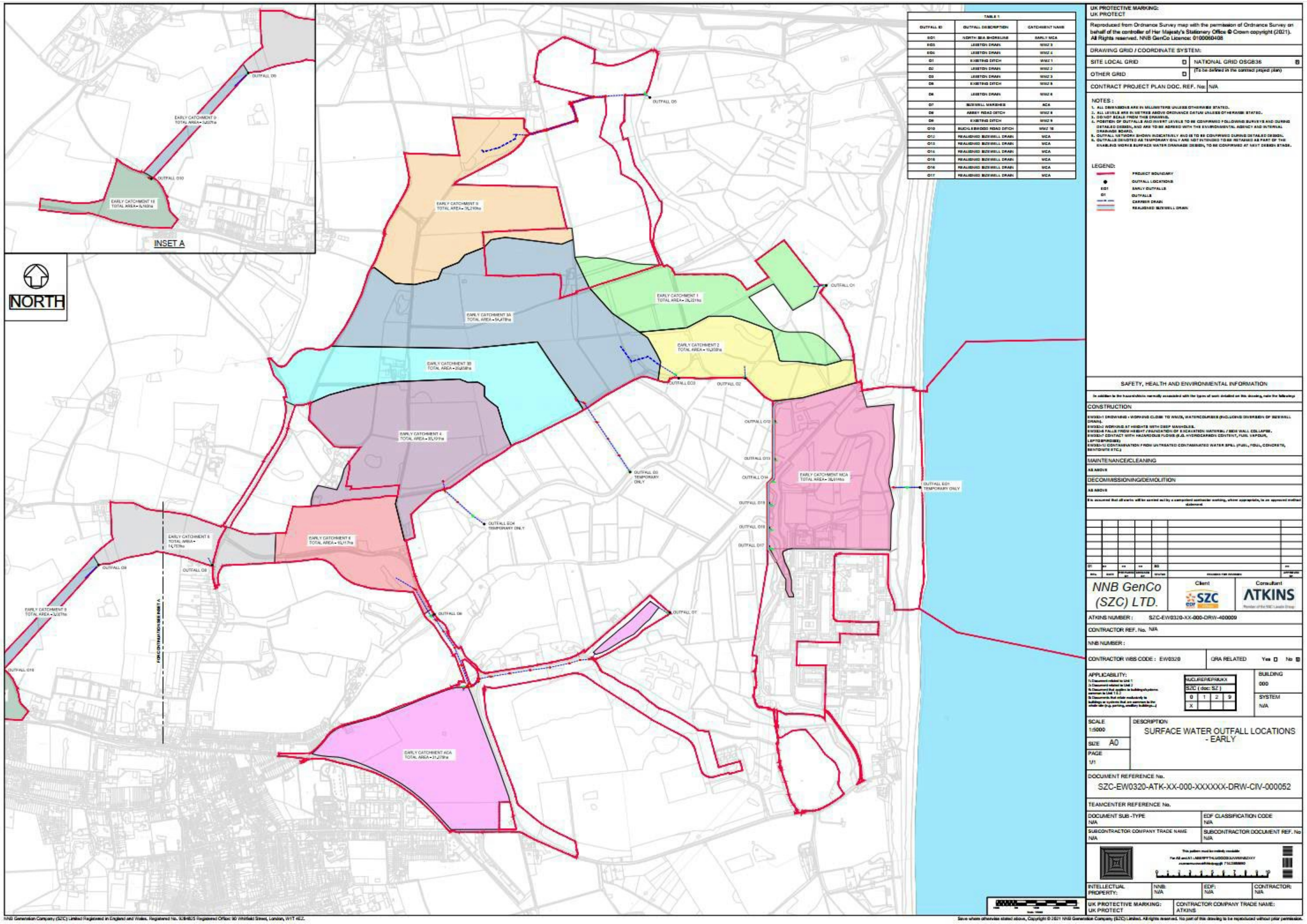
# The Sizewell C Project

## SZC Co.'s Response to the Secretary of State's Request for Further Information dated 18 March 2022: Appendix 3 - The Drainage Strategy Part 11 of 12

Revision: 2.0

April 2022

# APPENDIX B: EARLY CATCHMENTS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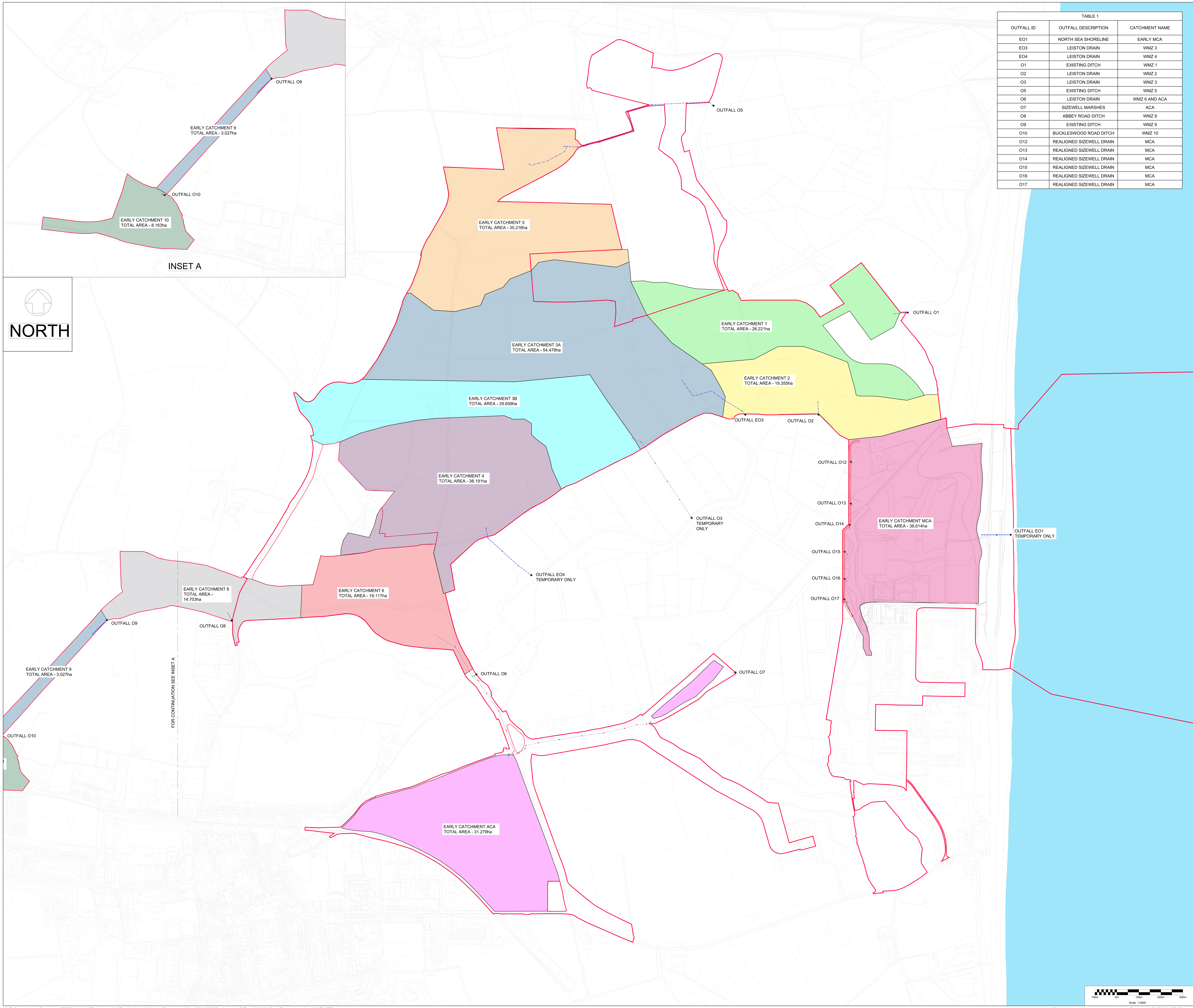
UK PROTECTIVE MARKING:
UK PROTECT
SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION
CONSTRUCTION
MAINTENANCE/CLEANING
DECOMMISSIONING/DEMOLITION
NNB GenCo
(SZC) LTD.
SZC
ATKINS
SURFACE WATER OUTFALL LOCATIONS
- EARLY
SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052




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

**UK PROTECTIVE MARKING:**
UK PROTECT



DRAWING GRID / COORDINATE SYSTEM:

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

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

NOTES :

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS OTHERWISE STATED.
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4. POSITION OF OUTFALLS ARE INDICATIVE AND SUBJECT TO DESIGN DEVELOPMENT. OUTFALL LOCATIONS TO BE CONFIRMED FOLLOWING SURVEYS AND TO BE AGREED WITH THE ENVIRONMENT AGENCY AND INTERNAL DRAINAGE BOARD.
5. OUTFALL NETWORK SHOWN INDICATIVELY AND IS TO BE CONFIRMED DURING DETAILED DESIGN.
6. OUTFALLS DENOTED AS TEMPORARY ONLY ARE NOT INTENDED TO BE RETAINED AS PART OF THE ENABLING WORKS SURFACE WATER DRAINAGE DESIGN, TO BE CONFIRMED AT NEXT DESIGN STAGE.

LEGEND:

- PROJECT BOUNDARY
- OUTFALL LOCATIONS
- EO1 EARLY OUTFALLS
- O1 OUTFALLS
- CD CARRIER DRAIN
- REALIGNED SIZEWELL DRAIN

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

EW302-1 DROWNING - WORKING CLOSE TO WMZS, WATERCOURSES (INCLUDING DIVERSION OF SIZEWELL DRAIN).
EW302-2 WORKING AT HEIGHTS WITH DEEP MANHOLES.
EW302-5 FALLS FROM HEIGHT / INUNDATION OF EXCAVATION MATERIAL / SIDE WALL COLLAPSE.
EW302-7 CONTACT WITH HAZARDOUS FLOWS (E.G. HYDROCARBON CONTENT, FUEL VAPOUR, LEPTOSPIROSIS)
EW302-12 CONTAMINATION FROM UNTREATED CONTAMINATED WATER SPILL (FUEL, FOUL, CONCRETE, BENTONITE ETC.)

MAINTENANCE/CLEANING

AS ABOVE

DECOMMISSIONING/DEMOLITION

AS ABOVE

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

| REV. | DATE | PREPARED BY | CHECKED BY | STATUS | REASONS FOR REVISION | APPROVED BY |
|---|---|---|---|---|---|---|
| 01 | 06/10/21 | NJ | MS | S3 | P2 - Initial Revision | KMJ |

NNB GenCo (SZC) LTD.

Client

Consultant

Member of the SNC-Lavalin Group

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

CONTRACTOR REF. No. N/A

NNB NUMBER :

CONTRACTOR WBS CODE : EW0320

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 | | | |
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SYSTEM
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DESCRIPTION
SURFACE WATER OUTFALL LOCATIONS - EARLY

DOCUMENT REFERENCE No.
SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052

TEAMCENTER REFERENCE No.

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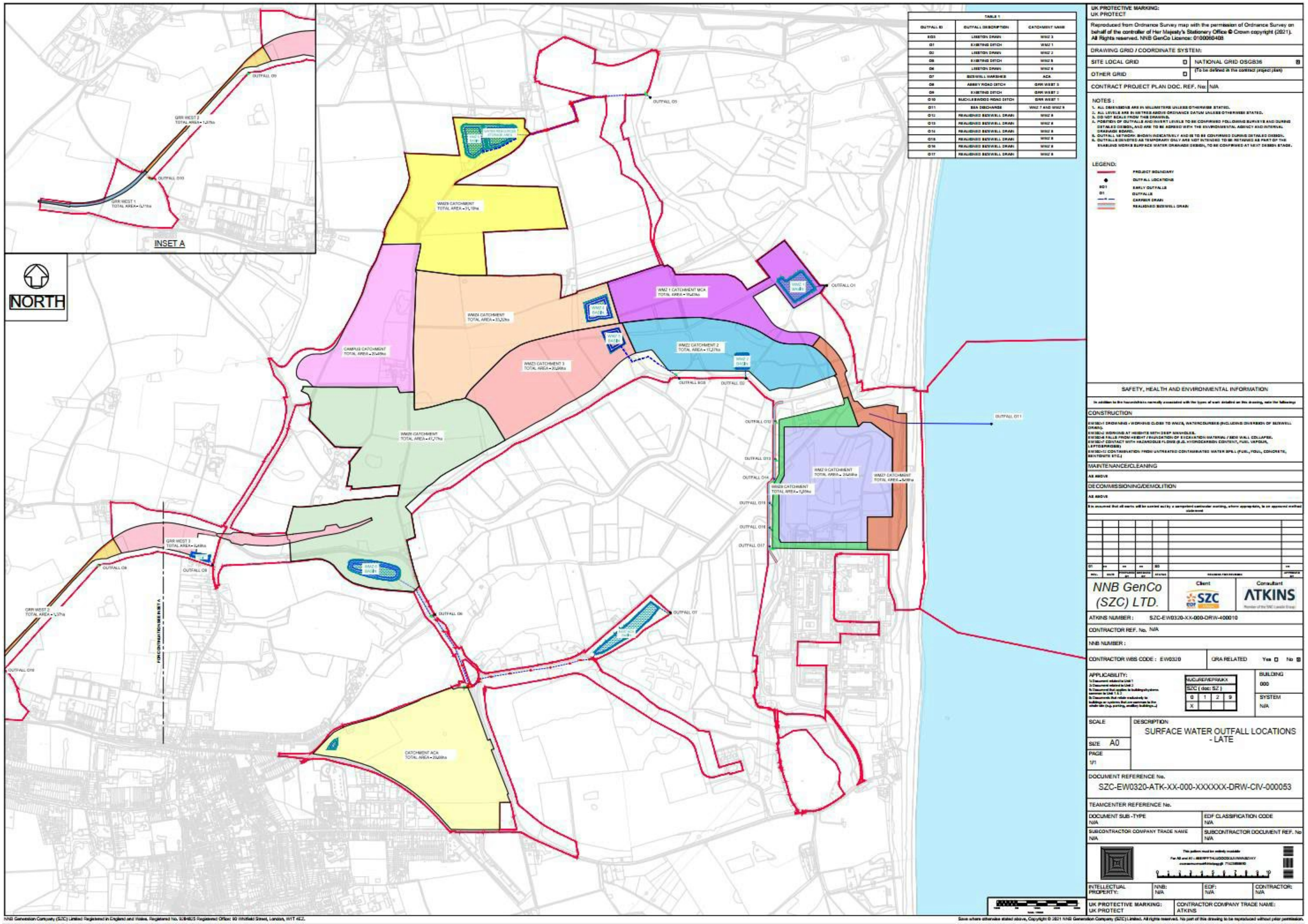
**UK PROTECTIVE MARKING:**
UK PROTECT

CONTRACTOR COMPANY TRADE NAME:
ATKINS

# APPENDIX C: LATE (ENABLING WORKS) CATCHMENTS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UK PROTECT
DRAWING GRID / COORDINATE SYSTEM:
SITE LOCAL GRID
NATIONAL GRID OSGB36
OTHER GRID
NOTES:
LEGEND:
SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION
CONSTRUCTION
MAINTENANCE/CLEANING
DECOMMISSIONING/DEMOLITION
NNB GenCo
(SZC) LTD.
Client
SZC
Consultant
ATKINS
ATKINS NUMBER : SZC-EW0320-XX-000-DRW-400010
CONTRACTOR REF. No. N/A
NNB NUMBER :
CONTRACTOR WBS CODE : EW0320
QRA RELATED Yes No
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TEAMCENTER REFERENCE No.
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EDF CLASSIFICATION CODE
N/A
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SUBCONTRACTOR DOCUMENT REF. No
N/A
INTELLECTUAL PROPERTY:
NNB:
N/A
EDF:
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CONTRACTOR:
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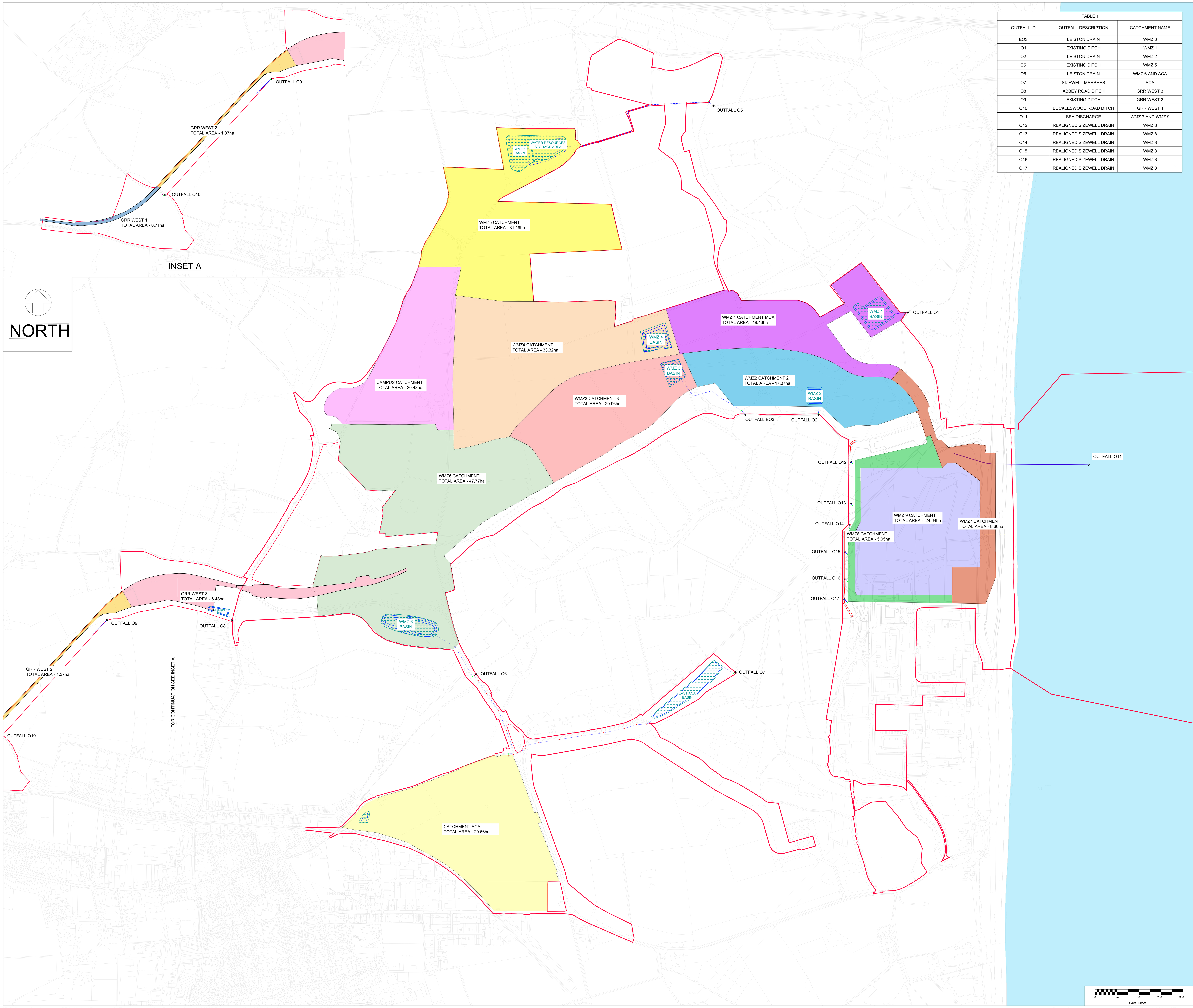
UK PROTECTIVE MARKING:
UK PROTECT
CONTRACTOR COMPANY TRADE NAME:
ATKINS




| TABLE 1 | | |
|---|---|---|
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| EO3 | LEISTON DRAIN | WMZ 3 |
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| O2 | LEISTON DRAIN | WMZ 2 |
| O5 | EXISTING DITCH | WMZ 5 |
| O6 | LEISTON DRAIN | WMZ 6 AND ACA |
| O7 | SIZEWELL MARSHES | ACA |
| O8 | ABBEY ROAD DITCH | GRR WEST 3 |
| O9 | EXISTING DITCH | GRR WEST 2 |
| O10 | BUCKLESWOOD ROAD DITCH | GRR WEST 1 |
| O11 | SEA DISCHARGE | WMZ 7 AND WMZ 9 |
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| O13 | REALIGNED SIZEWELL DRAIN | WMZ 8 |
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### MAINTENANCE/CLEANING

AS ABOVE

### DECOMMISSIONING/DEMOLITION

AS ABOVE

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| 01 | 06/10/21 | NJ | MS | S3 | P2 - Initial Revision | KMJ |

NNB GenCo (SZC) LTD.

Client: SZC

Consultant: ATKINS — Member of the SNC-Lavalin Group

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

CONTRACTOR REF. No. N/A

NNB NUMBER :

CONTRACTOR WBS CODE : EW0320

QRA RELATED Yes ☐ No ☒

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| NUCL/REP/EPR/UKX | | | |
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BUILDING
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DESCRIPTION

# Surface Water Outfall Locations – Late

DOCUMENT REFERENCE No.
SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000053

TEAMCENTER REFERENCE No.

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**UK PROTECTIVE MARKING:**
UK PROTECT

CONTRACTOR COMPANY TRADE NAME:
ATKINS

# ANNEX 2A.23: DRAINAGE INTENT STATEMENT SPORTS PITCHES AND NON NUCLEAR ISLAND OPERATIONAL DRAINAGE

# Drainage Intent Statement for Sports Pitches and Non Nuclear Island Operational Drainage

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# 1 PURPOSE

1.1.1 The purpose of this statement is to outline the intended drainage provision relating to the proposed Sports Pitches at Leiston Leisure Centre and at Sizewell C power station out with the Nuclear Licensed Site.

# 2 BACKGROUND INFORMATION

2.1.1 REP10-030, -31 and 032 Drainage Strategy outlines the fundamental principles to be applied together with the Flood Risk Assessment REP AS-018. These principles and assessments are submitted as part of the Development Consent Order EN10012 and contain significant evidence in the form of surveys, modelling, assessments and other information that should be read in conjunction with this summary statement.

