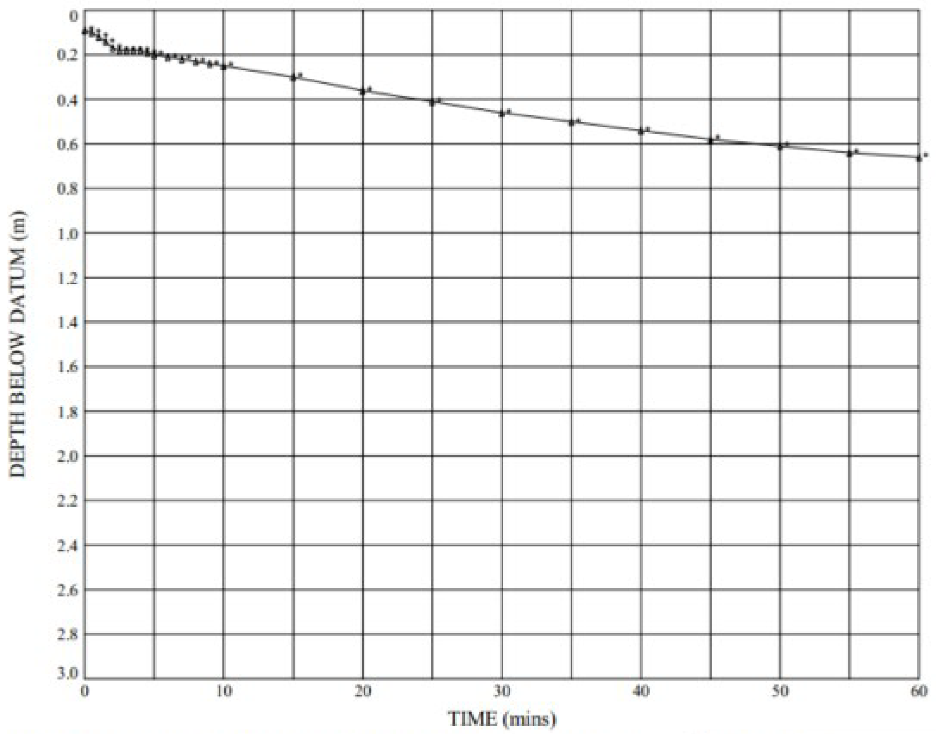
A.4.1. Infiltration testing in 2014 was carried out by Structural Soils Limited and presented in a Ground Investigation report issued on 14th November 2021. Relevant soakaway test results are extracted below.

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA2

Ground Level: **19.09** National Grid Co-ordinates: **E:645057.6 N:264534.2**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.090m to 0.660m).

Volume of water soaking away,	**0.010073**	m^3
Surface area,	**0.5973**	m^2
Time to drain,	**3600**	secs
Infiltration rate,	**4.68x10^{-6}**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **2.60**mBGL
Borehole diameter: **150**mm
Depth to base of casing: **1.37**mBGL
Height of datum above GL: **0.23**m

Legend:

Δ	Test 1	(15/04/2014)

	Compiled By	Date	Checked By	Date
STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	[redacted]	18/11/14	[redacted]	30/10/14
	Contract: **Sizewell Associated Developments**		Contract Ref: **762971**	

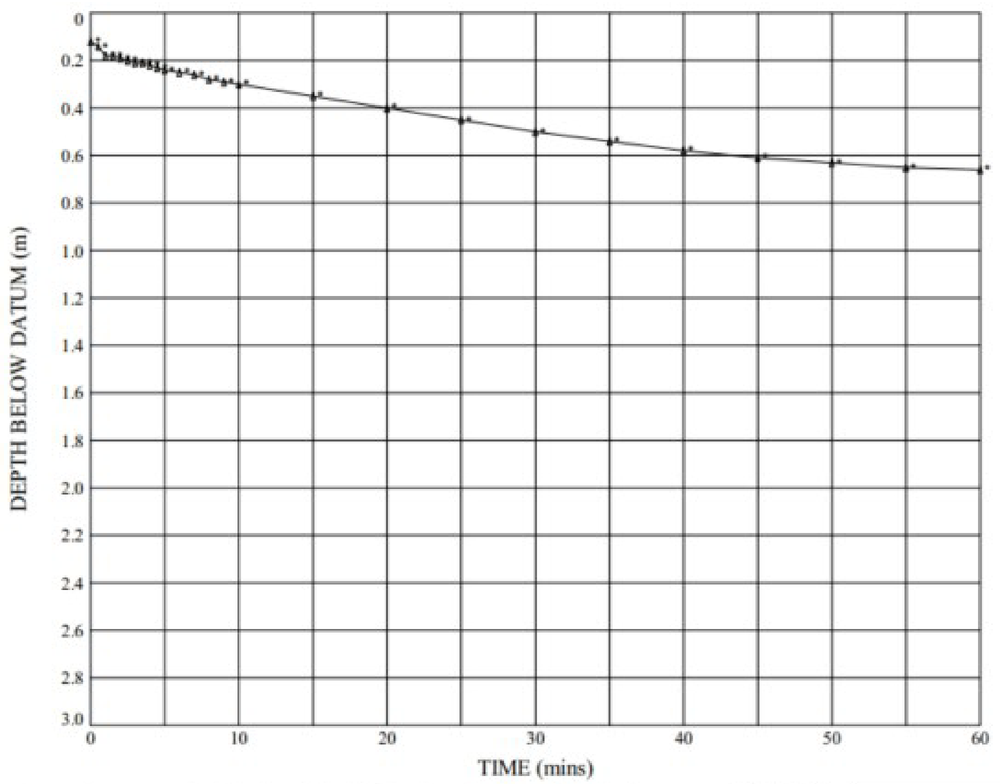
GINT_LIBRARY_V8_05.GLB LibVersion: v8_05 - Lib0004 PrjVersion: v8_05 - Core+Logs 0003 | Graph 1 - BH SOAKAWAY | 762971_SIZEWELL_C.GPJ - v8_05 | 18/11/14 - 13:47 | CH.

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA2

Ground Level: **19.09** National Grid Co-ordinates: **E:645057.6 N:264534.2**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.120m to 0.660m).

Volume of water soaking away,	**0.009543**	m^3
Surface area,	**0.5973**	m^2
Time to drain,	**3600**	secs
Infiltration rate,	**4.44×10^{-6}**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **2.60**mBGL
Borehole diameter: **150**mm
Depth to base of casing: **1.37**mBGL
Height of datum above GL: **0.23**m

Legend:

Δ Test 2 (15/04/2014)

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

Compiled By	Date	Checked By	Date
[redacted]	**18/11/14**	[redacted]	**30/10/14**

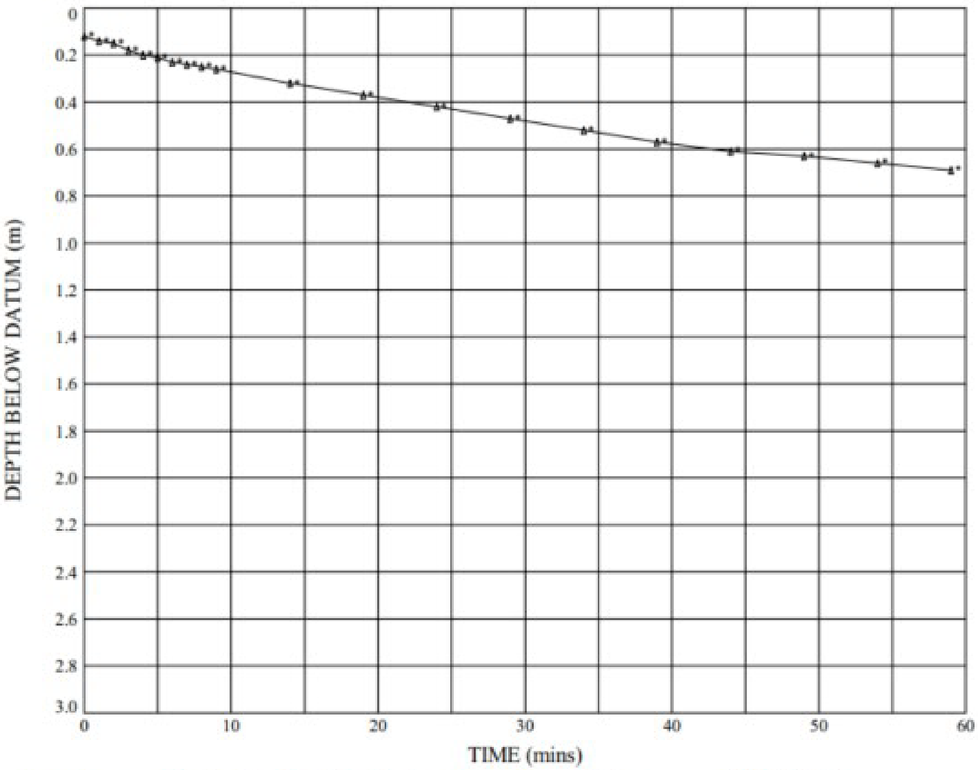
Contract	Contract Ref:
Sizewell Associated Developments	**762971**

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA2

Ground Level: **19.09** National Grid Co-ordinates: **E:645057.6 N:264534.2**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.120m to 0.690m).

Volume of water soaking away,	**0.010073**	m^3
Surface area,	**0.5973**	m^2
Time to drain,	**3540**	secs
Infiltration rate,	**$4.76x10^{-6}$**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **2.60**mBGL
Borehole diameter: **150**mm
Depth to base of casing: **1.37**mBGL
Height of datum above GL: **0.23**m

Legend:

Δ Test 3 (15/04/2014)

STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	Compiled By	Date	Checked By	Date
	[redacted]	18/11/14	[redacted]	30/10/14
	Contract: **Sizewell Associated Developments**		Cont[redacted] **762971**	

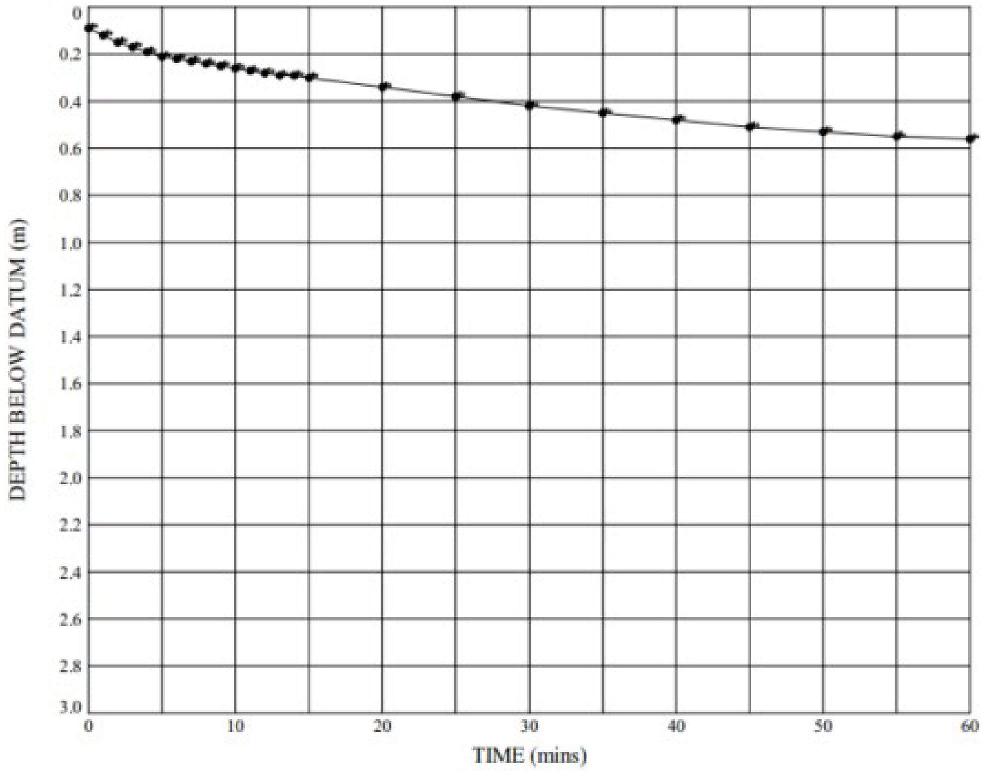
GINT_LIBRARY_V8_05.GLB LibVersion: v8_05 - Lib0004 PrjVersion: v8_05 - Core+Logs 0003 | Graph 1 - BH SOAKAWAY | 762971_SIZEWELL_C.GPJ - v8_05 | 18/11/14 - 13:47 | CH.

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA3

Ground Level: **16.30** National Grid Co-ordinates: **E:645256.9 N:264850.7**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.090m to 0.560m).

