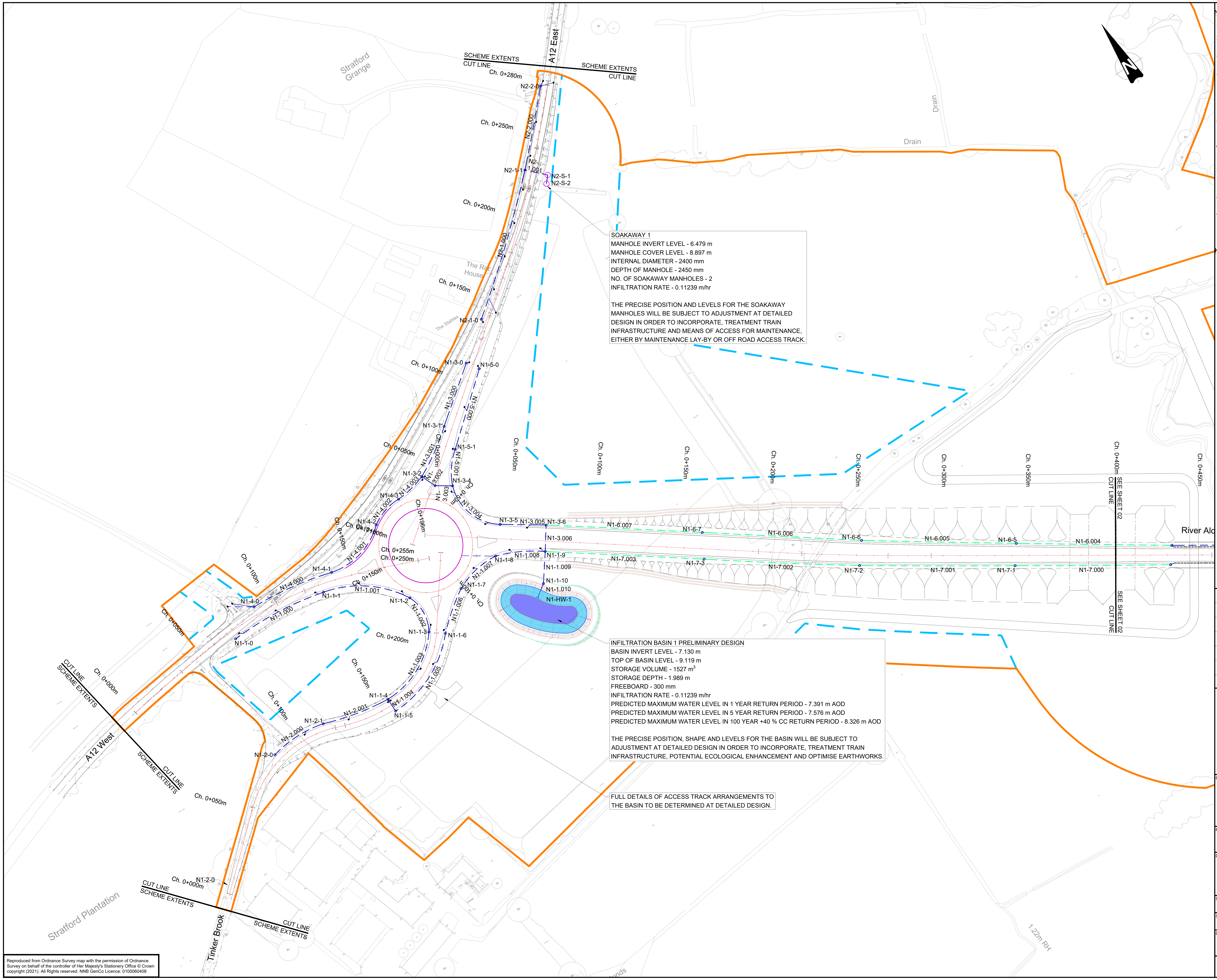
The Sizewell C Project

SZC Co.'s Response to the Secretary of State's Request for Further Information dated 18 March 2022: Appendix 3 - The Drainage Strategy Part 9 of 12

Revision: 2.0

April 2022

APPENDIX F: DRAINAGE NETWORK LAYOUT WITH HYDRAULIC MODEL LABELS



SOAKAWAY 1
MANHOLE INVERT LEVEL - 6.479 m
MANHOLE COVER LEVEL - 8.897 m
INTERNAL DIAMETER - 2400 mm
DEPTH OF MANHOLE - 2450 mm
NO. OF SOAKAWAY MANHOLES - 2
INFILTRATION RATE - 0.11239 m/hr

THE PRECISE POSITION AND LEVELS FOR THE SOAKAWAY MANHOLES WILL BE SUBJECT TO ADJUSTMENT AT DETAILED DESIGN IN ORDER TO INCORPORATE, TREATMENT TRAIN INFRASTRUCTURE AND MEANS OF ACCESS FOR MAINTENANCE, EITHER BY MAINTENANCE LAY-BY OR OFF ROAD ACCESS TRACK.

INFILTRATION BASIN 1 PRELIMINARY DESIGN
BASIN INVERT LEVEL - 7.130 m
TOP OF BASIN LEVEL - 9.119 m
STORAGE VOLUME - 1527 m^3
STORAGE DEPTH - 1.989 m
FREEBOARD - 300 mm
INFILTRATION RATE - 0.11239 m/hr
PREDICTED MAXIMUM WATER LEVEL IN 1 YEAR RETURN PERIOD - 7.391 m AOD
PREDICTED MAXIMUM WATER LEVEL IN 5 YEAR RETURN PERIOD - 7.576 m AOD
PREDICTED MAXIMUM WATER LEVEL IN 100 YEAR +40 % CC RETURN PERIOD - 8.326 m AOD

THE PRECISE POSITION, SHAPE AND LEVELS FOR THE BASIN WILL BE SUBJECT TO ADJUSTMENT AT DETAILED DESIGN IN ORDER TO INCORPORATE, TREATMENT TRAIN INFRASTRUCTURE, POTENTIAL ECOLOGICAL ENHANCEMENT AND OPTIMISE EARTHWORKS.

FULL DETAILS OF ACCESS TRACK ARRANGEMENTS TO THE BASIN TO BE DETERMINED AT DETAILED DESIGN.

Confidential

WSP Project Number: 70071213

NOTES:

1. DO NOT SCALE FROM THIS DRAWING.
2. ALL DIMENSIONS ARE SHOWN IN METRES UNLESS OTHERWISE STATED.
3. ALL LEVELS ARE IN METRES ABOVE ORDNANCE DATUM UNLESS OTHERWISE STATED.
4. THE WORKS SHALL BE CARRIED OUR IN ACCORDANCE WITH DESIGN MANUAL FOR ROADS AND BRIDGES, MANUAL OF CONTRACT DOCUMENTS FOR HIGHWAY WORKS, CIRIA C753 THE SuDS MANUAL AND SUFFOLK COUNTY COUNCIL SPECIFIC STANDARD DETAILS AND SPECIFICATIONS AS APPROPRIATE.
5. POSITION OF PROPOSED HIGHWAY GULLIES IS APPROXIMATE AND TO BE FIXED AT THE DETAILED DESIGN STAGE.
6. ALL MANHOLES SHALL BE CATCHPIT MANHOLES UNLESS OTHERWISE STATED. REFER TO MANHOLE SCHEDULES FOR MANHOLE DETAILS.
7. POLLUTION CONTROL MEASURES TO BE ADDED AT THE DETAILED DESIGN STAGE FOLLOWING RECOMMENDATIONS OF HEWRAT ASSESSMENT.
8. DRAINAGE PIPE AND MANHOLE REFERENCE RELATE TO THOSE CONTAINED IN HYDRAULIC MODEL.
9. CUTTING CUT OFF DRAINS ARE NOT SHOWN AT PRELIMINARY DESIGN STAGE AND WILL BE ADDED AT DETAILED DESIGN STAGE, IF REQUIRED.

KEY:

- PROPOSED CARRIER DRAIN
- PROPOSED CONCRETE CHANNEL WITH UNDERDRAIN CARRIER PIPE
- PROPOSED SWALE WITH UNDERDRAIN FILTER PIPE OF 150mm TO 300mm Ø
- PROPOSED COMBINED KERB DRAINAGE UNIT
- PROPOSED BRIDGE DECK UNIT
- ∘ PROPOSED CATCHPIT
- ‹ PROPOSED HEADWALL
- ○ PROPOSED SOAKAWAY MANHOLE
- •– PROPOSED GULLY WITH CONNECTION
- | PROPOSED WEIR
- ● PROPOSED INFILTRATION BASIN
- PROPOSED PERMANENT BOUNDARY
- PROPOSED DCO BOUNDARY

Rev.	Date	Description	App'd	Auth'd
P01	05/02/20	First Revision	CR	---

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Client

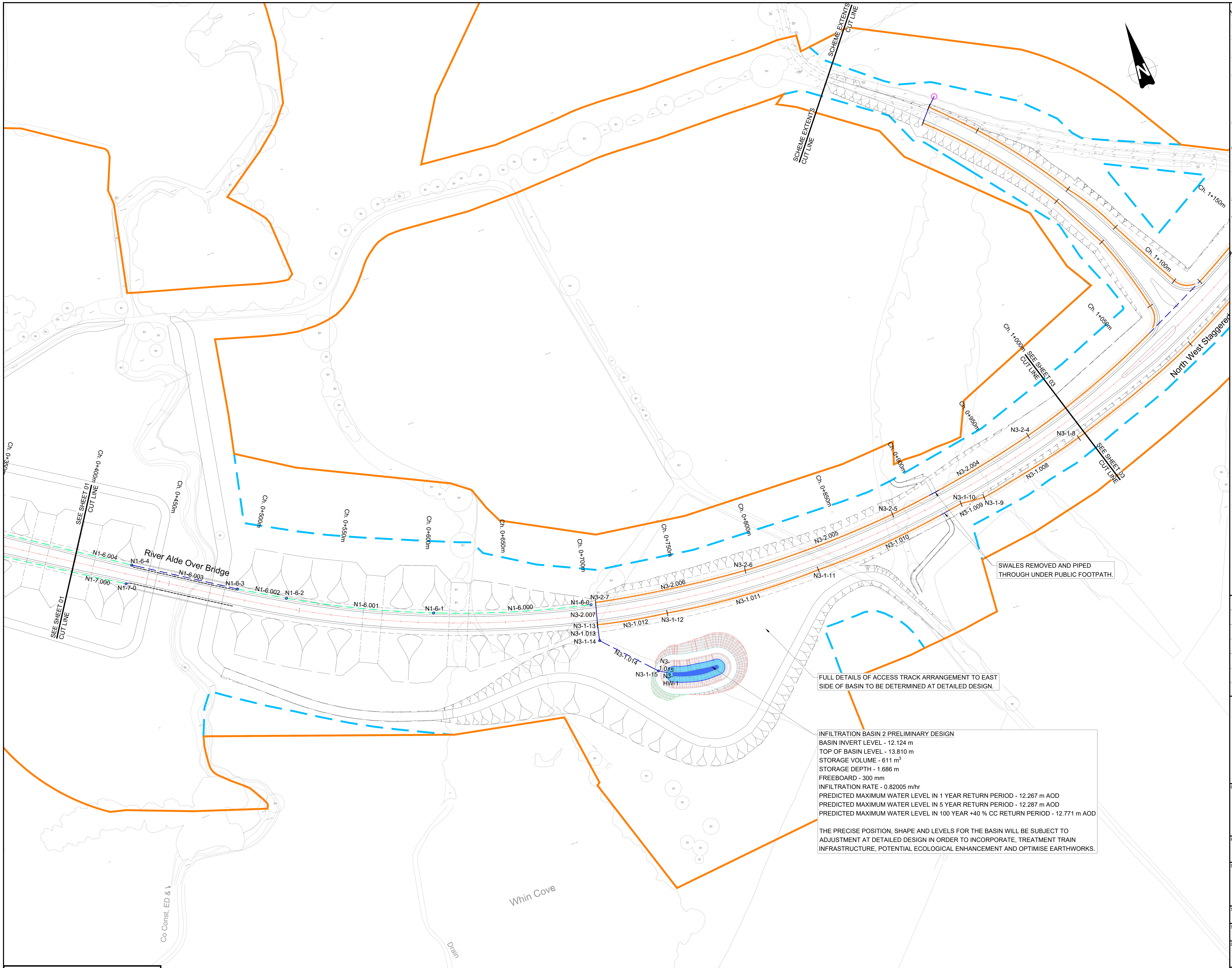
Project Title
AD3 MAJOR ROAD SCHEMES
AD0320 TWO VILLAGE BYPASS

Drawing Title
PRELIMINARY DESIGN
DRAINAGE HIGHWAYS NETWORK LAYOUT

SHEET 01

Scale @ A1	Date	Drawn	Check	Approved	Authorised
1:500	05/02/20	J.Silekar	D.Lord	C.Ritchley	---

Suitability: S2
Status: Suitable for Information
Stage: 3
Drawing Number: SZC-AD0320-WSP-TVBHDG-ZZ0000-DRW-HCD-305001
Revision: P01



SWALES REMOVED AND PIPED THROUGH UNDER PUBLIC FOOTPATH.

FULL DETAILS OF ACCESS TRACK ARRANGEMENT TO EAST SIDE OF BASIN TO BE DETERMINED AT DETAILED DESIGN.

INFILTRATION BASIN 2 PRELIMINARY DESIGN
BASIN INVERT LEVEL - 12.124 m
TOP OF BASIN LEVEL - 13.810 m
STORAGE VOLUME - 611 m^3
STORAGE DEPTH - 1.686 m
FREEBOARD - 300 mm
INFILTRATION RATE - 0.82005 m/hr
PREDICTED MAXIMUM WATER LEVEL IN 1 YEAR RETURN PERIOD - 12.267 m AOD
PREDICTED MAXIMUM WATER LEVEL IN 5 YEAR RETURN PERIOD - 12.287 m AOD
PREDICTED MAXIMUM WATER LEVEL IN 100 YEAR +40 % CC RETURN PERIOD - 12.771 m AOD

THE PRECISE POSITION, SHAPE AND LEVELS FOR THE BASIN WILL BE SUBJECT TO ADJUSTMENT AT DETAILED DESIGN IN ORDER TO INCORPORATE, TREATMENT TRAIN INFRASTRUCTURE, POTENTIAL ECOLOGICAL ENHANCEMENT AND OPTIMISE EARTHWORKS.