# 3 SPORTS PITCHES

3.1.1 AS-018 - Flood Risk Assessment – Exec Summary – Section e – states:

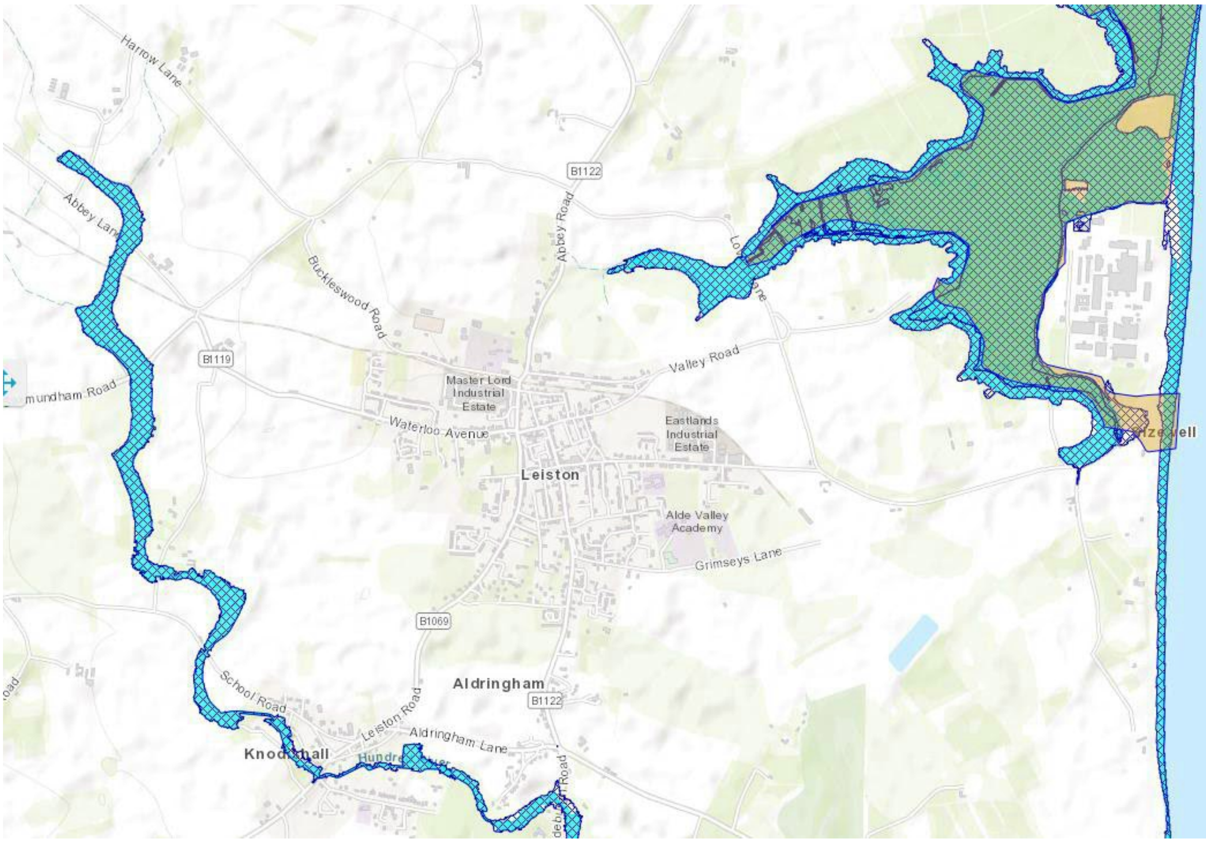
3.1.2 ***Off-site sports facilities***

3.1.3 *The off-site sports facilities are considered to be at low risk of flooding from groundwater, reservoirs, fluvial, coastal and breach.*

3.1.4 *The development of the off-site sports facilities would marginally increase the localised risk of flooding from surface water and sewers. The proposed embedded design approach for surface water and sewers provides suitable mitigation to maintain a low flood risk while the site is in use. Therefore, the mitigated surface water and sewer flood risk is considered to be low.*

3.1.5 *The off-site sports facilities are a permanent development in Leiston and would remain in use by the local community throughout the operation and decommissioning phases of the Sizewell C power station. The level of mitigated flood risk would remain unchanged due to the inclusion of climate change allowances in the design.*

**Plate 3.1: Flood Risk Zones for Leiston**




## 3.2 Leiston Sports Centre Pitches - dDCO Requirement 18 Sports Pitches

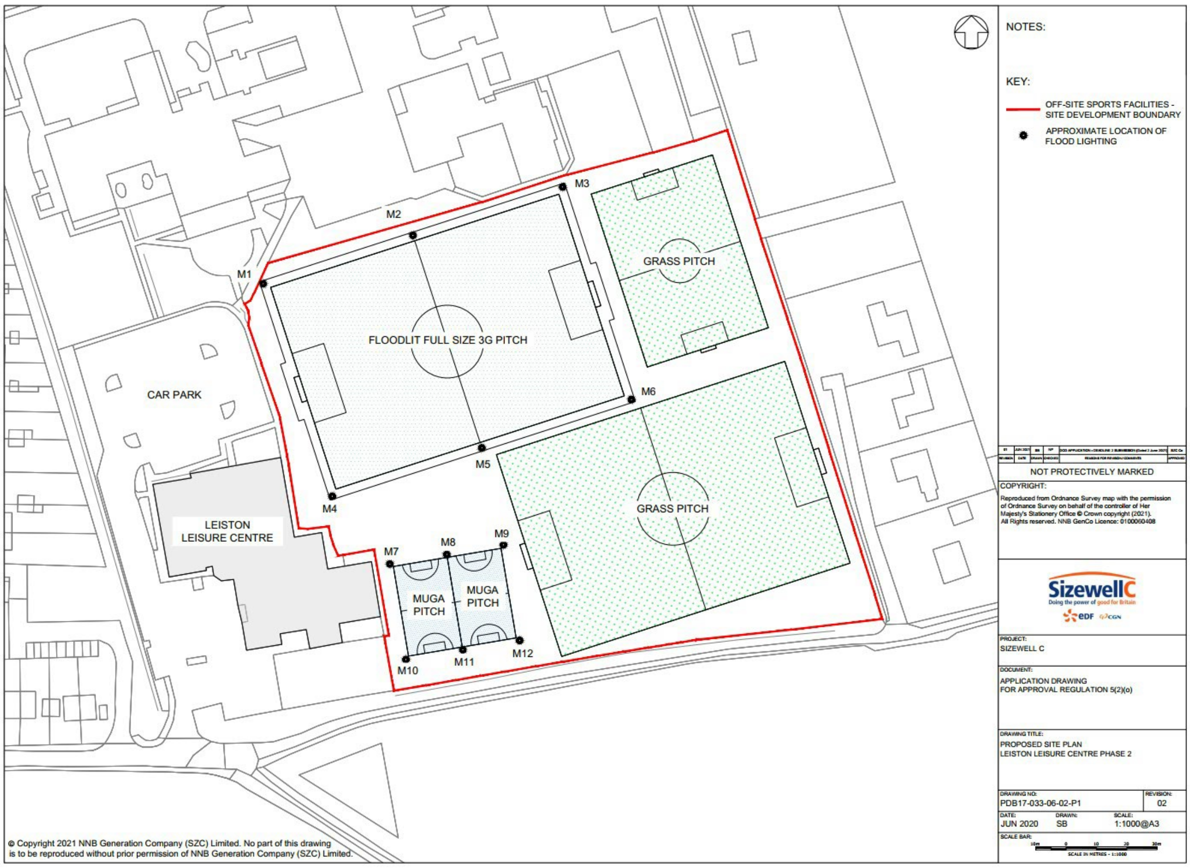
3.2.1 ***Sports Facilities: Reserved matters***

3.2.2 *(1) Construction of Work No. 5, must not commence until details of the layout, scale and external appearance of those buildings and associated landscape works have been submitted to and approved by East Suffolk Council.*

3.2.3 *(2) The details referred to in paragraph (1) must be in general accordance with Proposed Site Plan Leiston Leisure Centre Phase (PDB17-033-06-02-P1).*

3.2.4 *(3) Work No. 5 must be carried out in accordance with the approved details.*

**Plate 3.2: [REP2-011](REP2-011) - Proposed Site Plan Leiston Leisure Centre Phase (PDB17-033-06-02-P1)**




## 3.3 Drainage Strategy

3.3.1 [REP10-030](REP10-030) Drainage Strategy section 5-1:

3.3.2 *a) Leiston off-site sports facilities 5.1.1 Off-site sports facilities for use by the general public and the construction workforce are to be located in Leiston and retained for use after construction. A full-sized artificial grass pitch (AGP) and multi-use games areas (MUGA) are proposed on land between Leiston Leisure Centre and Alde Valley Academy. 5.1.2.*

3.3.3 *b) The base for an AGP and MUGA is typically a porous engineered construction consisting of two courses of open-textured bituminous macadam laid above a graded stone sub-base, which would allow the AGP and MUGA to be free-draining. Where infiltration is poor, a sub-surface drainage system may be required. The design of subsurface drainage will follow Sport England's Artificial Surfaces for Outdoor Sport*

*Design Guidance and employ SuDS techniques to attenuate and limit flow from the site to greenfield runoff rates. 5.1.3.*

3.3.4 The drainage intent for the addition of the proposed new sports pitches is to treat the site in a manner that ensures it behaves hydraulically as it does now with no increase in off-site flood risk. The development area comprises playing fields presently. As such, the two proposed grass pitches present no change in respect of how the site would drain. The proposed 3G pitch will be constructed with a porous subbase to simulate infiltration as grass; it is not considered that the small increases in hard surface from paths and floodlight installation will have negligible effect on site drainage. The Muga's will have a drainage system as hard paved, this will run to oversized land drains to incorporate storage and simulate green field run off rates.

3.3.5 In order to clarify the feasibility of the sports pitch, infiltration testing is proposed for the site. This will assist in determining the requirements for the surface water system. High infiltration will allow the pitches to operate without storage, lower infiltration (lowest SCC acceptable level) will require storage on site. There is adequate space for this storage requirement and slower infiltration.

3.3.6 Should infiltration testing reveal that infiltration is not an option for this site the proposal is to store the surface water on site and pump to local non-potable demand areas. This includes the local allotments.

# 4 RETAINED PERMANENT WORKS OUTSIDE OF NSL DRAINAGE

4.1.1 The Permanent SZC development works **Plate 3.2** will adhere to the Flood Risk Assessment and Drainage Strategy Outlined. The Main Development Site within the NSL as stated and which is marked on **Plate 4.2** will drain to the cooling out take structures.

### Plate 4.1: REP10-004 – 2.5 Main Development Site Permanent Development Site Proposed Landscape Masterplan

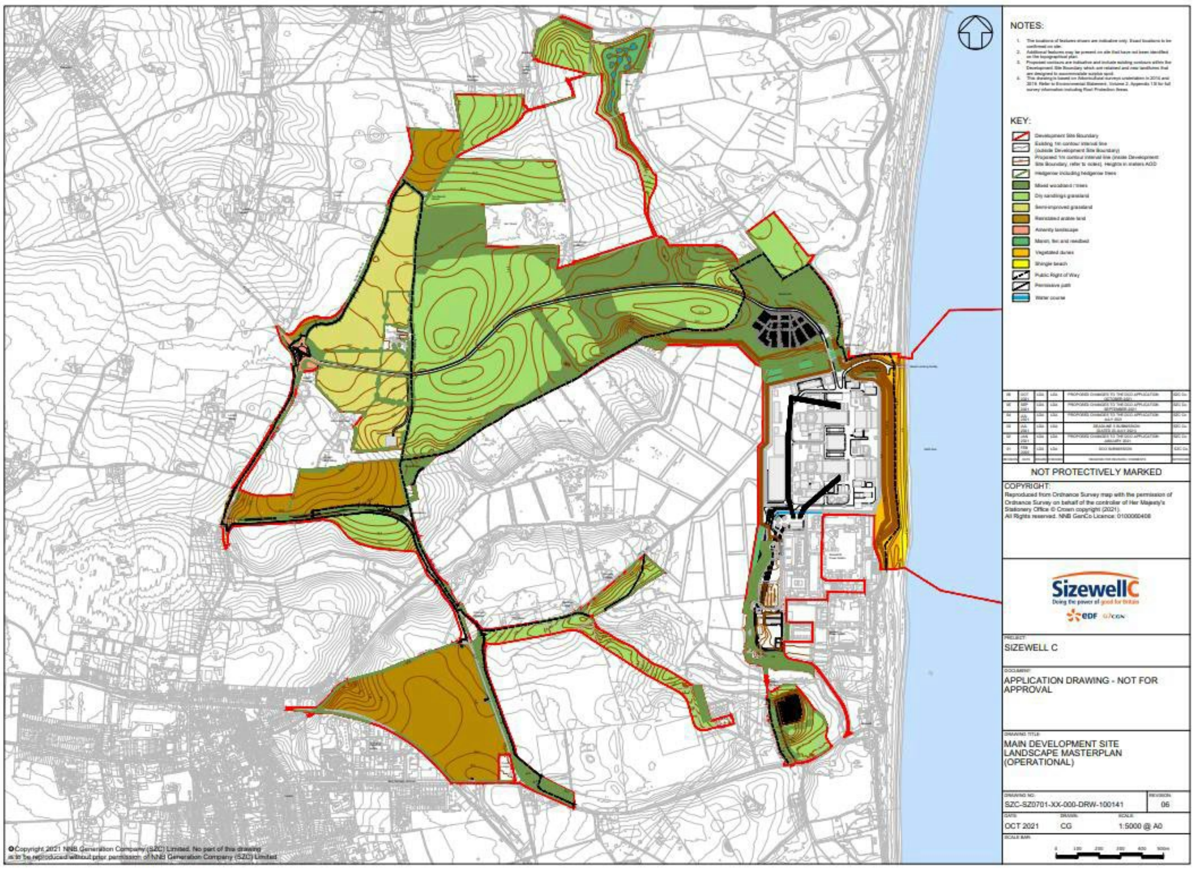
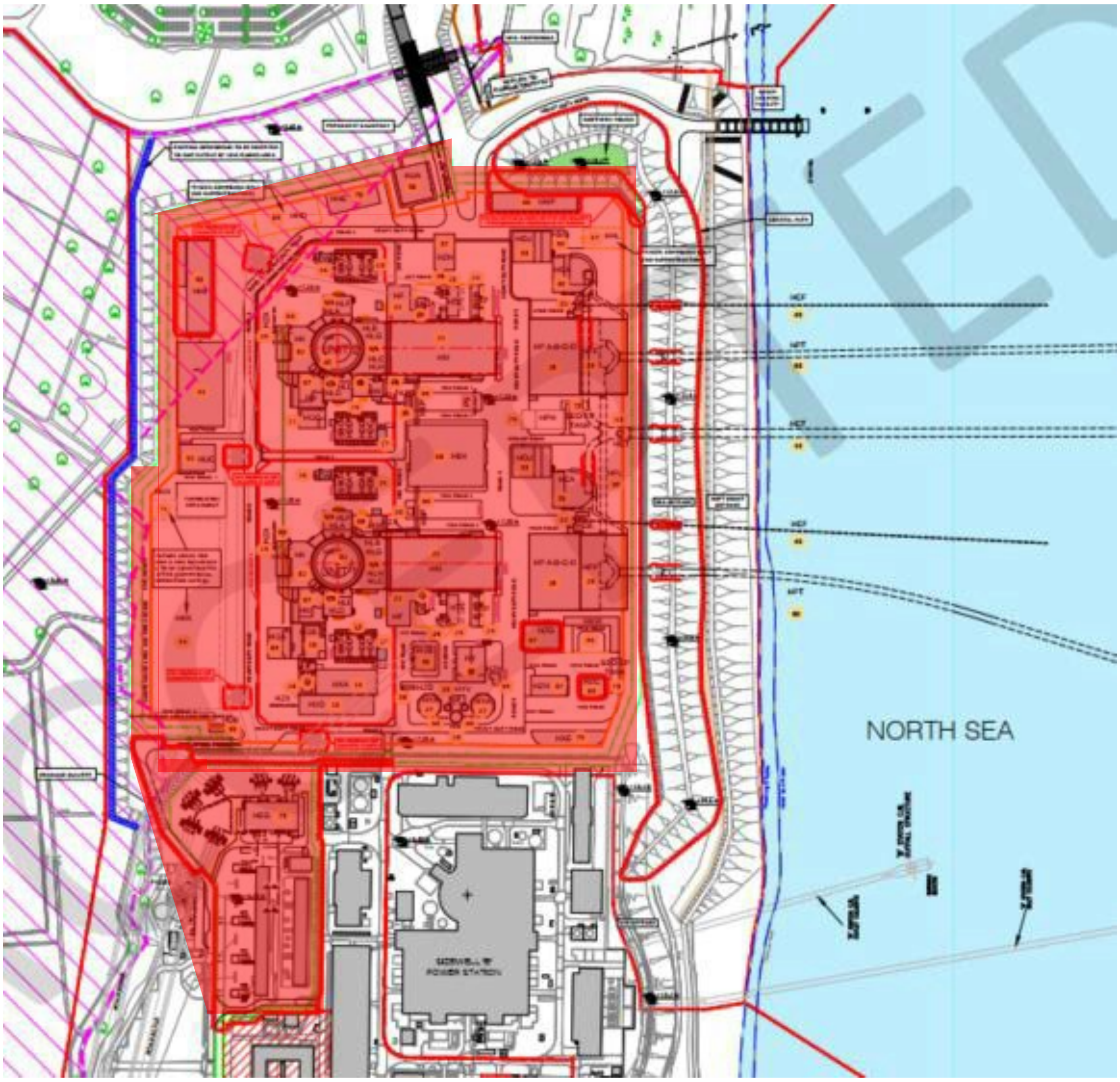


**Plate 4.2: Main Construction Area – Nuclear Site Licence (NSL) definition**




4.1.2 The remaining areas listed below will have the stated drainage strategy:

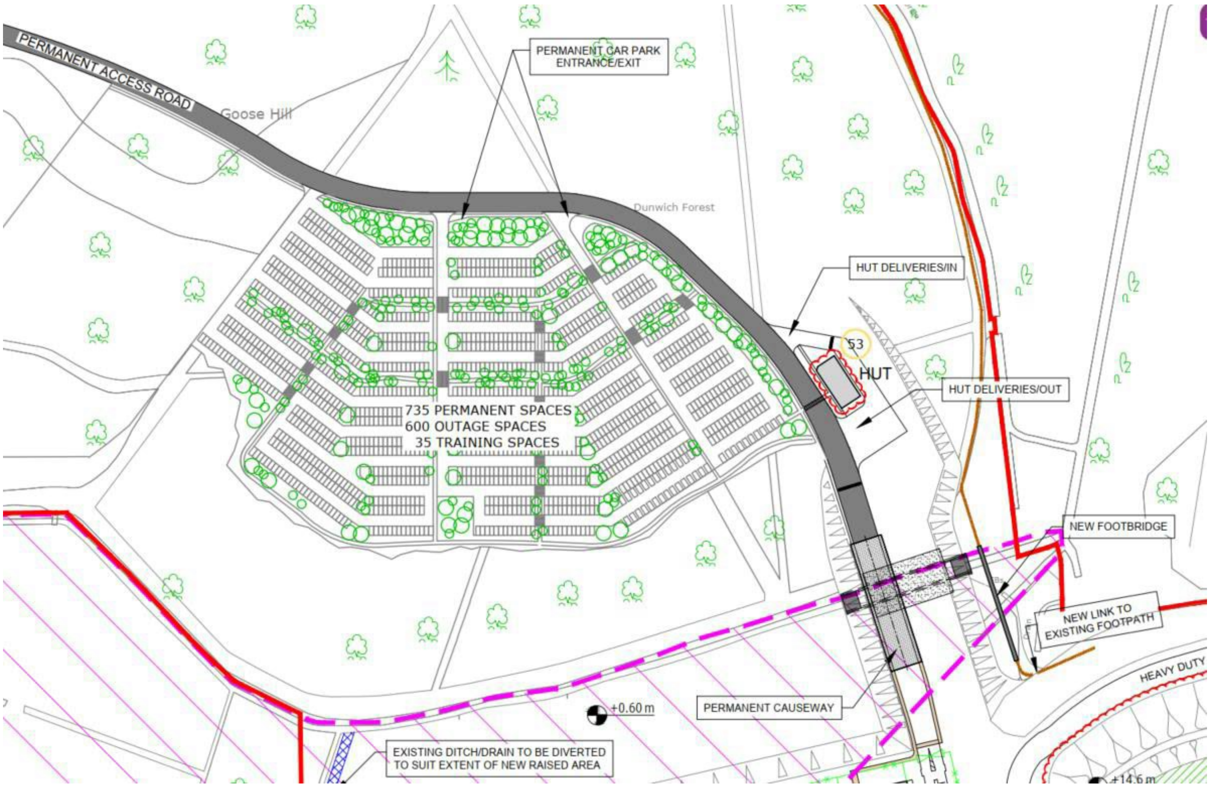
a) SSSI Crossing

4.1.3 The SSSI Crossing has a high point above Leiston Drain, as such the crossing to the South of this ridge will drain to the NSL platform. The area to the North of this will drain to the Permanent car park and be captured in this system within the car park.

### b) Permanent Car Park

4.1.4 The permanent car park will apply the Drainage Strategy and capture surface water for infiltration. This is proposed to be in the form of permeable paving and soft landscape areas with a retention sub-base to enable infiltration. This area will also collect and infiltrate the associated road and hardstandings from the car park to the SSSI Crossing.

**Plate 4.3: Permanent Car Park Layout**




### c) Permanent Access Road

4.1.5 The permanent access road will utilise filter strips an swales to infiltrate run off back to ground.

### d) 132KV Substation (HTR)

4.1.6 The 132KV substation South of Upper Abbey farm will include environmental protection measures (containment and treatment) prior to infiltration to ground.

### e) B1122 Entrance Roundabout

4.1.7 The entrance roundabout will be retained as per the strategy currently submitted in the DCO, utilising an infiltration basin to the South as shown in the submitted drawings.

### f) Restored Estate

4.1.8 The remaining site which is currently mainly under arable use will be restored to a mosaic of free draining semi natural habitats including dry grassland, heathland, scrub and deciduous woodland.

# ANNEX 2A.24: AD6 DRAINAGE DESIGN NOTE

# AD6 DRAINAGE DESIGN NOTE

## DOCUMENT CONTROL

This document is stored and approved in the AECOM Online SharePoint system.

| | | |
|---|---|---|
| **Author:** | Name: Derek Lord | Date: 7 February 2022 |
| **Reviewer(s):** | Name: Chris Uzzell | Date: |
| **Owner:** | Name: Mark Beaumont | Date: |

### Revision history / Record of comments

| Revision | Amendment | By | Date |
|---|---|---|---|
| 01 | First issue for review | Derek Lord | 07/02/2022 |
| 02 | Updated in response to SCC comments | Derek Lord | 02/03/2022 |
| 03 | Updated in response to SCC Comments | Derek Lord | 29/03/2022 |
| Revision * | * | * | Click or tap to enter a date. |
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# 1 INTRODUCTION

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

1.1.2 The AD6 highway development was originally submitted to the Planning Inspectorate (PINS) as part of the Application to build and operate a new nuclear power station to the north of Sizewell B.

1.1.3 SZC Co. has undertaken work to validate and develop the design of the AD6 highway facilities that were originally submitted as part of the Application. This document forms one of a series of design validation and evolution documents being provided in support of the **Outline Drainage Strategy** [REP2-033] and subsequent **Drainage Strategy** (submitted at Deadline 7).

1.1.4 The AD6 works consist of the upgrading and modification of the existing highways directly adjacent to the power station site, and in the Leiston area to accommodate traffic during the construction phase of Sizewell C. They are also required to provide site access and to accommodate a new railway crossing of the B1122 Abbey Road at its junction with Lovers Lane. A bridleway which passes through the temporary construction area (TCA), designated Bridleway 19 is diverted to a new route clear of the site.

1.1.5 With the exception of the ACA entrances which will be removed as part of site restoration and a small length of Bridleway 19 adjacent to Water Management Zone 6 (WMZ6) which will be moved to its permanent alignment when WMZ6 is removed, the modifications of the existing highway network are permanent and will be adopted as part of the highway network by Suffolk County Council (SCC). Bridleway 19 will in part be adopted by SCC and in part be located on private land but with public right of way access.