Volume of water soaking away,	**0.014765**	m^3
Surface area,	**0.6597**	m^2
Time to drain,	**3600**	secs
Infiltration rate,	**6.22×10^{-6}**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **2.30**mBGL
Borehole diameter: **200**mm
Depth to base of casing: **1.30**mBGL
Height of datum above GL: **0.24**m

Legend:

● Test 1 (16/04/2014)

STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	Compiled By	Date	Checked By	Date
	[redacted]	18/11/14	[redacted]	30/10/14
	Contract: **Sizewell Associated Developments**		**762971**	

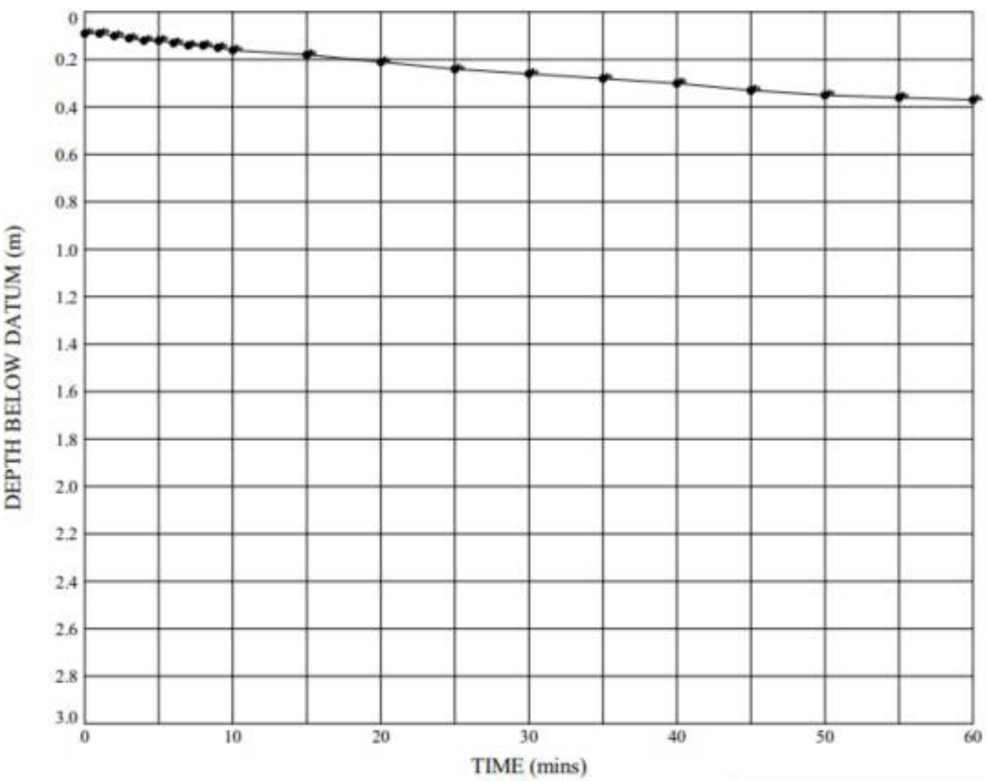
GINT_LIBRARY_V8_05.GLB LibVersion: v8_05 - Lib0004 PrjVersion: v8_05 - Core+Logs 0003 | Graph 1 - BH SOAKAWAY | 762971_SIZEWELL_C.GPJ - v8_05 | 18/11/14 - 13:47 | CH.

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA3

Ground Level: **16.30** National Grid Co-ordinates: **E:645256.9 N:264850.7**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.090m to 0.370m).

Volume of water soaking away,	**0.008796**	m^3
Surface area,	**0.6597**	m^2
Time to drain,	**3600**	secs
Infiltration rate,	**3.70×10^{-6}**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **2.30**mBGL
Borehole diameter: **200**mm
Depth to base of casing: **1.30**mBGL
Height of datum above GL: **0.24**m

Legend:

● Test 2 (17/04/2014)

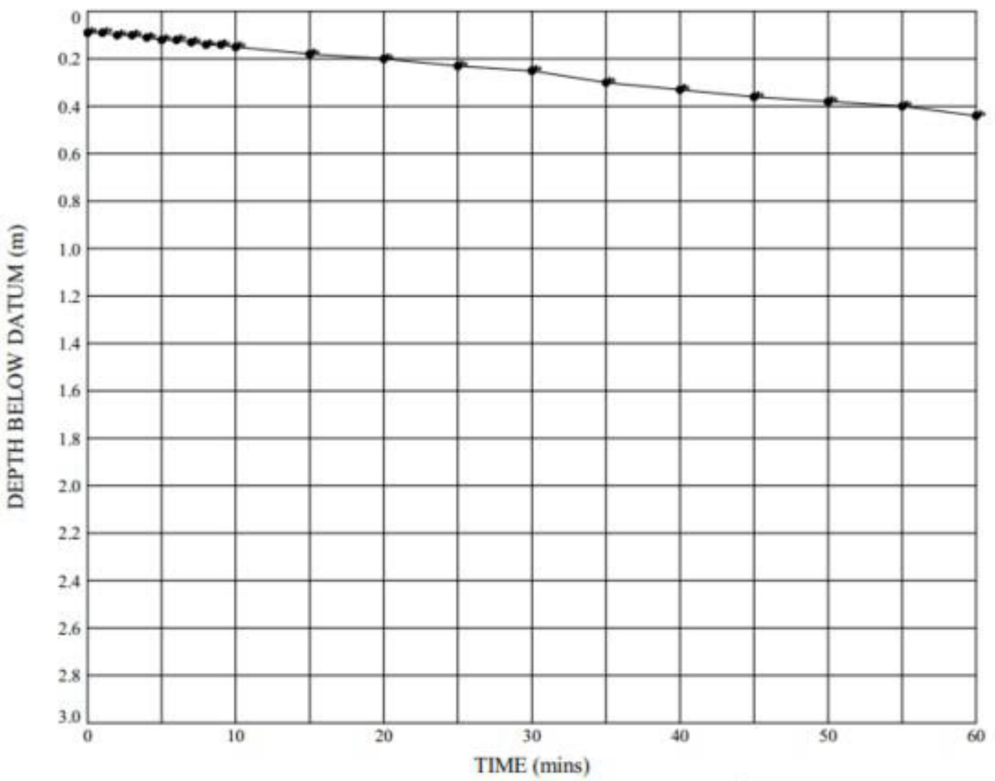
STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	Compiled By	Date	Checked By	Date
	[redacted]	18/11/14	[redacted]	30/10/14
	Contract: **Sizewell Associated Developments**		Contract Ref: **762971**	

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA3

Ground Level: **16.30** National Grid Co-ordinates: **E:645256.9 N:264850.7**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.090m to 0.440m).

Volume of water soaking away,	**0.010996**	m^3
Surface area,	**0.6597**	m^2
Time to drain,	**3600**	secs
Infiltration rate,	**4.63×10^{-6}**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **2.30**mBGL
Borehole diameter: **200**mm
Depth to base of casing: **1.30**mBGL
Height of datum above GL: **0.24**m

Legend:

● Test 3 (17/04/2014)

STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	Compiled By	Date	Checked By	Date
	[redacted]	18/11/14	[redacted]	30/10/14
	Contract: **Sizewell Associated Developments**		Contract Ref: **762971**	

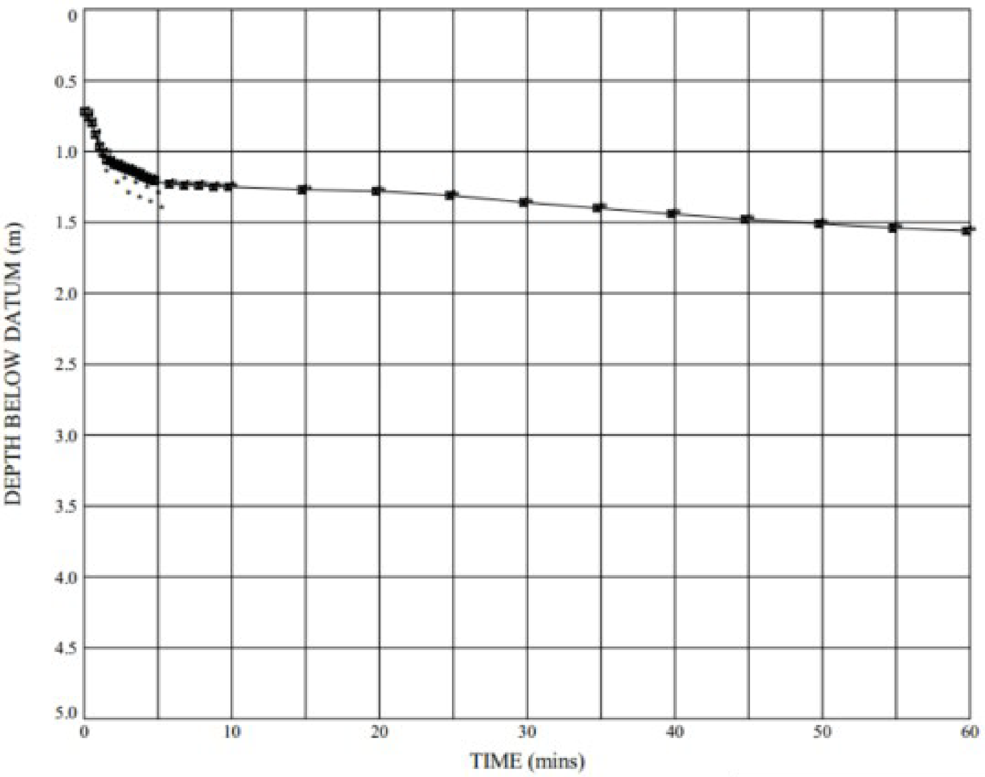
GINT_LIBRARY_V8_05.GLB LibVersion: v8_05 - Lib0004 PrjVersion: v8_05 - Core+Logs 0003 | Graph 1 - BH SOAKAWAY | 762971_SIZEWELL_C.GPJ - v8_05 | 18/11/14 - 13:47 | CH.

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA4

Ground Level: **17.10** National Grid Co-ordinates: **E:645257.8 N:265170.9**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.720m to 1.560m).

Volume of water soaking away,	**0.014844**	m^3
Surface area,	**0.4889**	m^2
Time to drain,	**3585**	secs
Infiltration rate,	**8.47×10^{-6}**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **3.20**mBGL
Borehole diameter: **150**mm
Depth to base of casing: **2.20**mBGL
Height of datum above GL: **1.10**m

Legend:

Test 1 (15/04/2014)

STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	Compiled By [redacted]	Date **18/11/14**	Checked By [redacted]	Date **30/10/14**
	Contract **Sizewell Associated Developments**		Contract Ref: **762971**	

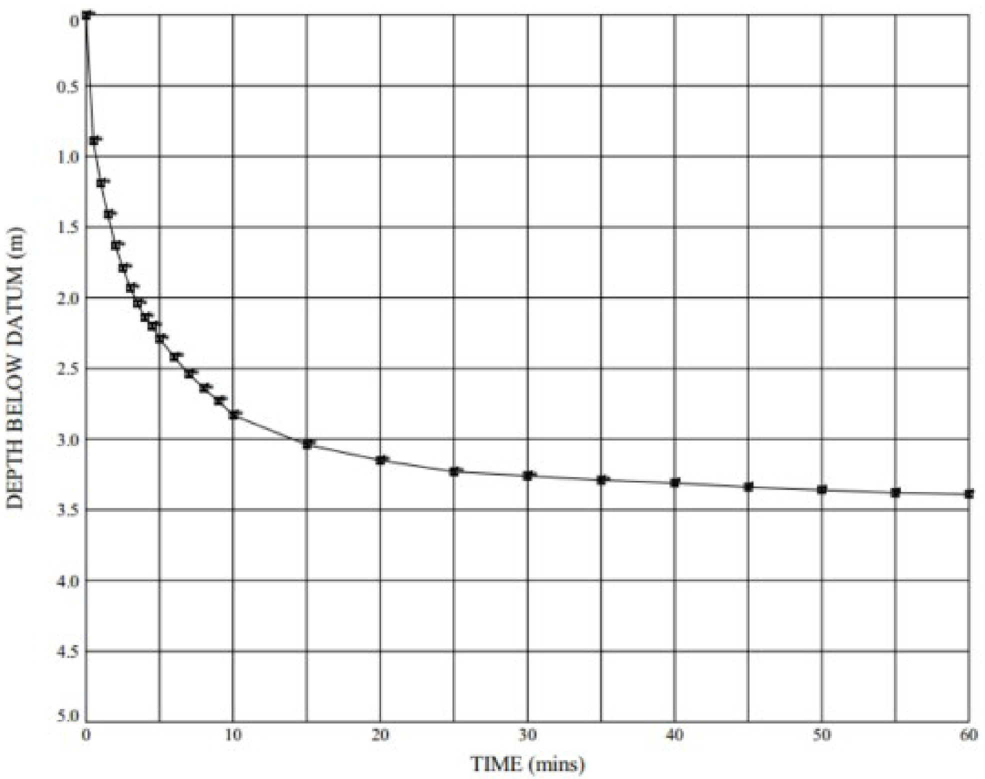
GINT_LIBRARY_V8_05.GLB LibVersion: v8_05 - 1.060004 PrjVersion: v8_05 - Core+Logs 0003 | Graph 1 - BH SOAKAWAY | 762971_SIZEWELL_C.GPJ - v8_05 | 18/11/14 - 13:47 | CH.

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA4

Ground Level: **17.10**   National Grid Co-ordinates: **E:645257.8 N:265170.9**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within the cased section (marked with a *) have been used (i.e. from 0.000m to 3.290m).

Volume of water soaking away,	**0.058133**	m^3
Surface area,	**0.4889**	m^2
Time to drain,	**2100**	secs
Infiltration rate,	**5.66×10^{-5}**	m/sec

Test water depth starts above base depth of casing but finishes below this base depth at end of test. Calculation of infiltration is performed using a subset of data where the test water readings are within the cased section (denoted by a * next to the datamarker in the graph frame above).