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KEY:

—→—	PROPOSED CARRIER DRAIN
—→—	PROPOSED CONCRETE CHANNEL WITH UNDERDRAIN CARRIER PIPE
━	PROPOSED SWALE WITH UNDERDRAIN FILTER PIPE OF 150mm TO 300mm Ø
━	PROPOSED COMBINED KERB DRAINAGE UNIT
------	PROPOSED BRIDGE DECK UNIT
○	PROPOSED CATCHPIT
‹	PROPOSED HEADWALL
○	PROPOSED SOAKAWAY MANHOLE
•—	PROPOSED GULLY WITH CONNECTION
|	PROPOSED WEIR
●	PROPOSED INFILTRATION BASIN
— —	PROPOSED PERMANENT BOUNDARY
━	PROPOSED DCO BOUNDARY

Rev.	Date	Description	App'd	Auth'd
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Client

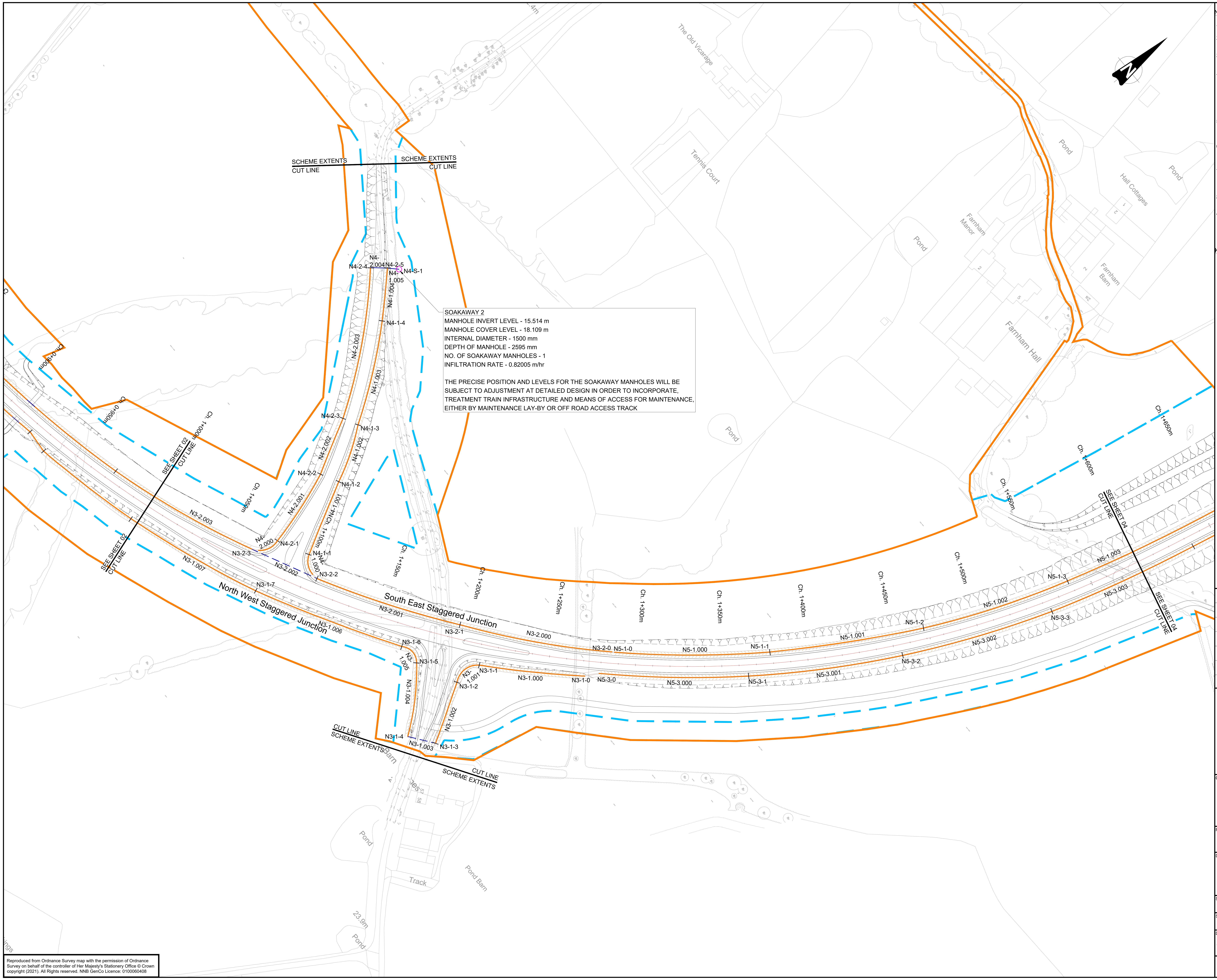
Project Title
AD3 MAJOR ROAD SCHEMES
AD0320 TWO VILLAGE BYPASS

Drawing Title
PRELIMINARY DESIGN
DRAINAGE HIGHWAYS NETWORK LAYOUT

SHEET 02

Scale @ A1	Date	Drawn	Check	Approved	Authorised
1:500	05/02/20	J.Silekar	D.Lord	C.Ritchley	---

Suitability: S2
Status: Suitable for Information
Stage: 3
Drawing Number: SZC-AD0320-WSP-TVBHDG-ZZ0000-DRW-HCD-305002
Revision: P01



SOAKAWAY 2
MANHOLE INVERT LEVEL - 15.514 m
MANHOLE COVER LEVEL - 18.109 m
INTERNAL DIAMETER - 1500 mm
DEPTH OF MANHOLE - 2595 mm
NO. OF SOAKAWAY MANHOLES - 1
INFILTRATION RATE - 0.82005 m/hr

THE PRECISE POSITION AND LEVELS FOR THE SOAKAWAY MANHOLES WILL BE SUBJECT TO ADJUSTMENT AT DETAILED DESIGN IN ORDER TO INCORPORATE, TREATMENT TRAIN INFRASTRUCTURE AND MEANS OF ACCESS FOR MAINTENANCE, EITHER BY MAINTENANCE LAY-BY OR OFF ROAD ACCESS TRACK

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━━━	PROPOSED COMBINED KERB DRAINAGE UNIT
------------	PROPOSED BRIDGE DECK UNIT
∘	PROPOSED CATCHPIT
‹	PROPOSED HEADWALL
○	PROPOSED SOAKAWAY MANHOLE
▪—	PROPOSED GULLY WITH CONNECTION
│	PROPOSED WEIR
⬬	PROPOSED INFILTRATION BASIN
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Drawing Title
PRELIMINARY DESIGN
DRAINAGE HIGHWAYS NETWORK LAYOUT

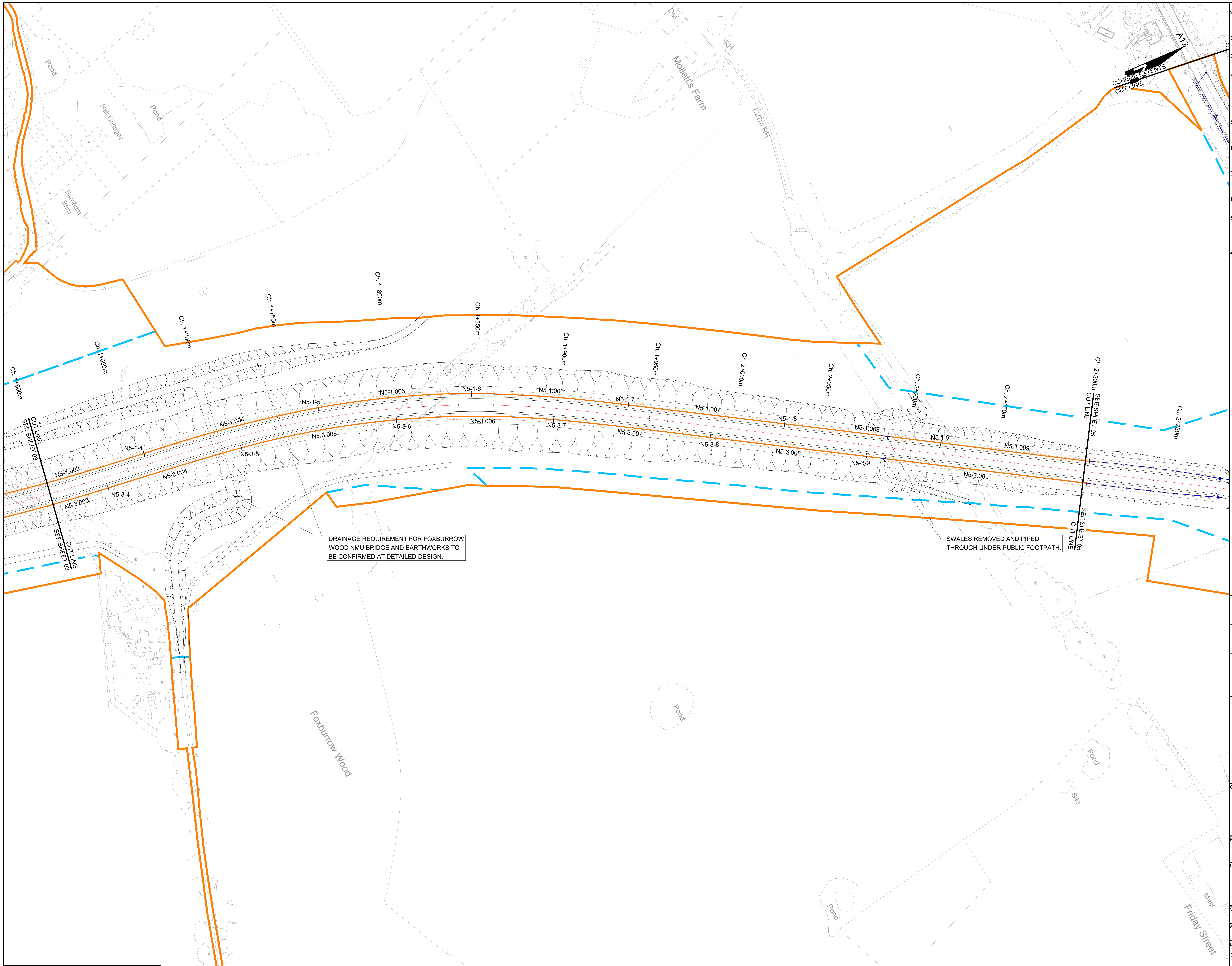
SHEET 03

Scale @ A1	Date	Drawn	Check	Approved	Authorised
1:500	05/02/20	J.Silekar	D.Lord	C.Ritchley	---

Suitability	Status	Stage
S2	Suitable for Information	3

Drawing Number	Revision
SZC-AD0320-WSP-TVBHDG-ZZ0000-DRW-HCD-305003	P01

WSP Project Number: 70071213



DRAINAGE REQUIREMENT FOR FOXBURROW WOOD NMU BRIDGE AND EARTHWORKS TO BE CONFIRMED AT DETAILED DESIGN.

SWALES REMOVED AND PIPED THROUGH UNDER PUBLIC FOOTPATH.

Confidential

WSP Project Number: 70071213

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PROPOSED PERMANENT BOUNDARY
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Client

Project Title

AD3 MAJOR ROAD SCHEMES
AD0320 TWO VILLAGE BYPASS

Drawing Title

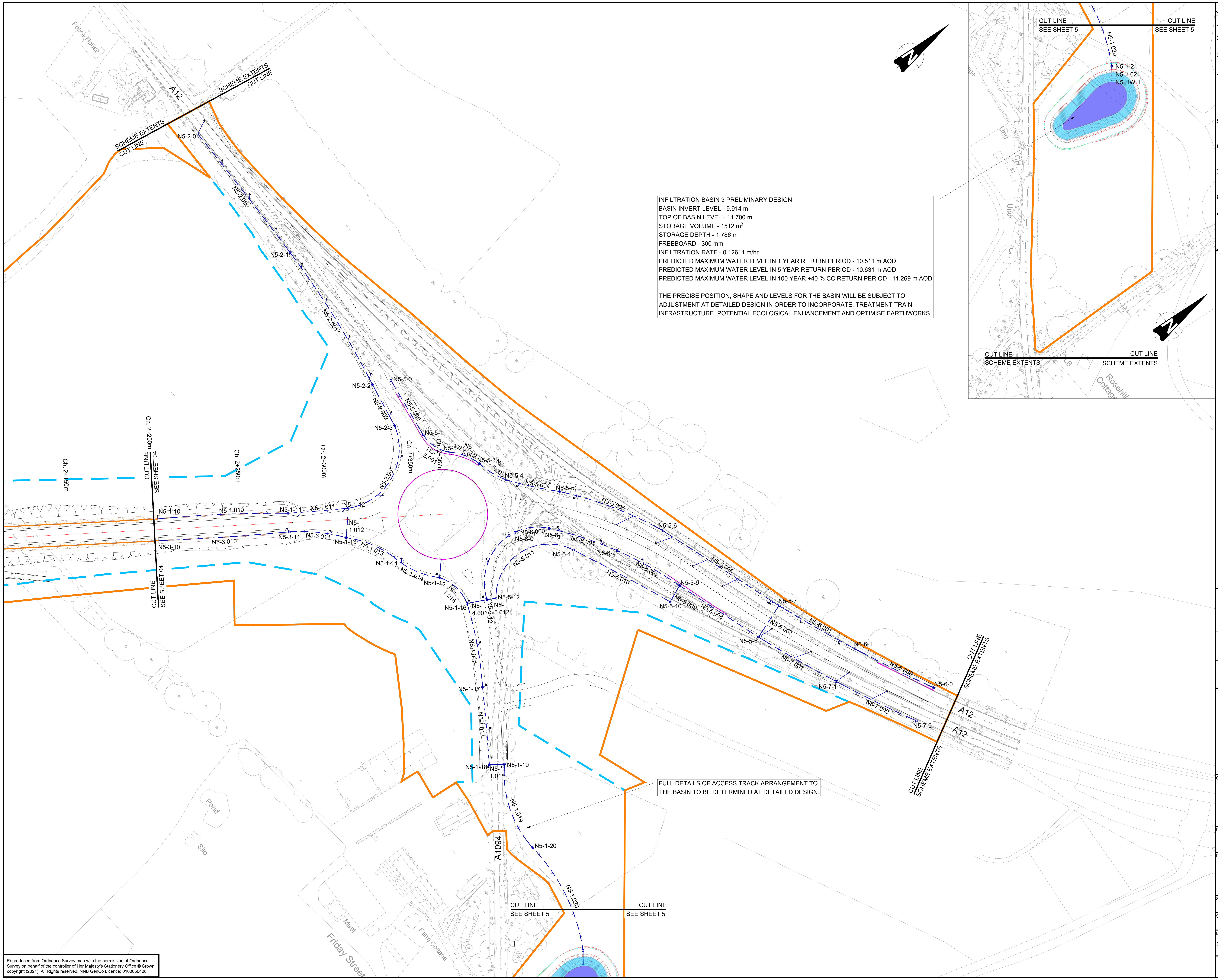
PRELIMINARY DESIGN
DRAINAGE HIGHWAYS NETWORK LAYOUT

SHEET 04

Scale @ A1	Date	Drawn	Check	Approved	Authorised
1:500	05/02/20	J.Silekar	D.Lord	C.Ritchley	---

Suitability	Status	Stage
S2	Suitable for Information	3

Drawing Number	Revision
SZC-AD0320-WSP-TVBHDG-ZZ0000-DRW-HCD-305004	P01



INFILTRATION BASIN 3 PRELIMINARY DESIGN
BASIN INVERT LEVEL - 9.914 m
TOP OF BASIN LEVEL - 11.700 m
STORAGE VOLUME - 1512 m^3
STORAGE DEPTH - 1.786 m
FREEBOARD - 300 mm
INFILTRATION RATE - 0.12611 m/hr
PREDICTED MAXIMUM WATER LEVEL IN 1 YEAR RETURN PERIOD - 10.511 m AOD
PREDICTED MAXIMUM WATER LEVEL IN 5 YEAR RETURN PERIOD - 10.631 m AOD
PREDICTED MAXIMUM WATER LEVEL IN 100 YEAR +40 % CC RETURN PERIOD - 11.269 m AOD

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FULL DETAILS OF ACCESS TRACK ARRANGEMENT TO THE BASIN TO BE DETERMINED AT DETAILED DESIGN.