1.1.6 The AD6 Leiston adoptable highways works are required at a number of locations in the vicinity of Leiston as listed below:

- Main Site Access Roundabout
- Abbey Road from Abbey Lane to new Lovers Lane junction
- Lovers Lane diversions

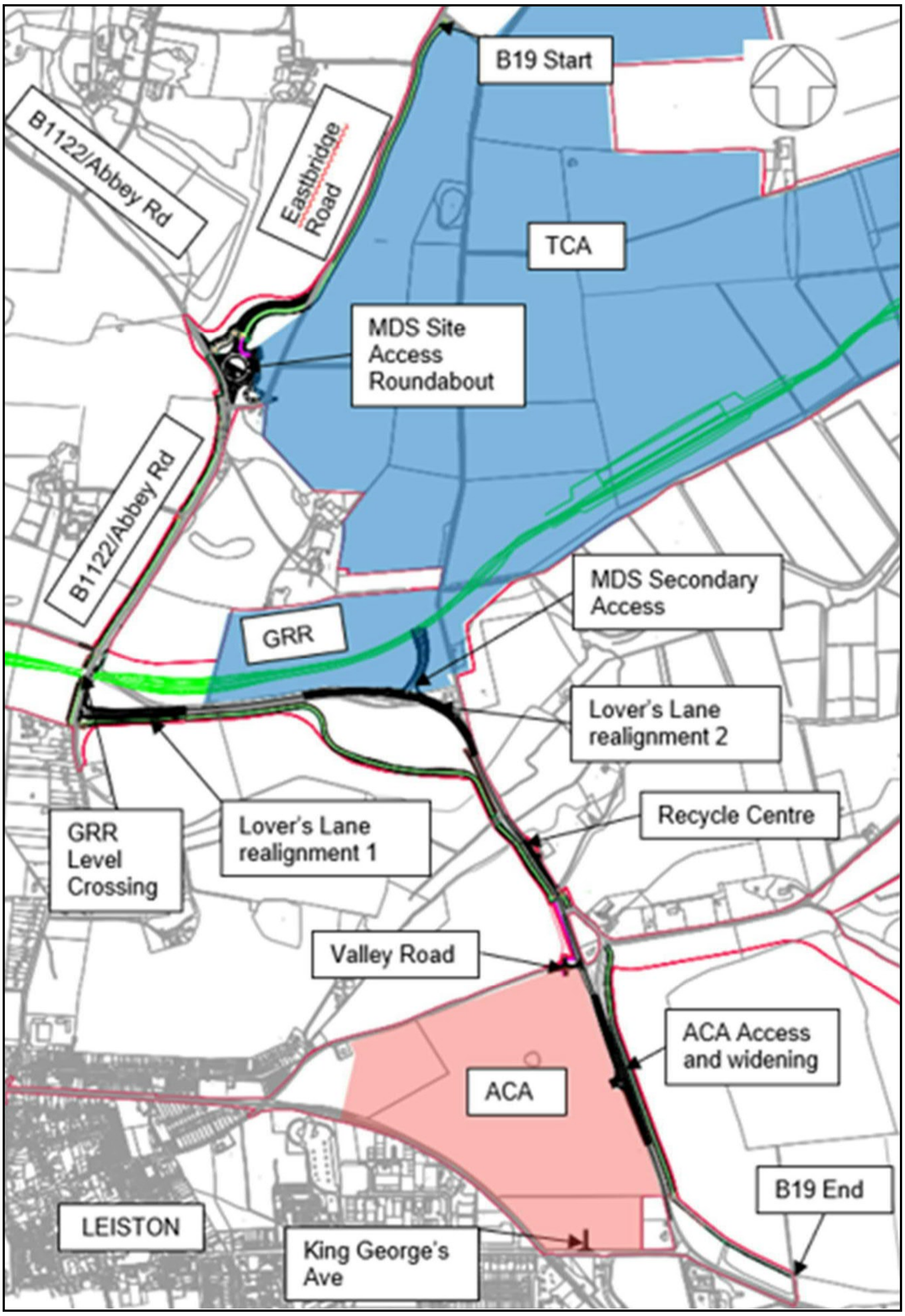
- Land East of Eastlands Industrial Estate (LEEIE) also known as Ancillary Construction Area (ACA) and Reclamation Centre site entrance accesses

- Bridleway 19

- ACA and Water Management Zone 6 (WMZ6) surface water outfalls to Leiston Drain

1.1.7 The locations of the various works are shown in **Plate 1**.

**Plate 1: Location of AD6 Works**




1.1.8 The AD6 highways and bridleway will be designed to (SCC's) adoptable standards **(Ref.1, 2, 3 and 4).**

1.1.9 AD6 will generate highway and bridleway surface water runoff which will require to be removed, treated as necessary and disposed by infiltration to ground. Where infiltration is not viable, runoff will be discharged to watercourse at attenuated greenfield runoff rates.

1.1.10 AD6 will require the diversion and modification of the upstream section of the Leiston Drain watercourse at B1122 Abbey Road.

1.1.11 AD6 Bridleway 19 will cross the Leiston Drain and its associated flood plain immediately upstream and to the west of Lovers Lane. Leiston Drain will need to be accommodated.

# 2 PURPOSE

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

- The effective removal of surface water runoff from the proposed AD6 highway, Bridleway 19 and its disposal;
- The Bridleway 19 crossing of the Leiston Drain and its associated floodplain; and
- The diversion of Leiston Drain to accommodate the railway crossing at Abbey Road.

2.1.2 This drainage strategy was developed in consultation with drainage regulators and local authorities, including SCC and the Environment Agency (EA). A number of workshops were held and the observations/requirements of drainage regulators were incorporated in the strategy. The proposed drainage infrastructure was described in the concept drainage design submitted as part of the Application. This concept design was based on data and information available at that time.

2.1.3 The purpose of this technical note is to provide details of data which validate the **Outline Drainage Strategy** [REP2-033] and subsequent **Drainage Strategy** (submitted at Deadline 7), a description of how the proposed concept drainage infrastructure is developing and evolving and to demonstrate that it continues to provide for the effective and satisfactory drainage of the AD6 road modification and associated bridleway 19, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution. This technical note takes account of new infiltration data that has become available and site inspection. It provides additional information and responses to points raised by SCC following their review during the DCO Examination Stage and submitted to SZC on 4 January 2022.

2.1.4 This technical note is updated at revision 02 to address comments raised by SCC following their review of revision 01. These are shown in Appendix F

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

# 3 DESCRIPTION OF DCO DRAINAGE DESIGN STRATEGY

3.1.1 The AD6 concept drainage at DCO stage was developed by SZC Co.

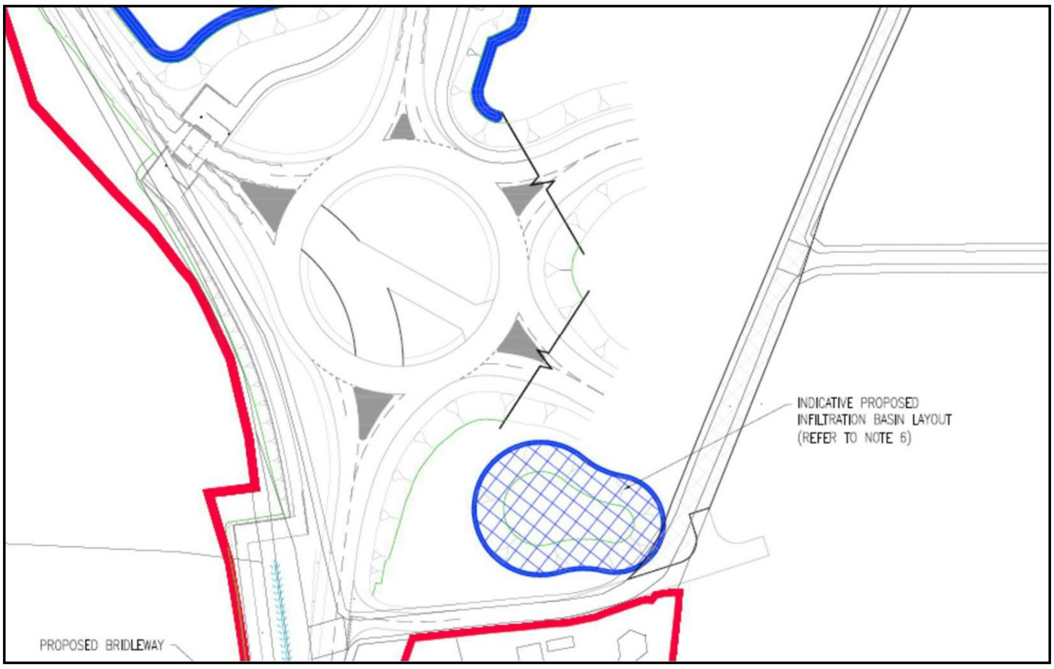
3.1.2 Proposals were developed for both highway and bridleway drainage together with the Leiston Drain watercourse crossing at Lovers Lane and its diversion at Abbey Road.

3.1.3 Subject to achievable infiltration rates, all surface water generated by roads would discharge to ground. Surface water runoff would be removed by “over the edge” discharge into swales and filter drains where flows would infiltrate to ground.

3.1.4 The bridleway would also discharge runoff to ground, both directly through its semi-permeable surface and by “over the edge” discharge into swales and filter drains where flows subsequently infiltrate to ground.

3.1.5 Traditional drainage with surface outlets, gullies, combined kerb drains (CKDs) etc would be provided at the Main Site Access (MSA) roundabout. These would discharge into carrier drains for removal. The carrier drains would discharge into an infiltration basin located to the south of the roundabout as shown in **Plate 2**.

### Plate 2: MSA Roundabout Proposed Infiltration Basin

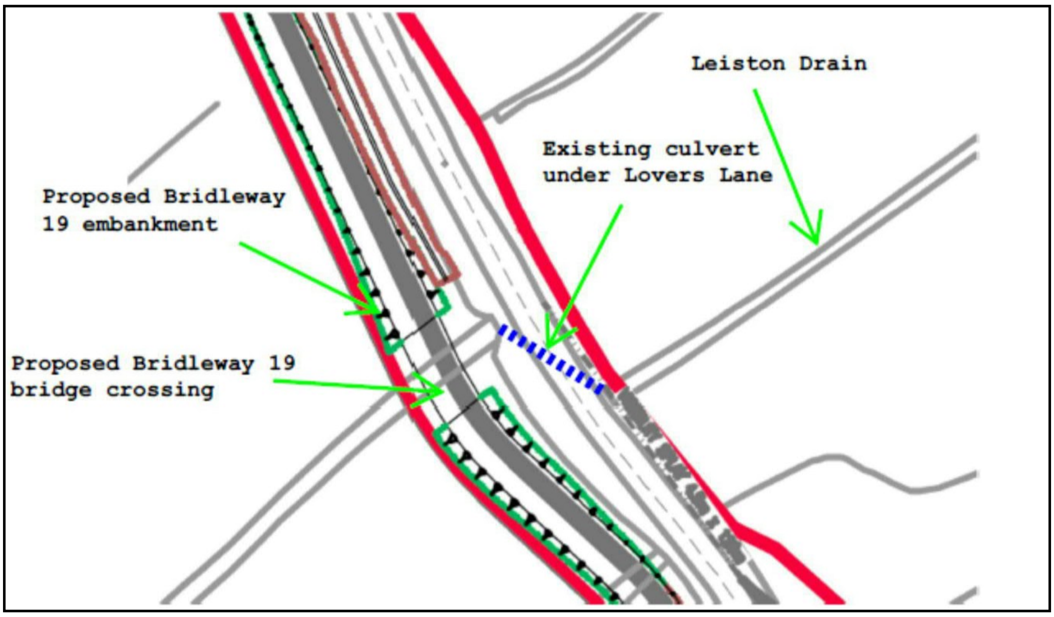



3.1.6 The basic design proposal for the bridleway crossing of the Leiston Drain and its floodplain is with an embankment and bridge. The embankment/bridge would run parallel but separate to the adjacent Lovers Lane embankment and existing Leiston Drain twin 750 mm culverts. The existing culverts would be retained in unchanged state.

3.1.7 The proposals were not evaluated by hydraulic modelling at this stage of design.

3.1.8 The proposed layout of the Bridleway 19 crossing of the Leiston Drain is shown in **Plate 3**.

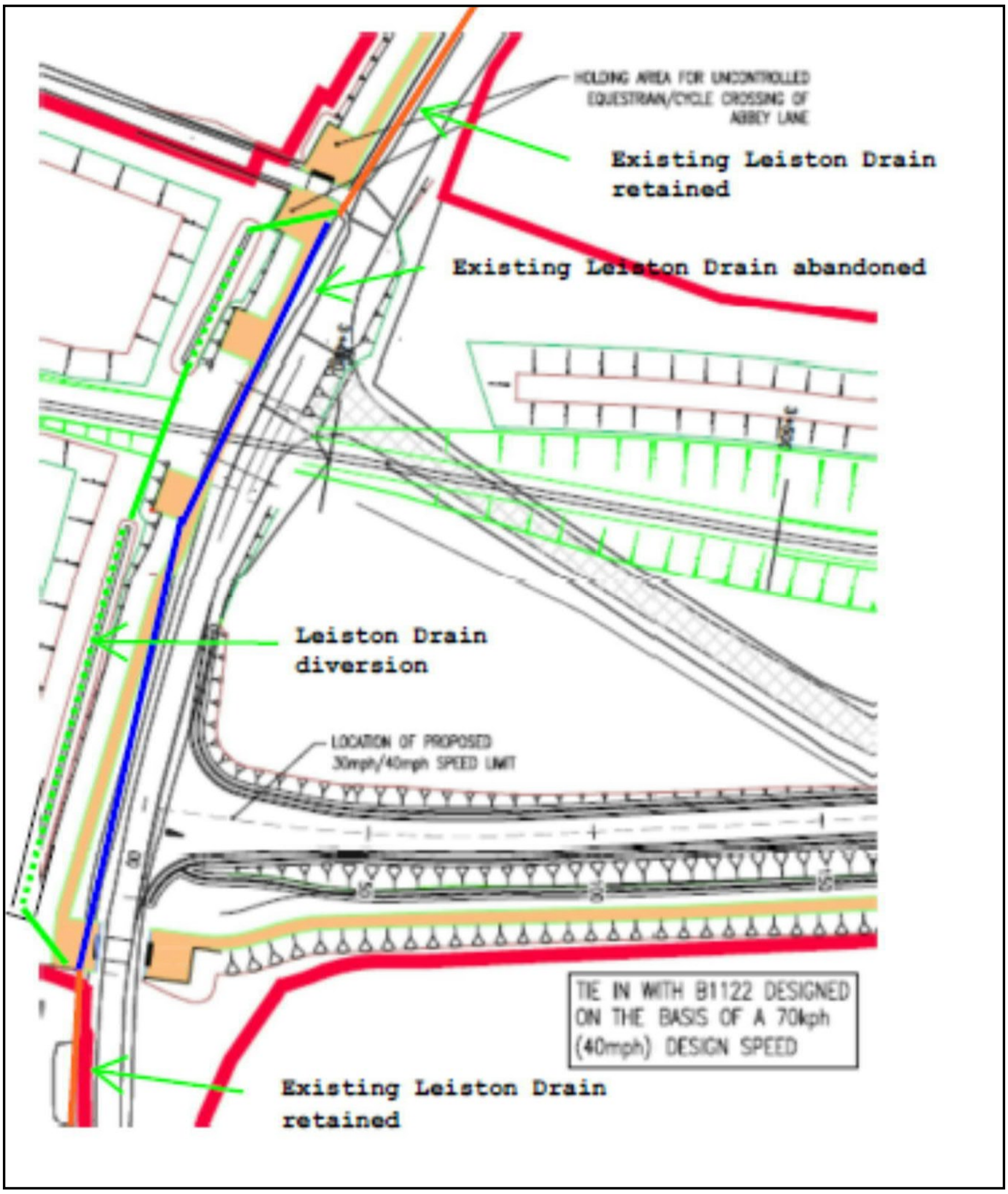
**Plate 3: Proposed Bridleway 19 Leiston Drain Crossing**




3.1.9 The upper reaches of the Leiston Drain require diverting between the existing culvert located beneath the drive of 105 Abbey Road immediately south of AD6 works and the existing culvert crossing located beneath Abbey Lane. The diversion is required in order to accommodate Bridleway 19. Where possible the diversion is in open channel, however, it is in culvert beneath the bridleway and the railway crossing.

3.1.10 The proposed Leiston Drain watercourse diversion layout is shown in **Plate 4**.

**Plate 4: Proposed Leiston Drain Watercourse Diversion**




# 4 ADDITIONAL INPUT DATA

4.1.1 The preliminary drainage design has been developed based on the DCO drainage design strategy but modified to take account of data which has become available since DCO submission.

4.1.2 The new data which informs the design is listed below:

- Drone Topographic Survey of AD6 route
- Topographic survey of watercourses within and adjacent to red line boundary
- Aerial view from drone flyover
- Ground Investigation and infiltration testing
- GPR survey
- Additional traditional topographic survey of critical locations
- Site visits and inspection of the full length of the AD6 route on 24 February and 5 August 2021
- Highways England Water Risk Assessment Tool (HEWRAT) results

4.1.3 The design development has also evolved through the Design Review meetings held with SCC and the EA. Comments and requirements confirmed by SCC and the EA have been recorded in minutes of the review meetings and taken into account.

4.1.4 SCC have advised SZC that the design should take account of the conclusions contained in the Leiston **Surface Water Management Plan (SWMP) Update** dated 2017.

4.1.5 The final draft preliminary design will be submitted to SCC as the intended adopting Highway Authority, to SCC as Lead Local Flood Authority, and the EA. Any final comments can be addressed in the preliminary design drawings and reports, prior to issue as final design.

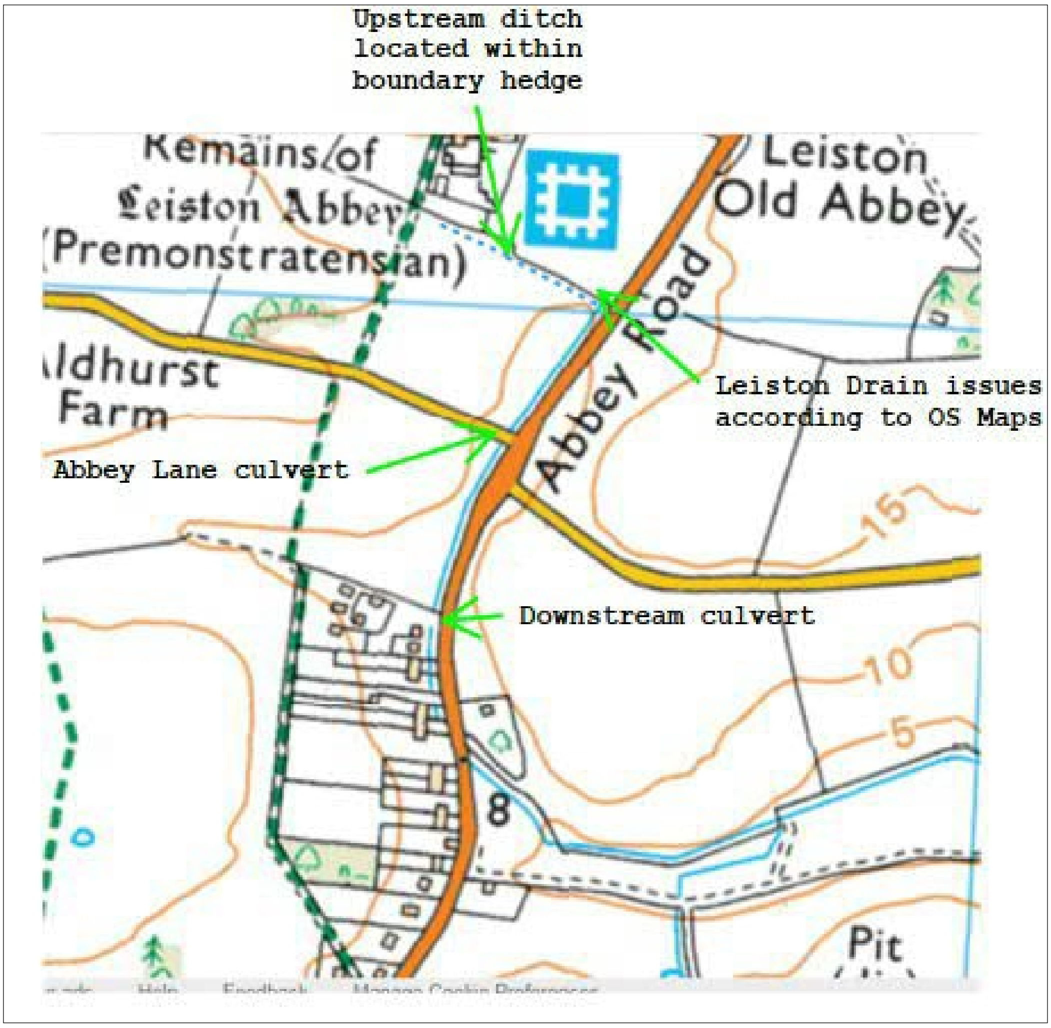
# 5 EXISTING WATERCOURSE ARRANGEMENTS

5.1.1 Following the site visits and review of survey data, details of existing highway drainage arrangements have been determined and are described below. The existing arrangements have been considered as part of Preliminary Design.

5.1.2 Leiston Drain is shown on OS based mapping to issue at a field boundary upstream of Abbey Lane. However, as a result of site inspection it can be confirmed that it continues upstream hidden within the boundary hedge. At the boundary with the B1122 Abbey Road, it turns and runs immediately behind the boundary hedge south to Abbey Lane. There is a 750 mm culvert crossing beneath Abbey Lane. The watercourse then continues to run in parallel to Abbey Road before passing under the access drive to 105 Abbey Road in a 750 mm culvert.

5.1.3 The Leiston Drain watercourse upper reaches at Abbey Road general layout is shown in **Plate 5**.

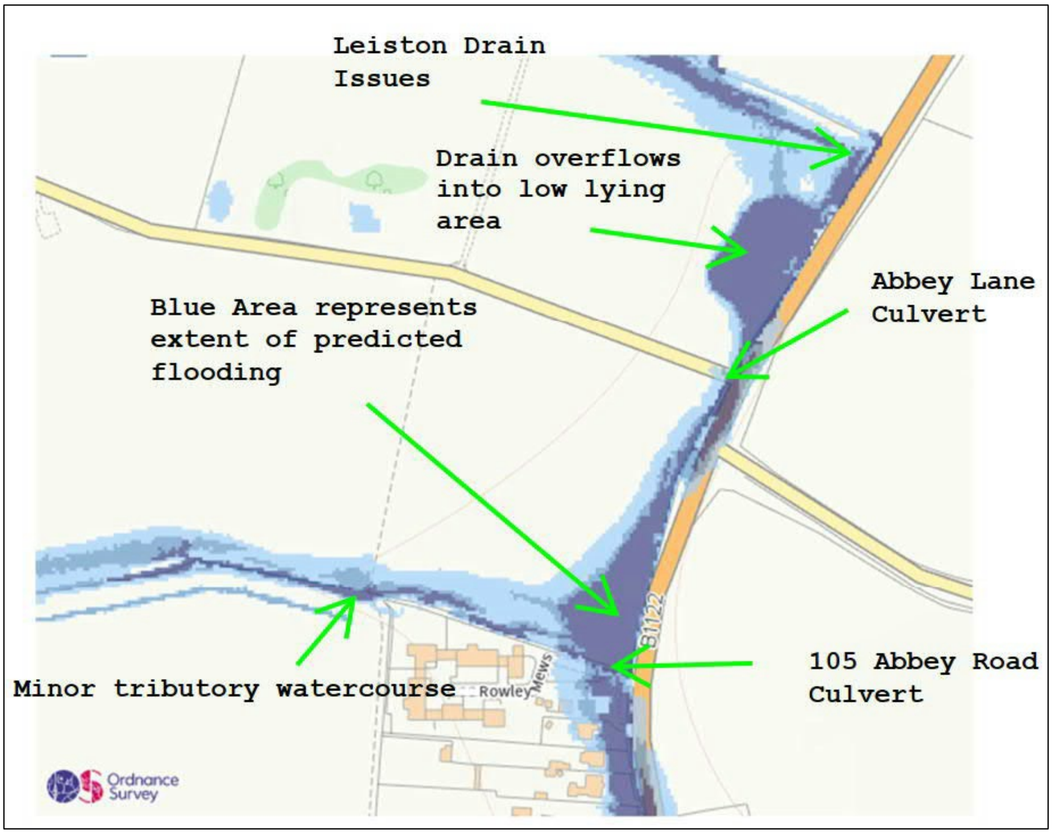
### Plate 5: Leiston Drain watercourse upper reaches




5.1.4 The Leiston Drain is currently heavily silted adjacent Abbey Road including through both the Abbey Lane culvert and the culvert beneath the drive of 105 Abbey Road. This silted state continues downstream where it runs within property gardens and through other access drive culverts.