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **3.20**mBGL
Borehole diameter: **150**mm
Depth to base of casing: **2.20**mBGL
Height of datum above GL: **1.10**m

Legend:

Test 2 (15/04/2014)

STRUCTURAL SOILS	Compiled By	Date	Checked By	Date
The Potteries, Pottery Street, Castleford, W. Yorkshire WF10 1NJ	[redacted]	18/11/14	[redacted]	30/10/14
	Contract: **Sizewell Associated Developments**		Contract Ref: **762971**	

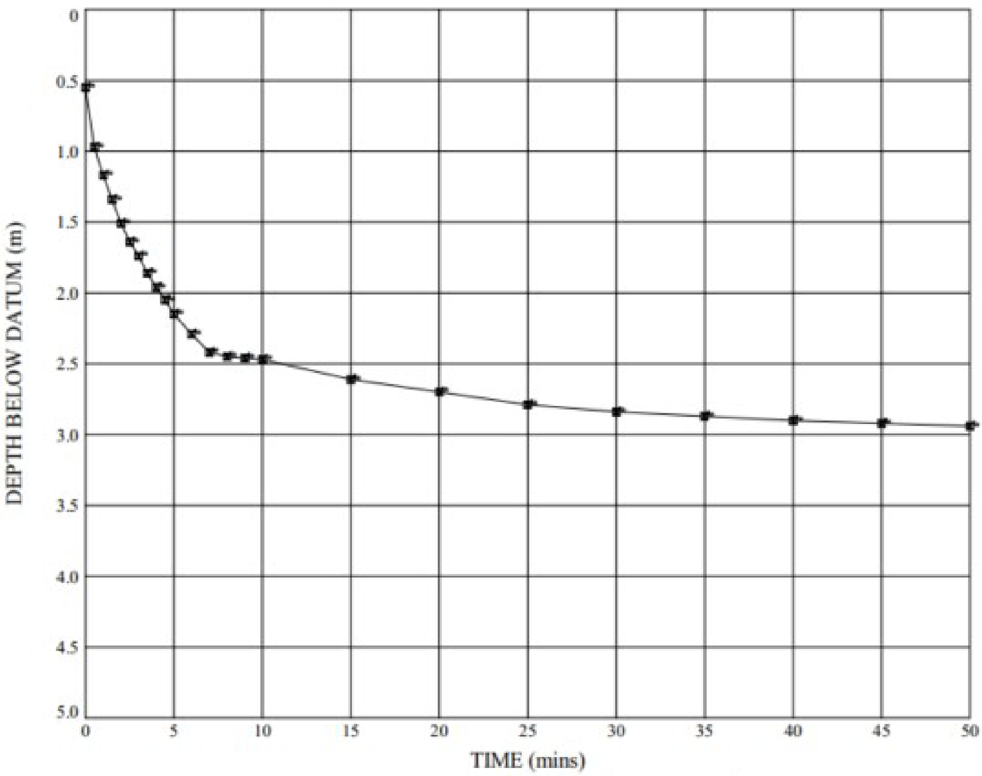
GINT_LIBRARY_V8_05.GLB LibVersion: v8_05 - Lib0004 PrjVersion: v8_05 - Core+Logs 0003 | Graph 1 - BH SOAKAWAY | 762971_SIZEWELL_C.GPJ - v8_05 | 18/11/14 - 13:47 | CH.

BOREHOLE SOAKAWAY TEST
NON STANDARD TEST

Test position: SA4

Ground Level: **17.10** National Grid Co-ordinates: **E:645257.8 N:265170.9**

PLOT OF DEPTH OF WATER BELOW DATUM AGAINST TIME



Due to the presence of casing in the test hole the infiltration rate has been calculated using a subset of the test data. Only test readings within cased section (marked with a *) have been used (i.e. from 0.550m to 2.940m).

Volume of water soaking away,	**0.042235**	m^3
Surface area,	**0.4889**	m^2
Time to drain,	**3000**	secs
Infiltration rate,	**2.88×10^{-5}**	m/sec

Test water depth remains above casing base depth throughout soakaway test.

Test hole details:

Measurement datum was **CASING**
Hole depth at start of test: **3.20**mBGL
Borehole diameter: **150**mm
Depth to base of casing: **2.20**mBGL
Height of datum above GL: **1.10**m

Legend:

Test 3 (15/04/2014)

STRUCTURAL SOILS The Potteries Pottery Street Castleford W. Yorkshire WF10 1NJ	Compiled By	Date	Checked By	Date
	[redacted]	**18/11/14**	[redacted]	**30/10/14**
	Contract: **Sizewell Associated Developments**		Contract Ref: **762971**	

GINT_LIBRARY_V8_05.GLB LibVersion: v8_05 - Lib0004 PrjVersion: v8_05 - Core+Logs 0003 | Graph 1 - BH SOAKAWAY | 762971_SIZEWELL_C.GPJ - v8_05 | 18/11/14 - 13:47 | CH.

APPENDIX B: GREENFIELD RUNOFF RATE ESTIMATION

HR Wallingford
Working with water

Greenfield runoff rate estimation for sites

www.uksuds.com | Greenfield runoff tool

Calculated by:	Dillon Hirani
Site name:	TCA Campus
Site location:	Sizewell TCA

Site Details

Latitude:	52.22707° N
Longitude:	1.58948° E
Reference:	2735935178
Date:	Sep 24 2021 07:19

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.

Runoff estimation approach IH124

Site characteristics

Total site area (ha): 50 (20.48)

Methodology

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

Soil characteristics	Default	Edited
SOIL type:	1	1
HOST class:	N/A	N/A
SPR/SPRHOST:	0.1	0.1

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?

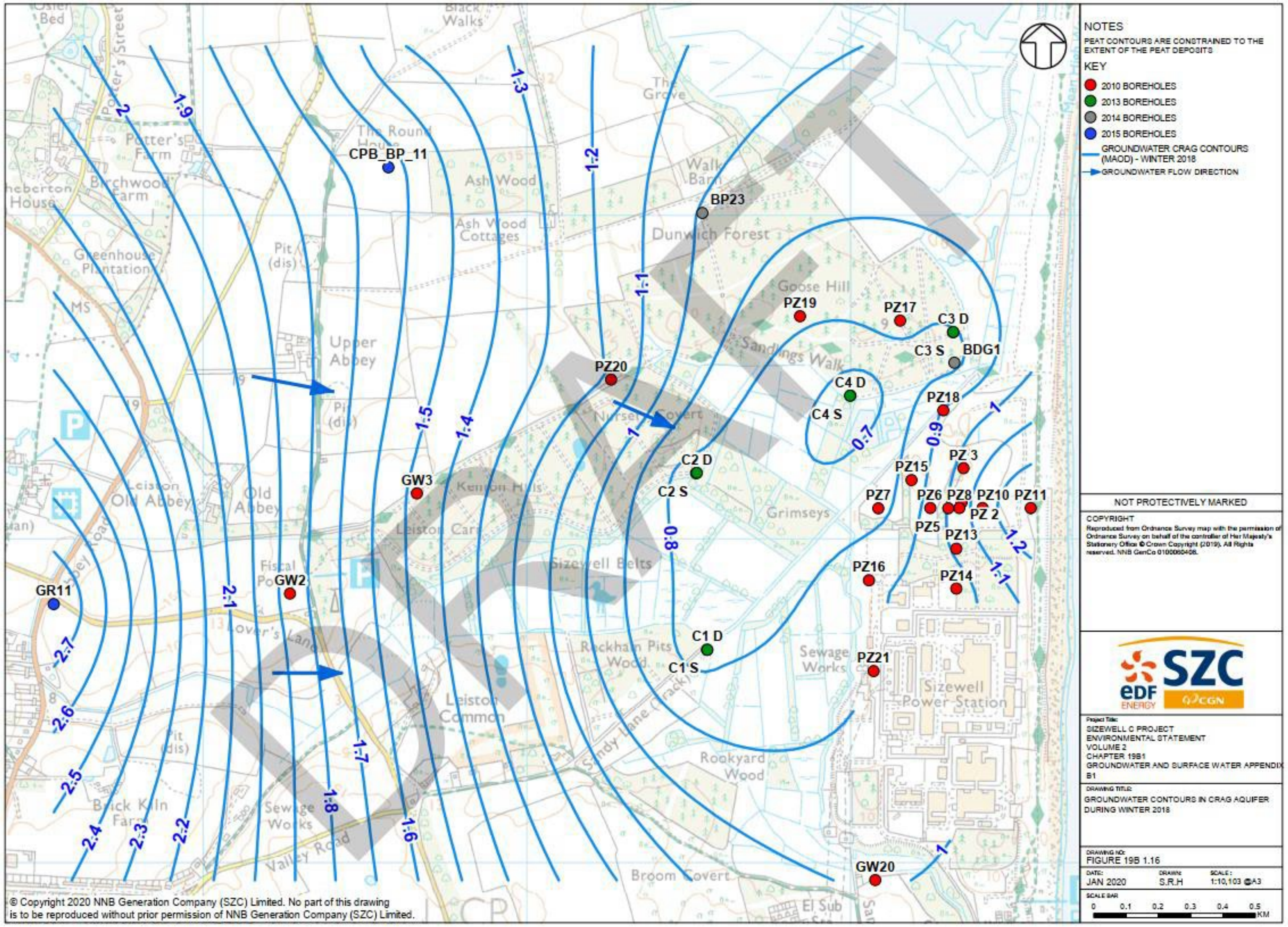
Where groundwater levels are low enough the use of soakaways to avoid discharge offsite would normally be preferred for disposal of surface water runoff.

Area Ratio: 0.4096

Greenfield runoff rates	Default	Edited	Interpolated
Q_{BAR} (l/s):	6.75	6.75	2.76
1 in 1 year (l/s):	5.88	5.88	2.41
1 in 30 years (l/s):	16.55	16.55	6.78
1 in 100 year (l/s):	24.05	24.05	9.85
1 in 200 years (l/s):	28.44	28.44	

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 C: GROUNDWATER CONTOUR DRAWING



APPENDIX D: SOURCE CONTROL (FEH 2013) – CAR PARK

Atkins (Epsom)		Page 1
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 19/01/2022 15:04 File Campus FEH13 _ Car Park....	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Half Drain Time : 7555 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	9.597	0.097	0.0	1.3	1.3	226.1	O K
30 min Summer	9.632	0.132	0.0	1.3	1.3	307.8	O K
60 min Summer	9.670	0.170	0.0	1.3	1.3	395.0	O K
120 min Summer	9.716	0.216	0.0	1.3	1.3	503.4	O K
180 min Summer	9.749	0.249	0.0	1.3	1.3	580.3	O K
240 min Summer	9.776	0.276	0.0	1.3	1.3	641.8	O K
360 min Summer	9.817	0.317	0.0	1.3	1.3	738.4	Flood Risk
480 min Summer	9.848	0.348	0.0	1.3	1.3	811.0	Flood Risk
600 min Summer	9.872	0.372	0.0	1.3	1.3	865.7	Flood Risk
720 min Summer	9.890	0.390	0.0	1.3	1.3	908.2	Flood Risk
960 min Summer	9.916	0.416	0.0	1.3	1.3	967.1	Flood Risk
1440 min Summer	9.942	0.442	0.0	1.3	1.3	1028.5	Flood Risk
2160 min Summer	9.954	0.454	0.0	1.3	1.3	1057.5	Flood Risk
2880 min Summer	9.954	0.454	0.0	1.3	1.3	1056.5	Flood Risk
4320 min Summer	9.938	0.438	0.0	1.3	1.3	1019.3	Flood Risk
5760 min Summer	9.917	0.417	0.0	1.3	1.3	971.2	Flood Risk
15 min Winter	9.605	0.105	0.0	1.3	1.3	244.3	O K
30 min Winter	9.643	0.143	0.0	1.3	1.3	332.6	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.080	0.0	95.8	27
30 min Summer	68.208	0.0	109.0	42
60 min Summer	43.872	0.0	219.0	72
120 min Summer	28.074	0.0	219.5	132
180 min Summer	21.656	0.0	215.8	192
240 min Summer	18.024	0.0	211.6	250
360 min Summer	13.906	0.0	202.0	370
480 min Summer	11.513	0.0	190.4	490
600 min Summer	9.884	0.0	184.6	610
720 min Summer	8.689	0.0	183.5	730
960 min Summer	7.018	0.0	185.4	970
1440 min Summer	5.090	0.0	183.9	1448
2160 min Summer	3.610	0.0	376.0	2168
2880 min Summer	2.801	0.0	369.0	2884
4320 min Summer	1.933	0.0	347.2	4324
5760 min Summer	1.483	0.0	725.1	5720
15 min Winter	100.080	0.0	100.9	27
30 min Winter	68.208	0.0	109.9	41

Atkins (Epsom)		Page 2
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		
Date 19/01/2022 15:04 File Campus FEH13 _ Car Park....	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
60 min Winter	9.683	0.183	0.0	1.3	1.3	426.8	O K
120 min Winter	9.734	0.234	0.0	1.3	1.3	544.1	O K
180 min Winter	9.770	0.270	0.0	1.3	1.3	627.4	O K
240 min Winter	9.798	0.298	0.0	1.3	1.3	694.1	O K
360 min Winter	9.843	0.343	0.0	1.3	1.3	799.1	Flood Risk
480 min Winter	9.877	0.377	0.0	1.3	1.3	877.5	Flood Risk
600 min Winter	9.902	0.402	0.0	1.3	1.3	936.7	Flood Risk
720 min Winter	9.922	0.422	0.0	1.3	1.3	982.8	Flood Risk
960 min Winter	9.950	0.450	0.0	1.3	1.3	1047.2	Flood Risk
1440 min Winter	9.979	0.479	0.0	1.3	1.3	1115.1	Flood Risk
2160 min Winter	9.994	0.494	0.0	1.3	1.3	1148.8	Flood Risk
2880 min Winter	9.994	0.494	0.0	1.3	1.3	1150.5	Flood Risk
4320 min Winter	9.979	0.479	0.0	1.3	1.3	1115.4	Flood Risk
5760 min Winter	9.959	0.459	0.0	1.3	1.3	1068.8	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.872	0.0	220.7	72
120 min Winter	28.074	0.0	218.7	130
180 min Winter	21.656	0.0	214.1	188
240 min Winter	18.024	0.0	208.9	248
360 min Winter	13.906	0.0	194.7	366
480 min Winter	11.513	0.0	187.0	486
600 min Winter	9.884	0.0	187.7	604
720 min Winter	8.689	0.0	190.3	722
960 min Winter	7.019	0.0	192.5	958
1440 min Winter	5.090	0.0	190.9	1430
2160 min Winter	3.610	0.0	390.7	2128
2880 min Winter	2.801	0.0	383.5	2828
4320 min Winter	1.932	0.0	361.4	4196
5760 min Winter	1.483	0.0	738.3	5528

Atkins (Epsom)		Page 3
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		
Date 19/01/2022 15:04 File Campus FEH13 _ Car Park....	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 647450 264900 TM 47450 64900
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.699
Cv (Winter)	0.755
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+20

Time Area Diagram

Total Area (ha) 1.299

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	0.433	4	8	0.433	8	12	0.433

Atkins (Epsom)		Page 4
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 19/01/2022 15:04 File Campus FEH13 _ Car Park....	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Model Details