Confidential

WSP Project Number: 70071213

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KEY:

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— —	PROPOSED CONCRETE CHANNEL WITH UNDERDRAIN CARRIER PIPE
▨	PROPOSED SWALE WITH UNDERDRAIN FILTER PIPE OF 150mm TO 300mm Ø
—	PROPOSED COMBINED KERB DRAINAGE UNIT
- - - -	PROPOSED BRIDGE DECK UNIT
○	PROPOSED CATCHPIT
‹	PROPOSED HEADWALL
○	PROPOSED SOAKAWAY MANHOLE
•–	PROPOSED GULLY WITH CONNECTION
|	PROPOSED WEIR
●	PROPOSED INFILTRATION BASIN
— —	PROPOSED PERMANENT BOUNDARY
—	PROPOSED DCO BOUNDARY

Rev.	Date	Description	App'd	Auth'd
P01	05/02/20	First Revision	CR	---

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Client

SZC edf ENERGY CGN

Project Title
AD3 MAJOR ROAD SCHEMES
AD0320 TWO VILLAGE BYPASS

Drawing Title
PRELIMINARY DESIGN
DRAINAGE HIGHWAYS NETWORK LAYOUT

SHEET 05

Scale @ A1	Date	Drawn	Check	Approved	Authorised
1:500	05/02/20	J.Silekar	D.Lord	C.Ritchley	---

Suitability: S2
Status: Suitable for Information
Stage: 3
Drawing Number: SZC-AD0320-WSP-TVBHDG-ZZ0000-DRW-HCD-305005
Revision: P01

APPENDIX G: RECORD OF SCC COMMENTS AND SZC ACTIONS

SCC Comments at Rev02	
The general principles of surface water drainage for the road schemes (Two Village Bypass, Sizewell Link Road and Yoxford Roundabout) and agreed between SZC Co and SCC.	
Plate 10 Infiltration rate stated: 0.11239m/hr (112.39mm/hr) Relevant test in Appendix A: TVTH201 Result of TVTH201: 60.12mm/hr Plate 14 Infiltration rate stated: 0.82005m/hr (820.05mm/hr) Relevant test in Appendix A: TVTH212A Result of TVTH212A: 363.6mm/hr Plate 16 Infiltration rate stated: 0.12611m/hr (126.11mm/hr) Relevant test in Appendix A: TVTH211 Result of TVTH211: 149.76mm/hr	The values in the Plates are those applicable at preliminary design. The change to the more conservative Fugro infiltration rates is confirmed in10.1.5
SCC Comments at Rev03	
8.1.4 – As per email on 21/02/2022 @ 13:44, when road is at grade or in cutting, shallow swales not required. Also, this isn't reflected in calculations, thus any storage in swale could be overestimated. 8.1.18 – Infiltration through swales has not been evidenced through the results of infiltration	Application of DMRB would imply the requirement for VRS if depth of swale is increased. If SCC as adopting authority is happy to remove the VRS requirement this could be done as a departure from standards. Infiltration viability is proven at the receiving infiltration basins.

testing along the corridor. Assuming that infiltration is available along the entire corridor at the same rate as achieved at the location of the proposed infiltration basins is not a conservative approach and is likely to underestimate the required land take of the proposed infiltration basins. Worth noting that BGS mapping identifies Lowestoft Formation along a significant part of the proposed route, where infiltration should not be expected. 10.1.3 – The lower values, which SCC agreed would be used, as stated, should be used at this stage of design development	The infiltration test results do show that for the portion of TVBP which is in cutting to the north of Hill Farm Road, infiltration is not viable. However, the swale/filter drain has a falling gradient towards the A12 northeast roundabout and hence runoff will be conveyed to basin 2 The hydraulic modelling results provided in Appendix C do use the lower Fugro infiltration rates.
Appendix A – It's not possible to use the plans that contain the locations of test results without context of the proposed scheme overlaid	Plan Added
Network 1 Infiltration rate used of 60.12mm/hr. This conflicts with Plate 10 but uses the right infiltration rate as far as SCC are concerned. Basin levels and modelled flood levels are different to that contained in Plate 10. Infiltration basin DS/PN is N1-1.010 with a weir overflow of 8.622m. Given this is an infiltration basin, I wouldn't expect to see any flow through this pipe but during 1:100+40% it is discharging at 12l/s. This is not in accordance with the proposed drainage strategy and does not represent the required attenuation volumes. In addition to the above, despite the offsite discharge, there is a cumulative flood volume of 96.661m3. This is a significant volume and I don't expect @Steve Merry would be content with this being retained on the road. Given the location next to the River Alde, it's likely this water would find its way to the river, thus increasing offsite flood risk, which is not something SCC can support.	See final comment. All Network Models will be updated at detailed design
Network 2 No comments as subject to change as per 8.1.10 of the report. Not ideal but I agree with the principles outlined in 8.1.10 and given the small area I'm content to leave this until detailed design	See final comment. All Network Models will be updated at detailed design

Network 3 Infiltration rate of 522mm/hr used. This conflicts with both Plate 14 and the results of TVTH212A. Where has this infiltration rate come from? Below comments are based on this aspect being addressed Basin levels and modelled flood levels are different to that contained in the relevant plate. This network model is very detailed, including losses through complex structures (swale/filter drains). Notwithstanding the comments made above in response to 8.1.18, if you're going to have a model with this much detail, you'll need to support it with plans and sections, this would include catchment extent, drainage strategy plans, swale and basin plans and sections. Without this information, we can't accept upstream losses. Whilst you haven't undertaken infiltration testing along the route away from proposed infiltration basins, I note there are trial pits. I would suggest there's some form of assessment of soil type in these trial pits, compared against that found at the infiltration test location to determine if the soil type is the same and therefore the infiltration rate achieved at TVTH212A may be suitable to be used elsewhere. But again, highlighting the point made in response to 8.1.18, this is not a conservative approach. Swale base infiltration rate wouldn't be natural soils so not correct to use same infiltration rate as for the filter drain. Any swales sections and plans should also reflect the use of V-notch weirs, which are also modelled At this stage we don't have the GI information to be modelling upstream losses to this extent, hence we usually only require source control calculations as this would demonstrate a worst-case scenario for attenuation requirements based on the limited GI undertaken to date. The current approach taken isn't very conservative in terms of attenuation volumes required and there's no justification for such an approach Cumulative flood volume of 44.46m3 for 1:100+40%. See comments on flood volumes in Network 1	See final comment. All Network Models will be updated at detailed design
Network 4 No comment as modelled network is not what is proposed	See final comment. All Network Models will be updated at detailed design

Network 5 Infiltration rate of 117mm/hr used. This conflicts with both Plate 16 and the results of TVTH 211. Where has this infiltration rate come from? Below comments are based on this aspect being addressed Technical comments similar to those as for Network 3 as similar level of detail provided DS/PN showing a pipe flow of 14.3l/s for 1:100+40%. Same issue as for Network 1 as this looks to be providing a positive discharge offsite and therefore not modelling as an infiltration only system Cumulative flood volumes of 86.37m3 for 1:100+40%. See comments on flood volumes in Network 1	See final comment. All Network Models will be updated at detailed design
Appendix D Confirm that invert levels, top levels, 1:100+40% levels and freeboard levels align with current calcs	
Email from SCC 23/02/2022 .Just picking up on the ‘comment response’ and ‘actions’ in relation to my comments on Two Village Bypass. It doesn’t look like you’re proposing any further work on this until detailed design? Slightly concerning from an SCC perspective as the infiltration rates used in design are not supported by the testing you have provided in the report and therefore the rates you use have no justification whatsoever. This is a fundamental point which we’ve been raising for quite some time now and I thought we’d bottomed out, see attached. SCC will not agree to a drainage strategy for TVBP which uses infiltration rates that are not evidenced, this is not a conservative design approach, especially when considered alongside comments RE modelled losses from infiltration through swales that won’t infiltrate as modelled. Could you please amend your comment responses and actions accordingly?	On 24/02/2022 Derek Lord and Matt Williams agreed that the Network 1, 3 and 5 infiltration basins are to be sized using source control with Fugro determined infiltration rates from Appendix A. This will give a highly conservative size since there will be no loss of runoff by infiltration upstream of the basins. Proof of basin fit within the red line boundary will validate design at this stage. Hydraulic model comments in rows above will then be addressed at detailed design.

ANNEX 2A.12: GREEN RAIL ROUTE DRAINAGE DESIGN NOTE

CONTENTS

PLATES

APPENDICES

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 SZC Co. has since undertaken work to validate and develop the design of the green rail route that was submitted as part of the Application. This document forms one of a series of design validation and evolution documents forming part of the **Drainage Strategy** (Doc. Ref. 6.3 2A(D)/10.14) submitted at Deadline 10.

1.1.3 The green rail route forms one of the Associated Developments (AD) which are required to mitigate traffic impacts arising from the Main Development Site construction. Its function is to provide a temporary railway facility which will be used for the delivery of bulky construction materials such as aggregates, cement, reinforcement steel and containerised goods to site. This will reduce the heavy goods vehicle traffic that is required to use local roads.

1.1.4 Full details of the green rail route facilities are contained in **Volume 9 Rail Chapter 2 Description of Rail** [APP-541] and are described in summary below.

1.1.5 The green rail route will be a single-track line commencing at a junction with the NR Leiston branch line located approximately 500 m to the east of Saxmundham Road level crossing. It will run over a distance of approximately 1.8 km across open country to the east side of Abbey Road where it will enter the Main Development Site. A total of four watercourses will be crossed and will require to be culverted beneath the railway. There are also level crossings which will be provided at B1122 Abbey Road and Buckleswood Road.

1.1.6 The green rail route will be constructed at approximately ground level between the junction and Buckleswood Road rising to a high point at 300 m from the junction and then falling continuously to B1122 Abbey Road crossing at a level of 9.5 mAOD. In order to provide a suitable operational track gradient, the line is largely in cutting between Buckleswood Road and Abbey Road.

1.1.7 The green rail route will continue to the east of Abbey Road running over a further distance of approximately 2.7 km within the Main Development Site.

There are no watercourse crossings. There will be one level crossing for an internal site road.

1.1.8 The gradient rises to the east of Abbey Road to a high point at approximately 510 m thus creating a low point at Abbey Road. Further to the east there is a fall in level to the TCA surface platform level on which the railway is horizontal.

1.1.9 The green rail route will not have 100% impermeable surface but will be less permeable than the current greenfield state with an assumption of 50% applied in design. It is assumed the green rail route will also cause change in existing overland flow routes for surface water runoff.

1.1.10 In accordance with Network Rail (NR) requirements railway drainage is required to keep the track bed and track support system dry such that it maintains its strength. There are 3 main causes of track bed becoming wet (wet bed). These are:

- Lack of infiltration rate which prevents rainwater from infiltrating to ground;
- Overland flow from adjacent areas onto the track; and
- High groundwater levels reaching the track bed surface.

1.1.11 Where none of these causes apply and the track bed remains dry at all times NR standards state that no drainage is required.

1.1.12 At drainage strategy concept stage, based on available infiltration test results which are variable along the green rail route and given evidence of overland flow paths crossing the line, a conservative approach has been taken. It is assumed that track drainage will be required.

1.1.13 The extent of required track drainage will be reviewed and updated as design proceeds through GRIP4 and 5 design stages.

1.1.14 The green rail route will remain in use and operation until commissioning of the SZC power station. Once no longer required, it will be removed over its full extent and the land returned to its current use.

2 PURPOSE

2.1.1 The **Outline Drainage Strategy** [REP2-033 page 93] identified at concept level the proposed drainage approach required for the effective removal of surface water runoff from the proposed green rail route together with its treatment and disposal.

2.1.2 The proposed drainage infrastructure was described in the concept drainage design submitted as part of the Application. This concept design was based on data and information available at that time. The design was supported by the submission of the **Rail Flood Risk Assessment** (FRA) [APP-143], the **Main Development Site FRA** [APP-093] and **Main Development Site FRA Addendum** [AS-157].

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

2.1.4 The purpose of this technical note is to provide details of data which validate the **Drainage Strategy** (Doc. Ref. 6.3 2A(D)/10.14) submitted at Deadline 10, provide a description of how the proposed concept drainage infrastructure is developing and evolving, and to demonstrate that its design continues to provide for the effective and satisfactory drainage of the green rail route that does not cause an unacceptable adverse impact on the water environment, both in terms of flood risk and pollution.

2.1.5 This technical note is updated at revision 02 to address comments raised by SCC following their review of revision 01 and the Green Rail Route - Updated DCO Drainage Strategy Statement February 2022 document. The comments are shown in **Appendix A**.

2.1.6 Because the Green Rail Route - Updated DCO Drainage Strategy Statement February 2022 document was intended to provide an update on the Drainage Strategy described in revision 01, it is included as **Appendix B.** This document contains relevant data so rather than repeat, where necessary references are made to Appendix B appendices in the body of this report.

2.1.7 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.

2.1.8

3 GREEN RAIL ROUTE: BASELINE DRAINAGE ARRANGEMENTS

3.1.1 The extent of the green rail route from the NR junction to Abbey Road is currently unpaved agricultural land. Buckleswood Road passes through this land and a level crossing will be provided. As shown in **Plate 2** an extract from DCO drawing 5.9 Rail Flood Risk Assessment Figure 4 [APP-144] there are limited locations with predicted surface water flood risk.