5.1.5 SCC advise that there are currently flooding problems with this section of Leiston Drain adjacent Abbey Road. The EA Surface Water flood map shows the predicted flooding extent as in **Plate 6**.

### Plate 6: Leiston Drain predicted extent of flooding at Abbey Road




5.1.6 A small tributary watercourse not shown on OS maps was identified during the site survey and is located along the boundary of 105 Abbey Road. Its position can be seen in **Plate 6**.

5.1.7 A desktop FEH catchment assessment to estimate flow rates in Leiston Drain at its point of issue and the tributary watercourse at its confluence with Leiston Drain has been undertaken the rates are shown in **Table 1.** The capacity of the 750 mm diameter Abbey Lane culvert and Access Drive culvert have also been determined and are shown.

**Table 1: Leiston Drain watercourse flow rates and culvert capacity**

| Catchment | 10 year return period rainfall | 20 year return period rainfall | 50 year return period rainfall |
|---|---|---|---|
| Leiston Drain | 0.46 $m^3/s$ | 0.55 $m^3/s$ | 0.70 $m^3/s$ |
| Tributary | 0.19 $m^3/s$ | 0.24 $m^3/s$ | 0.30 $m^3/s$ |
| Outfall | 0.65 $m^3/s$ | 0.79 $m^3/s$ | 1.00 $m^3/s$ |
| Abbey Lane culvert capacity 2.02 $m^3/s$ /s | | | |
| Abbey Road access drive culvert capacity 0.55 $m^3/s$ | | | |

5.1.8 From the above table it can be seen that the primary cause of flooding is a lack of capacity in the downstream outfall culvert as the calculated capacity at 0.55 m3/s is less than the flow rates ranging from 0.65 to 1.0 m3/s. This is exacerbated by the silting up of the watercourse.

5.1.9 During the August 2021 site visit it was noted that there is a low-lying part of the field upstream of Abbey Lane which contained a pool of water. It is apparent that during more extreme rainfall the Leiston Drain overflows and fills this low lying area as is shown in **Plate 6**. The effect of this is that the desktop calculated flow rate for the combined flow at the Abbey Road access drive culvert is overstated but it remains the case that the observed flooding is due to the culverts lack of adequate capacity.

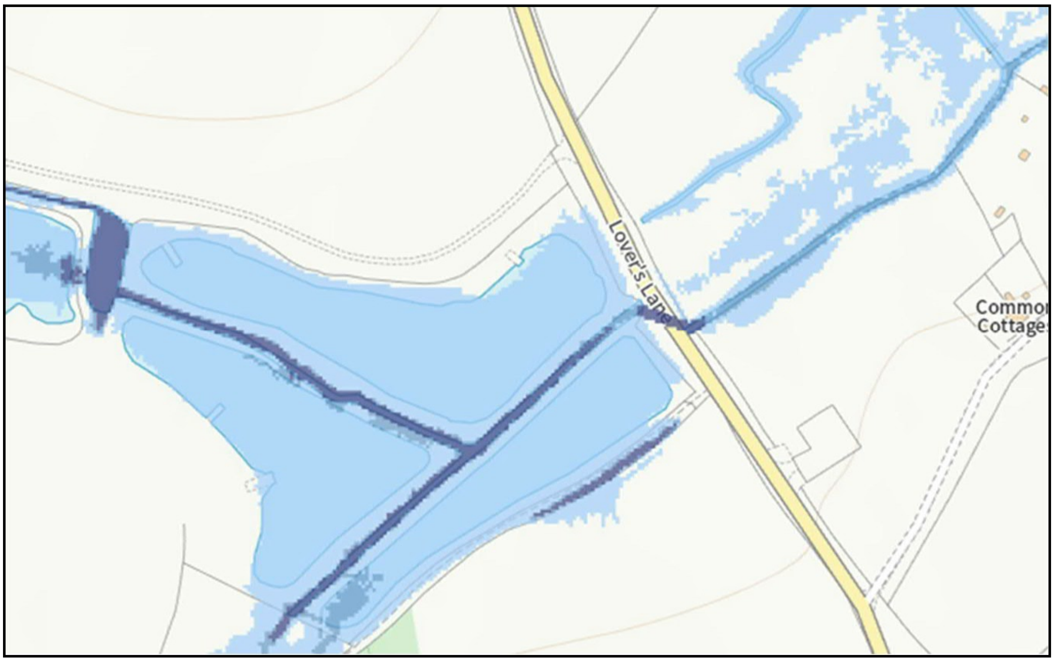
5.1.10 Based on observation, both Leiston Drain and its tributary watercourse are normally dry other than during or immediately after rainfall.

5.1.11 The Leiston Drain continues downstream from Abbey Road in an easterly direction in open channel running in open country through Aldhurst Farm to Lovers Lane. It crosses beneath Lovers Lane through twin 750 mm culverts.

5.1.12 The hydraulic performance of the twin culverts and flood extent in this area has been assessed as part of the Main Development Site Flood Risk Assessment undertaken by consultants for SZC.

5.1.13 As shown in **Plate 7** the twin culverts and the Lovers Lane embankment create a constriction causing a temporary impoundment of flood water upstream to the west of Lovers Lane. Based on the Main Site Flood Risk Assessment undertaken for SZC, the predicted maximum upstream flood water level immediately west of Lovers Lane is 2.0 mAOD during a 1 in 100 year flood including 35 % allowance for climate change. As Lovers Lane is on an embankment over the twin culverts with a minimum surface level of 3.2 mAOD which is above the predicted flood level there is no predicted risk of fluvial flooding of the road from the Leiston Drain.

**Plate 7: Leiston Drain predicted extent of flooding at Lovers Lane**




5.1.14 The Main Development Site Flood Risk Assessment determines fluvial (river) flood risk. SCC advise that a pluvial (surface) flooding assessment for the Lovers Lane area in vicinity of the culverts is contained in the Leiston SWMP. This concludes that the 1 in 100 year return period plus 40% climate change pluvial flood level reaches the surface of Lovers Lane. Since the pluvial flood level is greater than the fluvial, SCC expect this level to be used in design of the Bridleway 19 Leiston Drain crossing.

# 6 EXISTING HIGHWAY DRAINAGE ARRANGEMENTS

6.1.1 Following the site visit and review of survey data, details of existing highway drainage arrangements have been determined and are described below. The existing arrangements have been considered as part of Preliminary Design.

6.1.2 The existing section of the B1122 Abbey Road within the extent of the future MSA roundabout has no formal highway drainage other than at its junction with Eastbridge Road. It is assumed that highway surface runoff discharges “over the edge “to verges.

6.1.3 At Eastbridge Road junction there are kerb outlets which remove surface water highway runoff and discharge into swales. During previous site visits it has been noted that the swales were full of water and infiltration testing within the vicinity indicates that discharge by infiltration alone is not viable.

It is concluded that the swales will store runoff, and whilst there may be some infiltration to ground, removal will be very slow.

6.1.4 Other than on the first section of Eastbridge Road, from the junction with B1122 Abbey Road to the bend located some 70m to the east there is no formal highway drainage. It is assumed that highway surface runoff discharges “over the edge “to verges.

6.1.5 To the south of the B1122 Abbey Road/Eastbridge Road junction there is no formal highway drainage upstream of the point where Leiston Drain issues. South of this point the Abbey Road northbound carriageway is drained via gullies/kerb outlets into the Leiston Drain.

6.1.6 Lovers Lane has a steep falling longitudinal gradient downwards towards its junction with Abbey Road. There are highway gullies located on either side of the road immediately short of the junction which intercepts highway surface water runoff from Lovers Lane. The gullies have outfall pipes across and beneath Abbey Road which discharge into the Leiston Drain watercourse.

6.1.7 To the south of the junction with Lovers Lane the Abbey Road southbound carriageway has a fall to the north such that highway surface water runoff discharges into the Lovers Lane Road gulley. Gullies on the northbound carriageway discharge directly into the Leiston Drain watercourse.

6.1.8 On Lovers Lane from the B1122 Abbey Road junction eastwards from its high point to the junction with Bridleway 19 and Kenton Hills car park there are a small number of gullies which drain the road. It is assumed that they drain to soakaways.

6.1.9 Continuing southwards from the Kenton Hills car park junction to the junction with Sandy Lane /Valley Road, Lovers Lane has a fall both from north and south to a low point at the Leiston Drain culvert. There are a limited number of road gullies which drain the road most of which appear to discharge to soakaway but nearer to the Leiston Drain discharge is to watercourse.

6.1.10 There is no surface water drainage infrastructure visible in Valley Road. The road has a fall to the junction with Lovers Lane, so highway surface water runoff flows into Lovers Lane and is collected in a downstream road gulley.

6.1.11 The GPR survey has identified a local highway pipe network which drains Lovers Lane to the south of its junction with Valley Road. This network outfalls to the north but its downstream route could not be determined. A series of gullies drain both carriageways in this area.

6.1.12 The section of Lovers Lane further south and where an access road into the ACA is to be located is drained by gullies provided on both northbound and southbound carriageways. The northbound gullies discharge across the road and share soakaways with the southbound gullies.

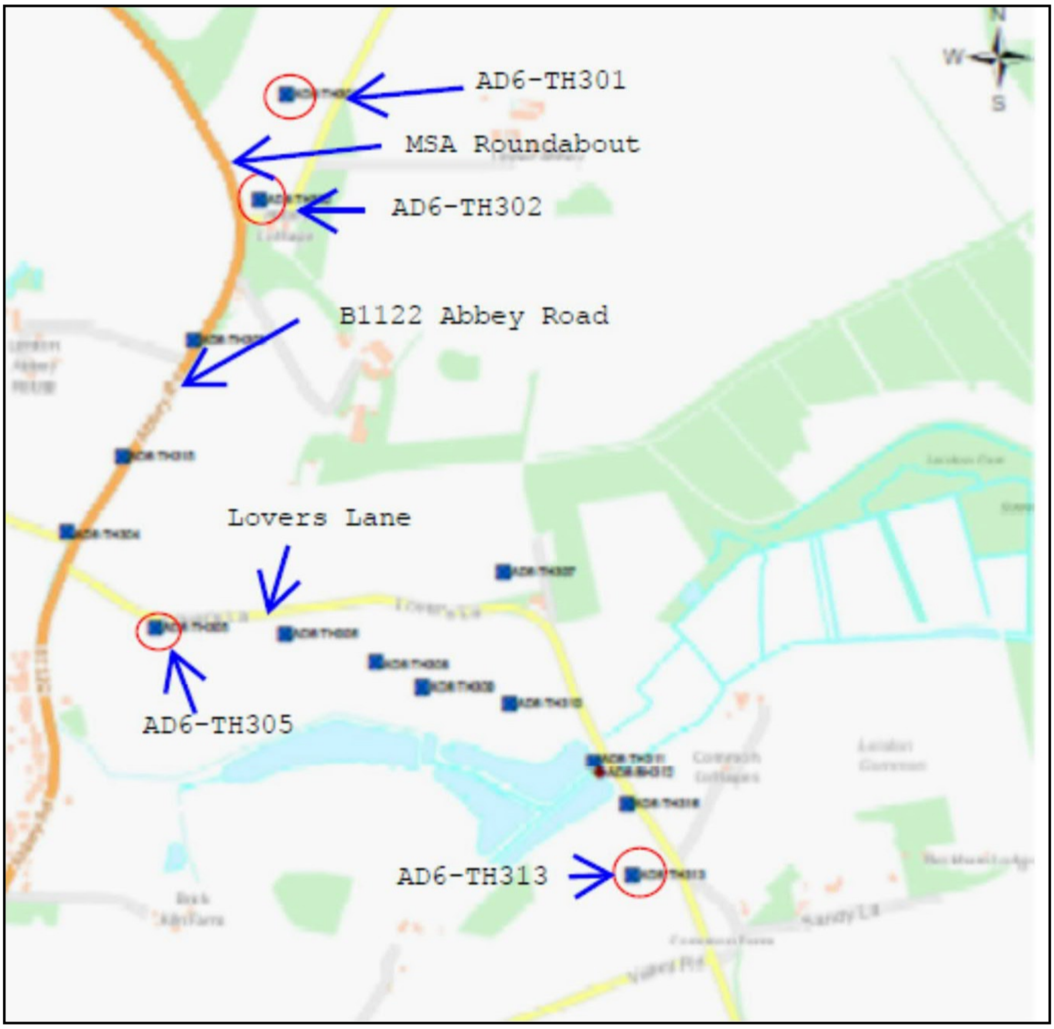
6.1.13 The eastern section of King Georges Avenue in vicinity to where an access road into the ACA is to be provided is drained by road gullies provided on both eastbound and westbound carriageways. The eastbound gullies discharge across the road to join the westbound gullies.

6.1.14 The GPR survey shows details of the highway drainage described above and it is recommended that this data is shared with SCC.

# 7 GROUND INVESTIGATION AND INFILTRATION TESTING RESULTS

7.1.1 Ground investigation and infiltration testing has been undertaken for the AD6 scheme. Infiltration testing was undertaken in accordance with BRE365 at the four locations shown in **Plate 8** and circled red.

**Plate 8: AD6 Infiltration Rate Test Locations**




7.1.2 Infiltration testing results are shown in **Table 2**.

**Table 2: AD6 Site Infiltration Test Trial Hole Results**

| Location | Test 1(m/s) | Test 2(m/s) | Test 3 (m/s) |
|---|---|---|---|
| TH301 | $7.86 \times 10^{-6}$ | $6.01 \times 10^{-6}$ | - |
| TH302 | Negligible | Negligible | Negligible |
| TH305 | $1.93 \times 10^{-5}$ | $1.56 \times 10^{-5}$ | $1.04 \times 10^{-5}$ |
| TH313 | $1.18 \times 10^{-5}$ | $1.20 \times 10^{-5}$ | $8.71 \times 10^{-6}$ |

7.1.3 These results demonstrate that in the areas of TH305 and TH313, disposal of surface water runoff by infiltration is viable. SCC consider that an infiltration rate in excess of 1.4 x 10-6 is viable for infiltration to ground. However, infiltration in the area of TH302 is confirmed to be unviable. Full details of borehole logs and infiltration measurements are shown in **Appendix A**. In the case of TH301, there are indications that infiltration may be possible. However, the results are incomplete and hence not BRE365 compliant. If reliance is placed on removal of runoff by infiltration, at this location further infiltration testing should be undertaken.

7.1.4 Infiltration testing at the site of WMZ6, undertaken as part of Main Development Site geotechnical investigation, confirmed an infiltration rate of 5.58X10-6 m/s.

7.1.5 Infiltration testing near to the diversion route for Eastbridge Road, undertaken as part of Main Development Site geotechnical investigation confirmed an infiltration rate of 1.62 X 10-6 m/s

# 8 UPDATED SURFACE WATER DRAINAGE DESIGN STRATEGY – HIGHWAY AND BRIDLEWAY 19 DRAINAGE

8.1.1 Based on the results of Geotechnical Investigation infiltration testing, other than at the locations listed below the proposed drainage strategy remains as removal of highway and bridleway surface water runoff and its disposal by infiltration.

- MSA roundabout
- Abbey Road to the south of Abbey LaneLovers Lane between Secondary Site Access Road and Valley Road
- ACA main access off Lovers Lane

8.1.2 The hydraulic design of drainage infrastructure uses the local infiltration rates to determine the rate of removal of runoff by infiltration from filter drains and the extent of additional underground storage required in the form of soakaway manholes pending full infiltration.

8.1.3 As agreed with SCC and the EA prior to DCO submission, the environmental impact of discharging highway runoff is to be assessed using the Highways England Water Risk Assessment Tool (HEWRAT) **(Ref. 3)** methodology. The assessment results confirm that a SuDS management train with the combination of swales, filter drains, attenuation and infiltration basins, for AD6 discharges to ground and at attenuated rates to watercourses are low risk and therefore acceptable. SCC has indicated that they will wish to review the management train at detailed design stage and may wish to see provision of additional treatment stages. A copy of the assessment is shown in **Appendix B**.

8.1.4 In addition to the AD6 drainage infrastructure proposed at DCO submission, the need for surface water outfall drains from the ACA site and WMZ6 have been identified. These are proposed to discharge to Leiston Drain and will only operate during the construction phase. These additional drainage requirements for surface water drains from the ACA site and WMZ6 have been incorporated in the AD6 drainage design and this has been agreed in principle with SCC. SCC agreement is subject to the design ensuring that self-cleansing velocities will continue to be provided when flow rates reduce after the removal of ACA and WMZ6 flows following completion of construction and commissioning of the SZC power station.

### *Bridleway 19 and Eastbridge Road Drainage*

8.1.5 The section of Eastbridge Road that is diverted, and due to levels cannot drain by gravity back to the MSA drainage, will be drained “over the edge” with highway surface water runoff discharging into a swale where it will be stored and gradually infiltrate to ground. The infiltration rate at this location is of the order of 1.62 X 10-6 m/s. Given the relatively low proven infiltration rate, additional storage will be provided by oversizing the swale.

8.1.6 Bridleway 19 which runs parallel to Eastbridge Road behind a boundary hedge from the MSA roundabout and terminates at the Upper Abbey access track will drain partly by direct infiltration though the semi-permeable surface and partly “over the edge” to a nominally lower grassed section. A filter drain will be located below the channel to rapidly remove runoff from the surface pending infiltration. Given the relatively low proven infiltration rate, additional storage is provided with the use of soakaway manhole chambers at intervals along the bridleway route.

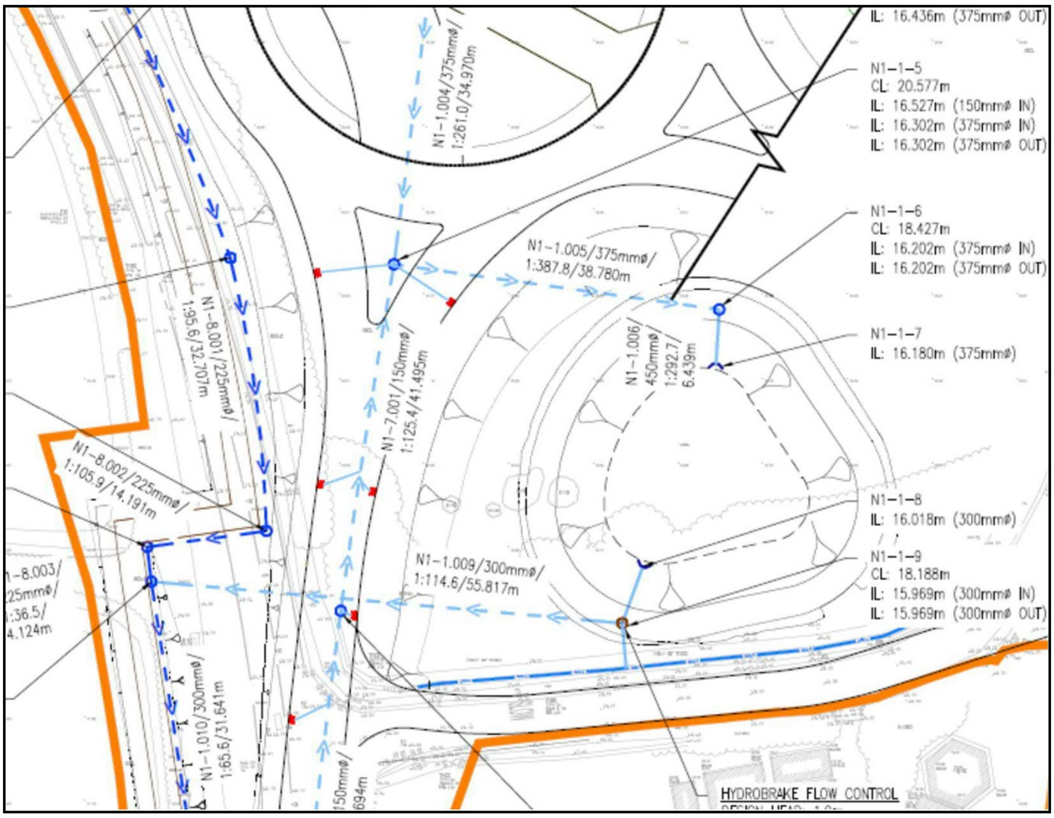
### *MSA Roundabout Drainage*

8.1.7 The drainage arrangements for the MSA roundabout remain as assumed for DCO submission, with the exception that the proposed infiltration basin located between the remaining stub section of Eastbridge Road and the roundabout is changed to an attenuation basin.

8.1.8 The existing swale on the north side of the retained section of Eastbridge Road will be connected into the proposed basin which will effectively drain this road. This retained road will provide access to the attenuation basin.

8.1.9 The attenuation basin layout is shown in **Plate 9** and its predicted hydraulic performance is shown in **Table 3**. The performance data is taken from the hydraulic modelling results. Full details are shown in **Appendix C.** The network layout with model labels is shown in **Appendix D.**

### Plate 9: MSA Roundabout Attenuation Basin




### Table 3: MSA Roundabout Attenuation Basin Predicted Hydraulic Performance

| Basin Dimensions |
|---|
| Invert level 16.018 mAOD |
| Top of bank level 18.118 mAOD |
| Basin Depth 1.8 m allowing for freeboard |
| Freeboard 0.3 m |
| Storage volume below top of bank 1,476 m3 |
| **Hydraulic performance – predicted maximum water level** |
| 1 in 1 year 16.388 mAOD |

| Basin Dimensions |
|---|
| 1 in 5 year 16.486 mAOD |
| 1 in 100 year plus 40% cc 17.499 mAOD |

8.1.10 The flow rate from the attenuation basin is restricted to a maximum of 5 l/s in accordance with SCC SuDS policy. However, a high level overflow will be provided to ensure exceedance rainfall does not cause the basin to overflow.

8.1.11 The attenuation basin outfall drain passes west and under the MSA roundabout southern arm and connects to the proposed drainage system running parallel with Bridleway 19/Abbey Road and continues to the south towards the head of the Leiston drain to the north of the junction with Abbey Lane.

8.1.12 The section of Bridleway 19 to the west of the MSA roundabout drains south and discharges into the same outfall manhole as the connection from the attenuation basin.

*Bridleway 19 and Abbey Road Drainage to Abbey Lane*

8.1.13 The outfall drain which collects controlled flow from the attenuation basin and uncontrolled flow from the bridleway, runs in parallel to Abbey Road behind the highway boundary hedge. It collects highway surface water runoff from the MSA roundabout southern arm.

8.1.14 The outfall is designed as a filter drain which potentially allows flow to leave the pipe and infiltrate to ground.

8.1.15 The outfall drain is located between the highway boundary hedge and the bridleway which allows it to collect runoff from the bridleway. The bridleway will drain partly by direct infiltration though the semi-permeable surface and partly “over the edge” to a nominally lower grassed section located above the outfall drain.

8.1.16 At the point where it cross’s the access road to Leiston Abbey, there is a second flow control chamber which limits discharge to 5 l/s.

8.1.17 The outfall drain discharges into the existing Leiston Drain to the north of Abbey Lane. Following the August 2021 site inspection, the need for a culvert for the bridleway crossing of the Leiston Drain has been identified. This will be included in the detailed design.