Storage is Online Cover Level (m) 10.100

Cellular Storage Structure

Invert Level (m) 9.500 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Inf. Area (m²)	Depth (m)	Area (m²)	Inf. Area (m²)
0.000	2450.0	0.0	0.500	2450.0	0.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0061-1300-0500-1300
Design Head (m)	0.500
Design Flow (l/s)	1.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	61
Invert Level (m)	9.500
Minimum Outlet Pipe Diameter (mm)	75
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.500	1.3	Kick-Flo®	0.334	1.1
Flush-Flo™	0.147	1.3	Mean Flow over Head Range	-	1.1

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	1.3	1.200	1.9	3.000	2.9	7.000	4.4
0.200	1.3	1.400	2.1	3.500	3.2	7.500	4.6
0.300	1.2	1.600	2.2	4.000	3.4	8.000	4.7
0.400	1.2	1.800	2.3	4.500	3.5	8.500	4.9
0.500	1.3	2.000	2.4	5.000	3.7	9.000	5.0
0.600	1.4	2.200	2.5	5.500	3.9	9.500	5.1
0.800	1.6	2.400	2.6	6.000	4.1		
1.000	1.8	2.600	2.7	6.500	4.2		

APPENDIX E: SOURCE CONTROL (FEH 2013) – PERMEABLE PAVING

Atkins (Epsom)		Page 1
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 19/01/2022 15:00 File Campus FEH13 _ v3.SRCX	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Half Drain Time : 5339 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	9.517	0.117	0.0	9.0	9.0	2390.0	O K
30 min Summer	9.559	0.159	0.0	14.5	14.5	3250.2	O K
60 min Summer	9.604	0.204	0.0	18.6	18.6	4164.0	O K
120 min Summer	9.660	0.260	0.0	19.1	19.1	5298.5	O K
180 min Summer	9.699	0.299	0.0	19.1	19.1	6102.2	O K
240 min Summer	9.731	0.331	0.0	19.1	19.1	6743.2	O K
360 min Summer	9.780	0.380	0.0	19.1	19.1	7744.9	O K
480 min Summer	9.816	0.416	0.0	19.1	19.1	8490.1	Flood Risk
600 min Summer	9.844	0.444	0.0	19.1	19.1	9051.7	Flood Risk
720 min Summer	9.865	0.465	0.0	19.1	19.1	9487.8	Flood Risk
960 min Summer	9.895	0.495	0.0	19.1	19.1	10091.3	Flood Risk
1440 min Summer	9.924	0.524	0.0	19.1	19.1	10692.8	Flood Risk
2160 min Summer	9.935	0.535	0.0	19.1	19.1	10920.1	Flood Risk
2880 min Summer	9.931	0.531	0.0	19.1	19.1	10831.1	Flood Risk
4320 min Summer	9.904	0.504	0.0	19.1	19.1	10287.0	Flood Risk
5760 min Summer	9.878	0.478	0.0	19.1	19.1	9761.4	Flood Risk
15 min Winter	9.530	0.130	0.0	10.7	10.7	2658.7	O K
30 min Winter	9.577	0.177	0.0	16.5	16.5	3615.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	100.080	0.0	604.6	27
30 min Summer	68.208	0.0	957.9	42
60 min Summer	43.872	0.0	2128.2	72
120 min Summer	28.074	0.0	2752.9	132
180 min Summer	21.656	0.0	3041.8	190
240 min Summer	18.024	0.0	3142.9	250
360 min Summer	13.906	0.0	3114.1	370
480 min Summer	11.513	0.0	3044.3	490
600 min Summer	9.884	0.0	2965.6	610
720 min Summer	8.689	0.0	2885.3	730
960 min Summer	7.018	0.0	2734.2	968
1440 min Summer	5.090	0.0	2526.6	1448
2160 min Summer	3.610	0.0	5455.9	2164
2880 min Summer	2.801	0.0	5158.8	2884
4320 min Summer	1.933	0.0	4579.7	4112
5760 min Summer	1.483	0.0	10426.2	4672
15 min Winter	100.080	0.0	713.1	27
30 min Winter	68.208	0.0	1109.8	41

Atkins (Epsom)		Page 2
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 19/01/2022 15:00 File Campus FEH13 _ v3.SRCX	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control (l/s)	Max Σ Outflow (l/s)	Max Volume (m³)	Status
60 min Winter	9.627	0.227	0.0	18.9	18.9	4635.1	O K
120 min Winter	9.689	0.289	0.0	19.1	19.1	5902.2	O K
180 min Winter	9.733	0.333	0.0	19.1	19.1	6799.9	O K
240 min Winter	9.768	0.368	0.0	19.1	19.1	7516.2	O K
360 min Winter	9.823	0.423	0.0	19.1	19.1	8637.9	Flood Risk
480 min Winter	9.864	0.464	0.0	19.1	19.1	9475.2	Flood Risk
600 min Winter	9.896	0.496	0.0	19.1	19.1	10109.0	Flood Risk
720 min Winter	9.920	0.520	0.0	19.1	19.1	10598.5	Flood Risk
960 min Winter	9.952	0.552	0.0	19.1	19.1	11270.7	Flood Risk
1440 min Winter	9.986	0.586	0.0	19.1	19.1	11949.3	Flood Risk
2160 min Winter	9.999	0.599	0.0	19.2	19.2	12229.1	Flood Risk
2880 min Winter	9.996	0.596	0.0	19.1	19.1	12163.2	Flood Risk
4320 min Winter	9.970	0.570	0.0	19.1	19.1	11633.1	Flood Risk
5760 min Winter	9.939	0.539	0.0	19.1	19.1	11002.4	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
60 min Winter	43.872	0.0	2409.8	70
120 min Winter	28.074	0.0	2992.3	130
180 min Winter	21.656	0.0	3173.8	188
240 min Winter	18.024	0.0	3173.6	248
360 min Winter	13.906	0.0	3102.5	366
480 min Winter	11.513	0.0	3005.6	484
600 min Winter	9.884	0.0	2904.1	602
720 min Winter	8.689	0.0	2831.7	720
960 min Winter	7.019	0.0	2749.6	956
1440 min Winter	5.090	0.0	2659.4	1426
2160 min Winter	3.610	0.0	5567.9	2120
2880 min Winter	2.801	0.0	5330.0	2800
4320 min Winter	1.932	0.0	4847.2	4116
5760 min Winter	1.483	0.0	10753.7	5312

Atkins (Epsom)		Page 3
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 19/01/2022 15:00 File Campus FEH13 _ v3.SRCX	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 647450 264900 TM 47450 64900
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.657
Cv (Winter)	0.731
Shortest Storm (mins)	15
Longest Storm (mins)	5760
Climate Change %	+20

Time Area Diagram

Total Area (ha) 14.577

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	4.859	4	8	4.859	8	12	4.859

Atkins (Epsom)		Page 4
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 19/01/2022 15:00 File Campus FEH13 _ v3.SRCX	Designed by RUTT6643 Checked by	
Innovyze	Source Control 2020.1.3	

Model Details

Storage is Online Cover Level (m) 10.100

Infiltration Basin Structure

Invert Level (m) 9.400 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 0.30
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	68000.0	0.600	68000.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0200-1920-0600-1920
Design Head (m)	0.600
Design Flow (l/s)	19.2
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	200
Invert Level (m)	9.400
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	0.600	19.2	Kick-Flo®	0.485	17.4
Flush-Flo™	0.292	19.1	Mean Flow over Head Range	-	15.0

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	6.9	1.200	26.7	3.000	41.5	7.000	62.3
0.200	18.6	1.400	28.8	3.500	44.7	7.500	64.5
0.300	19.1	1.600	30.7	4.000	47.7	8.000	66.7
0.400	18.6	1.800	32.5	4.500	50.5	8.500	68.8
0.500	17.6	2.000	34.1	5.000	53.2	9.000	70.8
0.600	19.2	2.200	35.7	5.500	55.7	9.500	72.7
0.800	22.0	2.400	37.3	6.000	58.1		
1.000	24.5	2.600	38.7	6.500	60.0		

ANNEX 2A.20: ACA WEST EXPLANATORY NOTE

ACA West – Explanatory Note

CONTENTS

TABLES

APPENDICES

1 INTRODUCTION

1.1.1 This short note provides a summary of the ACA West WMZ Basin sizing to date and the proposed changes to accommodate more onerous storm volumes. This note does not intend to provide detailed design of the West ACA catchment definition or basin sizing.

1.1.2 Section 8 of the Surface Water Drainage – SCC Explanatory Note (SZC-EW0300-ATK-XX-000-XXXXXX-NOT-CIV-000008 Rev 01) highlighted a storage volume shortfall for the ACA West WMZ basin of approximately 1000m^3 between the basin size dimensions allocated at the time of writing the SCC Explanatory Note and the maximum volume of surface water runoff for a 100yr RP plus 20% climate change allowance with a 1l/s/ha outflow (Source Control estimation).

1.1.3 A simplified analysis of a subsequent storm (10yr RP) was undertaken and demonstrated the ACA West WMZ basin did not have capacity to accept this additional storage volume. Furthermore, it was stated the West ACA WMZ basin could meet the 24-hour half drain time requirement but only through an increased discharge pump rate of approximately 4.71l/s/ha.

1.1.4 As described in the SCC Explanatory Note, 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.

Table 1.1: ACA West Basin estimated runoff for a 100yr RP plus 20% climate change allowance storm with no outflow

WMZ	Catchment Area (ha)	PIMP (%)	Infil-tration rate (m/hr)	Out-flow (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

1.1.5 The report concluded that the best solution for the ACA West area was to increase the overall size of the basin to accommodate the 100yr RP plus 20% climate change allowance together with no outflow to simulate a pump failure scenario. This would retain the existing pumping discharge rate of 1 l/s/ha and therefore there would be no increase in the overall discharge rate.

1.1.6 SCC requested a revised assessment and sizing of the ACA West basin for the 24-hour pump failure.

2 24-HOUR PUMP FAILURE

2.1.1 To manage the 24-hour pump failure volumes, a basin volume of 4445.4m^3 is required as summarised in **Table 1.1** above, which is approximately 1800m^3 larger than the basin size originally proposed in the ACA. The West ACA basin is constrained by several proposed features including but not limited to the security fence, security fence access track and the perimeter swale to the north, south and west and the topsoil compound to the east of the basin. Given these constraints, widening the total basin extents in 2D is not feasible. Therefore, the options available to meet the 24-hour pump failure volumes include:

- Increase the basin volume by decreasing the access track width and increasing the pond depth
- Alter the proposed ACA West area by reallocating a portion of the topsoil compound area to increase available area for the basin
- Alter the proposed ACA sub catchments by reallocating a portion of the topsoil compound to Catchment 1

2.1.2 CIRIA C753 The SuDS Manual does not state a minimum access track width for basins but advises a minimum access track width of 3.5m for ponds to facilitate operation and maintenance activities. The SuDS Manual also advises a maximum depth of 2m for basins for health and safety reasons. The access track width was decreased to 4m, as this should still provide adequate access and space for maintenance and operational purposes. This would need to be confirmed for example by confirming the width suitability for vehicle access using swept path analysis in detailed design. The basin depth to the freeboard level was increased to 2.2m on the assumption that deep water health and safety hazards are not as prevalent on construction sites. This would need to be confirmed through a thorough health and safety assessment in detailed design. These changes are summarised in **Table 2.1** below.

WMZ Basin	Access Track Width (m)	Area at base (m^2)	Base level of Basin (mAOD)	Area at freeboard level (m^2)
ACA West (SCC Explanatory Note)	8	659.6	5.700	1510.9
ACA West (Revised)	4	1156	5.500	2417.64
WMZ Basin	**Depth to freeboard level (m)**	**WMZ Basin Volume (m^3) Base to Freeboard Level**	**Area at top of basin (m^2) 300mm Freeboard**	**WMZ Basin Volume (m^3) including freeboard**
ACA West (SCC Explanatory Note)	2.000	2170.5	1667.8	2676.5
ACA West (Revised)	2.200	3826.4	2621.5	4565.8

Table 2.1: Comparison of ACA West WMZ Basin Sizes

2.1.3 As discussed in the SCC Explanatory Note, the storage volumes calculated using Source Control at this stage do not consider network volumes that would be taken into account in the detailed hydraulic model. Additional storage will be available upstream of the basin within pipes/swales that are proposed around the topsoil compound. This will likely reduce the storage volumes required in the basin in detailed design.

2.1.4 Altering the catchments by either encroaching on the topsoil compound or resizing the sub catchments are also options to consider at later design stages. For example, a simple Source Control calculation found that approximately 1.9ha would need to be redistributed from ACA West to ACA East to ensure the storm volume could be contained with the basin size referenced in the SCC Explanatory Note. Altering the catchments is not the preferred option and is therefore not explored in detail at this stage but is noted as an option for future design stages if required.

3 24-HOUR HALF DRAIN

3.1.1 To drain half a basin volume of 4445.4m^3 in 24 hours, an outflow of 5.79 l/s/ha (2,222.7 x 1000 / 24 x 3600 = 25.7 l/s for 4.44 ha) is required. 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. This flow would discharge to Outfall O6, subject to agreement from SCC, the Internal Drainage Board and the Environment Agency.

4 SUMMARY

4.1.1 The approach taken for the ACA West catchment has been to enlarge the basin size by reducing the access track width around the perimeter to 4.0m. The total depth to the top of the freeboard has also increased to 2.5m. This simple approach has delivered a basin volume of 4,565.8m^3, increased from 2,676.5m^3. A nominal additional volume is anticipated to represent other storage volumes within the network. In order to achieve a 24-hour half drain time the outflow will need to be increased above the restricted rate of 1 l/s/ha.

4.1.2 The total volume provided by the revised basin sizing is able to contain the 100yr RP plus 20% climate change allowance (3,581.3m^3) together with a 24-hour period during which no flows leave the basin (overall total of 4,445.4m^3). This arrangement is therefore sufficient to accommodate the storm flows, cope with a 24-hour power outage and adequately addresses the potential residential flooding risk in the area.