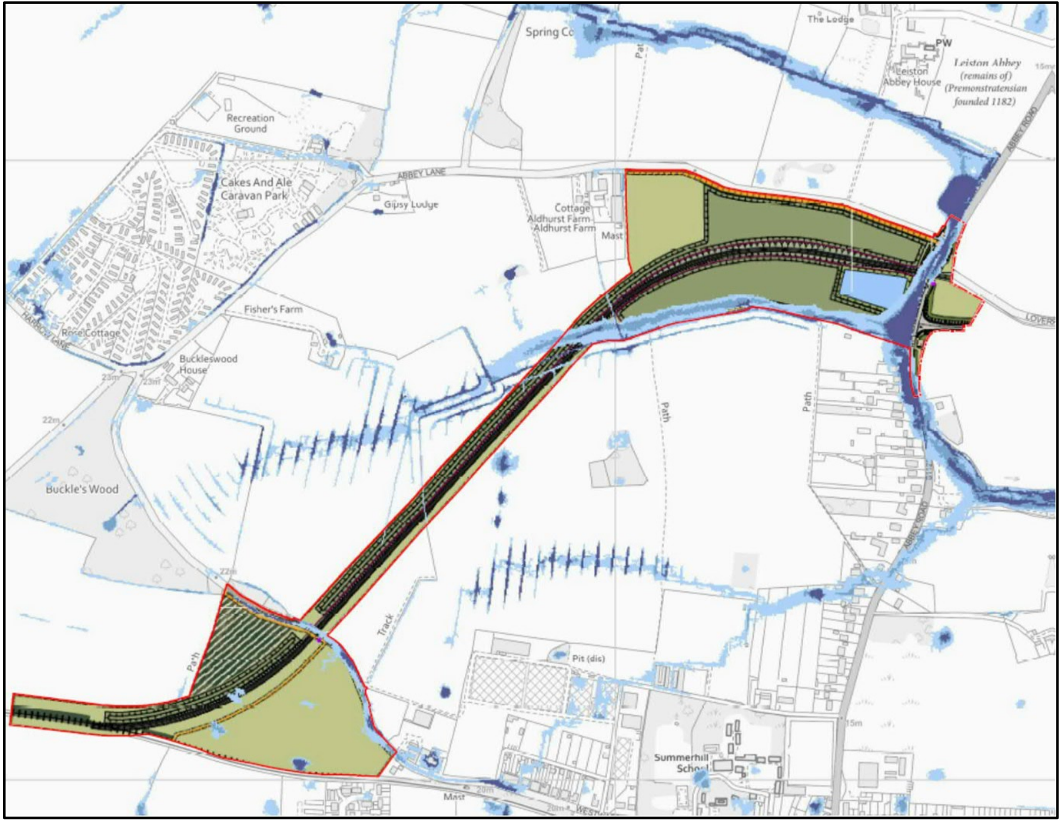
3.1.2 Local ditches are located on either side of Buckleswood Road. The ditches drain the road and, to an extent, the adjacent land. The green rail route level crossing will be required to accommodate these ditches which will need to be culverted.

3.1.3 The upper reaches of the Leiston Drain run parallel to Abbey Road in a field and close to the boundary hedge. Abbey Road highway drainage currently discharges into the Leiston Drain. The green rail route will cross both the Leiston Drain and the road with a level crossing required. Because the road level is required to be raised to accommodate the level crossing, it is necessary to modify the highway drainage.

3.1.4 As a result of the undertaking of the **Rail FRA** [APP-143] the presence of a surface water overland flow path and potential watercourse has been identified to the north of Leiston. The watercourse has been confirmed as a minor ditch at the point where it discharges into Leiston Drain. The flow will require to be accommodated with provision of a culvert or other drainage at the crossing point.

3.1.5 The risk of surface water flooding of the green rail route is shown in **Plate 1**.

Plate 1: Green rail route risk of surface water flooding



3.1.6 To the east of Abbey Road within the Main Development Site TCA the land to be occupied by the green rail route is currently unpaved and either in agricultural use or woodland. There are no known watercourses or areas shown on the EA surface water flood map showing flood risk. It is assumed that most rainwater will currently infiltrate to ground. To the extent that rainwater does not infiltrate, overland flow passes south and discharges into the Leiston Drain or its tributaries.

3.1.7 Since the whole area will be occupied by the TCA with the creation of construction platforms, any existing drainage will be replaced.

4 GROUND INVESTIGATION AND INFILTRATION TESTING RESULTS

4.1.1 This section describes the ground investigation undertaken prior to and informing the Application.

4.1.2 SZC Co. has undertaken ground investigation within the Main Development Site and this includes infiltration testing, some by borehole and some by BRE365 trial pit testing **(Ref. 1)**. Whilst the infiltration test results are variable across the TCA site, they are sufficient for use in development of the concept drainage and as noted in 3.1.9 and 3.1.10 above, average infiltration rates have been used in development of drainage infrastructure.

4.1.3 Infiltration testing along the line of green rail route between the NR junction and Abbey Road indicate results to the north of Leiston to be good and above the value of 1.4×10^{-6} m/s considered by SCC to be the minimum viable for infiltration to ground.

4.1.4 There is only one test result currently available near to Buckleswood Road level crossing and one near to the NR junction. These are both less than the value of 1.4×10^{-6} m/s. This would indicate that removal of runoff by infiltration alone is not viable.

4.1.5 Drainage records provided by NR show no recorded track drainage on the existing branch in proximity to the proposed green rail route junction. Physical observation of the branch at Saxmundham Road level crossing also confirms no obvious railway drainage. This would indicate that the existing branch is drained by infiltration.

4.1.6 Following completion of geotechnical investigation for the existing branch line and site walk, it can be confirmed that there is a sand bed drainage layer beneath the ballast, but no track drainage pipes. There is no indication of drainage problems. It is assumed that runoff which passes through the ballast to the sand bed layer migrates with the falling gradients and either ultimately finds its way to existing watercourses or does infiltrate slowly to ground.

4.1.7 The drainage design strategy at revision 01 was updated to reflect the outputs of these initial ground investigation and infiltration testing results.

4.1.8 Subsequently SZC Co. has continued to undertake ground investigation with infiltration testing to BRE365 standards. The results of this additional tests are discussed in **Appendix B Section 5** and shown in **Appendix B** of that document.

4.1.9 In summary, based on the overall infiltration data now available and following discussions with SCC, it is agreed that the Drainage Strategy will assume that no infiltration is achievable and that all runoff must be removed to an attenuation basin adjacent to Abbey Road from which it will be pumped at an agreed rate to and approved outfall.

5 DESCRIPTION OF DCO DRAINAGE CONCEPT DESIGN

5.1.1 This section describes the concept drainage design submitted as part of the Application.

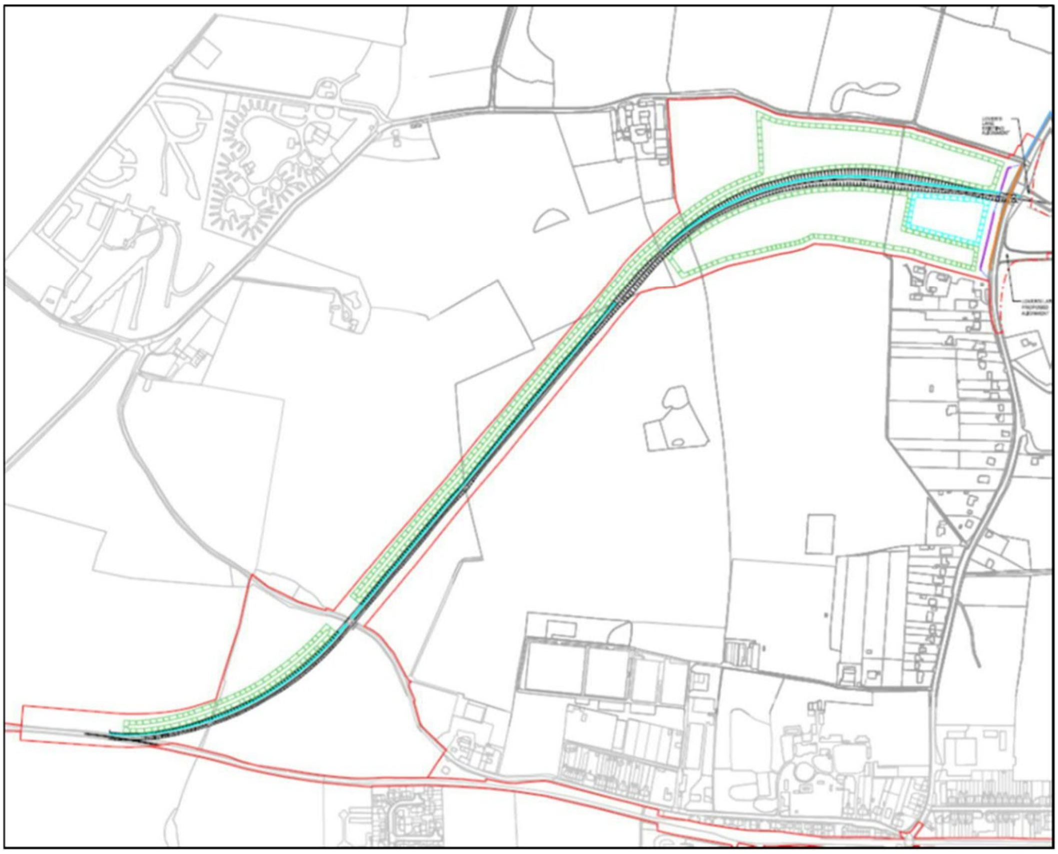
5.1.2 The design approach was conservative to provide assurance on land take and accommodating drainage infrastructure.

5.1.3 For the green rail route from the junction with the NR branch line to the B1122 Abbey Road level crossing, as described in in **Volume 9 Rail Chapter 2 Description of Rail** [APP-541 page 84], drainage in the form of a trackside swale was proposed. The swales would be located on the north side of the track and would be approximately 1 m wide and have a depth of 200 mm below the track bed. Based on ground investigation data, the assumed average infiltration rate within this catchment was 0.112 m/hr. If necessary additional temporary storage capacity would be provided by either providing a filter drain below the swale or increasing the width of the swale within the Order Limits.

5.1.4 Given the general fall in gradient from the junction to B1122 Abbey Road, any runoff from the railway or intercepted overland flow which does not infiltrate to ground would flow to Abbey Road. In order to remove such flow, the swale would discharge into an infiltration basin. The basin was located to the west of the Leiston Drain.

5.1.5 The proposed drainage layout and route of the green rail route was shown in **Plate 2**, an extract from DCO drawing **Chapter 2 Description of Rail Figure 2.6** [APP-543]. The full figure is shown in **Appendix A** of **Appendix B** of this report.

Plate 2: Green rail route from Leiston branch junction to B1122 Abbey Road showing concept drainage infrastructure



5.1.6 To the east of Abbey Road, the green rail route enters the Main Development Site TCA. The railway layout is shown in DCO drawings **Rail Plans For Approval** [APP-016]. The set of Figures is shown in **Appendix A of Appendix B** of this report.

5.1.7 Railway drainage was proposed in the form of filter drains located to the side of the track and sidings. These would collect railway runoff and effectively drain the railway. The filter drains would connect to the site construction surface water drainage network which would provide an outfall to remove excess runoff which does not infiltrate the ground.

5.1.8 The railway is located in two drainage catchments, although a section to the immediate east of Abbey Road drains back to the infiltration basin to the west of Abbey Road.

5.1.9 The first catchment would outfall to Water Management Zone (WMZ) 6 located to the south of Lovers Lane and outside of the TCA. The assumed average infiltration rate within this catchment was 0.112 m/hr. WMZ6 was designed as an infiltration basin but with an exceedance rainfall overflow facility which would discharge to Leiston Drain. As a result, other than in an exceedance rainfall event, all railway drainage would be effectively removed by infiltration with no discharge to local watercourse.

5.1.10 The second catchment would outfall to WMZ2 located within the TCA. The assumed average infiltration rate within this catchment was 0.0616 m/hr. WMZ2 was designed as an infiltration basin but with an exceedance rainfall overflow facility which would discharge via the Combined Drainage Outfall to sea.

6 UPDATED SURFACE WATER DRAINAGE DESIGN STRATEGY – GREEN RAIL ROUTE WEST OF ABBEY ROAD

6.1.1 The surface water arrangements for removal currently remain, in principle, as described in document **Environmental Statement Volume 9 Rail Chapter 2 Description of Rail** [APP-541] and shown in **Chapter 2 "Description of Rail Figure 2.6"** [APP-543], an extract of which is shown in **Plate 2** of this report.

6.1.2 The design assumes that the green rail route catchment area is 50% impermeable and that the average infiltration rate is 0.112 m/hr.

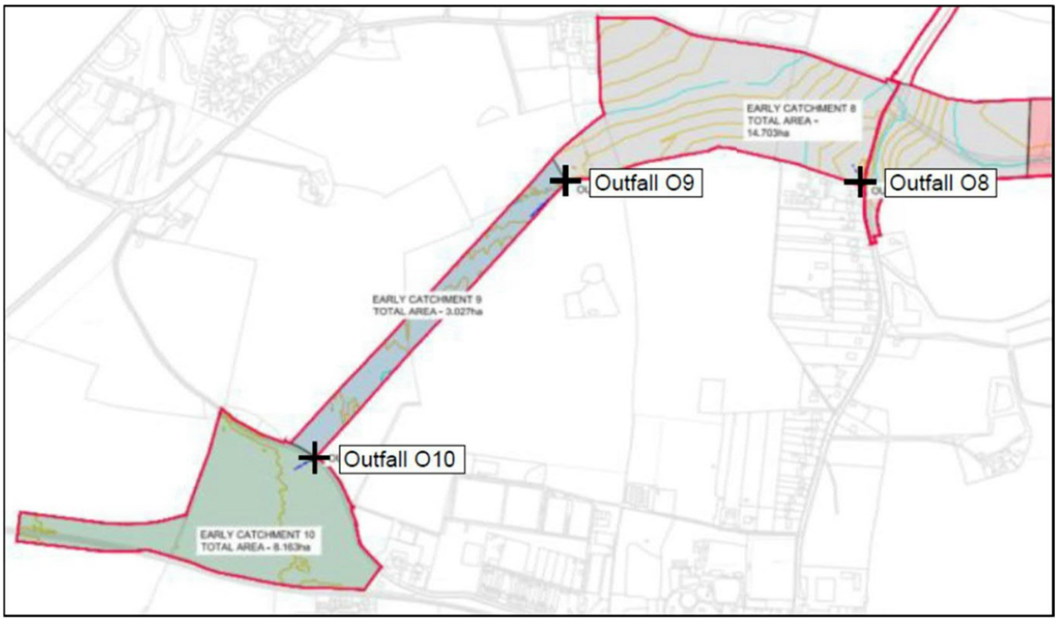
6.1.3 The design provides for a swale with filter drain which removes runoff from the track and adjacent strip together with cutting sides. In addition, cut off drains are provided at the top of cuttings to limit overland flow from adjacent land. The cut off drain's outfall into the trackside swale where the cuttings terminate.