8.1.18 To the south of this point over the edge runoff from the bridleway discharges into Leiston Drain.

8.1.19 There are no changes to existing Abbey Road highway drainage between the MSA roundabout southern arm tie in point and the modified Abbey Road tie in point just north of Abbey Lane. The existing gullies will continue to discharge into the Leiston Drain.

*Bridleway 19 and Abbey Road Drainage between Abbey Lane and 105 Abbey Road*

8.1.20 Whilst the alignment of Abbey Road remains unchanged its surface levels are modified and raised at the point of the railway crossing to 9.5 mAOD. This creates a fall from the railway crossing both to the north and south. The Leiston Drain is diverted to the west. As a result, the existing highway drainage is removed and replaced.

8.1.21 Highway surface water runoff will flow both north and south from the railway crossing and be collected by highway gullies. These will discharge the runoff into the Leiston Drain.

8.1.22 The existing Abbey Road junction with Lovers Lane is relocated southwards in order to accommodate the railway. As a result, the current discharge of highway surface water runoff from Lovers Lane is removed and there is a reduction in discharge to the Leiston Drain watercourse as the new junction runoff will be directed to the proposed infiltration basin.

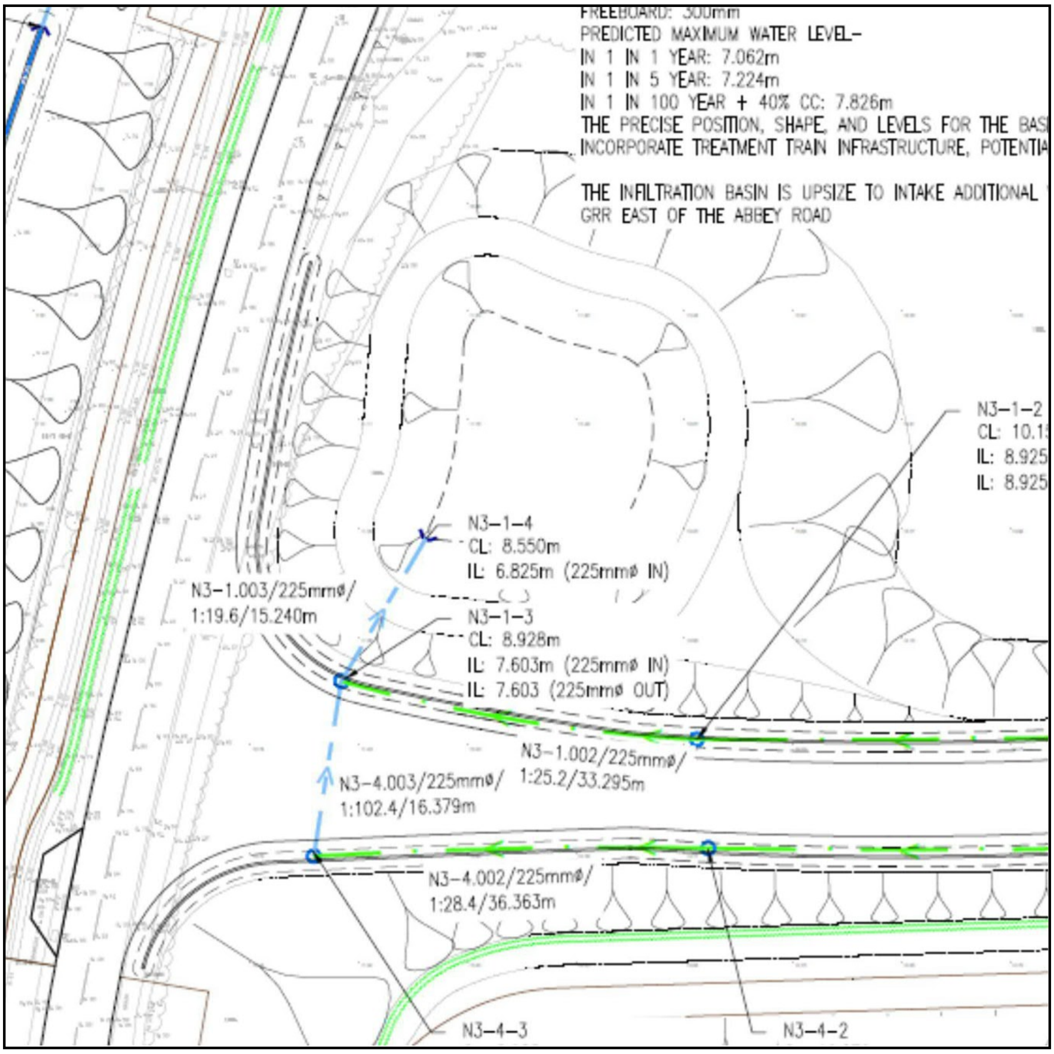
8.1.23 Bridleway 19 runs in parallel to Abbey Road behind a highway boundary hedge. The bridleway will drain partly by direct infiltration though the semi-permeable surface and partly “over the edge” into the Leiston Drain.

*Lovers Lane Diversion and new junction with Abbey Road including Bridleway 19*

8.1.24 Lovers Lane is diverted south of its existing junction with Abbey Road in order to accommodate the railway. It is noted that since the existing road discharges highway runoff into Leiston Drain, the effect of the diversion is to remove this discharge and its contribution to flow in the Leiston Drain.

8.1.25 The length of diverted road drains west towards Abbey Road. This section of road is drained “over the edge” into swales with filter drains on both sides of the road. These fall to a point just short of the Abbey Road junction. The westbound carriageway swale/filter drain is piped beneath Lovers Lane Road to join the eastbound carriageway swale/filter drain. There is a common outfall to an infiltration basin shown in **Plate 10** and its predicted hydraulic performance is shown in **Table 4**. The performance data is taken from the hydraulic modelling results. Full details are shown in **Appendix C.** The network layout with model labels is shown in **Appendix D.** The performance data shown in **Table 4** supersedes that contained in **Plate 10**. data

### Plate 10: Lovers Lane Infiltration Basin at Abbey Road/Lovers Lane Junction




### Table 4: Lovers Lane Infiltration Basin Predicted Hydraulic Performance

| Basin Dimensions |
|---|
| Invert level 6.825 mAOD |
| Top of bank level 8.550 mAOD |
| Basin Depth 1.725 m |
| Freeboard 0.3 m |

| Basin Dimensions |
|---|
| Storage volume top of bank 383 m3 |
| **Hydraulic performance – predicted maximum water level** |
| 1 in 1 year 7.121 mAOD |
| 1 in 5 year 7.223 mAOD |
| 1 in 100 year plus 40% cc 7.906 mAOD |

8.1.26 Whilst the hydraulic model calculations and data contained in Table 4 are for highway runoff from Lovers Lane and Bridleway 19, it is necessary to upsize the infiltration basin to receive runoff from a section of the new railway between Abbey Road and the Secondary Site Access Road located to the north. The required additional volume is 463 m3 and this is included in the basin size shown in **Plate 10**. The 463 m3 is based on an infiltration rate of 1.06 X $10^{-4}$ m/s which was measured by SZC during earlier infiltration testing. This is higher than the 1.04 x $10^{-5}$ measured at the infiltration basin but it is not accepted by SCC as it is not obtained in accordance with BRE365.

8.1.27 It is intended to undertake further infiltration testing and the lower value will be used in detailed design.

8.1.28 The railway inflow will be removed when the railway is decommissioned but it is proposed that the basin will not be modified and reduced in size at that stage since it will provide additional flood risk protection.

8.1.29 East of the Abbey Road/Lovers Lane junction the Bridleway 19 is located south of the westbound Lovers Lane carriageway. It will drain partly by direct infiltration though the semi-permeable surface and partly “over the edge” with runoff flowing into the roadside swale.

*Lovers Lane Diversion and Secondary Site Access junction, WMZ6 Outfall and including Bridleway 19*

8.1.30 A section of Lovers Lane located between the Lovers Lane/Abbey Road diversion to the west and the Lovers Lane/Secondary Site Access to the east, will remain in place and in unchanged state. No change to the existing limited highway drainage is proposed for this section of Lover Lane. Since there is a longfall in both directions and highway surface water runoff that is not removed by the existing gullies will flow to the proposed diversion swales for removal and disposal by infiltration.

8.1.31 The gradient of the Secondary Site Access Road will fall away from its junction with Lovers Lane so there will be no overland flow from the private road onto Lovers Lane. The junction will be removed on completion of SZC construction.

8.1.32 The section of Bridleway 19 that runs parallel to this section of Lovers Lane will drain partly by direct infiltration though the semi-permeable surface and partly “over the edge” with runoff into a nominally lower grassed section with underlying filter drain. The filter drain has a fall to the east and ultimately runoff that is not removed by local infiltration discharges into the Leiston Drain watercourse.

8.1.33 Within the Lovers Lane diversion area, the bridleway follows a route away from the road and runs to the south of WMZ6 before running parallel with the road corridor to the east. When SZC construction is complete, WMZ6 will be removed as part of reinstatement and the bridleway will be moved to its final position alongside the road.

8.1.34 The Lovers Lane Road diversion is drained “over the edge” into swales with filter drains initially on both sides of the road to the west but only on the westbound carriageway on the bend next to WMZ6. Drainage is not required to the eastbound carriageway because the road crossfall is to the westbound side of the road.

8.1.35 The Lovers Lane Road westbound swale/filter drain discharges to a common outfall with Bridleway 19, to the south of WMZ6.

8.1.36 WMZ6 is an infiltration basin that receives surface water runoff from within the SZC construction site and the secondary site access road. The runoff is treated and infiltrates to ground. During design it has been established that there is insufficient space to provide a storage volume to contain the runoff from a 1 in 100 year return period rainfall event plus allowance for climate change, pending infiltration. As a result, a high level overflow is provided to the Leiston Drain that will only operate during a the 1 in 100 year event. This will discharge at a controlled flow rate of 47 l/s.

8.1.37 The WMZ6 infiltration basin overflow will discharge into the common outfall with Bridleway 19 and the road to the Leiston Drain watercourse.

8.1.38 The common outfall will be in the form of a filter drain. This will encourage infiltration to ground whilst allowing runoff that does not infiltrate to discharge into Leiston Drain. The filter drain is located adjacent to the bridleway and under the nominally lower grassed channel which will collect bridleway runoff that does not directly infiltrate to ground through its semi - permeable surface.

8.1.39 The outfall will terminate in a headwall adjacent to the point where the Bridleway 19 boardwalk bridge commences. From this point it will continue as an open channel to point of discharge into Leiston Drain.

8.1.40 The section of Lovers Lane Road between the diverted section and the low point at the crossing of the Leiston Drain remains in an unaltered state and no changes are proposed to its existing drainage arrangements. Runoff will continue to be by "over the edge" to adjacent land at the toe of the Lovers lane embankment.

*Lovers Lane junction with Valley Road, ACA Outfall and Bridleway 19*

8.1.41 During design it has been established that given the extent of development within the ACA site and the low infiltration rates that have been established by testing, it is not possible to remove all surface water runoff generated, within the site by infiltration to ground. Accordingly, surface water runoff will be removed by discharge offsite to watercourse at controlled rates. Two outfalls are proposed. The main ACA drainage outfall is via Sandy Lane which ultimately discharges at controlled attenuated rates to the Leiston Drain some distance downstream. The Sandy Lane outfall is not considered further within this report as it is not within the AD6 highway design scope area and there is no interface.

8.1.42 The second outfall is for surface water runoff that is generated in a low lying part of the ACA that can't discharge to the main Sandy Lane outfall by gravity. This smaller catchment drains to a pumping station where flows are pumped at a rate of 15 l/s to the outfall manhole which is within the ACA site next to the junction with Lovers Lane and Valley Road. The outfall is required to discharge into Leiston Drain. As noted in Section 8.1.3 SCC have accepted that a common outfall, which discharges runoff from roads, bridleway and ACA is acceptable provided that self -cleansing velocities are maintained once the ACA site is decommissioned.

8.1.43 The junction of Valley Road and Lovers Lane is to be modified and will include for an entrance access into the ACA. The road and entrance will be drained by gullies which will discharge into a carrier drain. The ACA outfall drain will also discharge into this carrier drain.

8.1.44 The provision of the carrier drain arrangement will therefore capture highway runoff which currently flows out of Valley Road into Lovers Lane and thus will reduce local flood risk on the existing highway drainage network.

8.1.45 The carrier drain will outfall from Valley Road through the adjacent land and then follow a route parallel to Lovers Lane adjacent to the boundary hedge before discharging into the Leiston Drain watercourse. Within the land between Valley Road and the Leiston Drain, the outfall will take the form of a filter drain which will be located beneath the nominally lower grassed section next to the bridleway thus providing dual use.

8.1.46 The provision of a filter drain will encourage infiltration to ground whilst allowing runoff that does not infiltrate to discharge into Leiston Drain.

8.1.47 The outfall will terminate in a headwall adjacent to the point where the Bridleway 19 boardwalk bridge commences. From this point it will continue as an open channel to point of discharge into Leiston Drain.

8.1.48 Other than the provision of the bridleway crossing of Lovers Lane at Sandy Way and local widening at the Leiston Recycling Centre, there are no changes to Lovers Lane that require the provision of additional drainage. There are existing gullies at the Recycling Centre entrance and if required additional gullies can be provided.

*Lovers Lane ACA entrance*

8.1.49 Lovers Lane northbound carriageway will be widened to provide the main vehicular access junction into the ACA site. The widened road will drain to a swale with underlying filter drain where highway surface water runoff will infiltrate to ground. Because the reported infiltration rates within the ACA are relatively low, in order to ensure effective removal and disposal of runoff from the highway, a high-level connection will be made to the ACA internal drainage network.

8.1.50 Since the ACA is to be restored to current greenfield state on completion of SZC construction, it is assumed that, whilst the swale will drain the public highway, it will not be operated and maintained by SCC as highway authority.

8.1.51 Any existing gullies located within the widened entrance will be left in place and available for use following removal of the entrance.

*King Georges Avenue ACA entrance*

8.1.52 An access will be provided but with no change to the existing carriageway, other than removal of kerbs. The access entrance will be drained by gullies which will discharge into a local carrier drain. The fall of the entrance road is into the ACA such that there will be no surface water runoff from the site onto the highway.

8.1.53 The carrier drain will discharge runoff into soakaway manholes located to the side of the road where the runoff will infiltrate to ground.

*Bridleway 19 between Sandy Lane and Sizewell Gap*

8.1.54 Bridleway 19 follows a route away from Lovers Lane passing through heathlands. There is no drainage interface with the highway and no runoff onto the highway. The bridleway will drain partly by direct infiltration though the semi-permeable surface and partly “over the edge” to a nominally lower grassed section. A filter drain will be located below the channel to rapidly remove runoff from the surface pending infiltration. Given the proven infiltration rate additional storage is provided with the use of soakaway manhole chambers at intervals along the bridleway route.

# 9 REVISED DRAINAGE DESIGN STRATEGY – LEISTON DRAIN CROSSING

9.1.1 The position of the proposed Bridleway 19 approach embankment with an opening for the crossing of Leiston Drain is shown indicatively in DCO submitted drawings. The arrangement is shown in **Plate 3.** The embankment and crossing are located within the Leiston Drain flood plain, the extent of which is shown in **Plate 7.** There are requirements that the structures should be located wholly within the development red line boundary. There is also a standard regulatory requirement that where a development is permitted within a river flood plain that it should not cause any unacceptable increase of flood risk to 3rd party land.

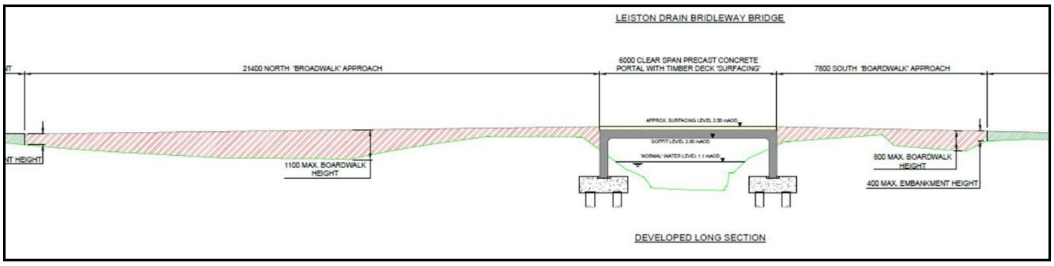
9.1.2 As part of design development, liaison has taken place with both SCC and the EA. As a result of this liaison SCC has confirmed that they expect that the bridleway should ideally remain dry and available for use for pluvial flood events up to the 1 in 100 year return period plus 40% allowance for climate change. The EA has confirmed that the crossing must not obstruct the mammal corridor which links the Aldhurst Farm wetlands upstream of Lovers Lane with the downstream wetlands.

9.1.3 An initial design consisting of a 3 span bridge and approach embankments was developed. The arrangement was subject to testing using the Main Development Site Flood Risk Assessment hydraulic model with the bridge deck set above the 1 in 100 year return period flood level of 2.0 mAOD. This confirmed that the proposed crossing had no adverse impact in increasing flood levels.

9.1.4 However, whilst the arrangement was satisfactory in terms on hydraulic impact, following the provision of topographic data which included details of the Aldhurst Farm lagoons, it was determined that the approach embankments would not fit within available space if constructed as natural embankments. In order to fit the embankments, it would be necessary to lower the level of the crossing and this was not acceptable since the bridleway would become subject to flooding.

9.1.5 In order to deliver a bridleway crossing that does not flood up to the 1 in 100 year return period event and contain the crossing structure in the space between the lagoons and the existing Lovers Lane embankment, it is necessary to modify the DCO proposal. It is proposed that the original undimensioned bridge be replaced with a portal culvert with a width of approximately 6.0 m. This structure will straddle the Leiston Drain. The embankments are replaced by a boardwalk deck supported on piers set into the ground. The arrangement is shown in **Plate 11**. The space shown shaded with red stripes represents the open space below the boardwalk and above existing ground level. This arrangement area continues to maximise flood storage volume.

**Plate 11: Bridleway 19 Crossing of Leiston Drain at Lovers Lane – Proposed Long Section**




9.1.6 With this arrangement, existing ground levels are maintained. The revised design has not been evaluated by hydraulic modelling at this time but it is apparent that the volume of flood plain removed by this arrangement is less than the original proposal with embankments. In addition, the arrangement allows unimpeded mammal movement to continue.

9.1.7 The proposed portal culvert/boardwalk design was submitted to SCC for technical approval. Upon review SCC advised that the level of the crossing is unacceptable based on the predicted 1 in 100 year rainfall return period rainfall event plus 40% climate change for pluvial (surface water) flooding which is at the lowest road level on Lovers Lane.

9.1.8 In subsequent discussions the outcome of which was confirmed in an email exchange dated 16 September 2021 it was agreed that the minimum surface level for the Bridleway 19 shall be 3.2 mAOD being the lowest road level of Lovers Lane and that the boardwalk must tie into existing ground levels at no less than 3.2 mAOD.

9.1.9 A revised design compliant with these requirements has been submitted to SCC for Approval in Principle.

# 10 REVISED DRAINAGE DESIGN STRATEGY – LEISTON DRAIN DIVERSION ABBEY ROAD

10.1.1 The proposed layout of the required Leiston Drain diversion between the existing culverts under Abbey Lane and the access drive of 105 Abbey Road as shown in **Plate 4** remains unchanged but has been developed further.

10.1.2 As noted in section 5.1.7 above estimates of existing flow rates in Leiston Drain for rainfall return periods of up to 1 in 100 years, shown in **Table 1**, have been established and used for the watercourse diversion design.

10.1.3 It should be noted that since it has been established from site inspection that upstream of Abbey Lane, the Leiston Drain overflows into the field, which is at a lower level, the predicted peak flow rate for more extreme rainfall return periods will be overstated. However, for the purpose of design, at this stage the predicted flow rates are used.

10.1.4 The presence of a tributary watercourse has been identified as a result of a topographic survey. As noted in section 5.1.7 above estimates of existing flow rates in this watercourse for rainfall return periods of up to 1 in 100 years have also been established.

10.1.5 The sizes and levels of the Abbey Lane and 105 Abbey Road culverts have been obtained as part of the topographic survey.

10.1.6 Based on the above data, it is confirmed that the Abbey Lane culvert capacity is greater than flows generated by the 1 in 100 year return period rainfall event plus 40% allowance for climate change. As a result, this culvert is to be retained and has been incorporated in design.

10.1.7 The 105 Abbey Road culvert is confirmed to be undersized such that its pipe full capacity is less that the predicted flows generated by a 1 in 10 year return period rainfall event. It can be concluded that the lack of capacity in this culvert is the primary cause of the reported flooding issues in this area, the predicted footprint of which is shown in **Plate 6**. This culvert is located immediately downstream of the DCO red line boundary.

10.1.8 The Leiston Drain watercourse diversion design which assumes a constant watercourse gradient of 1 in 250 is provided between the Abbey Lane culvert outlet and the 105 Abbey Road culvert inlet. The diversion route provides an open watercourse to the maximum extent with a channel bed width of 500 mm and natural side slopes of 1 in 3.

10.1.9 The position of the diversion channel is set by the position of the adjacent bridleway embankment.

10.1.10 It is necessary for the watercourse to be in culvert where it passes under the proposed railway and also under the bridleway crossing holding areas. The proposed culverts are 900 mm in diameter to ensure adequate capacity and also provide a good transition with the width of the open channel.

10.1.11 The requirement for a watercourse diversion and the proposed layout has been discussed with both SCC and the EA in design review meetings. The requirement for diversion and partial culverting has been accepted in principle and following validation of design will be the subject of regulatory consent.

10.1.12 SCC have confirmed that as a point of principle, the proposed highway drainage discharges and watercourse diversion must not create any increase in flood risk. In addition, as noted in section 5.1.5 above, there are known existing flooding problems at this location.

10.1.13 The outfall drain from the Main Site Access roundabout and B1122 road drains an area which forms part of the existing catchment Leiston Drain catchment will have a discharge rate limited to 5 l/s. The section of Lovers Lane described in section 6.1.6 above which currently discharges to Leiston Drain will be removed thus reducing flows into the watercourse.

10.1.14 The diverted watercourse will have a wider channel and have greater storage capacity.

10.1.15 Based on these changes it is apparent that there will be no increased flood risk as a result of the proposed highway works and watercourse diversion.