APPENDIX A: SOURCE CONTROL (FEH 2013) – WEST ACA BASIN

Atkins (Epsom)		Page 1
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 27/01/2022 13:53 File ACA West FEH13 (new).SRCX	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Half Drain Time exceeds 7 days.

Critical storm may not be identified, please run longer storm durations.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	6.182	0.682	0.0	902.1	O K
30 min Summer	6.382	0.882	0.0	1212.6	O K
60 min Summer	6.579	1.079	0.0	1539.7	O K
120 min Summer	6.809	1.309	0.0	1951.0	O K
180 min Summer	6.964	1.464	0.0	2244.4	O K
240 min Summer	7.083	1.583	0.0	2479.9	O K
360 min Summer	7.263	1.763	0.0	2854.0	O K
480 min Summer	7.395	1.895	0.0	3141.3	O K
600 min Summer	7.495	1.995	0.0	3365.8	O K
720 min Summer	7.574	2.074	0.0	3546.2	O K
960 min Summer	7.686	2.186	0.0	3812.0	O K
1440 min Summer	7.820	2.320	0.0	4140.7	Flood Risk
15 min Winter	6.226	0.726	0.0	968.5	O K
30 min Winter	6.437	0.937	0.0	1301.8	O K
60 min Winter	6.644	1.144	0.0	1653.0	O K
120 min Winter	6.886	1.386	0.0	2094.6	O K
180 min Winter	7.048	1.548	0.0	2409.6	O K
240 min Winter	7.172	1.672	0.0	2662.4	O K
360 min Winter	7.360	1.860	0.0	3064.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	106.848	0.0	27
30 min Summer	71.808	0.0	42
60 min Summer	45.588	0.0	72
120 min Summer	28.884	0.0	132
180 min Summer	22.152	0.0	192
240 min Summer	18.357	0.0	252
360 min Summer	14.084	0.0	372
480 min Summer	11.627	0.0	492
600 min Summer	9.966	0.0	612
720 min Summer	8.750	0.0	732
960 min Summer	7.054	0.0	972
1440 min Summer	5.109	0.0	1452
15 min Winter	106.848	0.0	27
30 min Winter	71.808	0.0	42
60 min Winter	45.588	0.0	72
120 min Winter	28.884	0.0	132
180 min Winter	22.152	0.0	192
240 min Winter	18.357	0.0	252
360 min Winter	14.084	0.0	372

Atkins (Epsom)		Page 2
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 27/01/2022 13:53 File ACA West FEH13 (new).SRCX	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Summary of Results for 100 year Return Period (+20%)

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
480 min Winter	7.498	1.998	0.0	3372.5	O K
600 min Winter	7.602	2.102	0.0	3613.5	O K
720 min Winter	7.684	2.184	0.0	3807.1	O K
960 min Winter	7.800	2.300	0.0	4092.6	Flood Risk
1440 min Winter	7.940	2.440	0.0	4445.4	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
480 min Winter	11.627	0.0	492
600 min Winter	9.966	0.0	612
720 min Winter	8.750	0.0	732
960 min Winter	7.054	0.0	972
1440 min Winter	5.108	0.0	1452

Atkins (Epsom)		Page 3
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 27/01/2022 13:53 File ACA West FEH13 (new).SRCX	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	2013
Site Location	GB 647050 262950 TM 47050 62950
Data Type	Catchment
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.761
Cv (Winter)	0.817
Shortest Storm (mins)	15
Longest Storm (mins)	1440
Climate Change %	+20

Time Area Diagram

Total Area (ha) 4.438

Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)	Time From:	(mins) To:	Area (ha)
0	4	1.479	4	8	1.479	8	12	1.479

Atkins (Epsom)		Page 4
Woodcoste Grove Ashley Road, Epsom Surrey, KT18 5BW		Micro Drainage
Date 27/01/2022 13:53 File ACA West FEH13 (new).SRCX	Designed by HIRA5452 Checked by	
Innovyze	Source Control 2020.1.3	

Model Details

Storage is Online Cover Level (m) 8.000

Infiltration Basin Structure

Invert Level (m) 5.500 Safety Factor 1.5
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 1.00
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	1156.0	2.200	2417.6	2.500	2621.5

ANNEX 2A.21: WMZS 7, 8, 9 SURFACE WATER DISCHARGE TECHNICAL NOTE

WMZ's 7, 8, 9 Surface Water Discharges Technical Note

CONTENTS

TABLES

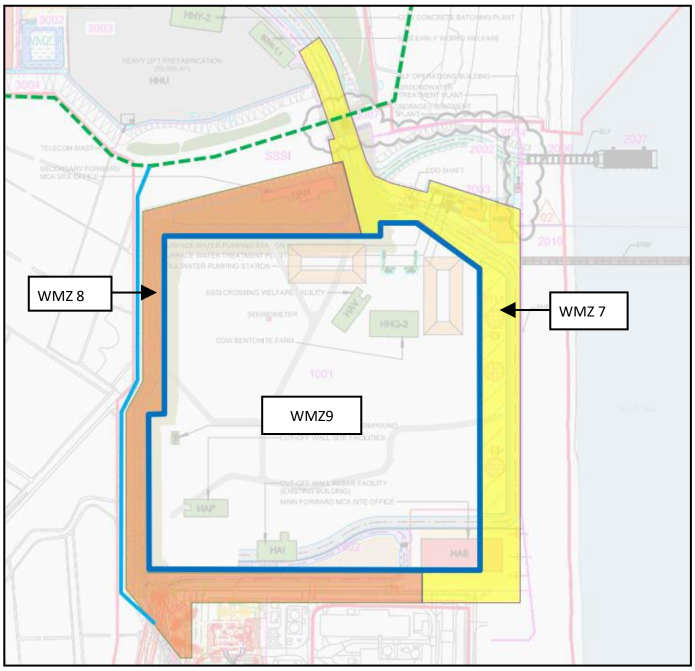
PLATES

1 WMZ'S 7, 8, 9 SURFACE WATER DISCHARGES

1.1 Introduction

1.1.1 This document aims to clarify the surface water arrangements for Water Management Zones (WMZs) 7, 8 and 9. These three WMZs form the area that includes the permanent position of the power plant (WMZ 9) with WMZ 7 and 8 lying to the east and west respectively (**Plate 1.1**). The project undergoes a number of phases during which the surface water is controlled in different ways. The approach recognises the varying nature of the pollution risk.

Plate 1.1: WMZ 7, 8 and 9



1.2 Background

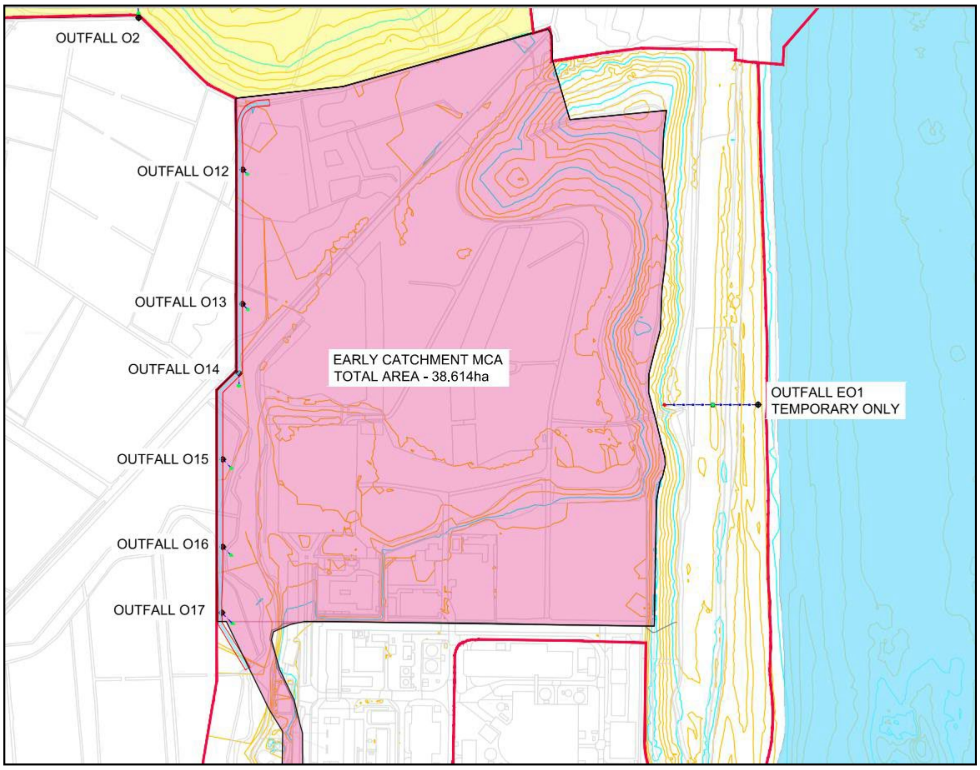
1.2.1 As part of the Enabling Works, most of the site within the red line boundary will be stripped of topsoil and regraded. Prior to the construction of the overall surface water network, and before earthworks/topsoil stripping commences, provision of early surface water management will be required. The drainage strategy varies across the site and is influenced by the planned activity and phasing of works within individual land parcels.

1.2.2 For the Main Construction Area (MCA), an Early catchment area was defined based on the existing levels and contour information. This total catchment, circa 38.6 ha, approximates where surface water would generally flow with some consideration of where runoff may be diverted/captured as a result of the initial earthworks (**Plate 1.2**). A number of early outfalls had been proposed but these are not being actively progressed and the surface water from this area is to be preferentially discharged through the Temporary Marine Outfall (TMO) once it is constructed. A fuller description with sketches is shown later in the document.

1.2.3 Initially a larger number of outfalls had been proposed along the western edge of WMZ 8 but this has been simplified to two outfalls only: Outfall 14 (O14) and Outfall 17 (O17). O14 is proposed to discharge the flows from WMZ 8. The discharge to O17 is proposed to discharge the water associated with the SZB link road and excess flows from SZB.

1.2.4 The three WMZs 7, 8 & 9 will be incorporated at the permanent Sizewell C power station and the permanent operational control of storm water is not described in detail in this note.

Plate 1.2: Early MCA catchment, temporary marine outfall EO1 and permanent construction outfalls O12 to O17 (SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000047)



a) Outfall O14

1.2.5 The surface water across the WMZ 8 is proposed to be controlled by cut off ditches along the western side of the site. These flows initially are to be pumped from the low point close to O14. The pumped flows are to be directed towards the surface water treatment plant where the solids load will be reduced before being discharged to sea.

1.2.6 The earliest times in the project will produce surface water that will be a high pollution risk to enter the Sizewell Drain and therefore these flows are to be treated and discharged to sea. It is only during the later stages that the pollution risk will be deemed low enough to allow discharge to the Sizewell Drain.

1.2.7 The O14 discharge will have passed through filter strips and filter drain along the access track running north-south.

1.2.8 It is anticipated that the flows through O14 will be limited to a rate of 5.0 l/s (equivalent to 1 l/s/ha for WMZ 8) and attenuation is required to achieve this restricted rate. The rate is subject to agreement from Environment Agency (EA), East Suffolk Internal Drainage Board (ESIDB) and Suffolk County Council (SCC).

b) Outfall O17

1.2.9 The excess surface water from Sizewell B (SZB) is proposed to be discharged to the Sizewell Drain. The area being drained consists of a limited part of the SZB site that is 100% impermeable. This water is made up of two components. The first being runoff that exceeds the capacity of the existing SZB drainage network (assumed to be 1:10 year event), and the second which forms the flows from a 1:10,000 event.

1.2.10 The surface water flows are proposed to be caught in the cut off drainage channel, which runs east to west along the northern boundary of the SZB site. Initially constructed as a ditch the final construction is proposed to be a formed concrete channel.

1.3 Discharge rate and storage

1.3.1 The estimated greenfield runoff rate using the IH124 method is low as shown below in **Table 1.1**. Initially, a number of outfalls along the western edge of the MCA to the Sizewell Drain were considered, to control the flow efficiently along the length of the drain and retain the existing un-developed runoff characteristics. The number and position of these outfalls is subject to discussions with the Environment Agency, Internal Drainage Board and Suffolk County Council.

1.3.2 It is anticipated that an overall better approach is to use the proposed TMO as early as possible and to significantly reduce reliance on the outfalls originally proposed along the western side of the area.

Table 1.1: Early MCA catchment greenfield runoff rates (initial proposal for multiple outfalls)

Catchment	Area (ha)	Outfall ID	Discharge Rate (l/s)					
			1 in 1 yr	1 in 30 yr	1 in 100 yr	Q_{BAR}	1 l/s/ha	Initial proposal
Early MCA	38.61	O12, O13, O14, O15, O16 & O17	4.51	12.07	18.46	5.18	38.61	6.44 per outfall

1.3.3 **Table 1.2** below shows the estimated storage required to contain the 1:100yr critical storm event including a 20% allowance for climate change for each WMZ. The storage estimates are based on a restricted flow equivalent to 1 l/s/ha. The critical storm is a 2-day duration, and suggests that circa 40,000m^3 is required across the MCA.

Table 1.2: MCA Sub Catchment Maximum Storage Volumes

WMZ	Area (ha)	PIMP (%)	Infilt-ration rate (m/hr)	Out-flow (l/s)	Maximum Volume (m^3)			Critical Storm Event (100RP + 20%CC)		
					FSR	FEH 1999	FEH 2013	FSR	FEH 1999	FEH 2013
7	8.66	100	0	8.66	6944.8	8897.8	**9087.3**	4320 min Winter	2880 min Winter	2880 min Winter
8	5.05			5.05	4084.3	5208.6	**5314.5**	4320 min Winter	2880 min Winter	2880 min Winter
9	24.64			24.64	19383.4	25390.5	**25890.5**	4320 min Winter	2880 min Winter	2880 min Winter

1.3.4 The discharge from WMZ 8 into the Sizewell Drain through O14 has a greenfield rate of 5.0 l/s. This rate is low and requires a significant storage in this area. It may not be possible to achieve the estimated storage volume in this area during the construction phase due to space limitations and therefore the upper acceptable discharge rate could be reviewed to reflect these constraints.