6.1.4 The concept design included for an infiltration basin to be provided to the west of Abbey Road given the assumption that removal of water to maintain a dry trackbed by infiltration alone would not be viable. Basic hydraulic modelling which includes the average infiltration rate of 0.112 m/hr validates this assumption and the requirement for additional infrastructure.

6.1.5 Since the route of the green rail route crosses the local watercourses at Buckleswood Road and to the north of Leiston, these would form a barrier to the trackside swale/filter drain and could potentially prevent a gravity outfall to the Abbey Road infiltration basin. However, the watercourses do have the potential to provide an alternative outfall. Accordingly, the concept design has been updated to include for discharge to these watercourses.

6.1.6 The proposed outfalls are shown in **Plate 3**.

Plate 3: Green rail route proposed outfalls west of Abbey Road



6.1.7 In order to minimise impact, as a design assumption, it is proposed that discharge rate into each of these watercourses will be limited to 5 l/s by installation of a flow control device. In due course the rate will be subject to refinement and agreement through the environmental permitting regime and supported by the detailed design, subsequent to the DCO.

6.1.8 It is noted that at approximately 250 m from NR junction the green rail route level rises to a high point with an increase in level of 2 m before falling towards Outfall 010 at Buckleswood Road. As part of detailed design development two drainage options will be considered.

6.1.9 The length of green rail route which falls towards the junction may be treated as a separate catchment with all runoff discharging by infiltration to ground. Notwithstanding the results of our initial testing, since the NR branch does not have track drainage it is very likely that infiltration will work for this section of the green rail route. However, we will continue to undertake further investigations and additional underground storage can be provided if necessary.

6.1.10 Alternatively, it will be possible for the filter drain to be laid at greater depth through the high point with all runoff that does not infiltrate discharges to outfall 010.

6.1.11 The actual bed levels for the Buckleswood Road watercourses are not currently available but they will be subject to full topographic survey to inform the preliminary design. It is assumed that the depth of the watercourse on the south side of Buckleswood will be sufficient to allow connection of the swale and filter drain. However, if it is too shallow then it does not provide an obstruction and the filter drain can be extended under Buckleswood Road and discharge will pass to outfall 09.

6.1.12 It is noted that for flow control devices to work as intended they should not be subject to surcharge. In this case the 1 in 100 year return period rainfall event runoff with no allowance for infiltration is estimated to be 5.8 l/s therefore if the flow control device fails and does not limit the flow to 5 l/s this will not result in substantial additional flow to the watercourse.

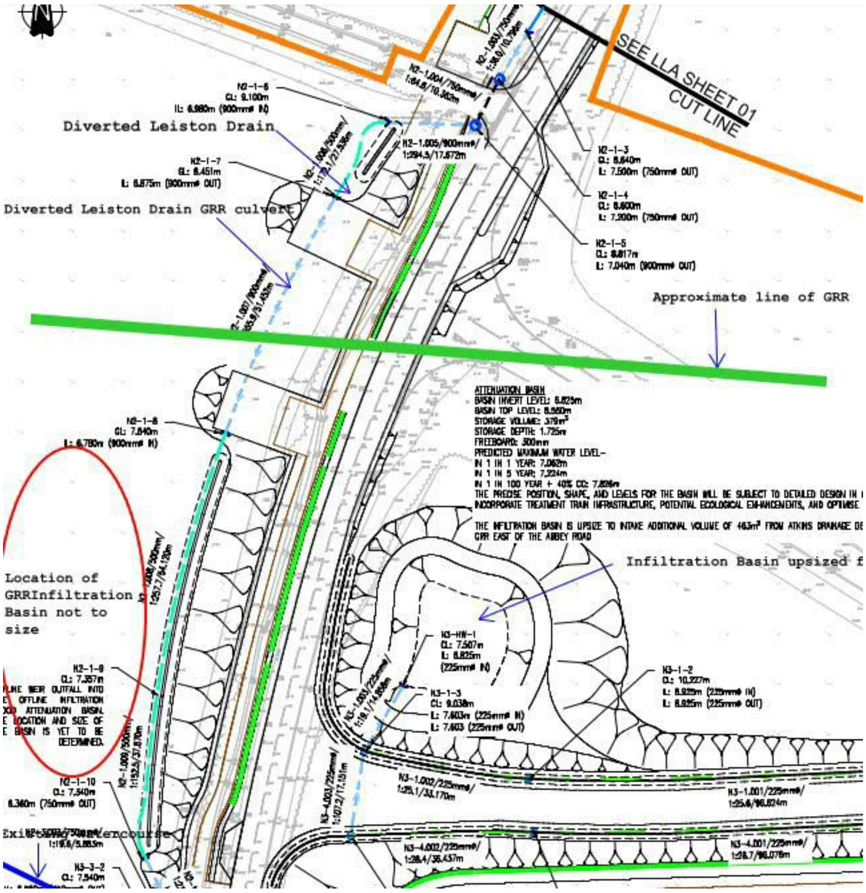
6.1.13 At outfall 09, the green rail route will cross the line of the local watercourse or the upstream flow path that feeds the watercourse. The green rail route is in cutting at this location and may be at a lower level than the watercourse. If this is the case then then the watercourse will have to be diverted on the upstream side of the green rail route and run in parallel to its north side towards Abbey Road until such time as either it can cross under the green rail route or until it reaches the Leiston Drain. If the cutting is upstream of the watercourse, then the cut off drain will intercept and divert the overland flow.

6.1.14 The green rail route outfall 09 is only available if the watercourse is at a lower level than the swale/filter drain at the crossing point. If outfall 09 is not achievable all runoff that does not infiltrate to ground will discharge to the infiltration basin at the low point to the west of Leiston Drain. It should be noted that as shown in **Plate 2** the concept design submitted with the DCO did assume all runoff that does not infiltrate via the swale/filter drains would discharge into this basin.

6.1.15 Outfall 08 with discharge to Leiston Drain at a controlled flow rate of 5 l/s is now proposed because the concept hydraulic modelling indicates that there is insufficient space to provide a basin of adequate size. This is in part due to additional constraints on land availability.

6.1.16 The allocated space for the basin did not take account of the local watercourse downstream of outfall 08 which must remain in place. The basin must also be coordinated with the Leiston Drain which is diverted to the west in order to accommodate the new Bridleway 19 which will run parallel to Abbey Road. The position of the diverted Leiston Drain with its green rail route culvert is shown in **Plate 4** which is an extract from AD6 Leiston Adoptable Highways drawing SZC-AD0600-WSP-LLAHDG-ZZ0000-DRW-HCD-305002.

Plate 4: Green rail route infiltration basin constraints west of Abbey Road



6.1.17 As shown in in **Plate 5** the proposed location of the basin is also close to an area of surface water flood risk.

Plate 5: Environment Agency predicted surface water flood risk extent

6.1.18 The predicted flood risk has been confirmed by SCC. The infiltration basin will therefore be located such that it is outside the surface water flooding footprint.

6.1.19 The predicted flooding could also constrain free outfall into the Leiston Drain and thus the proposed outfall 08 flow control device. If, following design development, it is confirmed that a discharge to Leiston Drain is required it may be necessary for it to be either at high level or pumped.

7 UPDATED SURFACE WATER DRAINAGE DESIGN STRATEGY – GREEN RAIL ROUTE EAST OF ABBEY ROAD WITHIN MAIN DEVELOPMENT SITE

7.1.1 The surface water arrangements for removal currently remain, in principle, as described in section 5 above. The green rail route will be drained by filter drains which will remove runoff by infiltration to ground. Runoff which does not infiltrate to ground will discharge to outfalls for removal and disposal. Initial conservative hydraulic modelling based on railway 50% impermeability and average infiltration rates demonstrates a requirement for outfalls.

7.1.2 The green rail route to the east of Abbey Road enters the Main Development Site TCA. The green rail route has a falling gradient through a cutting back from the Secondary Site Access Road level crossing over a distance of approximately 700 m. The drainage concept design for this length of green rail route provided for a filter drain which would run west, crossing under Abbey Road and discharge into the infiltration basin. This has been reviewed as part of design development. The route to the infiltration basin would require a crossing of the Leiston Drain which creates a barrier. Whilst it may be possible for a very shallow filter drain to pass over the culverted section of watercourse, as noted in Section 6 above, there are constraints on the footprint available for the infiltration basin. Based on current hydraulic modelling this results in a need to discharge to Leiston Drain when the storage capacity of the infiltration basin is exceeded. Given these constraints, an alternative option has been developed.

7.1.3 In parallel to the design of the green rail route, SZC Co. is also working up detailed design proposals for Abbey Road and Lovers Lane in their junction area and at the proposed green rail route level crossing. Lovers Lane is diverted south and will be drained by swales and filter drains. The current discharge of runoff to Leiston Drain, which is subject to flooding is no longer proposed.

7.1.4 Hydraulic modelling has demonstrated that an infiltration basin is required to supplement the swales and filter drains. This is located as shown in AD6 Leiston Adoptable Highways drawing SZC-AD0600-WSP-LLAHDG-ZZ0000-DRW-HCD-305002 shown in **Plate 4.** This infiltration basin is located in proximity to the green rail route and does not have similar space constraints to the infiltration basin to the west of Abbey Road.

7.1.5 Based on preliminary design hydraulic modelling, this infiltration basin requires a temporary storage capacity of 379 m3 to accommodate a 1 in 100 year return period rainfall event plus 40% climate change. Hydraulic modelling of the green rail route indicates a requirement for a temporary storage capacity of 463 m3. It is proposed to increase the size of the infiltration basin to accommodate both Lovers Lane and green rail route runoff.

7.1.6 The infiltration basin will be permanent and form part of the adopted highway network. The green rail route will be removed on completion of SZC construction. At that point the size of the basin would be reduced although offers a future opportunity for enhanced flood risk protection and treatment of highway runoff.

7.1.7 Within the TCA a 1200 m length of green rail route is located running east from the Secondary Site Access level crossing. A filter drain is proposed to remove runoff from both the track and the adjacent landscaping mound. This filter drain will discharge into the construction surface water drainage network which discharges to WMZ6. As such it is included in the catchment wide hydraulic model.

7.1.8 Based on an assessment of potential pollution at the sidings an oil separator is proposed to remove the risk of pollution.

7.1.9 Following design development and hydraulic modelling, WMZ6 is proposed to be an infiltration basin with no discharge to watercourse for rainfall events more frequent that the 1 in 100 year rainfall event.

7.1.10 Based on current hydraulic modelling and average infiltration rates WMZ6 will not have temporary storage capacity to contain the 1 in 100 year rainfall event. Accordingly, it is proposed that a high level overflow will be provided. This will discharge at a rate of 37 l/s during the 1 in 100 year event plus 40% climate change.

7.1.11 The overflow will discharge into a swale and filter drain which is being proposed to drain part of Lovers Lane and Bridleway 19. Some flow which passes through the swale and filter drain will infiltrate to ground. The remainder will discharge into Leiston Drain upstream of Lovers Lane culvert.

7.1.12 The remaining 800 m of green rail route to the east drain to catchment 5. A filter drain is proposed to remove runoff from both the track and the adjacent landscaping mound. This filter drain will discharge into the construction surface water drainage network which discharges to WMZ2. As such it is included in the catchment wide hydraulic model.

7.1.13 Following design development and hydraulic modelling, WMZ2 is proposed to be an infiltration basin with discharge to watercourse. However, a high-level overflow is also proposed for exceedance rainfall events. This will discharge to the Combined Drain Outfall which is proposed to discharge to sea.

8 FINAL UPDATE SURFACE WATER DRAINAGE DESIGN STRATEGY – GREEN RAIL ROUTE WEST OF ABBEY ROAD

8.1.1 As noted in Section 4 it is now agreed with SCC that for the purpose of achieving a viable drainage strategy no infiltration will be assumed. A swale/filter drain will be provided along the full length of the GRR starting at the junction with the branch line and falling by gravity to Abbey Road where it will discharge into an attenuation basin. Full details are provided in **Appendix B section 7.** Hydraulic calculations are provided in **Appendix D of Appendix B** to determine the size of the required basin assuming no pump out or infiltration and that all runoff is stored. Options of locations for the required basin size are shown in **Appendix E of Appendix B**.

8.1.2 Options for point of discharge for the pumped outfall are identified in **Appendix B section 7.8.** In summary it is assumed that pump out to the adjacent Leiston Drain watercourse will not be permitted and the rising main will be required to pump up to the Temporary Construction Area to the east of the Secondary Site Access Road where it will discharge into the TCA drainage network.

8.1.3 There is a potential to discharge runoff from the GRR upstream of Buckleswood Road proposed level crossing into the existing watercourse. The calculated greenfield rate for the 1 in 100 year return period rainfall event is approximately 8 l/s. However, give the local nature of this watercourse and lack of knowledge reqarding its outfall and any existing flood risk, this is currently discounted.

8.1.4 Topographic survey data has now been obtained for the existing watercourses and road level in Buckleswood Road. Based on an assumption that the watercourses will be culverted beneath the railway and that culvert size will be 450 mm it will be necessary to raise the existing road level by approximately 0.52 m to 22.86 mAOD. The GRR filter drain will require to pass under the culverted watercourses.

8.1.5 It can also be confirmed that based on the latest information it now appears likely that the local watercourse referred to in section 6.1.13 can be culverted beneath the GRR but as noted, if necessary, it is feasible to divert the watercourse to the north of the GRR such that it connects into the Leiston Drain further upstream.

8.1.6 There is a potential to discharge runoff from the GRR into this local watercourse shown as outfall 09 in **Plate 3**. upstream of Buckleswood Road proposed level crossing into the existing watercourse. The calculated greenfield rate for the 1 in 100 year return period rainfall event is approximately 13 l/s or 22 l/s if the area upstream of Buckleswood Road is included. However, give the local nature of this watercourse and that it discharges into the area subject to flood risk at Abbey Road, this will not be pursued.

9 FINAL UPDATE SURFACE WATER DRAINAGE DESIGN STRATEGY – GREEN RAIL ROUTE EAST OF ABBEY ROAD

9.1.1 The length of GRR considered in this section extends from Abbey Road to the Secondary Site Access Road. GRR drainage to the east of the

Secondary Site Access Road is excluded as it forms part of the TCA drainage infrastructure. This length of GRR falls towards Abbey Road where infiltration has been proved to be viable. It was originally assumed that this length of GRR would discharge into the basin to the west of Abbey Road but this is not practical due to the need to cross Leiston Drain.