# 11 POTENTIAL FLOOD ALLEVIATION LEGACY BENEFIT

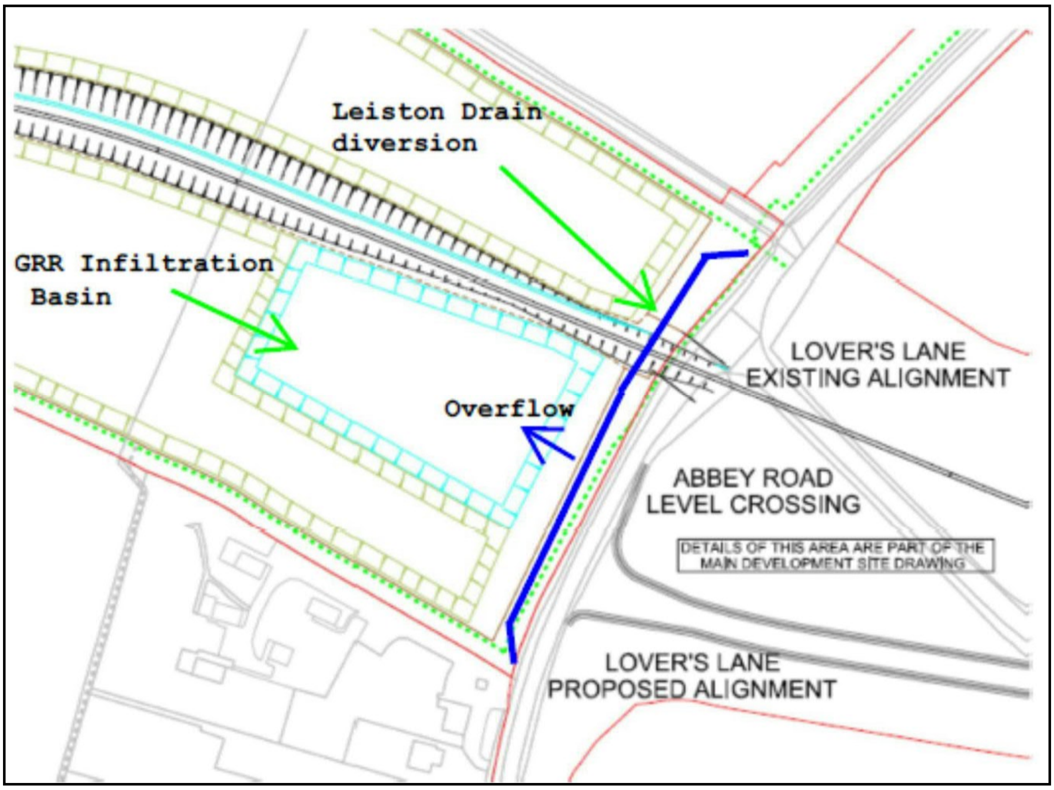
11.1.1 During liaison, SCC requested that SZC give consideration to including additional measures for flood relief as part of the design. This would potentially provide a legacy benefit. Although the cause of the flooding problems is lack of capacity in existing culverts downstream of the diversion, SZC agreed to evaluate options for flood relief which could be incorporated in design.

11.1.2 Two options were identified. One option is to provide a new diversion downstream of the inlet to the culvert at 105 Abbey Road. The diversion would be in the form of a culvert which would need to be located beneath Abbey Road and run over a distance of approximately 110 m. The existing open watercourse which is located within private properties would be abandoned.

11.1.3 This option has been considered by SCC and rejected since it would be wholly in culvert and also have adverse ecological impacts.

11.1.4 The second option that has been considered in more detail is to control the maximum flow rate upstream of the 105 Abbey Road culvert so that it does not greatly exceed the pipe full capacity of the culvert. A side weir would be provided to the open channel and as water levels increase the overflow weir would operate thus controlling water level in the watercourse. The arrangement is shown in **Plate 12**.

**Plate 12: Leiston Drain Flood Relief Option**




11.1.5 The weir would allow excess flow to pass into the proposed GRR railway infiltration basin located to the west of the diverted watercourse. This basin is provided to collect railway runoff that does not infiltrate to ground via the railway filter drain network. The required size of the basin for railway runoff and its location has not yet been confirmed. As a result, it is not known whether there is sufficient space for additional storage volume to accept inflow from Leiston Drain.

11.1.6 It is noted that the GRR infiltration basin is a temporary structure required only whilst the railway is in operation. Under the DCO proposals it is removed upon closure of the railway and the land returned to the landowner following restoration. On this basis the option of permanent use of the basin for downstream flood relief is not available and has not been included in the DCO proposals. However, SZC would be willing to discuss this option with stakeholders at a later stage post consent.

11.1.7 If this option is available all flows which pass into the basin would either infiltrate to ground or be stored thus controlling water levels in the Leiston Drain. At a later stage when inflow from the GRR is removed there would be a greater level of flood protection provided for the downstream area of Abbey Road.

# 12 VALIDATION OF UPDATED DCO DRAINAGE STRATEGY

12.1.1 In accordance with the drainage hierarchy, the **Outline Drainage Strategy** [REP2-033] proposed the primary use of infiltration, with additional use of attenuation techniques (e.g. ponds and swales) to manage water quality and to further promote infiltration. The strategy acknowledged the need for discharge to watercourse where infiltration rates were insufficient to support a primarily infiltration-led approach.

12.1.2 The approach in the **Outline Drainage Strategy** [REP2-033] is validated by the completed preliminary design, which has demonstrated that were viable, infiltration remains the primary means for removal of runoff but where not proposes the attenuated discharge of water to watercourses.

12.1.3 The preliminary design documents will be made available for review and acceptance by SCC and the EA with respect to potential adoption of the Sizewell link road by SCC and for required regulatory consents.

# 13 SUMMARY AND CONCLUSION

13.1.1 The purpose of this technical note is to provide details of how the DCO Drainage Strategy has needed to evolve and develop as a result of provision of new information. The proposed design development has been discussed with SCC and the EA in liaison and through Design Review Meetings.

13.1.2 The highway drainage has been designed in accordance with Design Manual for Roads and Bridges, **(Ref 3),** the CIRIA SuDS Manual C753 (**Ref. 4)** and to comply with stated requirements of SCC contained in their SuDS Local Design Guide Appendix A **(Ref 2).**

13.1.3 At this preliminary design stage, it is considered that the design provides for the effective removal, treatment and disposal of highway surface water runoff without adversely increasing flood risk to or from watercourses or impacting on third parties. Discharge to watercourse would not adversely impact on water quality. The flood risk performance of the highway is as specified in DMRB and SCC guidance3 **(Ref 1.2 & 3).**

# REFERENCES

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

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

3. Highways Agency et al. (2009). Volume 11, Section 3, Part 10: Road Drainage and the Water Environment, HD45/09. [redacted]

4. The SUDs Manual (C753), CIRIA, 2015, ISBN 978-0-86017-760-9.

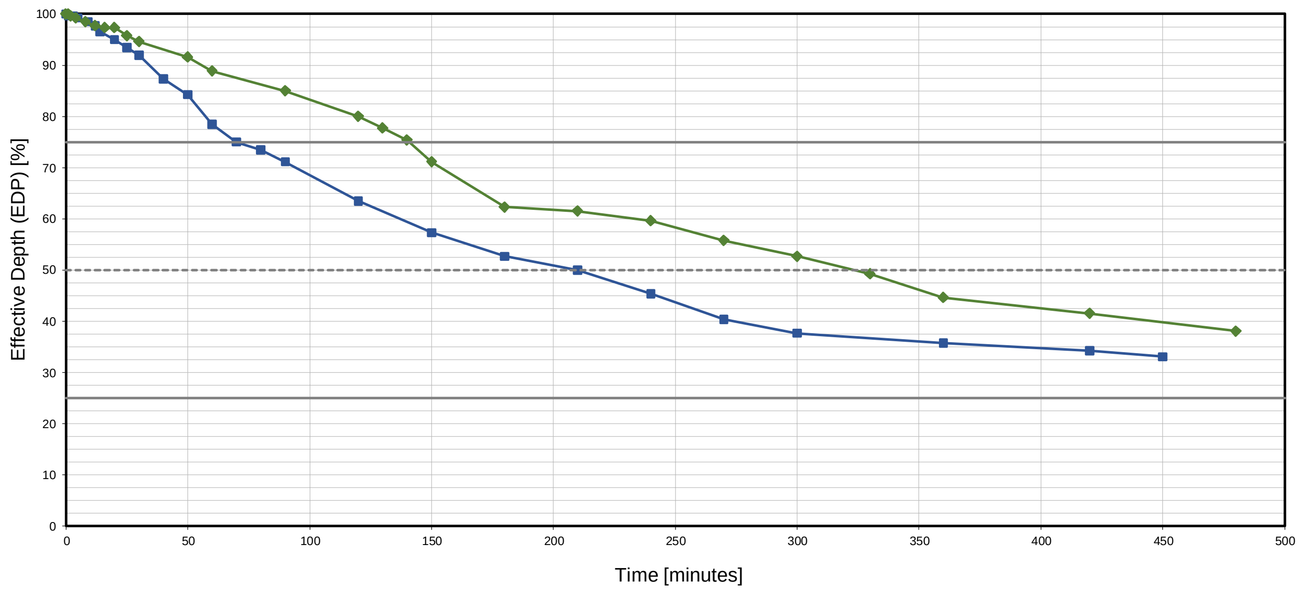
# APPENDIX A: INFILTRATION TEST DATA AND RESULTS

## F.3 Infiltration/Soakaway Tests

| Title | |
|---|---|
| Soakaway Test Results | Referenced by Location ID |

**NNB GENERATION COMPANY (SZC) LIMITED**
**AD6 ROAD SCHEMES**

| SOAKAWAY TEST RECORD | | | | | | | Method BRE 365 2016 |
|---|---|---|---|---|---|---|---|
| Date | 11/03/2021 | Operator | AD | Pit Depth | 3.00 m | Hole No. | AD6-TH301 |




| Test 1 ■ | | | Test 2 ◆ | | | Test 3 ▲ | | |
|---|---|---|---|---|---|---|---|---|
| Start Time = | 09:31 | | Start Time = | 08:49 | | Start Time = | | |
| Test Top = | 0.40 | m BGL | Test Top = | 0.40 | m BGL | Test Top = | | m BGL |
| Test Base = | 3.00 | m BGL | Test Base = | 3.00 | m BGL | Test Base = | | m BGL |
| 75% EDP = | 1.05 | m BGL | 75% EDP = | 1.05 | m BGL | 75% EDP = | | m BGL |
| 50% EDP = | 1.70 | m BGL | 50% EDP = | 1.70 | m BGL | 50% EDP = | | m BGL |
| 25% EDP = | 2.35 | m BGL | 25% EDP = | 2.35 | m BGL | 25% EDP = | | m BGL |

| Elapsed Time [hr:min:sec] | Water [m BDL] | Water [m BGL] | EDP [%] | Elapsed Time [hr:min:sec] | Water [m BDL] | Water [m BGL] | EDP [%] | Elapsed Time [hr:min:sec] | Water [m BDL] | Water [m BGL] | EDP [%] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 00:00:00 | | 0.40 | 100 | 00:00:00 | | 0.40 | 100 | 00:00:00 | | | |
| 00:01:00 | | 0.41 | 100 | 00:01:00 | | 0.40 | 100 | 00:00:30 | | | |
| 00:03:00 | | 0.41 | 100 | 00:02:00 | | 0.41 | 100 | 00:01:00 | | | |
| 00:05:00 | | 0.42 | 99 | 00:04:00 | | 0.42 | 99 | 00:01:30 | | | |
| 00:09:00 | | 0.44 | 98 | 00:08:00 | | 0.44 | 98 | 00:02:00 | | | |
| 00:12:00 | | 0.46 | 98 | 00:12:00 | | 0.46 | 98 | 00:03:00 | | | |
| 00:14:00 | | 0.49 | 97 | 00:16:00 | | 0.47 | 97 | 00:04:00 | | | |
| 00:20:00 | | 0.53 | 95 | 00:20:00 | | 0.47 | 97 | 00:05:00 | | | |
| 00:25:00 | | 0.57 | 93 | 00:25:00 | | 0.51 | 96 | 00:06:00 | | | |
| 00:30:00 | | 0.61 | 92 | 00:30:00 | | 0.54 | 95 | 00:07:00 | | | |
| 00:40:00 | | 0.73 | 87 | 00:50:00 | | 0.62 | 92 | 00:08:00 | | | |
| 00:50:00 | | 0.81 | 84 | 01:00:00 | | 0.69 | 89 | 00:09:00 | | | |
| 01:00:00 | | 0.96 | 78 | 01:30:00 | | 0.79 | 85 | 00:10:00 | | | |
| 01:10:00 | | 1.05 | 75 | 02:00:00 | | 0.92 | 80 | 00:12:00 | | | |
| 01:20:00 | | 1.09 | 73 | 02:10:00 | | 0.98 | 78 | 00:14:00 | | | |
| 01:30:00 | | 1.15 | 71 | 02:20:00 | | 1.04 | 75 | 00:16:00 | | | |
| 02:00:00 | | 1.35 | 63 | 02:30:00 | | 1.15 | 71 | 00:18:00 | | | |
| 02:30:00 | | 1.51 | 57 | 03:00:00 | | 1.38 | 62 | 00:20:00 | | | |
| 03:00:00 | | 1.63 | 53 | 03:30:00 | | 1.40 | 62 | 00:25:00 | | | |
| 03:30:00 | | 1.70 | 50 | 04:00:00 | | 1.45 | 60 | 00:30:00 | | | |
| 04:00:00 | | 1.82 | 45 | 04:30:00 | | 1.55 | 56 | 00:40:00 | | | |
| 04:30:00 | | 1.95 | 40 | 05:00:00 | | 1.63 | 53 | 00:50:00 | | | |
| 05:00:00 | | 2.02 | 38 | 05:30:00 | | 1.72 | 49 | 01:00:00 | | | |
| 06:00:00 | | 2.07 | 36 | 06:00:00 | | 1.84 | 45 | | | | |
| 07:00:00 | | 2.11 | 34 | 07:00:00 | | 1.92 | 42 | | | | |
| 07:30:00 | | 2.14 | 33 | 08:00:00 | | 2.01 | 38 | | | | |

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

Input by AH 24/03/2021

Checked by CAY 13/05/2021

Contract No. F181386

FUGRO

**NNB GENERATION COMPANY (SZC) LIMITED**
**AD6 ROAD SCHEMES**

| SOAKAWAY TEST RECORD | | | | | | | Method BRE 365 2016 |
|---|---|---|---|---|---|---|---|
| Date | 11/03/2021 | Operator | AD | Pit Depth | 3.00 m | Hole No. | AD6-TH301 |

## Test Details

| | |
|---|---|
| Datum (-ve denotes AGL) = 0.00 m BGL | Well Screen<br>Well screen not used |
| Pit Length = 2.58 m<br>Pit Width = 0.65 m<br>Pit Depth = 3.00 m BGL | Filter Material<br>Assumed Solid Fraction = 64.41 %<br>Assumed Porosity = 35.59 % |

Weather: Cold, heavy rain, strong wind, dry ground

Geology: CLAY/SAND

Remarks
Test 1 undertakn on 11/03/2021; Test 2 undertaken on 12/03/2021.
Test termination agreed with client representative; Test 3 not required.
Water level did not reach 25% EDP; infiltration rate\s given based on data between 75% EDP and 50% EDP only.
Volume of gravel fraction assumed to be 64.41% of the total volume of gravel filled space, giving an estimated porosity of 35.59%.
Gravel fill up to 0.40m BGL to support test pit.
Water added to the pit to 0.40m BGL (Test 1 and Test 2).

## Calculation

| Test 1 | | | Test 2 | | | Test 3 | | |
|---|---|---|---|---|---|---|---|---|
| Start Time = | 09:31 | | Start Time = | 08:49 | | Start Time = | | |
| Test Top = | 0.40 | m BGL | Test Top = | 0.40 | m BGL | Test Top = | | m BGL |
| Test Base = | 3.00 | m BGL | Test Base = | 3.00 | m BGL | Test Base = | | m BGL |
| EDP = | 2.60 | m | EDP = | 2.60 | m | EDP = | | m |
| 75% EDP = | 1.05 | m BGL | 75% EDP = | 1.05 | m BGL | 75% EDP = | | m BGL |
| 50% EDP = | 1.70 | m BGL | 50% EDP = | 1.70 | m BGL | 50% EDP = | | m BGL |
| 25% EDP = | 2.35 | m BGL | 25% EDP = | 2.35 | m BGL | 25% EDP = | | m BGL |
| V = | 4.36 | $m^3$ | V = | 4.36 | $m^3$ | V = | | $m^3$ |
| Vg = | 2.81 | $m^3$ | Vg = | 2.81 | $m^3$ | Vg = | | $m^3$ |
| Vp = | 1.55 | $m^3$ | Vp = | 1.55 | $m^3$ | Vp = | | $m^3$ |
| Vp75-50 = | 0.39 | $m^3$ | Vp75-50 = | 0.39 | $m^3$ | Vp75-25 = | | $m^3$ |
| ap = | 5.88 | $m^2$ | ap = | 5.88 | $m^2$ | ap = | | $m^2$ |
| Tp75 = | 4200 | s | Tp75 = | 8460 | s | Tp75 = | | s |
| Tp50 = | 12600 | s | Tp50 = | 19440 | s | Tp25 = | | s |
| Infiltration Rate, f = | 7.86E-06 | m/s | Infiltration Rate, f = | 6.01E-06 | 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.

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

or

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

Input by AH 24/03/2021

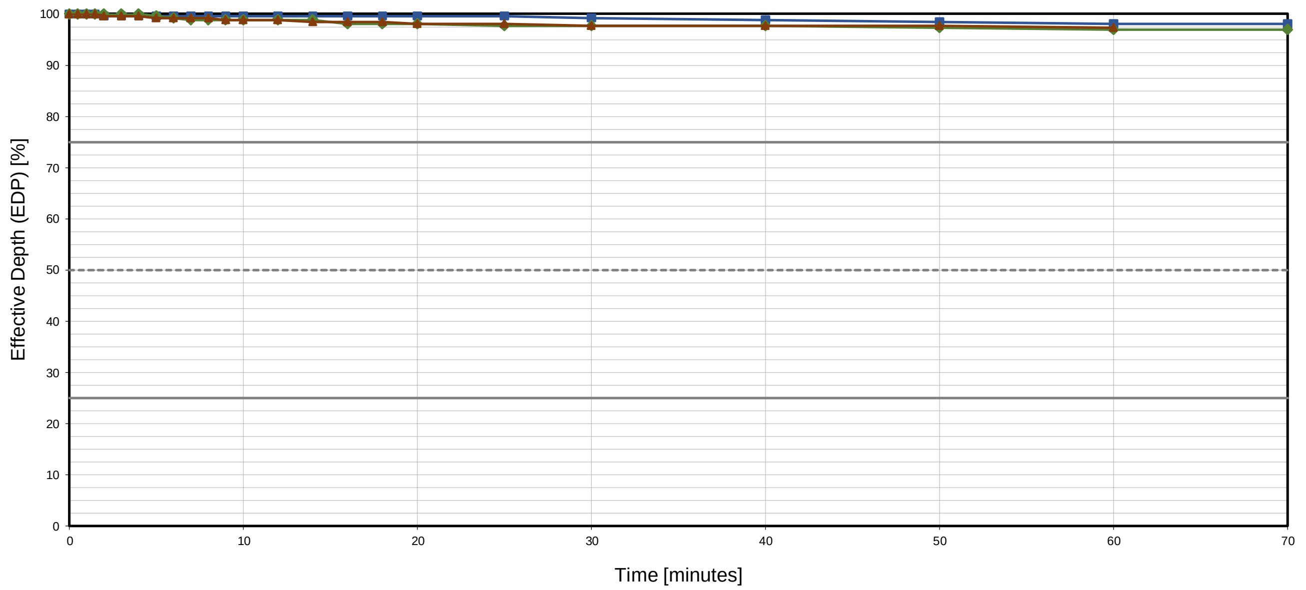
Checked by CAY 13/05/2021

Contract No. F181386

FUGRO

**NNB GENERATION COMPANY (SZC) LIMITED**
**AD6 ROAD SCHEMES**

| SOAKAWAY TEST RECORD | | | | | | | Method BRE 365 2016 |
|---|---|---|---|---|---|---|---|
| Date | 11/03/2021 | Operator | AD | Pit Depth | 3.00 m | Hole No. | AD6-TH302 |




| Test 1 ■ | | | | Test 2 ◆ | | | | Test 3 ▲ | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Start Time = | 14:35 | | | Start Time = | 10:05 | | | Start Time = | 11:32 | | |
| Test Top = | 0.42 | m BGL | | Test Top = | 0.40 | m BGL | | Test Top = | 0.40 | m BGL | |
| Test Base = | 3.00 | m BGL | | Test Base = | 3.00 | m BGL | | Test Base = | 3.00 | m BGL | |
| 75% EDP = | 1.07 | m BGL | | 75% EDP = | 1.05 | m BGL | | 75% EDP = | 1.05 | m BGL | |
| 50% EDP = | 1.71 | m BGL | | 50% EDP = | 1.70 | m BGL | | 50% EDP = | 1.70 | m BGL | |
| 25% EDP = | 2.36 | m BGL | | 25% EDP = | 2.35 | m BGL | | 25% EDP = | 2.35 | 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 | | 0.42 | 100 | 00:00:00 | | 0.40 | 100 | 00:00:00 | | 0.40 | 100 |
| 00:00:30 | | 0.42 | 100 | 00:00:30 | | 0.40 | 100 | 00:00:30 | | 0.40 | 100 |
| 00:01:00 | | 0.42 | 100 | 00:01:00 | | 0.40 | 100 | 00:01:00 | | 0.40 | 100 |
| 00:01:30 | | 0.42 | 100 | 00:01:30 | | 0.40 | 100 | 00:01:30 | | 0.40 | 100 |
| 00:02:00 | | 0.43 | 100 | 00:02:00 | | 0.40 | 100 | 00:02:00 | | 0.41 | 100 |
| 00:03:00 | | 0.43 | 100 | 00:03:00 | | 0.40 | 100 | 00:03:00 | | 0.41 | 100 |
| 00:04:00 | | 0.43 | 100 | 00:04:00 | | 0.40 | 100 | 00:04:00 | | 0.41 | 100 |
| 00:05:00 | | 0.43 | 100 | 00:05:00 | | 0.41 | 100 | 00:05:00 | | 0.42 | 99 |
| 00:06:00 | | 0.43 | 100 | 00:06:00 | | 0.42 | 99 | 00:06:00 | | 0.42 | 99 |
| 00:07:00 | | 0.43 | 100 | 00:07:00 | | 0.43 | 99 | 00:07:00 | | 0.42 | 99 |
| 00:08:00 | | 0.43 | 100 | 00:08:00 | | 0.43 | 99 | 00:08:00 | | 0.42 | 99 |
| 00:09:00 | | 0.43 | 100 | 00:09:00 | | 0.43 | 99 | 00:09:00 | | 0.43 | 99 |
| 00:10:00 | | 0.43 | 100 | 00:10:00 | | 0.43 | 99 | 00:10:00 | | 0.43 | 99 |
| 00:12:00 | | 0.43 | 100 | 00:12:00 | | 0.43 | 99 | 00:12:00 | | 0.43 | 99 |
| 00:14:00 | | 0.43 | 100 | 00:14:00 | | 0.43 | 99 | 00:14:00 | | 0.44 | 98 |
| 00:16:00 | | 0.43 | 100 | 00:16:00 | | 0.45 | 98 | 00:16:00 | | 0.44 | 98 |
| 00:18:00 | | 0.43 | 100 | 00:18:00 | | 0.45 | 98 | 00:18:00 | | 0.44 | 98 |
| 00:20:00 | | 0.43 | 100 | 00:20:00 | | 0.45 | 98 | 00:20:00 | | 0.45 | 98 |
| 00:25:00 | | 0.43 | 100 | 00:25:00 | | 0.46 | 98 | 00:25:00 | | 0.45 | 98 |
| 00:30:00 | | 0.44 | 99 | 00:30:00 | | 0.46 | 98 | 00:30:00 | | 0.46 | 98 |
| 00:40:00 | | 0.45 | 99 | 00:40:00 | | 0.46 | 98 | 00:40:00 | | 0.46 | 98 |
| 00:50:00 | | 0.46 | 98 | 00:50:00 | | 0.47 | 97 | 00:50:00 | | 0.46 | 98 |
| 01:00:00 | | 0.47 | 98 | 01:00:00 | | 0.48 | 97 | 01:00:00 | | 0.47 | 97 |
| 01:10:00 | | 0.47 | 98 | 01:10:00 | | 0.48 | 97 | | | | |

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

Input by AH 24/03/2021

Checked by CAY 13/05/2021

Contract No. F181386

## SOAKAWAY TEST RECORD

Method BRE 365 2016

| Date | 11/03/2021 | Operator | AD | Pit Depth | 3.00 m | Hole No. | AD6-TH302 |
|---|---|---|---|---|---|---|---|

### Test Details

| | | | |
|---|---|---|---|
| Datum (-ve denotes AGL) = | 0.00 m BGL | Well Screen | |
| | | Well screen not used | |
| Pit Length = | 3.62 m | Filter Material | |
| Pit Width = | 0.78 m | Assumed Solid Fraction = | 64.41 % |
| Pit Depth = | 3.00 m BGL | Assumed Porosity = | 35.59 % |

Weather: Cold, heavy rain, strong wind, dry ground.