1.3.5 The discharge to the Sizewell Drain is being considered only in the later stages of construction when the pollution risk is low. During this period the majority of the work is within the station boundary and has little effect outside. It is proposed that some areas can be made available for storage but this would be unlikely to meet the full 5,314 m^3.

1.3.6 Currently the WMZ 8 has been nominally calculated as the entire area to the redline in the west. This does not accurately reflect the area being drained for construction as it includes the Sizewell Drain itself and western bank and therefore needs to be revised. The exact areas and their corresponding PIMP values are also poorly understood at this stage. The storage volume stated represents the upper limit of storage requirements and this may reduce during the detailed design phase. A mutually agreed increase in the discharge rate for O14 would ease the storage requirement in this area.

1.4 Overview of construction stages

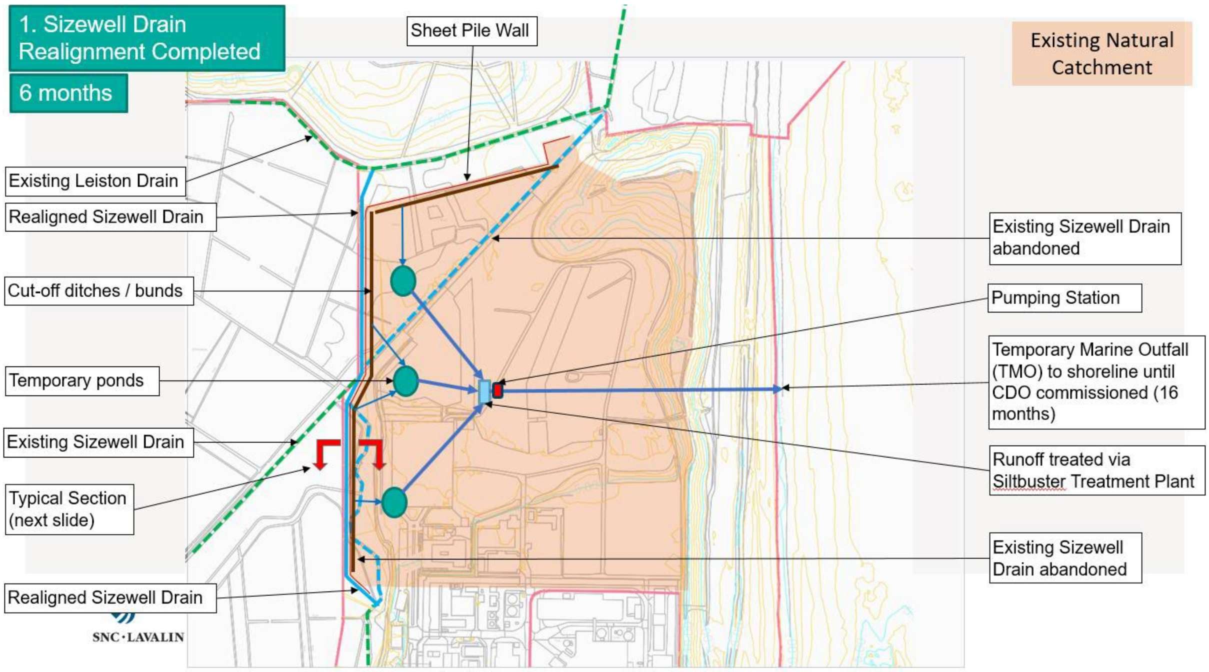
1.4.1 This section provides a high-level overview of the construction stages considered in defining the surface water drainage catchments within the Main Construction Area. The durations stated within each section are indicative only and subject to change following review of the construction programme. A detailed design of each drainage network is not shown and will be developed in consultation with East Suffolk Internal Drainage Board (ESIDB), Environment Agency (EA), Suffolk County Council (SCC) and Natural England.

a) Stage 1 – Sizewell Drain realignment

1.4.2 This first stage on site involves the Sizewell Drain realignment. No surface water is proposed to be discharged to the Sizewell Drain but rather a series of temporary ditches/bunds and sediment basins would locally collect stormwater, treat it on site through a proprietary plant before discharge through the Temporary Marine Outfall (TMO) to the sea (**Plate 1.3**). The location of the sediment basins will be placed away from Sizewell Drain and not disrupt the maintenance regime associated with the drain.

1.4.3 Currently the SZB overland flows infiltrate onto the SZC area and it is thought these flows will be captured in the surface water arrangements at this first stage.

Plate 1.3: Stage 1 – Indicative surface water drainage management during the Sizewell Drain Realignment works



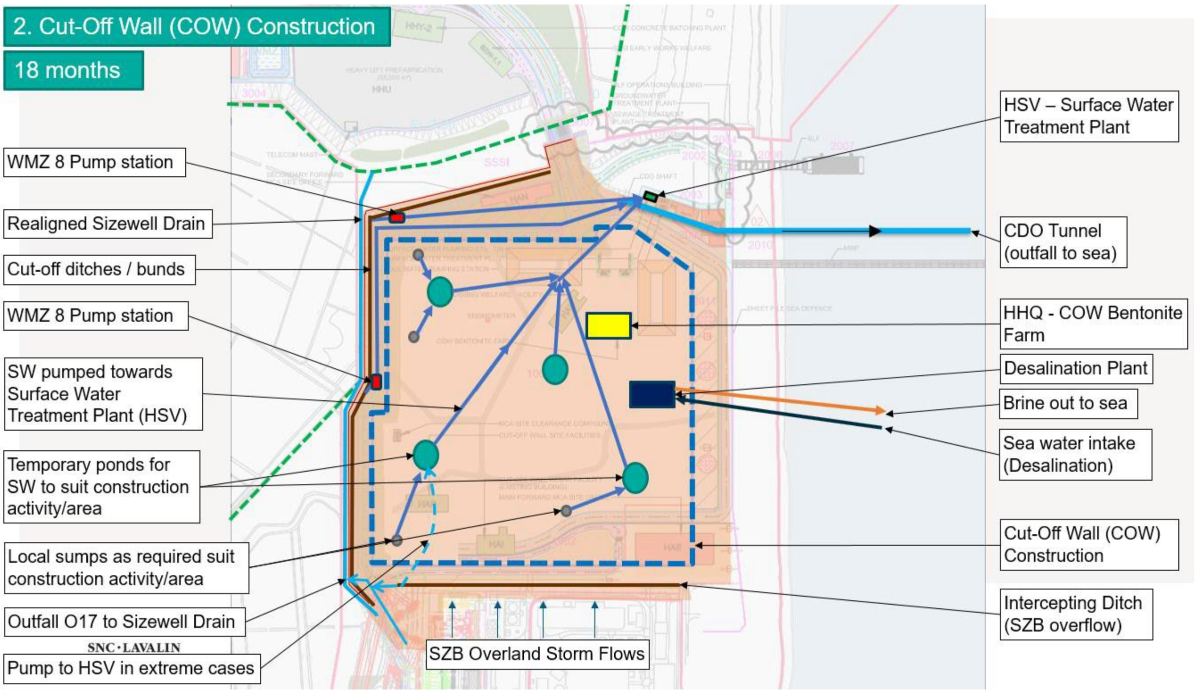
b) Stage 2 – Cut-Off Wall (COW) construction

1.4.4 This stage in the project involves the construction of the Cut Off Wall (COW), which entails digging down and supporting the excavation with Bentonite clay (**Plate 1.4**). The clay is then replaced using concrete to build the wall. The displaced Bentonite is then recycled for use in the next section of the wall.

1.4.5 During this time no water is proposed to be discharged to the Sizewell Drain. Stormwater is to be collected in semi-permanent ponds which are pumped to the proprietary treatment plant before being discharged to sea via the Construction Drainage Outfall (CDO).

1.4.6 Overland surface flows from SZB are to be prevented from flowing onto the SZC by means of a temporary ditch later to be a permanent cut off drainage channel. This will direct flows towards O17, which drains into the Sizewell Drain. It is anticipated that these flows present a low pollution risk as the flows generated are not from a construction site and are from large rainfall events.

Plate 1.4: Stage 2 - Indicative surface water drainage management during construction of the Cut-Off Wall (COW)



c) Stage 3 – Excavation and dewatering (WMZ9)

1.4.7 This stage is initiated when the COW is complete (**Plate 1.5**). This allows the material inside the wall to be excavated. To allow the excavation to take place the area needs to be dewatered. The groundwater is pumped out and passed through a Groundwater Treatment plant before being discharge via the CDO to the sea.

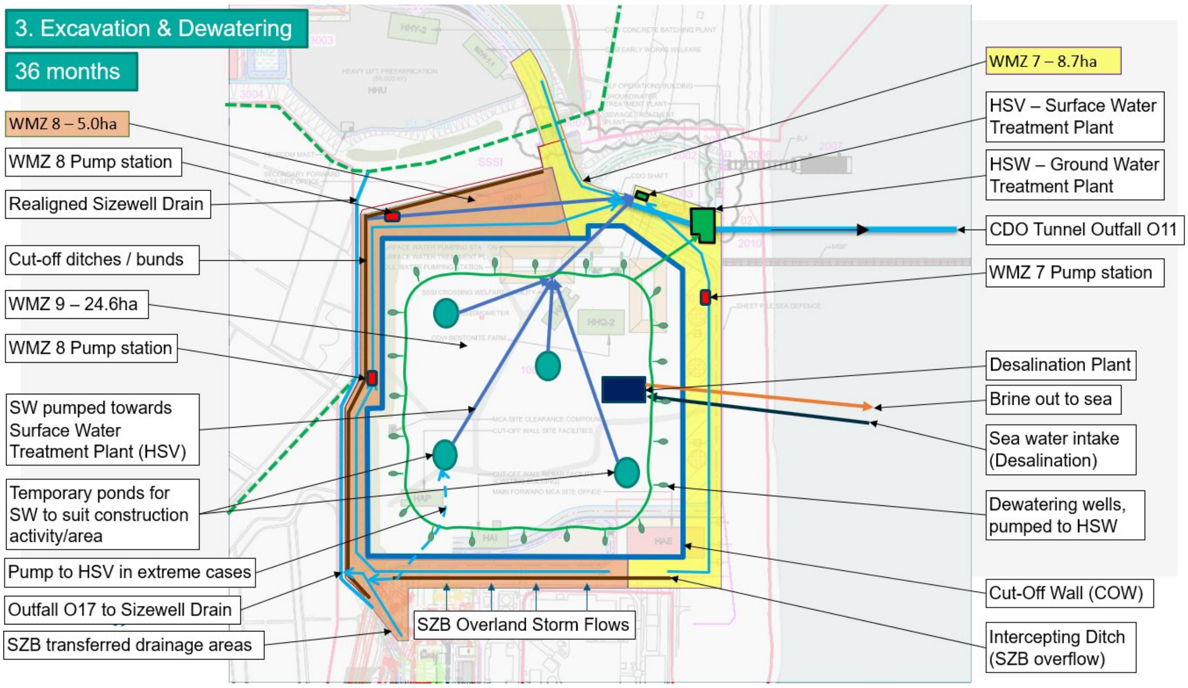
1.4.8 Once the COW is constructed this defines the 3 WMZs as separate areas. It is proposed that surface water collecting inside the COW (WMZ 9) would again be collected in semi-permanent ponds which are pumped via a proprietary treatment plant before being discharge via the CDO to the sea. It is expected that some surface water will infiltrate the ground within WMZ 9 and be pumped out through the dewatering route.

1.4.9 The stormwater in WMZ 7 (eastern) is surrounded by the COW to the west and the sheet pile sea defence wall to the east it therefore does not present any external risk of flooding. The stormwater is to be collected and discharged via a pumping station through a treatment plant and then via the CDO to the sea.

1.4.10 No water is planned to be discharged from WMZ 8 (western) to the Sizewell Drain but rather the stormwater would be pumped to the treatment plant before being discharged via the CDO to the sea. It is anticipated that the station would have multiple pumps to ensure that the risk of all pumps failing at once is minimised. Some temporary storage is to be provided to buffer flows to the pumps.

1.4.11 Overland surface flows from SZB are to be prevented from flowing onto the SZC by means of a temporary ditch later to be a permanent cut off drainage channel. This will direct flows towards O17, which drains into the Sizewell Drain. It is anticipated that these flows present a low pollution risk as the flows generated are not from a construction site and are from large rainfall events.

Plate 1.5: Stage 3 - Indicative surface water drainage management on completion of the Cut-Off Wall (COW)



d) Stage 4 – Construction of main platform area

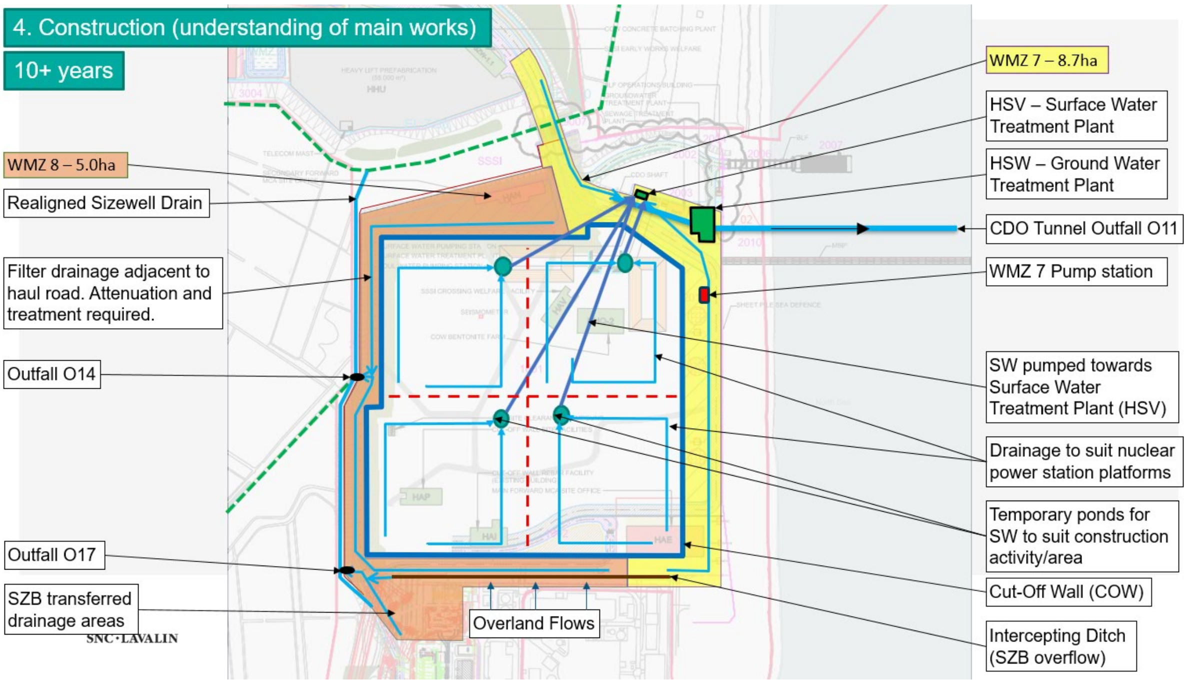
1.4.12 Once the excavation is complete within WMZ 9 the main power station can be constructed. It is anticipated that this is carried out in areas approximating to quadrants. Each area would control its own stormwater with a semi-permanent pond, which would be pumped to a treatment plant before being discharged via the CDO to the sea (**Plate 1.6**).