9.1.2 As an alternative, discharge into an infiltration basin to the east of Abbey Road has been proposed. Since an infiltration basin is also proposed for the highway runoff from Lovers Lane it is intended that this basin be increased in size to accommodate the GRR runoff.

9.1.3 Initial source control calculations have been undertaken to establish the required increase in storage volume and these are shown in **Appendix F of Appendix B.** The required additional volume was estimated to be 463 m3 which would need to be added to the highway volume of 383 m3 to give a total of 846 m3. In their review of the Appendix F calculation SCC noted that a nominal outfall flow was allowed for in the source control calculations. A revised calculation has been undertaken and this is shown in **Appendix D.** This supersedes **Appendix F of Appendix B.** The effect of removing the nominal outfall flow is to increase the required storage by 16 m3.

9.1.4 In their review of **Appendix B** SCC have noted that the design parameters used in the **Appendix F** calculations are not aligned to those used for the AD6 infiltration basin. The **Appendix F** calculations assume an infiltration value of 1.06 x 10-4 m/s. The AD6 infiltration basin calculations use the lower infiltration value of 1.04 x 10-5 m/s which is taken from BRE365 compliant testing. The use of the lower BRE validated infiltration rate would result in an increased requirement for the GRR volume. However, this is countered by the fact that the GRR calculations assume no infiltration through the bed of the basin. The revised calculations are shown in **Appendix C.**

9.1.5 It is accepted that at detailed design it will be necessary to determine the required volume for highway and railway runoff using consistent data. However, for the purpose of validating the updated Drainage Strategy at this stage, given the space available for the infiltration basin, within the red line boundary it is considered that the combined highway/railway infiltration basin provides a viable solution for the disposal of runoff.

10 SUMMARY AND CONCLUSION

10.1.1 The purpose of this technical note is to provide details of data which validate the **Drainage Strategy** (Doc. Ref. 6.3 2A(D)/10.14) submitted at Deadline 10. It describes how the concept design is evolving to provide for the effective drainage of the green rail route. It also identifies aspects which will require to be addressed as design develops to preliminary and detailed stages, as secured by Requirement 5.

10.1.2 Subject to DCO consent being granted for the Sizewell C project and acceptance of the drainage design strategy principles contained in this report, the drainage designs will be developed to preliminary design stage.

10.1.3 The green rail route design for track drainage will be in accordance with Network Rail Railway Drainage Systems Manual NR/L2/CIV/005 (Ref. 2). Infiltration basin design will be based on CIRIA C753 SuDS Manual (Ref. 3).

10.1.4 As preliminary design progresses SZC Co. will liaise with SCC and the EA through Design Review Meetings and provide evidence for validation of design to enable acceptance of the drainage infrastructure submitted for approval to East Suffolk Council (Requirement 5) prior to relevant works commencing, and to SCC (Requirement 13A) to demonstrate compliance with regulatory requirements and environmental permits.

REFERENCES

1. BRE Digest Soakaway design: DG 365 – 2016, BRE, 2016
[redacted]
2. NR/L2/CIV/005. 1st Edition, June 2, 2018 - Drainage Systems Manual. Network Rail.
3. The SUDs Manual (C753), CIRIA, 2015, ISBN 978-0-86017-760-9.

APPENDIX A: RECORD OF SCC COMMENTS AND SZC ACTIONS

SCC Comments on Rev 01	SZC Response
The current Annex contains a description of the strategy with no supporting information such as suitably scaled plans, sections and supporting calculations.	SCC comments Rev01 largely addressed in the Green Rail Route Updated Drainage Strategy other than as stated below
You essentially put forward two options. Option 1 being discharge to intercepting watercourses (O9 & O10) and the Abbey Road infiltration basin. You need to demonstrate you have suitable land at each attenuation location, with supporting plans and calculations	
Option 2 is required if levels do not allow you to discharge to the intercepting watercourses. Is there a risk that by the time the furthest point reaches the Abbey Road infiltration basin (as a worst case scenario) that it could be lower than the basin invert? If so, would pumping be required? If so, the appropriate assessment will need to be undertaken and it may be more suitable to keep the catchments separate and pump into the intercepting watercourses. Will need to discuss further if this is the case	
A discharge rate of 5l/s is proposed to discharge into the adjacent watercourse at Abbey Road as a worst-case scenario. Given the existing surface water flood risk here we need to be a bit careful. What is the greenfield runoff rate from your area of works (not entire red line boundary) into this watercourse at the moment? If it's less than 5l/s, then you'd technically be proposing an increase in SW flood risk in an area of high risk – which we wouldn't support. The need for this discharge is stated to be due to a lack of space, as previously stated by SCC, this is not an approach we would support Is the basin now proposed on the east side of Abbey Road rather than west, or is this in addition to the west basin?	
Flows east of Abbey Road are said to be dealt with by WMZs. I don't recall seeing additional areas being allowed for in the relevant WMZ	

designs? Again, do levels support this approach or will any pumping be required?	
You state that infiltration is likely at the NR junction. I wouldn't agree with this statement. There has been a recent development by Persimmon just east of the junction you refer to. This development struggled to infiltrate their surface water, and with no other available alternative, had to resort to deep infiltration through boreholes. At the moment you've not set out any firm proposals to manage and dispose of this surface water. With the above in mind RE likelihood of infiltration, you'll need to identify your options and demonstrate deliverability within your order limits.	
There's a mention of needing to divert a watercourse that the green rail route will intersect whilst in cutting. Connecting this to the Abbey Road watercourse has the potential to increase surface water flood risk. You'll need to have a think about this. It will certainly require detailed hydraulic modelling at detailed design. But ahead of that, you'll need to have a think about what mitigation could be implemented to ensure there is no increase in offsite flood risk and ensure you have the available land to deliver this	
There's a mention of the Abbey Road basin being adapted by SZC and adopted by Suffolk Highways post-development.	
SCC comments on Green Rail Route Updated Drainage Strategy	
3.3 is a repeat of 3.2 5.5 states an infiltration rate achieved of 1.06x10-4 (381.6mm/hr). It looks like this is what you have used for the design of the east basin. If you're going to use this rate, you need to support it with the results of testing as it's a magnitude of 10 higher than the nearby rate which you have evidenced in AD6-TH305 of 1.05x10-5 (37.44mm/hr). Also, using the highest of two rates from tests close to one another isn't the conservative approach encouraged by SCC LLFA or national guidance.	Error agreed The value shows the viability of infiltration but is not BRE365 compliant. The AD6 value is BRE365 compliant hence used. The calculations are used to get a high level estimate of volume required for GRR runoff which will discharge into the AD6 infiltration basin

Your calculations for this basin also utilise an offsite discharge through a hydrobrake at 2.2l/s in the critical event, but this is not mentioned in Section 8 or shown in Plate 5? Hydrobrake and basin invert levels do not correspond with Plate 5.	Calculations updated for no overflow of flow control.
Plate 5 contains some errors. The basin invert and top levels are consistent but the predicted maximum water levels look wrong and don't match the calculations provided in Appendix F.	Plate 5 not in error as provides the AD6 performance not the Source Control calculations in Appendix F
The calcs in Appendix F show a volume of 463m3 storage provided. This accords with AD6 Technical Note, but 8.1.26 of this document states that an 'additional 463m3' is required. So, should it be 463m3 in addition to the volumes already required, in which case you need more than the 463m3 modelled? Table 4 of AD6 Technical Note only notes a 'storage volume top of bank' of 383m3.	As stated on Plate 5 a volume of 463 m3 is provided in the AD6 basin for GRR runoff
The information contained in AD6 Technical Note and GRR Technical Note in relation to the basin East of Abbey Road should be the same as it is serving both areas, but there's no consistency and I can't say with any certainty what the cumulative attenuation volume requirements are, let alone confirm that sufficient attenuation is provided. The plans provided in both documents aren't consistent either.	The text in AD6 and GRR is modified for clarity to demonstrate that information is aligned
Approach for area west of Abbey Road with no outfall is conservative and leaves options for infiltration or pumping to MDS WMZs. Good.	

APPENDIX B: GREEN RAIL ROUTE – UPDATED DCO DRAINAGE STRATEGY

GREEN RAIL ROUTE – UPDATED DCO DRAINAGE STRATEGY STATEMENT FEBRUARY 2022

1. INTRODUCTION

1.1. Sizewell Co. (SZC) is developing the design of the Green Rail Route (GRR) that was submitted to the Planning Inspectorate as part of a Development Consent Order (DCO) application to build and operate a new nuclear power station to the north of Sizewell B.

1.2. The green rail route forms one of the Associated Developments (AD) which are required to mitigate traffic impacts arising from the Main Development Site construction. Its function is to provide a temporary railway facility which will be used for the delivery of bulky construction materials such as aggregates, cement, reinforcement steel and containerised goods to site. This will reduce the heavy goods vehicle traffic that is required to use local roads.

1.3. The Green Rail Route Drainage Strategy was produced as one of a series of design validation and evolution documents forming part of the **Drainage Strategy** (Doc. Ref. 6.3 2A(D)/10.14) submitted at Deadline 10.

1.4. Following Examination liaison has taken place with Suffolk County Council (SCC) who require more detail of the strategy and in particular want to see evidence that residual issues on storage volumes in respect to 1 in 100 +CC storm have been resolved. SCC require the evidence in the form of hydraulic modelling to demonstrate that there is a viable drainage solution for the removal of runoff from the railway and its disposal.

2. PURPOSE

2.1 The purpose for this note is to provide infiltration test data, hydraulic modelling calculations and layout drawings required to demonstrate that a viable technically achievable drainage solution is capable of delivery within the red line boundary.

2.2 The Green Rail Route commences at the junction with the existing Sizewell Branch line and terminates at sidings within the Temporary Construction Area (TCA) of the Main Development Site. A series of plans showing the route are shown for reference in **Appendix A**

3. DESCRIPTION OF THE DCO DRAINAGE DESIGN STRATEGY

3.1 For the green rail route from the junction with the NR branch line to the B1122 Abbey Road level crossing, as described in in **Volume 9 Rail Chapter 2 Description of Rail** [APP-541], drainage in the form of a trackside swale is proposed. The swales would be located on the north side of the track and would be approximately 1 m wide and have a depth of 200 mm below the track bed. Based on ground investigation data, the assumed average infiltration rate within this catchment was 0.112 m/hr. If necessary additional temporary storage capacity would be provided by either providing a filter drain below the swale or increasing the width of the swale within the Order Limits.

3.2 Given the general fall in gradient from the junction with the existing branch line to the B1122 Abbey Road, any runoff from the railway or intercepted overland flow which does not infiltrate to ground would flow to Abbey Road. In order to remove such flow, the swale would discharge into an infiltration basin. The basin is located to the west of the Leiston Drain.

3.3 Given the general fall in gradient from the junction with the existing branch line to the B1122 Abbey Road, any runoff from the railway or intercepted overland flow which does not infiltrate to ground would flow to Abbey Road. In order to remove such flow, the swale would discharge into an infiltration basin. The basin is located to the west of the Leiston Drain.

3.4 To the east of Abbey Road, the green rail route enters the Main Development Site TCA. Railway drainage is proposed in the form of filter drains located to the side of the track and sidings. These would collect railway runoff and effectively drain the railway. The filter drains would connect to the site construction surface water drainage network which would provide an outfall to remove excess runoff which does not infiltrate the ground.

3.5 This report considers the catchment between Abbey Road and the TCA Secondary Site Access Road drains back to Abbey Road where flows which have not infiltrated to ground would discharge into an infiltration basin.

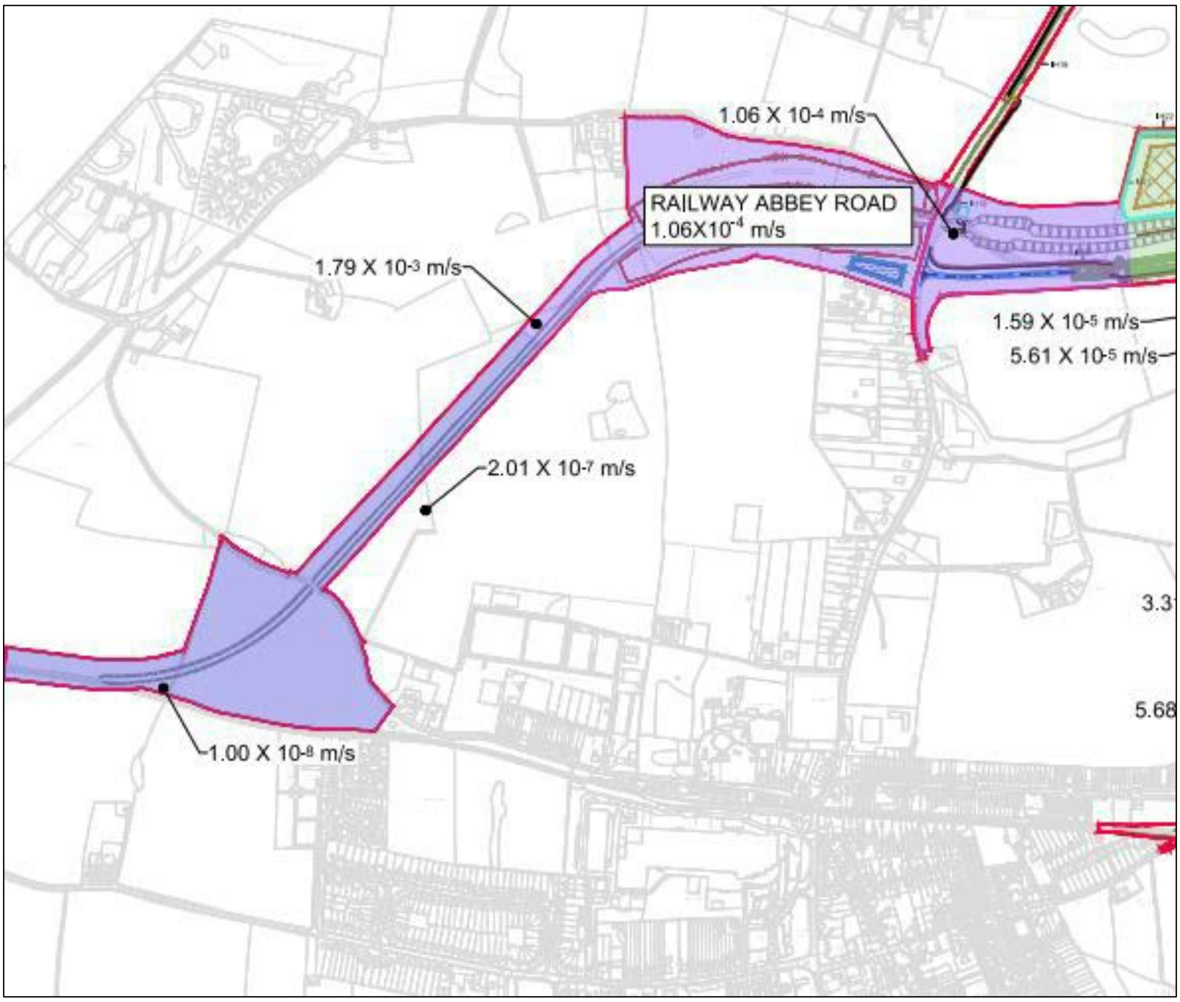
4. SCOPE OF THIS REPORT

4.1 The extent of Green Rail Route considered in this report is the length from the junction with the existing branch line to Abbey Road and from Abbey Road to the Secondary Site Access Road level crossing. The remaining portion of the railway within the TCA to the east of the Secondary Access Road is excluded from consideration in this report as it drains to Main Development Site drainage infrastructure which has been covered in other reports.