Geology: CLAY

Remarks

Test 1 undertaken on 11/03/2021; Test 2 and Test 3 undertaken on 12/03/2021.
Negligible discharge observed. Test termination agreed with client representative.
Water level did not reach 75%, 50% or 25% EDP; infiltration rates cannot be given.
Volume of gravel fraction assumed to be 64.41% of the total volume of gravel filled space, giving an estimated porosity of 35.59%.
Gravel fill up to 0.40m BGL to support test pit.
Water added to the pit to 0.42m BGL (Test 1) and 0.40m BGL (Test 2); water was noted at 0.48m BGL before Test 2 started.

### Calculation

| Test 1 | | | Test 2 | | | Test 3 | | |
|---|---|---|---|---|---|---|---|---|
| Start Time = | 14:35 | | Start Time = | 10:05 | | Start Time = | 11:32 | |
| Test Top = | 0.42 | m BGL | Test Top = | 0.40 | m BGL | Test Top = | 0.40 | m BGL |
| Test Base = | 3.00 | m BGL | Test Base = | 3.00 | m BGL | Test Base = | 3.00 | m BGL |
| EDP = | 2.58 | m | EDP = | 2.60 | m | EDP = | 2.60 | m |
| 75% EDP = | 1.07 | m BGL | 75% EDP = | 1.05 | m BGL | 75% EDP = | 1.05 | m BGL |
| 50% EDP = | 1.71 | m BGL | 50% EDP = | 1.70 | m BGL | 50% EDP = | 1.70 | m BGL |
| 25% EDP = | 2.36 | m BGL | 25% EDP = | 2.35 | m BGL | 25% EDP = | 2.35 | m BGL |
| V = | 7.28 | $m^3$ | V = | 7.34 | $m^3$ | V = | 7.34 | $m^3$ |
| Vg = | 4.73 | $m^3$ | Vg = | 4.73 | $m^3$ | Vg = | 4.73 | $m^3$ |
| Vp = | 2.56 | $m^3$ | Vp = | 2.61 | $m^3$ | Vp = | 2.61 | $m^3$ |
| Vp75-25 = | 1.28 | $m^3$ | Vp75-25 = | 1.31 | $m^3$ | Vp75-25 = | 1.31 | $m^3$ |
| ap = | 14.18 | $m^2$ | ap = | 14.26 | $m^2$ | ap = | 14.26 | $m^2$ |
| Tp75 = | | s | Tp75 = | | 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.

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

or

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

Input by AH 24/03/2021

Checked by CAY 13/05/2021

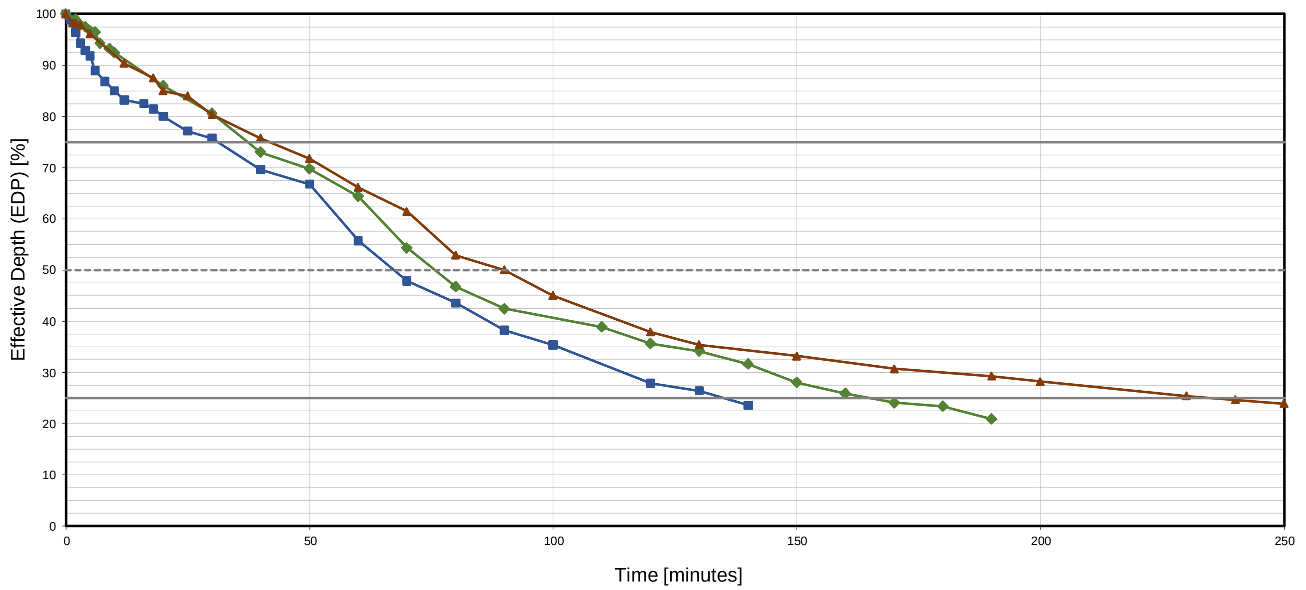
Contract No. F181386

**NNB GENERATION COMPANY (SZC) LIMITED**
**AD6 ROAD SCHEMES**

## SOAKAWAY TEST RECORD

Method BRE 365 2016

| Date | 09/03/2021 | Operator | AD | Pit Depth | 3.10 m | Hole No. | AD6-TH305 |
|---|---|---|---|---|---|---|---|




| Test 1 ■ | | | Test 2 ◆ | | | Test 3 ▲ | | |
|---|---|---|---|---|---|---|---|---|
| Start Time = | 11:34 | | Start Time = | 13:55 | | Start Time = | 09:33 | |
| Test Top = | 0.30 | m BGL | Test Top = | 0.32 | m BGL | Test Top = | 0.30 | m BGL |
| Test Base = | 3.10 | m BGL | Test Base = | 3.10 | m BGL | Test Base = | 3.10 | m BGL |
| 75% EDP = | 1.00 | m BGL | 75% EDP = | 1.02 | m BGL | 75% EDP = | 1.00 | m BGL |
| 50% EDP = | 1.70 | m BGL | 50% EDP = | 1.71 | m BGL | 50% EDP = | 1.70 | m BGL |
| 25% EDP = | 2.40 | m BGL | 25% EDP = | 2.41 | m BGL | 25% EDP = | 2.40 | 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 | | 0.30 | 100 | 00:00:00 | | 0.32 | 100 | 00:00:00 | | 0.30 | 100 |
| 00:01:00 | | 0.33 | 99 | 00:02:00 | | 0.35 | 99 | 00:01:30 | | 0.35 | 98 |
| 00:01:30 | | 0.34 | 99 | 00:03:00 | | 0.38 | 98 | 00:02:00 | | 0.35 | 98 |
| 00:02:00 | | 0.40 | 96 | 00:04:00 | | 0.39 | 97 | 00:03:00 | | 0.36 | 98 |
| 00:03:00 | | 0.46 | 94 | 00:05:00 | | 0.41 | 97 | 00:05:00 | | 0.41 | 96 |
| 00:04:00 | | 0.50 | 93 | 00:06:00 | | 0.42 | 96 | 00:12:00 | | 0.57 | 90 |
| 00:05:00 | | 0.53 | 92 | 00:07:00 | | 0.48 | 94 | 00:18:00 | | 0.65 | 88 |
| 00:06:00 | | 0.61 | 89 | 00:09:00 | | 0.51 | 93 | 00:20:00 | | 0.72 | 85 |
| 00:08:00 | | 0.67 | 87 | 00:10:00 | | 0.53 | 92 | 00:25:00 | | 0.75 | 84 |
| 00:10:00 | | 0.72 | 85 | 00:20:00 | | 0.71 | 86 | 00:30:00 | | 0.85 | 80 |
| 00:12:00 | | 0.77 | 83 | 00:30:00 | | 0.86 | 81 | 00:40:00 | | 0.98 | 76 |
| 00:16:00 | | 0.79 | 83 | 00:40:00 | | 1.07 | 73 | 00:50:00 | | 1.09 | 72 |
| 00:18:00 | | 0.82 | 81 | 00:50:00 | | 1.16 | 70 | 01:00:00 | | 1.25 | 66 |
| 00:20:00 | | 0.86 | 80 | 01:00:00 | | 1.31 | 64 | 01:10:00 | | 1.38 | 61 |
| 00:25:00 | | 0.94 | 77 | 01:10:00 | | 1.59 | 54 | 01:20:00 | | 1.62 | 53 |
| 00:30:00 | | 0.98 | 76 | 01:20:00 | | 1.80 | 47 | 01:30:00 | | 1.70 | 50 |
| 00:40:00 | | 1.15 | 70 | 01:30:00 | | 1.92 | 42 | 01:40:00 | | 1.84 | 45 |
| 00:50:00 | | 1.23 | 67 | 01:50:00 | | 2.02 | 39 | 02:00:00 | | 2.04 | 38 |
| 01:00:00 | | 1.54 | 56 | 02:00:00 | | 2.11 | 36 | 02:10:00 | | 2.11 | 35 |
| 01:10:00 | | 1.76 | 48 | 02:10:00 | | 2.15 | 34 | 02:30:00 | | 2.17 | 33 |
| 01:20:00 | | 1.88 | 44 | 02:20:00 | | 2.22 | 32 | 02:50:00 | | 2.24 | 31 |
| 01:30:00 | | 2.03 | 38 | 02:30:00 | | 2.32 | 28 | 03:10:00 | | 2.28 | 29 |
| 01:40:00 | | 2.11 | 35 | 02:40:00 | | 2.38 | 26 | 03:20:00 | | 2.31 | 28 |
| 02:00:00 | | 2.32 | 28 | 02:50:00 | | 2.43 | 24 | 03:50:00 | | 2.39 | 25 |
| 02:10:00 | | 2.36 | 26 | 03:00:00 | | 2.45 | 23 | 04:00:00 | | 2.41 | 25 |
| 02:20:00 | | 2.44 | 24 | 03:10:00 | | 2.52 | 21 | 04:10:00 | | 2.43 | 24 |

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

Input by AH 24/03/2021

Checked by CAY 13/05/2021

Contract No. F181386

| SOAKAWAY TEST RECORD | | | | | | | Method BRE 365 2016 |
|---|---|---|---|---|---|---|---|
| Date | 09/03/2021 | Operator | AD | Pit Depth | 3.10 m | Hole No. | AD6-TH305 |

## Test Details

| | | | |
|---|---|---|---|
| Datum (-ve denotes AGL) = | 0.00 m BGL | Well Screen | |
| | | Well screen not used | |
| Pit Length = | 2.58 m | Filter Material | |
| Pit Width = | 0.62 m | Assumed Solid Fraction = | 64.41 % |
| Pit Depth = | 3.10 m BGL | Assumed Porosity = | 35.59 % |

Weather: Warm, dry, light wind, dry ground

Geology: SAND

Remarks
Test 1 and Test 2 undertaken on 09/03/2021; Test 3 undertaken on 10/03/2021.
Volume of gravel fraction assumed to be 64.41% of the total volume of gravel filled space, giving an estimated porosity of 35.59%.

Gravel filled up to 1.22m BGL to support test pit.
Water added to the pit to 0.30m BGL (Test 1), 0.32m BGL (Test 2) and 0.30m BGL (Test 3).

## Calculation

| Test 1 ■ | | | Test 2 ◆ | | | Test 3 ▲ | | |
|---|---|---|---|---|---|---|---|---|
| Start Time = | 11:34 | | Start Time = | 13:55 | | Start Time = | 09:33 | |
| Test Top = | 0.30 | m BGL | Test Top = | 0.32 | m BGL | Test Top = | 0.30 | m BGL |
| Test Base = | 3.10 | m BGL | Test Base = | 3.10 | m BGL | Test Base = | 3.10 | m BGL |
| EDP = | 2.80 | m | EDP = | 2.78 | m | EDP = | 2.80 | m |
| 75% EDP = | 1.00 | m BGL | 75% EDP = | 1.02 | m BGL | 75% EDP = | 1.00 | m BGL |
| 50% EDP = | 1.70 | m BGL | 50% EDP = | 1.71 | m BGL | 50% EDP = | 1.70 | m BGL |
| 25% EDP = | 2.40 | m BGL | 25% EDP = | 2.41 | m BGL | 25% EDP = | 2.40 | m BGL |
| V = | 4.48 | $m^3$ | V = | 4.45 | $m^3$ | V = | 4.48 | $m^3$ |
| Vg = | 1.94 | $m^3$ | Vg = | 1.94 | $m^3$ | Vg = | 1.94 | $m^3$ |
| Vp = | 2.54 | $m^3$ | Vp = | 2.51 | $m^3$ | Vp = | 2.54 | $m^3$ |
| Vp75-25 = | 1.27 | $m^3$ | Vp75-25 = | 1.25 | $m^3$ | Vp75-25 = | 1.27 | $m^3$ |
| ap = | 10.56 | $m^2$ | ap = | 10.50 | $m^2$ | ap = | 10.56 | $m^2$ |
| Tp75 = | 1878 | s | Tp75 = | 2256 | s | Tp75 = | 2520 | s |
| Tp25 = | 8100 | s | Tp25 = | 9900 | s | Tp25 = | 14100 | s |
| Infiltration Rate, f = | 1.93E-05 | m/s | Infiltration Rate, f = | 1.56E-05 | m/s | Infiltration Rate, f = | 1.04E-05 | m/s |

Notes
Pit sides are assumed to be vertical; dimensions at mid-depth of pit used in general.
m AGL/BGL = metres above / below ground level;
m BDL = metres below datum level.

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

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

or

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

Input by AH 24/03/2021

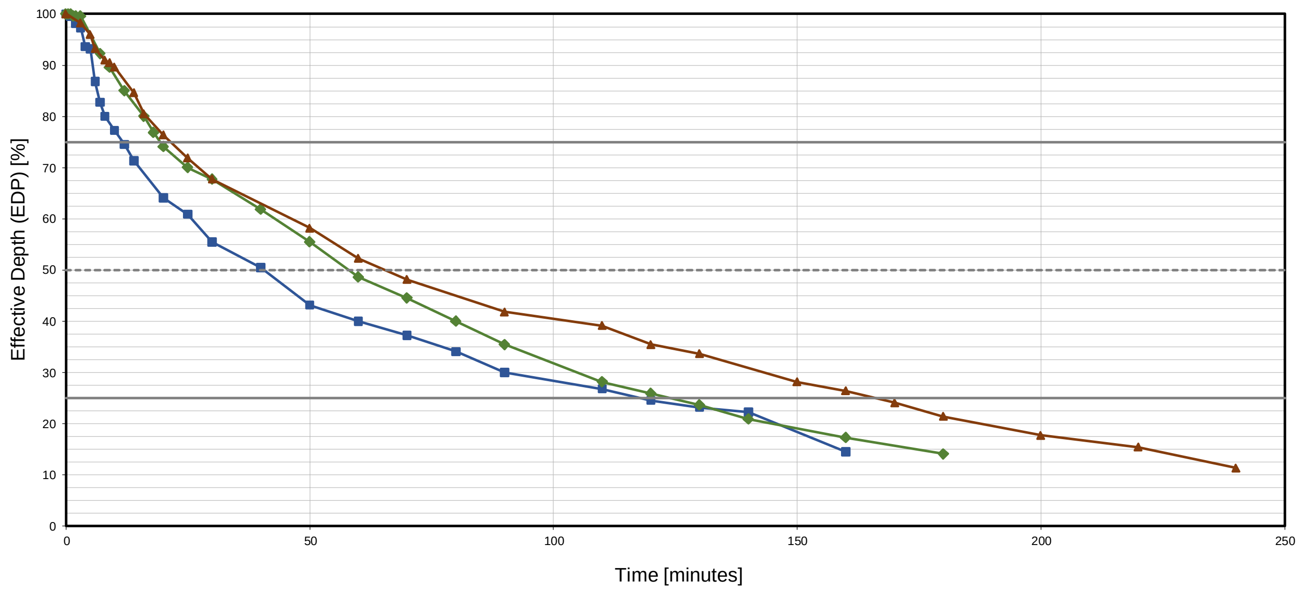
Checked by CAY 13/05/2021

Contract No. F181386

FUGRO

**NNB GENERATION COMPANY (SZC) LIMITED**
**AD6 ROAD SCHEMES**

| SOAKAWAY TEST RECORD | | | | | | | Method BRE 365 2016 |
|---|---|---|---|---|---|---|---|
| Date | 13/03/2021 | Operator | AD | Pit Depth | 2.50 m | Hole No. | AD6-TH313 |




| Test 1 ■ | | | | Test 2 ◆ | | | | Test 3 ▲ | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Start Time = | 10:28 | | | Start Time = | 13:14 | | | Start Time = | 08:42 | | |
| Test Top = | 0.30 | m BGL | | Test Top = | 0.30 | 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 = | 0.85 | m BGL | | 75% EDP = | 0.85 | m BGL | | 75% EDP = | 0.85 | m BGL | |
| 50% EDP = | 1.40 | m BGL | | 50% EDP = | 1.40 | m BGL | | 50% EDP = | 1.40 | m BGL | |
| 25% EDP = | 1.95 | m BGL | | 25% EDP = | 1.95 | 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 | | 0.30 | 100 | 00:00:00 | | 0.30 | 100 | 00:00:00 | | 0.30 | 100 |
| 00:01:00 | | 0.31 | 100 | 00:00:30 | | 0.30 | 100 | 00:03:00 | | 0.34 | 98 |
| 00:02:00 | | 0.34 | 98 | 00:01:00 | | 0.30 | 100 | 00:05:00 | | 0.39 | 96 |
| 00:03:00 | | 0.36 | 97 | 00:01:30 | | 0.31 | 100 | 00:06:00 | | 0.45 | 93 |
| 00:04:00 | | 0.44 | 94 | 00:02:00 | | 0.31 | 100 | 00:08:00 | | 0.50 | 91 |
| 00:05:00 | | 0.45 | 93 | 00:03:00 | | 0.31 | 100 | 00:09:00 | | 0.51 | 90 |
| 00:06:00 | | 0.59 | 87 | 00:07:00 | | 0.47 | 92 | 00:10:00 | | 0.53 | 90 |
| 00:07:00 | | 0.68 | 83 | 00:09:00 | | 0.53 | 90 | 00:14:00 | | 0.64 | 85 |
| 00:08:00 | | 0.74 | 80 | 00:12:00 | | 0.63 | 85 | 00:16:00 | | 0.73 | 80 |
| 00:10:00 | | 0.80 | 77 | 00:16:00 | | 0.74 | 80 | 00:20:00 | | 0.82 | 76 |
| 00:12:00 | | 0.86 | 75 | 00:18:00 | | 0.81 | 77 | 00:25:00 | | 0.92 | 72 |
| 00:14:00 | | 0.93 | 71 | 00:20:00 | | 0.87 | 74 | 00:30:00 | | 1.01 | 68 |
| 00:20:00 | | 1.09 | 64 | 00:25:00 | | 0.96 | 70 | 00:50:00 | | 1.22 | 58 |
| 00:25:00 | | 1.16 | 61 | 00:30:00 | | 1.01 | 68 | 01:00:00 | | 1.35 | 52 |
| 00:30:00 | | 1.28 | 55 | 00:40:00 | | 1.14 | 62 | 01:10:00 | | 1.44 | 48 |
| 00:40:00 | | 1.39 | 50 | 00:50:00 | | 1.28 | 55 | 01:30:00 | | 1.58 | 42 |
| 00:50:00 | | 1.55 | 43 | 01:00:00 | | 1.43 | 49 | 01:50:00 | | 1.64 | 39 |
| 01:00:00 | | 1.62 | 40 | 01:10:00 | | 1.52 | 45 | 02:00:00 | | 1.72 | 35 |
| 01:10:00 | | 1.68 | 37 | 01:20:00 | | 1.62 | 40 | 02:10:00 | | 1.76 | 34 |
| 01:20:00 | | 1.75 | 34 | 01:30:00 | | 1.72 | 35 | 02:30:00 | | 1.88 | 28 |
| 01:30:00 | | 1.84 | 30 | 01:50:00 | | 1.88 | 28 | 02:40:00 | | 1.92 | 26 |
| 01:50:00 | | 1.91 | 27 | 02:00:00 | | 1.93 | 26 | 02:50:00 | | 1.97 | 24 |
| 02:00:00 | | 1.96 | 25 | 02:10:00 | | 1.98 | 24 | 03:00:00 | | 2.03 | 21 |
| 02:10:00 | | 1.99 | 23 | 02:20:00 | | 2.04 | 21 | 03:20:00 | | 2.11 | 18 |
| 02:20:00 | | 2.01 | 22 | 02:40:00 | | 2.12 | 17 | 03:40:00 | | 2.16 | 15 |
| 02:40:00 | | 2.18 | 15 | 03:00:00 | | 2.19 | 14 | 04:00:00 | | 2.25 | 11 |

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

Input by AH 24/03/2021 Checked by CAY 13/05/2021

Contract No. F181386

| SOAKAWAY TEST RECORD | | | | | | | Method BRE 365 2016 |
|---|---|---|---|---|---|---|---|
| Date | 13/03/2021 | Operator | AD | Pit Depth | 2.50 m | Hole No. | AD6-TH313 |

## Test Details

| | | | |
|---|---|---|---|
| Datum (-ve denotes AGL) = | 0.00 m BGL | Well Screen | |
| | | Well screen not used | |
| Pit Length = | 2.11 m | Filter Material | |
| Pit Width = | 0.70 m | Assumed Solid Fraction = | 64.41 % |
| Pit Depth = | 2.50 m BGL | Assumed Porosity = | 35.59 % |

Weather: Cold, strong wind, dry ground.

Geology: SAND

Remarks
Test 1 and Test 2 undertaken on 13/03/2021; Test 3 undertaken on 14/03/2021.
Volume of gravel fraction assumed to be 64.41% of the total volume of gravel filled space, giving an estimated porosity of 35.59%.

Gravel filled up to 0.30m BGL to support test pit.
Water added to the pit to 0.30m BGL (Test 1, Test 2 and Test 3).

## Calculation

| Test 1 ■ | | | Test 2 ◆ | | | Test 3 ▲ | | |
|---|---|---|---|---|---|---|---|---|
| Start Time = | 10:28 | | Start Time = | 13:14 | | Start Time = | 08:42 | |
| Test Top = | 0.30 | m BGL | Test Top = | 0.30 | 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 = | 2.20 | m | EDP = | 2.20 | m | EDP = | 2.20 | m |
| 75% EDP = | 0.85 | m BGL | 75% EDP = | 0.85 | m BGL | 75% EDP = | 0.85 | m BGL |
| 50% EDP = | 1.40 | m BGL | 50% EDP = | 1.40 | m BGL | 50% EDP = | 1.40 | m BGL |
| 25% EDP = | 1.95 | m BGL | 25% EDP = | 1.95 | m BGL | 25% EDP = | 1.95 | m BGL |
| V = | 3.25 | $m^3$ | V = | 3.25 | $m^3$ | V = | 3.25 | $m^3$ |
| Vg = | 2.09 | $m^3$ | Vg = | 2.09 | $m^3$ | Vg = | 2.09 | $m^3$ |
| Vp = | 1.16 | $m^3$ | Vp = | 1.16 | $m^3$ | Vp = | 1.16 | $m^3$ |
| Vp75-25 = | 0.58 | $m^3$ | Vp75-25 = | 0.58 | $m^3$ | Vp75-25 = | 0.58 | $m^3$ |
| ap = | 7.66 | $m^2$ | ap = | 7.66 | $m^2$ | ap = | 7.66 | $m^2$ |
| Tp75 = | 690 | s | Tp75 = | 1152 | s | Tp75 = | 1290 | s |
| Tp25 = | 7080 | s | Tp25 = | 7470 | s | Tp25 = | 9960 | s |
| Infiltration Rate, f = | 1.18E-05 | m/s | Infiltration Rate, f = | 1.20E-05 | m/s | Infiltration Rate, f = | 8.71E-06 | 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.