1.4.13 Flows within WMZ 7 would continue as before and discharge after treatment to the sea.

1.4.14 Flows from WMZ 8 would again be restricted from entering the Sizewell Drain during construction. Flows are to be pumped for treatment and thereafter discharged to sea. Some temporary storage is to be provided to buffer flows to the pumps.

1.4.15 Overland flows from SZB are anticipated along the SZC southern boundary. These flows are to be collected in the permanent cut off drainage channel where they would be directed to the Sizewell Drain via Outfall 17 (O17). It is anticipated that these SZB flows would be after the first flush and only constitute flows greater than 1:10 year storms and therefore be of an acceptable quality to discharge to the Sizewell Drain.

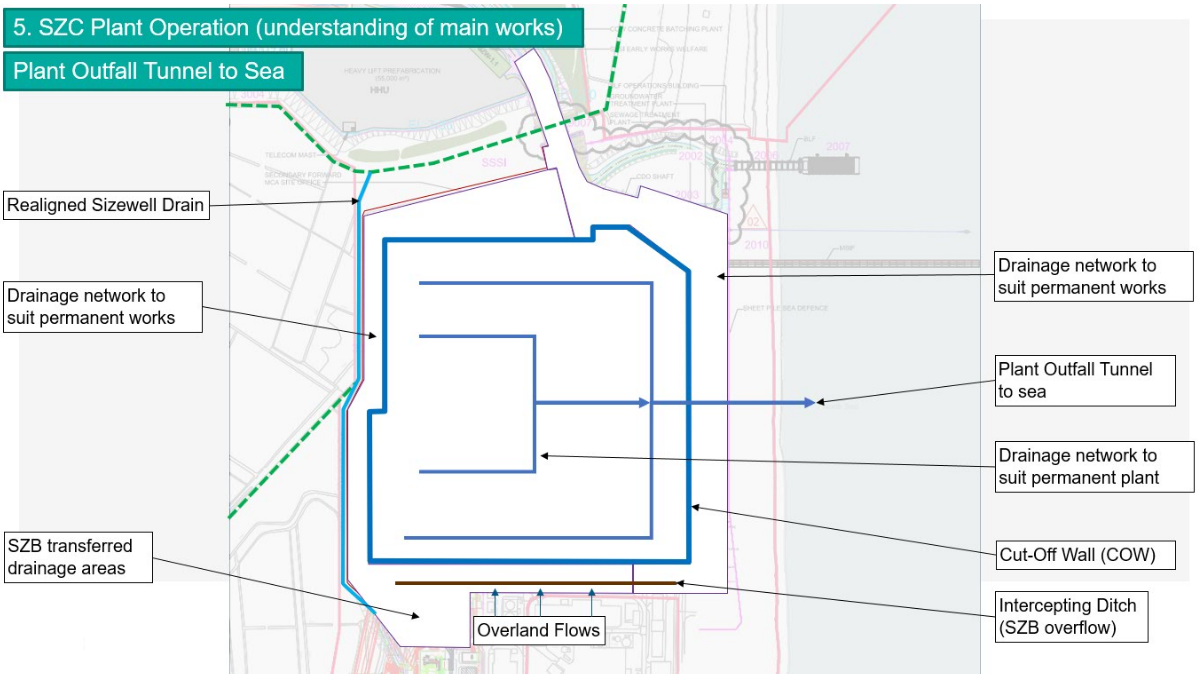
Plate 1.6: Stage 4 - Indicative surface water management during construction of the main platform area works



e) Stage 5 – SZC Plant operation

1.4.16 The final operational stage of the station would manage the stormwater flows from WMZs 7, 8 and 9. These flows are to be directed via the plant outfall tunnels to the sea. Little detailed design work has been carried out on these permanent power station networks (**Plate 1.7**). The overland flows from SZB would continue to be collected in the permanent cut off drainage channel where they would be directed towards the Sizewell C network.

Plate 1.7: Stage 5 – Indicative surface water management during SZC plant operation



1.5 Sizewell Drain – Maintenance

1.5.1 There is a maintenance track proposed alongside the Sizewell Drain. The current design width varies along its length. The ESIDB have suggested a minimum track width of 6m adjacent to the Sizewell Drain. The enlargement of this track from 2m to 6m is a significant increase and affects a number of structures. It is not proposed therefore to identify a complete solution in this document.

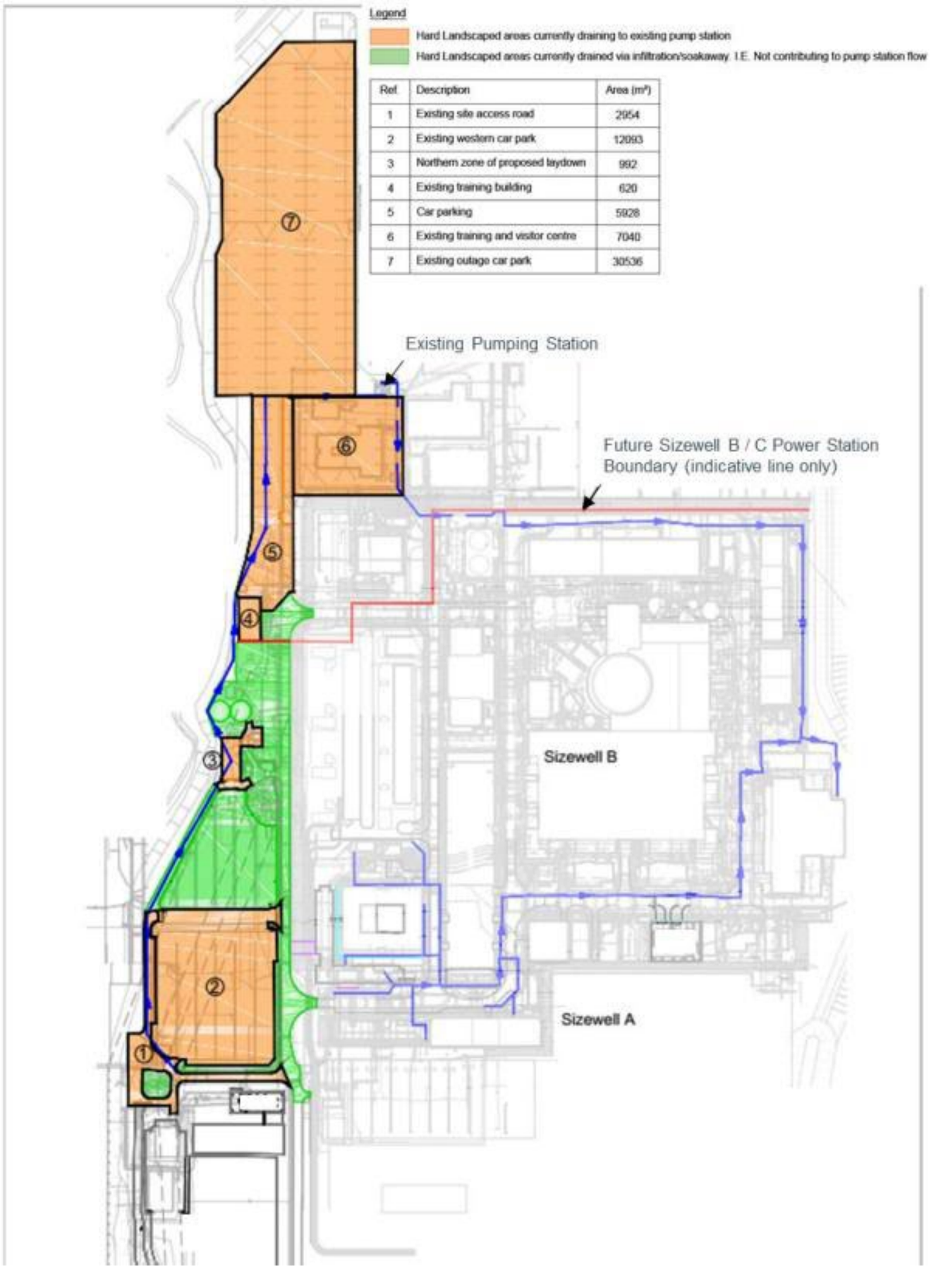
1.5.2 The material supporting the track is to be constructed to maintain stability at the 1:1 side slope.

1.6 Sizewell B (SZB) – Overland flows

1.6.1 The risk of overland flows originates from a limited area within SZB, where runoff flows northwards onto the SZC area. The risk of overland flows from SZB would be for design events greater than which the existing surface water network within the SZB was designed for, due to changes in surface water drainage design guidance for construction in 1990. Some areas in SZB are to have their control transferred to SZC, which is shown in **Plate 1.8** with the red line. These areas constitute the catchment that will discharge to the Sizewell C catchment.

1.6.2 The flows from the SZB site consist of impermeable areas and contain only those flows greater than capacity of the existing network, therefore it is anticipated that the pollution risk would be very low and the treatment requirements would be minimal. No detailed design has currently been carried out on the expected flows generated from this area.

Plate 1.8: Areas external to the SZB main platform fence line draining to the existing pumping station (extract from Sizewell B Relocated Facilities Environmental Statement Appendix 3.2 Surface Water Drainage Strategy April 2019)



1.7 Conclusion

1.7.1 This document provides a high-level overview of how surface water runoff can be managed across WMZ 7, 8 and 9 across various stages of construction. Given that low greenfield runoff rates are estimated, the attenuation demands are significant. Further work is required to develop the drainage design with particular attention to the required attenuation and permitted discharge rate associated with each catchment.

ANNEX 2A.22: TOPOGRAPHICAL CATCHMENT NARRATIVE

Topographical Catchment Narrative

CONTENTS

APPENDICES

1 SIZEWELL C NATURAL OVERLAND SURFACE WATER FLOWS

1.1.1 This short note provides a summary of the existing and proposed surface water flows from the Sizewell C Enabling Works development. This note does not intend to provide detailed design of catchment drainage and their definition.

1.2 Overview

1.2.1 The SZC construction area is large and develops surface water flows in a number of directions. The following describes in overall terms how the development of the construction water management zones do not adversely affect the natural overland surface water patterns.

1.2.2 The overland surface water flows result from storms where rainfall is greater than the available infiltration rate. This condition during a storm event is likely to cover the whole SZC construction site even during a localised storm. During these times water would be discharged into the ditches and streams feeding into the surrounding wetland areas. The natural flows are more likely to enter water drainage routes in a diffuse way. The construction areas plan to discharge the surface runoff through localised outfalls.

1.2.3 The construction area can be discussed as individual water management zones (WMZ) and the natural and proposed flow directions compared. Two phases of construction should be recognised described as Early and Late. The WMZs numbering is similar but do not represent entirely the same areas. For clarification reference should be made to the following drawings:

- Early catchments SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052
- Late catchments SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000053

1.2.4 The existing ground contours are shown in **Appendix A**. Early Catchments and proposed outfalls are shown in **Appendix B**. Late (enabling works) catchments and proposed outfalls are shown in **Appendix C**.

2 INDIVIDUAL WATER MANAGEMENT ZONES

2.1 WMZ 1 (Early & Late)

2.1.1 The natural flows of water from the WMZ 1 area are directed eastwards and join the Sizewell Drain just north of the proposed SSSI crossing. The proposal for this WMZ is to have a single outfall O1 discharging to the east in a ditch that flows to the Sizewell Drain in the north.

2.2 WMZ 2 (Early & Late)

2.2.1 The flows from this WMZ flow naturally to the south and enter the natural drainage in the Leiston Drain. The proposed flows from WMZ 2 are to be into an outfall O2 draining south into the Leiston Drain.

2.3 WMZ 3a & 3b (Early)

2.3.1 The natural drainage from the early catchment areas (3a & 3b) drains to the south entering the Leiston Drain. It is proposed that WMZ 3a is drained by outfall O3 and WMZ 3b is drained by outfall EO3, which enter the Leiston Drain.

2.4 WMZ 3 (Late) & WMZ 4 (Late)

2.4.1 WMZ 4 Late is wholly formed from parts of Early WMZ 3a & Early 3b. The natural drainage pattern follows that in WMZ 3a and WMZ 3b described above.

2.4.2 The outfall O3 is no longer used in this late phase. Both WMZ 3 Late and WMZ 4 Late discharge through a single outfall O4 into the Leiston Drain.

2.5 WMZ 5

2.5.1 This area naturally flows to the north and into the northern wetland area. It is proposed that this area would discharge into the northern wetland area via outfall O5.