5. INFILTRATION DATA

5.1 Prior to the submission of the DCO Drainage Strategy SZC undertook a campaign of geotechnical investigation which included infiltration testing. Some limited testing was undertaken along the route of the railway as shown in **Plate 1**.

Plate 1: Early Infiltration Test Results



5.2 These results indicated that from the junction with the existing branch line, through Buckleswood Road and to the minor watercourse crossing north of Leiston, infiltration would not be viable. Downstream towards Abbey Road and to the east of Abbey Road the infiltration rates would indicate that infiltration is viable.

5.3 Subsequent to the issue of the Green Rail Route DCO Drainage Strategy further infiltration testing has been undertaken along the line of the Green Rail Route from the junction with the existing branch line to Abbey Road. These tests have been undertaken in accordance with the requirements of BRE365. The location of testing and the results are shown in **Appendix B.**

5.4 The results validate the conclusion that removal of runoff by infiltration is not viable until immediately adjacent to Abbey Road. The test at TP05 suggests that infiltration may be viable but is not a complete test.

5.5 For the length of Green Rail Route between Abbey Road and the Secondary Site Access Road, as shown in **Plate 1**, there is a single infiltration test result of 1.06 x 10-4 m/s which would indicate the viability of infiltration.

5.6 Subsequent to the issue of the Green Rail Route DCO Drainage Strategy a single further infiltration test has been undertaken in this area as part of the investigation for the separate highway scheme. The results are also shown in **Appendix B**.

5.7 The infiltration test result at AD6-TH305 is 1.04 x10-5 which also indicates the viability of infiltration.

6. LEISTON DRAIN INTERACTION WITH GREEN RAIL ROUTE

6.1 The Green Rail Route Flood Risk Assessment provides details of flood risk in the vicinity of Leiston Drain and which would impact the railway. Because the area is in the upper reaches of the Leiston Drain no EA fluvial assessment of flood risk is available. However, the EA surface water flood map indicates a high risk of flooding of land at Abbey Road. The extent is shown in **Plate 2.**

Plate 2: Predicted Surface Water flooding at Abbey Road

6.2 It can be seen that the predicted flooding extent would interact with the railway and the DCO proposed location for an infiltration basin. However, the railway would not be at risk of surface water flooding since it will be on an embankment. The location of the infiltration basin requires to be moved to avoid interaction with any flooding from Leiston Drain.

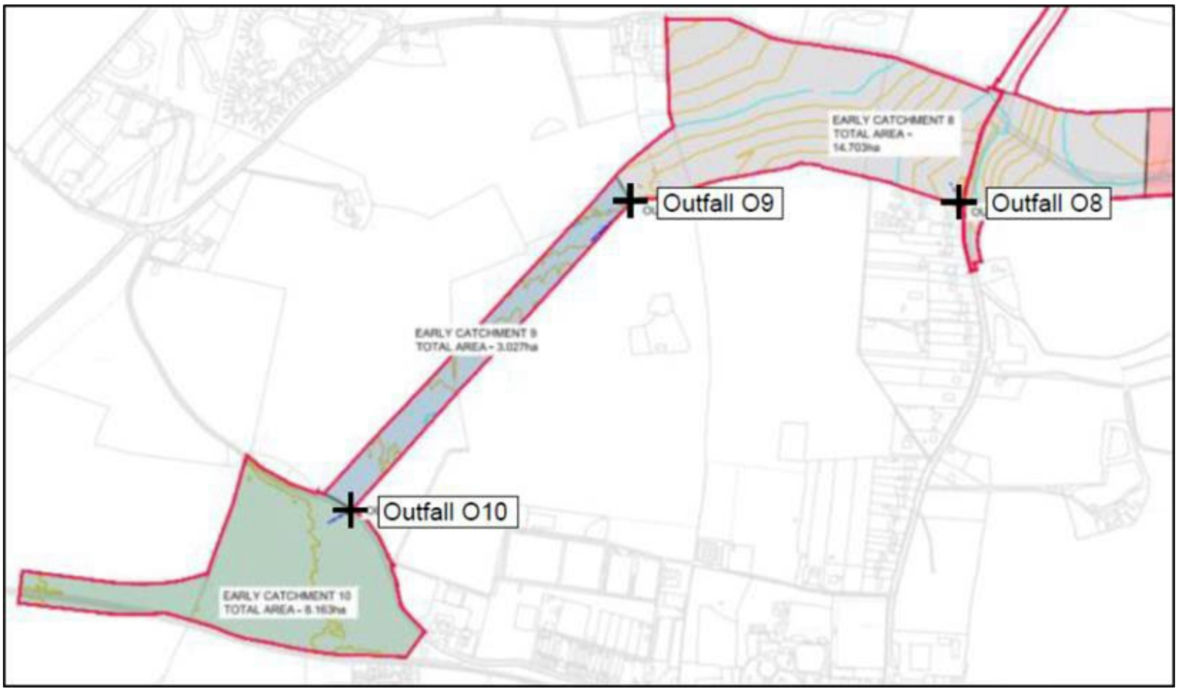
6.3 Flooding at Abbey Road is confirmed by SCC and the cause of flooding has been confirmed as part of SZC design for a diversion of the Leiston Drain required for road improvements associated with the railway level crossing of Abbey Road. Whilst the preliminary design for the watercourse diversion will ensure a reduction in flood risk, SCC have made clear and it is accepted that discharge of runoff from the railway via the infiltration basin into the Leiston Drain at this location will not be consented.

7. DESCRIPTION OF THE UPDATED DRAINAGE DESIGN STRATEGY – EXISTING BRANCH LINE TO ABBEY ROAD

7.1 The DCO drainage strategy described in **Section 3** above was based on the provision of a swale and filter drain which would remove runoff and dispose it by infiltration to ground. The infiltration basin shown schematically on layout plans and located at the lowest elevation would remove any runoff which does not infiltrate to ground along the line of the railway swale/filter drain.

7.2 As part of a review of the DCO drainage strategy the presence of watercourse crossings at O09 and Buckleswood Road, Outfall O10 have been identified as shown in **Plate 3**.

Plate 3: Watercourse Crossings and Potential Outfalls



7.3 Since it is concluded that infiltration is not viable, except potentially in vicinity to Abbey Road, the potential discharge of runoff from the railway filter drain into these watercourses at greenfield runoff rate has been identified. If this is possible and discharges can be agreed this would reduce the required size of the infiltration basin. However, since this option can’t be confirmed at this stage it is assumed that all runoff from railway and adjacent land will discharge into the infiltration basin with no loss of volume enroute.

7.4 Hydraulic modelling has been undertaken to determine the storage volume required at the infiltration basin. The extent of the assumed contributing area is shown in **Appendix C**.

7.5 The required volume for a number of scenarios has been determined using MicroDrainage Source Control. These include varying the effective contributing area (PIMP value) and use of the minimum infiltration rate considered viable by SCC. However, for the purpose of obtaining agreement with SCC on the acceptability of Green Railway Route drainage provision the assumed infiltration basin volume of 7,700 m3 has been calculated using a PIMP value of 100% and no removal of volume from the basin. The source control model with results is shown in **Appendix D.** Since there is no outfall and all runoff which enters the basin remains there, this effectively represents a scenario in which there is a 24 hour power outage.

7.6 As shown in the model input data, the basin volume of 7,700 m3 assumes a plan crest area of 7,433 m2 for a basin depth of 1.5 m. The basin needs to be located at the low point adjacent to Leiston Drain but not within the area predicted to be at risk of surface water flooding. Two location options have been identified and are shown in **Appendix E** and **Plate 4** in red**.**

Plate 4 Potential Locations for Infiltration Basin at Abbey Road

7.7 The basin can be located on either side of the railway or split with smaller basins on either side.

7.8 On the conservative assumption that all railway runoff reaches the basin and there is no removal of water by infiltration, it is necessary to remove the volume of stored runoff by pumping out since discharge into Leiston Drain will not be consented. It is intended that flows will be pumped up and into the TCA area at a suitable rate to be determined. From the TCA and the route of downstream disposal would be via WMZ6 to Leiston Drain outfall O6. The pumping station and rising main will be located entirely within the red line boundary.

8. DESCRIPTION OF THE UPDATED DRAINAGE DESIGN STRATEGY – ABBEY ROAD TO SECONDARY SITE ACCESS

8.1 The DCO drainage strategy described in **Section 3** above was based on the provision of a swale and filter drain which would remove runoff and dispose it by infiltration to ground. The infiltration basin shown schematically on layout plans and located at the lowest elevation to the west of Abbey Road and described in **Section 7**, would remove any runoff which does not infiltrate to ground along the line of the railway swale/filter drain.

8.2 As part of design development, it has been determined that to the east of Abbey Road infiltration is viable. Space of an infiltration basin is available adjacent to the railway and within the red line boundary.

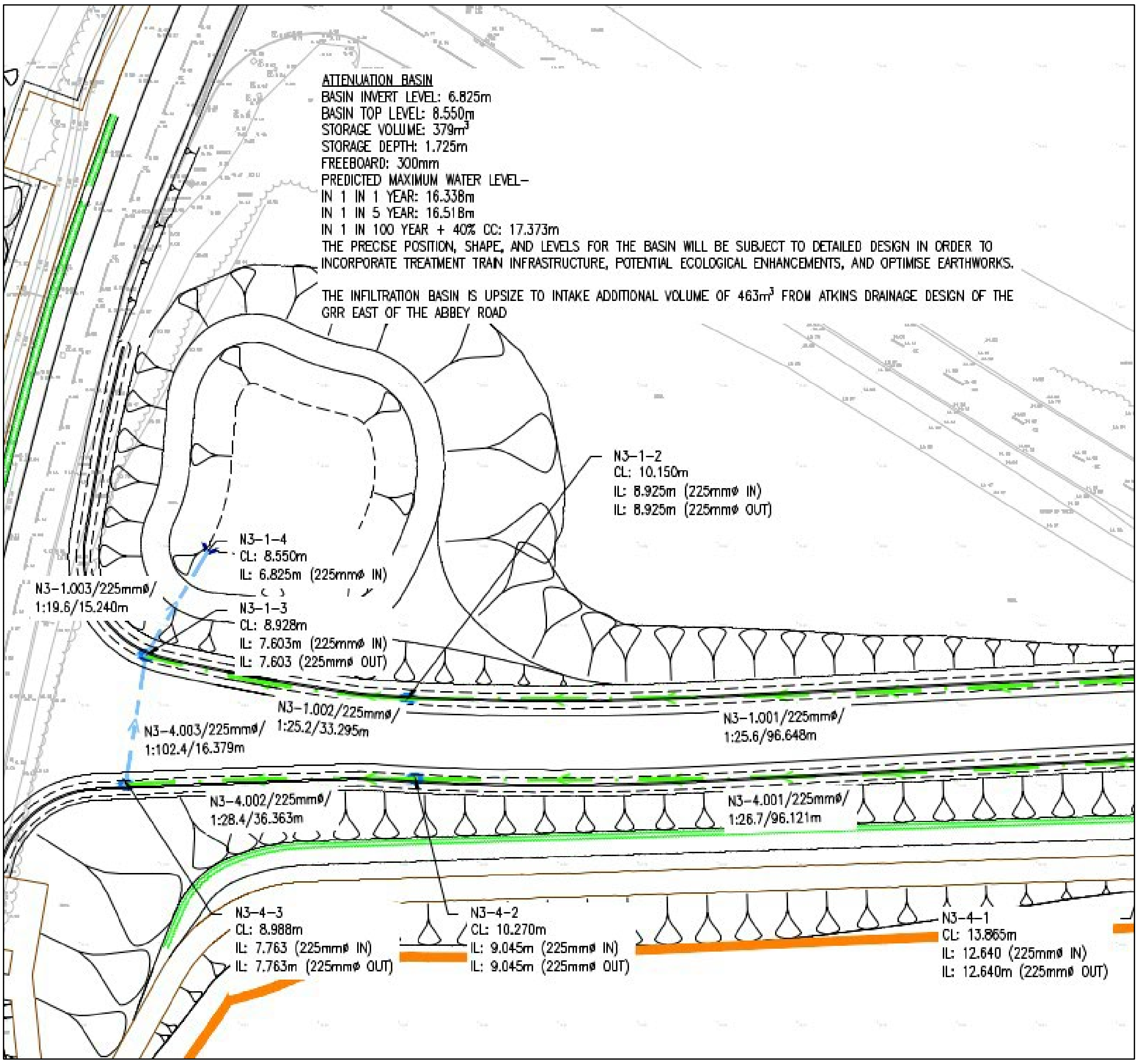
8.3 By discharging into an infiltration basin to the east of Abbey Road, the runoff from the east into the infiltration basin to the west of Abbey Road is removed. This enables that basin described in **Section 7** to be reduced in size. It also removes the problem of needing to cross over the Leiston Drain which is in culvert beneath the railway.

8.4 The required volume for the infiltration basin has been determined using MicroDrainage Source Control. The source control model with results is shown in **Appendix F.**

8.5 The calculations show that allowing for infiltration from the basin, a temporary storage volume of 463 m3 is required.

8.6 There is a need to modify the local road network at Abbey Road and Lovers Lane to accommodate the railway. It is proposed to drain the diverted Lover Lane to an infiltration basin and this would be located in the same area as the one required for the railway. According and as advised to SCC the Lovers Lane highway infiltration basin has been increased in size to accommodate the railway runoff volume. The arrangement is shown in **Plate 5.**

Plate 5: Combined Railway and Road Infiltration Basin to the East of Abbey Road



8.7 The infiltration basin will be required permanently to receive highway flows from Lovers Lane. The flows from the railway will be removed when the railway is demolished. At this stage the infiltration basin could be reduced in size but it will be more beneficial to leave it unaltered and provide a greater level of resilience against exceedance flood risk.

9. SUMMARY AND CONCLUSION

9.1 This note covers the Green Rail Route between its junction with the existing branch line and the Secondary Site Access Road. Its purpose is to provide details of data which validate the Drainage Strategy (Doc. Ref. 6.3 2A(D)/10.14) submitted at Deadline 10.