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

Input by AH 24/03/2021 Checked by CAY 13/05/2021

Contract No. F181386

| | | | | |
|---|---|---|---|---|
| FUGRO | Contract Name | AD6 Road Schemes | | Location ID |
| | Client | NNB Generation Company (SZC) Limited | | **AD6-TH301** |
| | Fugro Reference | F181386 | | |
| | Coordinates (m) | E644905.06 N264639.03 | Ground Elevation (m Datum) 18.11 | Sheet 1 of 1 |
| | Hole Type | Trial Pit | | Status: Draft |

## Equipment

| Depth From (m) | Depth To (m) | Hole Type | Date From | Date To | Equipment | Core Barrel | Core Bit | Drilling Crew | Logged By | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 3.00 | TP | 10/03/2021 | 12/03/2021 | Machine excavated : 14 tonne | | | AD+ST+LL | AD | |

## Progress

| Date (dd/mm/yyyy) | Time (hh:mm) | Hole Depth (m) | Casing Depth (m) | Water Depth (m) | Weather |
|---|---|---|---|---|---|
| 10/03/2021 | 16:00 | 0.00 | | | Cold/ Rain |
| 10/03/2021 | 17:00 | 3.00 | | Dry | |

## Rotary Details

| Depth From (m) | Depth To (m) | Flush Type | Flush Return (%) | Flush Colour | Run Time (hh:mm) |
|---|---|---|---|---|---|

## Core Details

| Depth From (m) | Depth To (m) | Diameter (mm) |
|---|---|---|

## Hole and Casing

| Depth To (m) | Hole Diameter (mm) | Depth To (m) | Casing Diameter (mm) |
|---|---|---|---|

## Chiselling / Slow Progress

| Depth From (m) | Depth To (m) | Duration (hh:mm) | Tool / Remark |
|---|---|---|---|

## Water Strike

| Strike At (m) | Rise To (m) | Time Elapsed (mins) | Casing Depth (m) | Depth Sealed (m) |
|---|---|---|---|---|

## Water Added

| Depth From (m) | Depth To (m) |
|---|---|

## Water Strike Remarks

Groundwater not encountered during excavation.

## General Remarks

1. A PAS 128 survey (Category Type B) was undertaken at setting out of positions.
2. Prior to excavation, a Cable Avoidance Tool (CAT) survey was undertaken; services were not located.
3. Soakaway testing was carried out on completion of excavation; results reported separately.
4. Dynamic Cone Penetrometer testing (DCP) was carried out adjacent to the trial pit; results reported separately.
5. As-built coordinates and level were not requested; the setting out coordinates and level obtained prior to intrusive works were used.

## Installation

| Type | Tip Depth / Distance (m) | Response Zone Top (m) | Response Zone Base (m) | Installation Date |
|---|---|---|---|---|

## Pipe

| ID | Top Depth (m) | Base Depth (m) | Diameter (mm) | Type |
|---|---|---|---|---|

## Backfill

| Depth From (m) | Depth To (m) | Backfill Material | Date |
|---|---|---|---|
| 0.00 | 3.00 | Arisings | 12/03/2021 |

## Notes

- Abbreviations and results data defined in 'Exploratory Location Records Keysheets'

| | | | | | |
|---|---|---|---|---|---|
| Checked By | CAY/KES | Elevation Datum | Ordnance Datum (Newlyn) | Grid Coordinate System | OSGB |

| | | | |
|---|---|---|---|
| FUGRO | Contract Name | AD6 Road Schemes | Location ID: **AD6-TH301** |
| | Client | NNB Generation Company (SZC) Limited | |
| | Fugro Reference | F181386 | |
| | Coordinates (m) | E644905.06 N264639.03 — Ground Elevation (m Datum) 18.11 | Sheet 1 of 1 |
| | Hole Type | Trial Pit / Trench | Status: Draft |

**Sampling and In Situ Testing**

| Depth (m) | Type | No. | Test Results |
|---|---|---|---|
| 0.00 - 0.40 | B | 1 | |
| 0.10 - 0.20 | D | 2 | |
| 0.20 | HVane | | 22 kPa (*10 kPa*) |
| 0.20 | HVane | | 26 kPa (*14 kPa*) |
| 0.20 | HVane | | 32 kPa (*12 kPa*) |
| 0.30 - 0.40 | ES | 3 | |
| 0.30 | PID | | < 0.1 ppm |
| 0.40 - 0.75 | B | 4 | |
| 0.50 - 0.60 | D | 5 | |
| 0.50 - 0.60 | ES | 6 | |
| 0.50 | PID | | < 0.1 ppm |
| 0.75 - 2.10 | B | 7 | |
| 1.00 - 1.10 | D | 8 | |
| 1.20 - 1.30 | ES | 9 | |
| 1.20 | PID | | < 0.1 ppm |
| 2.10 - 2.70 | B | 10 | |
| 2.20 - 2.30 | ES | 12 | |
| 2.20 | PID | | < 0.1 ppm |
| 2.50 - 2.60 | D | 11 | |
| 2.70 - 3.00 | B | 13 | |
| 2.80 - 2.90 | D | 14 | |
| 2.80 - 2.90 | ES | 15 | |
| 2.80 | PID | | < 0.1 ppm |

**Strata Details**

| Strata Descriptions | Depth (Thickness) (m) | Level (m Datum) |
|---|---|---|
| TOPSOIL. Brown slightly gravelly CLAY. Gravel is subangular and subrounded fine to coarse of flint. | (0.40) 0.40 | 17.71 |
| Soft brown mottled orangish brown slightly gravelly sandy CLAY. Sand is fine and medium. Gravel is subangular and subrounded fine and medium of flint. | (0.35) 0.75 | 17.36 |
| Firm yellowish brown slightly gravelly sandy CLAY. Sand is fine and medium. Gravel is angular to subrounded fine to coarse of chalk and flint. | (1.35) 2.10 | 16.01 |
| Stiff yellowish brown slightly gravelly sandy CLAY. Sand is fine. Gravel is angular to subrounded fine to coarse of chalk and flint. | (0.60) 2.70 | 15.41 |
| Yellowish brown slightly gravelly SAND. Sand is fine and medium. Gravel is angular to subrounded fine to coarse of flint and quartzite. | (0.30) 3.00 | 15.11 |
| End of Trial Pit / Trench at 3.00 m | | |

Groundwater: Water Strike | Backfill / Installation

| Notes | Pit Stability | Plan |
|---|---|---|
| - Abbreviations and results data defined on 'Notes on Exploratory Position Records' | Stable | 2.58 m × 0.65 m, 94° |

Template: FGSL/HBSI/FGSL Trial Pit.hbt/Config Fugro Rev5/05/12/2019/TS-AW | Print Date 13/05/2021

| | | | | |
|---|---|---|---|---|
| FUGRO | Contract Name | AD6 Road Schemes | | Location ID |
| | Client | NNB Generation Company (SZC) Limited | | **AD6-TH302** |
| | Fugro Reference | F181386 | | |
| | Coordinates (m) | E644859.02 N264459.89 | Ground Elevation (m Datum) 18.28 | Sheet 1 of 1 |
| | Hole Type | Trial Pit | | Status: Draft |

## Equipment

| Depth From (m) | Depth To (m) | Hole Type | Date From | Date To | Equipment | Core Barrel | Core Bit | Drilling Crew | Logged By | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 3.00 | TP | 11/03/2021 | 12/03/2021 | Machine excavated : 14 tonne | | | AD+ST+LL | AD | |

## Progress

| Date (dd/mm/yyyy) | Time (hh:mm) | Hole Depth (m) | Casing Depth (m) | Water Depth (m) | Weather |
|---|---|---|---|---|---|
| 11/03/2021 | 12:45 | 0.00 | | | Cold /Rain |
| 11/03/2021 | 13:45 | 3.00 | | Dry | |

## Rotary Details

| Depth From (m) | Depth To (m) | Flush Type | Flush Return (%) | Flush Colour | Run Time (hh:mm) |
|---|---|---|---|---|---|

## Core Details

| Depth From (m) | Depth To (m) | Diameter (mm) |
|---|---|---|

## Hole and Casing

| Depth To (m) | Hole Diameter (mm) | Depth To (m) | Casing Diameter (mm) |
|---|---|---|---|

## Chiselling / Slow Progress

| Depth From (m) | Depth To (m) | Duration (hh:mm) | Tool / Remark |
|---|---|---|---|

## Water Strike

| Strike At (m) | Rise To (m) | Time Elapsed (mins) | Casing Depth (m) | Depth Sealed (m) |
|---|---|---|---|---|

## Water Added

| Depth From (m) | Depth To (m) |
|---|---|

## Water Strike Remarks

Groundwater not encountered during excavation.

## General Remarks

1. A PAS 128 survey (Category Type B) was undertaken at setting out of positions.
2. Prior to excavation, a Cable Avoidance Tool (CAT) survey was undertaken; services were not located.
3. Soakaway testing was carried out on completion of excavation; results reported separately.
4. Dynamic Cone Penetrometer testing (DCP) was carried out adjacent to the trial pit; results reported separately.
5. As-built coordinates and level were not requested; the setting out coordinates and level obtained prior to intrusive works were used.

## Installation

| Type | Tip Depth / Distance (m) | Response Zone Top (m) | Response Zone Base (m) | Installation Date |
|---|---|---|---|---|

## Pipe

| ID | Top Depth (m) | Base Depth (m) | Diameter (mm) | Type |
|---|---|---|---|---|

## Backfill

| Depth From (m) | Depth To (m) | Backfill Material | Date |
|---|---|---|---|
| 0.00 | 3.00 | Arisings | 12/03/2021 |

## Notes

- Abbreviations and results data defined in 'Exploratory Location Records Keysheets'

| Checked By | CAY/KES | Elevation Datum | Ordnance Datum (Newlyn) | Grid Coordinate System | OSGB |
|---|---|---|---|---|---|

| | Contract Name | AD6 Road Schemes | Location ID |
|---|---|---|---|
| | Client | NNB Generation Company (SZC) Limited | **AD6-TH302** |
| | Fugro Reference | F181386 | |
| | Coordinates (m) | E644859.02 N264459.89 / Ground Elevation (m Datum) 18.28 | Sheet 1 of 1 |
| | Hole Type | Trial Pit / Trench | Status: Draft |

**Sampling and In Situ Testing**

| Depth (m) | Type | No. | Test Results |
|---|---|---|---|
| 0.00 - 0.40 | B | 1 | |
| 0.10 - 0.20 | ES | 3 | |
| 0.20 - 0.30 | D | 2 | |
| 0.20 | PID | | < 0.1 ppm |
| 0.20 | HVane | | 20 kPa (*8 kPa*) |
| 0.20 | HVane | | 22 kPa (*8 kPa*) |
| 0.20 | HVane | | 28 kPa (*12 kPa*) |
| 0.40 - 0.50 | ES | 6 | |
| 0.40 - 0.75 | B | 4 | |
| 0.40 | PID | | < 0.1 ppm |
| 0.50 - 0.60 | D | 5 | |
| 0.75 - 1.15 | B | 7 | |
| 0.90 - 1.00 | D | 8 | |
| 1.00 - 1.10 | ES | 9 | |
| 1.00 | PID | | < 0.1 ppm |
| 1.15 - 2.10 | B | 10 | |
| 1.50 - 1.60 | ES | 12 | |
| 1.50 | PID | | < 0.1 ppm |
| 1.60 - 1.70 | D | 11 | |
| 2.10 - 2.60 | B | 13 | |
| 2.20 - 2.30 | D | 14 | |
| 2.50 - 2.60 | ES | 15 | |
| 2.50 | PID | | < 0.1 ppm |
| 2.60 - 3.00 | B | 16 | |
| 2.70 - 2.80 | D | 17 | |
| 2.90 - 3.00 | ES | 18 | |
| 2.90 | PID | | < 0.1 ppm |

**Strata Details**

| Strata Descriptions | Depth (Thickness) (m) | Level (m Datum) | Legend | Water Strike | Backfill / Installation |
|---|---|---|---|---|---|
| TOPSOIL. Dark brown slightly sandy slightly gravelly CLAY with occasional rootlets and roots (<20mm x 100mm). Sand is fine and medium. Gravel is subangular and subrounded fine to coarse of flint. | (0.40) 0.40 | 17.88 | | | |
| Stiff dark brown slightly sandy slightly gravelly CLAY. Sand is fine and medium. Gravel is fine to coarse of chalk and flint. | (0.35) 0.75 | 17.53 | | | |
| Stiff dark orangish brown mottled light grey slightly sandy slightly gravelly CLAY. Sand is fine to coarse. Gravel is subangular and subrounded fine to coarse of chalk and flint. | (1.35) 2.10 | 16.18 | | | |
| Stiff brown slightly sandy slightly gravelly CLAY. Sand is fine and medium. Gravel is subangular and subrounded fine to coarse of chalk. | (0.50) 2.60 | 15.68 | | | |
| Firm becoming stiff orangish brown slightly sandy slightly gravelly CLAY. Gravel is subangular and subrounded fine to coarse of chalk. | (0.40) 3.00 | 15.28 | | | |
| End of Trial Pit / Trench at 3.00 m | | | | | |

| Notes | Pit Stability | Plan |
|---|---|---|
| - Abbreviations and results data defined on 'Notes on Exploratory Position Records' | Stable | 2.62 m; 0.78 m; 95° |

Template: FGSL/HBSI/FGSL Trial Pit.hbt/Config Fugro Rev5/05/12/2019/TS-AW

Print Date 13/05/2021

| | | | | |
|---|---|---|---|---|
| | Contract Name | AD6 Road Schemes | | Location ID: AD6-TH305 |
| | Client | NNB Generation Company (SZC) Limited | | |
| | Fugro Reference | F181386 | | |
| | Coordinates (m) | E644682.04 N263726.04 | Ground Elevation (m Datum) 14.78 | Sheet 1 of 1 |
| | Hole Type | Trial Pit | | Status: Draft |

## Equipment

| Depth From (m) | Depth To (m) | Hole Type | Date From | Date To | Equipment | Core Barrel | Core Bit | Drilling Crew | Logged By | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 3.10 | TP | 09/03/2021 | 10/03/2021 | Machine excavated : 14 tonne | | | AD+ST+LL | AD | |

## Progress

| Date (dd/mm/yyyy) | Time (hh:mm) | Hole Depth (m) | Casing Depth (m) | Water Depth (m) | Weather |
|---|---|---|---|---|---|
| 09/03/2021 | 09:45 | 0.00 | | | Dry/Sunny |
| 09/03/2021 | 11:00 | 3.10 | | Dry | |

## Rotary Details

| Depth From (m) | Depth To (m) | Flush Type | Flush Return (%) | Flush Colour | Run Time (hh:mm) |
|---|---|---|---|---|---|

## Core Details

| Depth From (m) | Depth To (m) | Diameter (mm) |
|---|---|---|

## Hole and Casing

| Depth To (m) | Hole Diameter (mm) | Depth To (m) | Casing Diameter (mm) |
|---|---|---|---|

## Chiselling / Slow Progress

| Depth From (m) | Depth To (m) | Duration (hh:mm) | Tool / Remark |
|---|---|---|---|

## Water Strike

| Strike At (m) | Rise To (m) | Time Elapsed (mins) | Casing Depth (m) | Depth Sealed (m) |
|---|---|---|---|---|

## Water Added

| Depth From (m) | Depth To (m) |
|---|---|

## Water Strike Remarks

Groundwater not encountered during excavation.

## General Remarks

1. A PAS 128 survey (Category Type B) was undertaken at setting out of positions.
2. Prior to excavation, a Cable Avoidance Tool (CAT) survey was undertaken; services were not located.
3. Hand vane tests not undertaken as material unsuitable.
4. Soakaway testing was carried out on completion of excavation; results reported separately.
5. Dynamic Cone Penetrometer testing (DCP) was carried out adjacent to the trial pit; results reported separately.
6. As-built coordinates and level were not requested; the setting out coordinates and level obtained prior to intrusive works were used.

## Installation

| Type | Tip Depth / Distance (m) | Response Zone Top (m) | Response Zone Base (m) | Installation Date |
|---|---|---|---|---|

## Pipe

| ID | Top Depth (m) | Base Depth (m) | Diameter (mm) | Type |
|---|---|---|---|---|

## Backfill

| Depth From (m) | Depth To (m) | Backfill Material | Date |
|---|---|---|---|
| 0.00 | 3.10 | Arisings | 10/03/2021 |

## Notes

- Abbreviations and results data defined in 'Exploratory Location Records Keysheets'

| Checked By | CAY/KES | Elevation Datum | Ordnance Datum (Newlyn) | Grid Coordinate System | OSGB |
|---|---|---|---|---|---|

| | | | |
|---|---|---|---|
| FUGRO | Contract Name | AD6 Road Schemes | Location ID: **AD6-TH305** |
| | Client | NNB Generation Company (SZC) Limited | |
| | Fugro Reference | F181386 | |
| | Coordinates (m) | E644682.04 N263726.04 — Ground Elevation (m Datum) 14.78 | Sheet 1 of 1 |
| | Hole Type | Trial Pit / Trench | Status: Draft |

**Sampling and In Situ Testing**

| Depth (m) | Type | No. | Test Results |
|---|---|---|---|
| 0.00 - 0.30 | B | 1 | |
| 0.10 - 0.20 | D | 2 | |
| 0.20 - 0.30 | ES | 3 | |
| 0.20 | PID | | < 0.1 ppm |
| 0.30 - 0.80 | B | 4 | |
| 0.40 - 0.50 | D | 5 | |
| 0.50 - 0.60 | ES | 6 | |
| 0.50 | PID | | < 0.1 ppm |
| 0.80 - 1.10 | B | 7 | |
| 0.90 - 1.00 | ES | 9 | |
| 0.90 | PID | | < 0.1 ppm |
| 1.00 - 1.10 | D | 8 | |
| 1.10 - 2.20 | B | 10 | |
| 1.50 - 1.60 | ES | 12 | |
| 1.50 | PID | | < 0.1 ppm |
| 1.80 - 1.90 | D | 11 | |
| 2.20 - 2.80 | B | 13 | |
| 2.40 - 2.50 | ES | 15 | |
| 2.40 | PID | | < 0.1 ppm |
| 2.50 - 2.60 | D | 14 | |

**Strata Details**

| Strata Descriptions | Depth (Thickness) (m) | Level (m Datum) | Legend | Water Strike | Backfill / Installation |
|---|---|---|---|---|---|
| TOPSOIL. Brown slightly gravelly SAND with occasional rootlets (<10mm x 50mm). Sand is fine and medium. Gravel is subangular and subrounded fine and medium of flint. [TOPSOIL] | (0.30) 0.30 | 14.48 | | | |
| Dark orangish brown slightly gravelly SAND. Sand is fine and medium. Gravel is subangular and subrounded fine to coarse of flint. | (0.50) 0.80 | 13.98 | | | |
| Orangish brown slightly gravelly to gravelly SAND with cobbles (<100mm x 120mm x 140mm). Sand is fine to coarse. Gravel is subrounded and subangular fine to coarse of chalk and flint. | (2.00) 2.80 | 11.98 | | | |
| Light creamy yellow slightly gravelly SAND. Sand is fine and medium. Gravel is subangular to rounded fine to coarse of flint and quartzite. | (0.30) 3.10 | 11.68 | | | |
| End of Trial Pit / Trench at 3.10 m | | | | | |

| Notes | Pit Stability | Plan |
|---|---|---|
| - Abbreviations and results data defined on 'Notes on Exploratory Position Records' | Stable | 2.58 m × 0.62 m, 93° |

Template: FGSL/HBSI/FGSL Trial Pit.hbt/Config Fugro Rev5/05/12/2019/TS-AW — Print Date 13/05/2021

| FUGRO | Contract Name | AD6 Road Schemes | | | Location ID |
|---|---|---|---|---|---|
| | Client | NNB Generation Company (SZC) Limited | | | **AD6-TH313** |
| | Fugro Reference | F181386 | | | |
| | Coordinates (m) | E645496.04 N263305.05 | Ground Elevation (m Datum) | 9.91 | Sheet 1 of 1 |
| | Hole Type | Trial Pit | | | Status: Draft |

## Equipment

| Depth From (m) | Depth To (m) | Hole Type | Date From | Date To | Equipment | Core Barrel | Core Bit | Drilling Crew | Logged By | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| 0.00 | 2.50 | TP | 13/03/2021 | 14/03/2021 | Machine excavated : 14 tonne | | | AD+ST+J | AD | |

## Progress

| Date (dd/mm/yyyy) | Time (hh:mm) | Hole Depth (m) | Casing Depth (m) | Water Depth (m) | Weather |
|---|---|---|---|---|---|
| 13/03/2021 | 09:45 | 0.00 | | | Cold/Dry |
| 13/03/2021 | 10:30 | 2.50 | | Dry | |

## Rotary Details

| Depth From (m) | Depth To (m) | Flush Type | Flush Return (%) | Flush Colour | Run Time (hh:mm) |
|---|---|---|---|---|---|

## Core Details

| Depth From (m) | Depth To (m) | Diameter (mm) |
|---|---|---|

## Hole and Casing

| Depth To (m) | Hole Diameter (mm) | Depth To (m) | Casing Diameter (mm) |
|---|---|---|---|

## Chiselling / Slow Progress

| Depth From (m) | Depth To (m) | Duration (hh:mm) | Tool / Remark |
|---|---|---|---|

## Water Strike

| Strike At (m) | Rise To (m) | Time Elapsed (mins) | Casing Depth (m) | Depth Sealed (m) |
|---|---|---|---|---|

## Water Added

| Depth From (m) | Depth To (m) |
|---|---|

## Water Strike Remarks

Groundwater not encountered during excavation.

## General Remarks

1. A PAS 128 survey (Category Type B) was undertaken at setting out of positions.
2. Prior to excavation, a Cable Avoidance Tool (CAT) survey was undertaken; services were not located.
3. Soakaway testing was carried out on completion of excavation; results reported separately.
4. Dynamic Cone Penetrometer testing (DCP) was carried out adjacent to the trial pit; results reported separately.
5. As-built coordinates and level were not requested; the setting out coordinates and level obtained prior to intrusive works were used.

## Installation

| Type | Tip Depth / Distance (m) | Response Zone Top (m) | Response Zone Base (m) | Installation Date |
|---|---|---|---|---|

## Pipe

| ID | Top Depth (m) | Base Depth (m) | Diameter (mm) | Type |
|---|---|---|---|---|

## Backfill

| Depth From (m) | Depth To (m) | Backfill Material | Date |
|---|---|---|---|
| 0.00 | 2.50 | Arisings | 14/03/2021 |

## Notes

- Abbreviations and results data defined in 'Exploratory Location Records Keysheets'

| Checked By | CAY/KES | Elevation Datum | Ordnance Datum (Newlyn) | Grid Coordinate System | OSGB |
|---|---|---|---|---|---|