2.6 WMZ 10 (Accommodation Campus)

2.6.1 This WMZ, initially formed from parts of WMZ 3a and WMZ 3b, is relatively flat but gently slopes west to east and therefore would flow onto WMZ 4 Late. Based on the existing catchment definition, it is proposed that this water would join the WMZ 4 outfall. This part of the project is undergoing design development and will be reviewed, along with alternative options such as discharging south towards WMZ 6 (late) subject to discussion and approval from Suffolk County Council (SCC). This discharge location is not significantly different from the natural flow path.

2.7 WMZ 6 (WMZ 4 Early & WMZ 6 Early)

2.7.1 Early WMZ 4 and Early WMZ 6 together form Late WMZ 6. The natural water flow routes across early WMZ 4 are to the south to enter the Leiston Drain. The flows within early WMZ 6 also drain to the south and discharge more generally to the Leiston Drain. Early WMZ 4 drains to outfall EO4. This outfall discharging to the Leiston Drain only operates during this early stage. Early WMZ 6 discharges to outfall O6.

2.7.2 During the late stage the late WMZ 6 (comprising both early WMZ 4 & early WMZ 6) discharges only to outfall O6 into the Leiston Drain and outfall EO4 is no longer to be used. As described above, flows from the Campus may join the discharge into the outfall O6.

2.8 ACA West

2.8.1 The small catchment in the west of the ACA area slopes to the west with natural flows heading north and then east, eventually making their way to the Leiston Drain. The basin in the west ACA is pumped to the outfall O6 that discharges to the Leiston Drain slightly northwards.

2.9 ACA East

2.9.1 The main area of the ACA naturally flows to the east and draining to the Leiston Drain. It is proposed that flows generated on the ACA East would be discharged via the outfall O7 into the Sizewell Marshes, following discussions with Environment Agency, East Suffolk Council, SCC and Internal Drainage Board in December 2020.

2.10 Railway

2.10.1 The Green Railway route has limited infiltration in its western area and therefore the overland flows have been directed to the eastern part to a basin. The eastern part of the area has much higher infiltration values and therefore the basin is to be designed to work on infiltration alone with no outfall. The natural flows in this area are small due to the size of the Green Rail route. It is not thought that any disruption is caused to the natural drainage pattern and all flows in this arrangement would be returned to the groundwater.

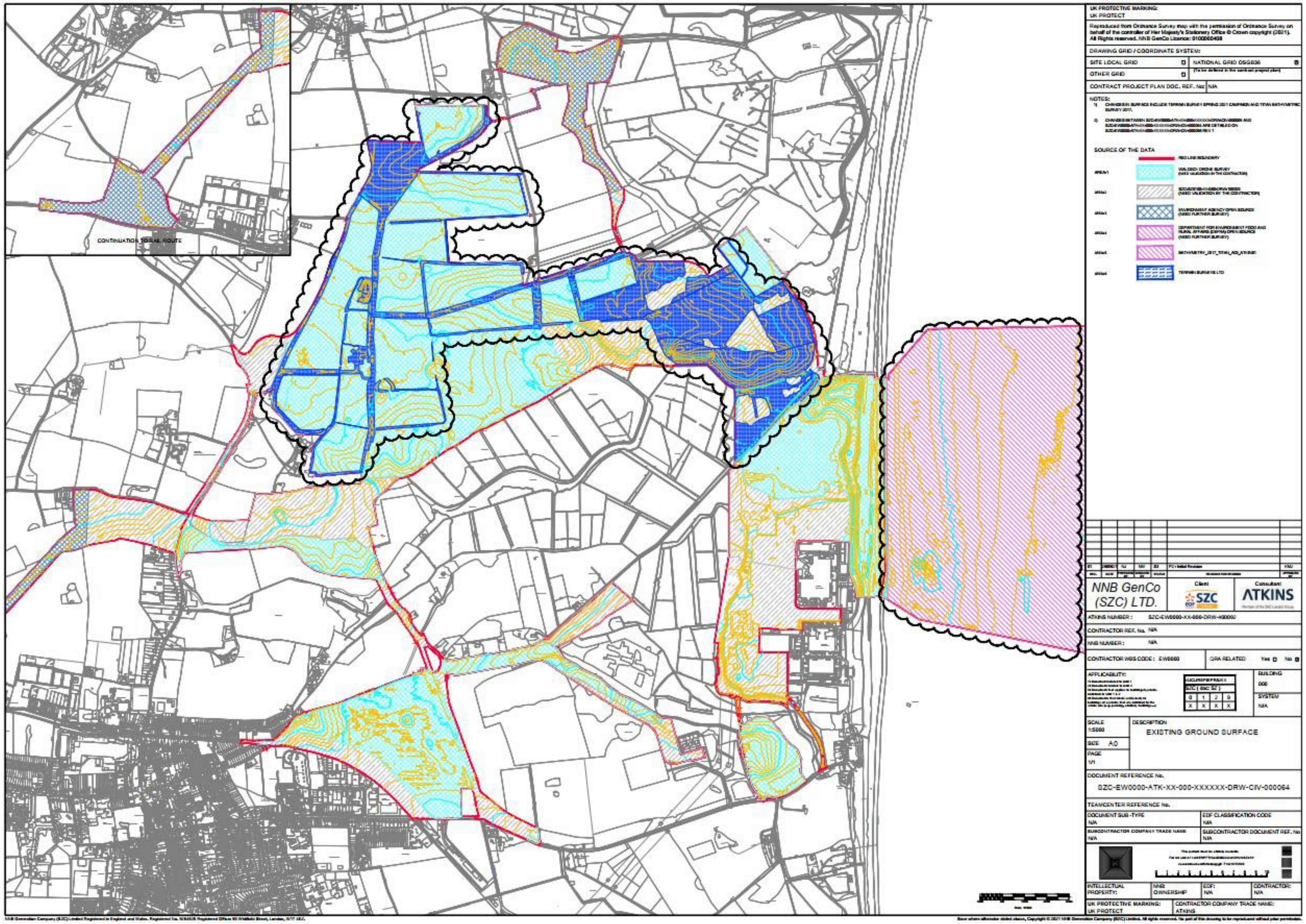
2.10.2 This part of the project is still undergoing design as infiltration figures are being assessed. Should the infiltration figures be found to be inadequate for an infiltration solution an outfall may be necessary and the pumped flows would be directed to the Leiston Drain via outfall O6. This discharge location represents the natural direction of the flows from the railway area.

3 SUMMARY

3.1.1 It is recognised that natural flows will be much more diffuse than the flows from outfall point discharge locations. In discussing each WMZ it has been shown that the final destination of the overland flows are to a large extent the same natural drainage areas. This has been intentional and indeed the natural contours have been the guidance in defining the WMZ areas and their outfall discharge locations.

3.1.2 It is important to note that exact outfall locations and their associated discharge rates are subject to change based on future hydraulic modelling. All discharges are to be modelled as part of the wider catchment to ensure they do not increase flood risk. Ongoing engagement with environmental stakeholders to determine satisfactory discharge rates and locations to reduce environmental impact will continue to be an important part of the design process.

APPENDIX A: EXISTING GROUND CONTOURS


CONTINUATION TO RAIL ROUTE
UK PROTECTIVE MARKING:
UK PROTECT
Reproduced from Ordnance Survey map with the permission of Ordnance Survey on behalf of the controller of Her Majesty's Stationery Office © Crown copyright (2021). All Rights reserved. NNB GenCo Licence: 0100060408
DRAWING GRID / COORDINATE SYSTEM:
SITE LOCAL GRID
NATIONAL GRID OSGB36
OTHER GRID
CONTRACT PROJECT PLAN DOC. REF. No: N/A
NOTES:
SOURCE OF THE DATA
NNB GenCo (SZC) LTD.
Client
SZC
Consultant
ATKINS
ATKINS NUMBER : SZC-EW0000-XX-000-DRW-400090
CONTRACTOR REF. No. N/A
NNB NUMBER : N/A
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QRA RELATED Yes No
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TEAMCENTER REFERENCE No.
DOCUMENT SUB-TYPE
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N/A
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CONTRACTOR:
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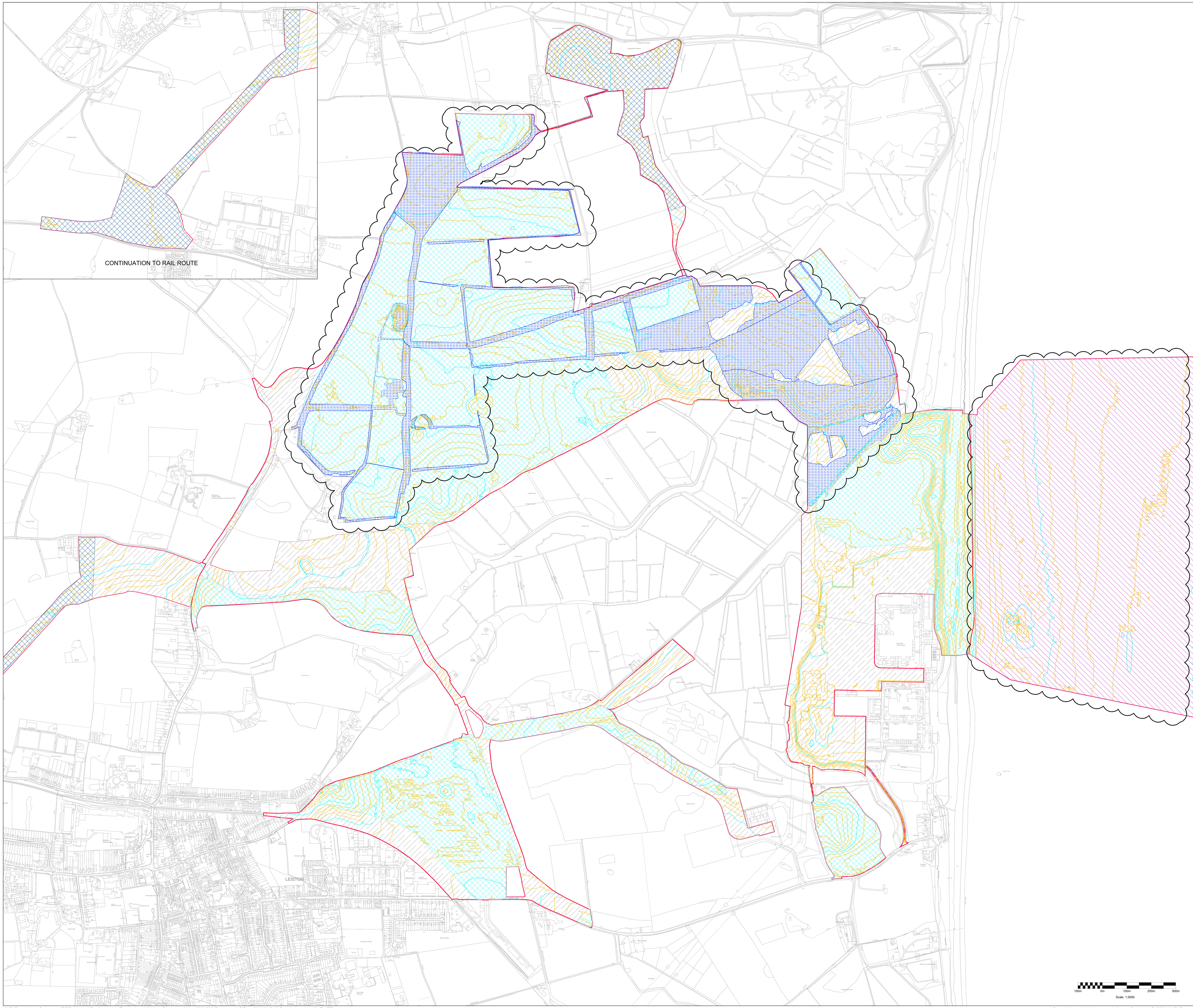

UK PROTECTIVE MARKING:
UK PROTECT

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) CHANGES IN SURFACE INCLUDE TERRAIN SURVEY SPRING 2021 CAMPAIGN AND TITAN BATHYMETRIC SURVEY 2017.

2) CHANGES BETWEEN SZC-EW0000-ATK-XX-000-XXXXXX-DRW-CIV-000009 AND SZC-EW0000-ATK-XX-000-XXXXXX-DRW-CIV-000064 ARE DETAILED ON SZC-EW0000-ATK-XX-000-XXXXXX-DRW-CIV-000058 REV 1

SOURCE OF THE DATA

	RED LINE BOUNDARY
AREA-1	WALDECK DRONE SURVEY (NEED VALIDATION BY THE CONTRACTOR)
AREA-2	SZC-SZ0100-XX-000-DRW-100003 (NEED VALIDATION BY THE CONTRACTOR)
AREA-3	ENVIRONMENT AGENCY OPEN SOURCE (NEED FURTHER SURVEY)
AREA-4	DEPARTMENT FOR ENVIRONMENT FOOD AND RURAL AFFAIRS (DEFRA) OPEN SOURCE (NEED FURTHER SURVEY)
AREA-5	BATHYMETRY_2017_TITAN_AOI_ATKINS1
AREA-6	TERRAIN SURVEYS LTD

REV.	DATE	PREPARED BY	CHECKED BY	STATUS	REASONS FOR REVISION	APPROVED BY
01	28/09/21	NJ	NM	S3	P2 - Initial Revision	KMJ

NNB GenCo (SZC) LTD.

Client: SZC

Consultant: ATKINS

ATKINS NUMBER : SZC-EW0000-XX-000-DRW-400092

CONTRACTOR REF. No. N/A

NNB NUMBER : N/A

CONTRACTOR WBS CODE : EW0000

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			
SZC (doc: SZ)			
0	1	2	9
X	X	X	X

BUILDING
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SYSTEM
N/A

SCALE
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SIZE A0

PAGE
1/1

DESCRIPTION
EXISTING GROUND SURFACE

DOCUMENT REFERENCE No.
SZC-EW0000-ATK-XX-000-XXXXXX-DRW-CIV-000064

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

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UK PROTECTIVE MARKING: UK PROTECT	CONTRACTOR COMPANY TRADE NAME: ATKINS