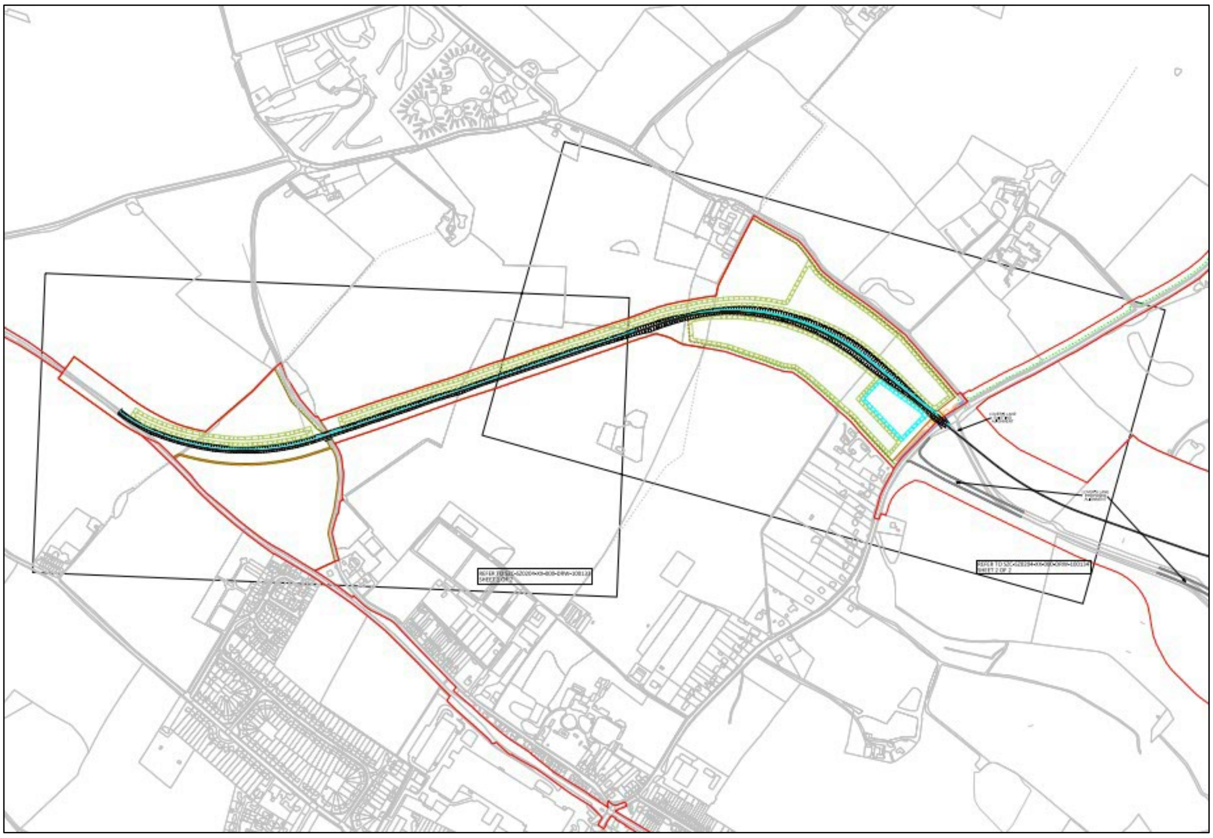
9.2 It describes how the concept design is evolving to provide for the effective drainage of the green rail route. It also identifies aspects which will require to be addressed as design develops to preliminary and detailed stages, as secured by Requirement 5.

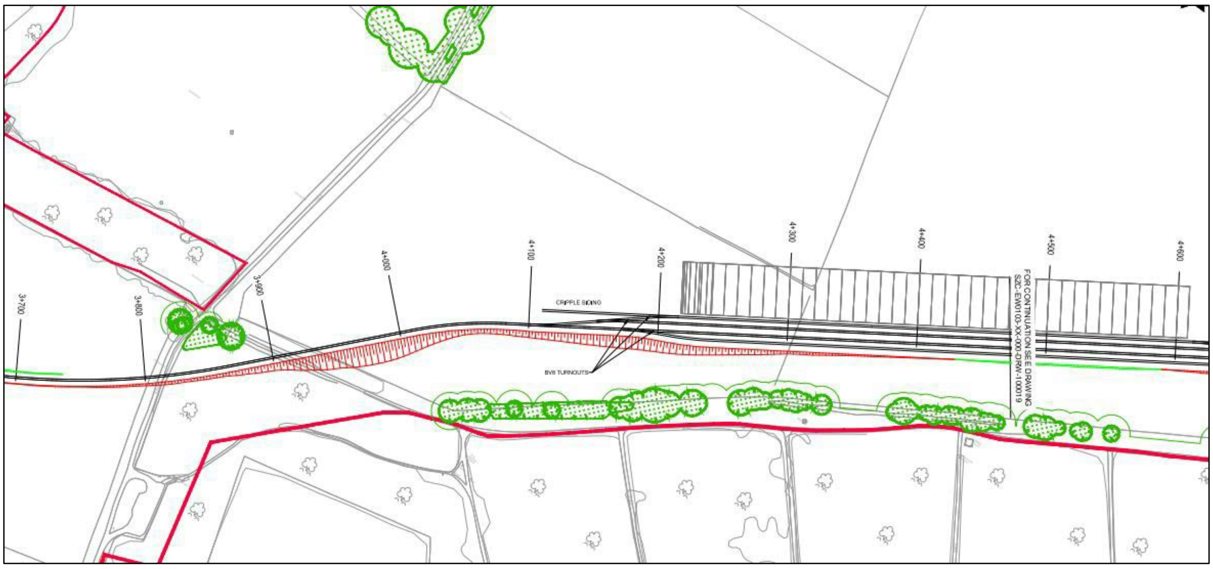
9.3 At this stage it provides evidence to enable SCC to confirm that an achievable drainage solution, compliant with the Drainage Strategy, can be delivered within the red line boundary.

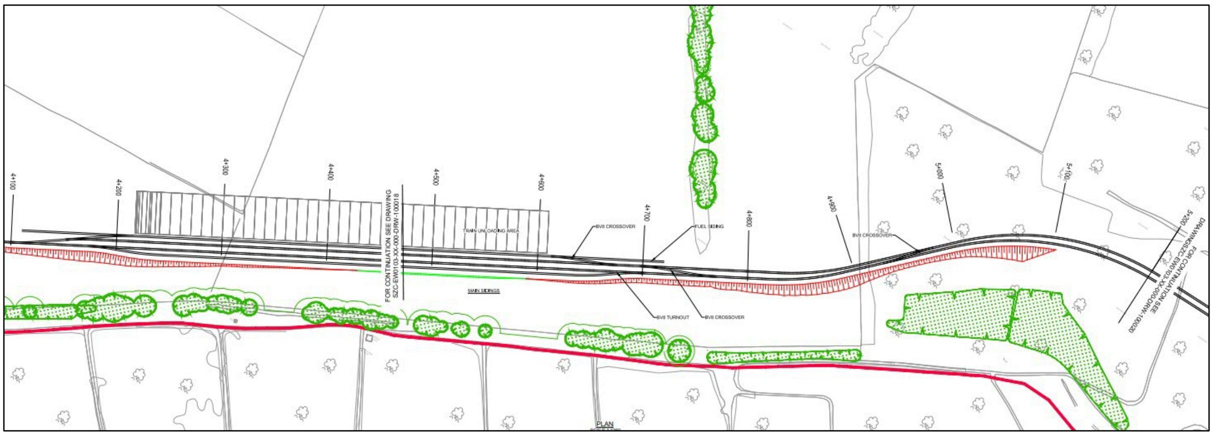
APPENDIX A

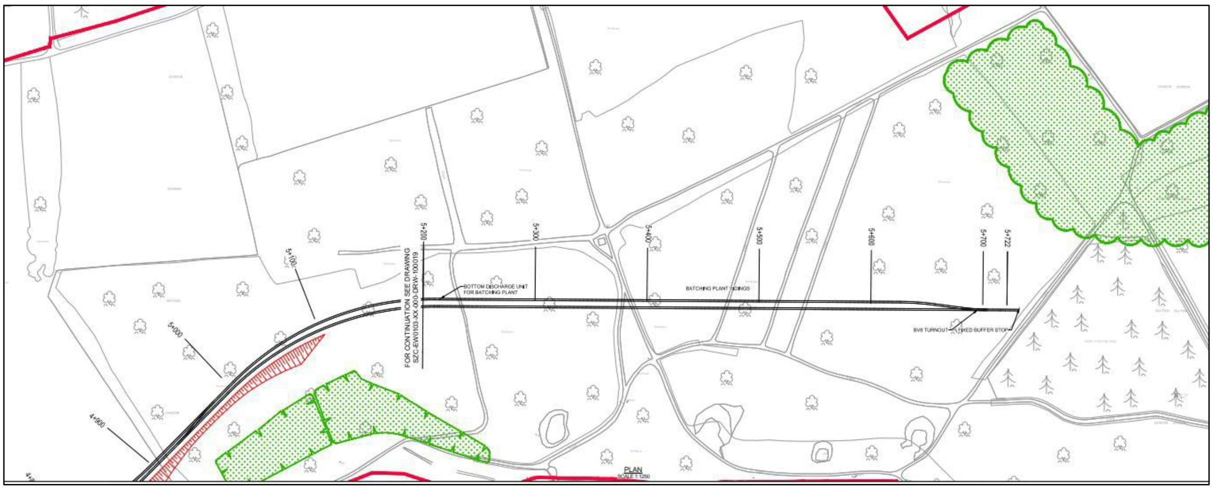
GREEN RAIL ROUTE LOCATION PLAN

Proposed Route and Extent of Green Rail




3+700
3+800
3+900
4+000
4+100
4+200
4+300
4+400
4+500
4+600
CRIPPLE SIDING
BV8 TURNOUTS
FOR CONTINUATION SEE DRAWING
SZC-EW0103-XX-000-DRW-100019


4+100
4+200
4+300
4+400
4+500
4+600
4+700
4+800
4+900
5+000
5+100
5+200
FOR CONTINUATION SEE DRAWING
SZC-EW0103-XX-000-DRW-100018
TRAIN UNLOADING AREA
MAIN SIDINGS
BV8 CROSSOVER
FUEL SIDING
BV8 TURNOUT
BV8 CROSSOVER
BV8 CROSSOVER
FOR CONTINUATION SEE
DRAWING SZC-EW0103-XX-000-DRW-100020
PLAN


4+900
5+000
5+100
5+200
5+300
5+400
5+500
5+600
5+700
5+722
FOR CONTINUATION SEE DRAWING
SZC-EW0103-XX-000-DRW-100019
BOTTOM DISCHARGE UNIT
FOR BATCHING PLANT
BATCHING PLANT SIDINGS
BV8 TURNOUT
FIXED BUFFER STOP
PLAN
SCALE 1:1250

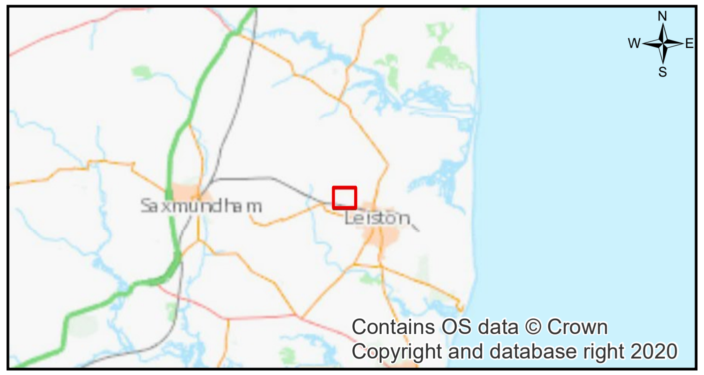
APPENDIX B

GREEN RAIL ROUTE INFILTRATION TEST DATA

B.2 Exploratory Location Plans

Title	Reference
Exploratory Location Plans (GRR)	B.2.1 to B.2.7

Location



Scale: 1:250,000

0 2.5 5 10 15 20 Kilometres

Legend

Location Type

●	Cable Percussion Borehole
▼	Cone Penetration Test Location (with Inspection Pit)
☒	Trial Pit

Notes

The ground investigation data are presented as two separate reports according to the investigation areas. Please see separate reports for other exploratory locations.

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

Rev.	Date	Description	Initials
1	01/03/2021	Draft Issue	LCB
2	04/05/2021	Final Issue	LCB

Scale: 1:2,500

0 25 50 100 150 200 Metres

Coordinate System

British National Grid

Client

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

Investigation Supervisor

Atkins

Contractor

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

Project Title

Sizewell C - Onshore 2020 Ground Investigation Campaign - SZC Green Rail Route

Drawing Title

Exploratory Location Plan (GRR)

Drawing Number

B.2.1

Drawn By	Checked By	Issued On	Project No.	Sheet Size	Rev.
LCB	CAY	04/05/2021	G200032U	A3	2

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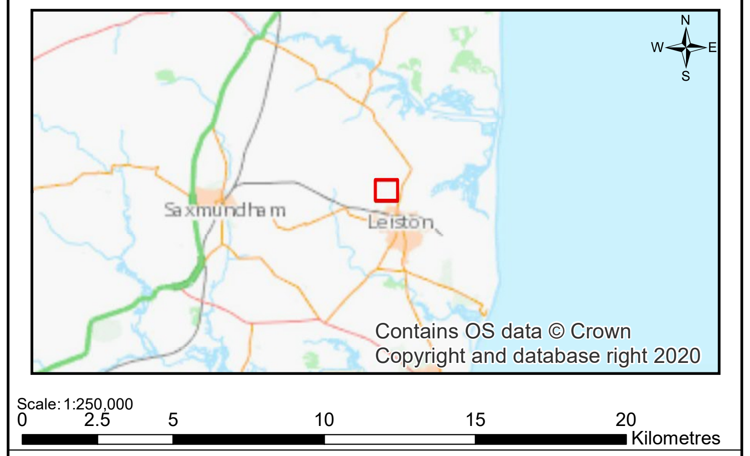
Location



Contains OS data © Crown Copyright and database right 2020

Legend

Location Type

⊕ Cable Percussion Borehole

▼ Cone Penetration Test Location (with Inspection Pit)

☒ Trial Pit

Notes

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

GRR-CPT03-EW and GRR-CPT03A-EW were undertaken within their same inspection pit; Same set of grid coordinates have been used.

GRR-CPT07-EW and GRR-CPT07A-EW were undertaken within their same inspection pit; Same set of grid coordinates have been used.

Rev.	Date	Description	Initials
1	01/03/2021	Draft Issue	LCB
2	04/05/2021	Final Issue	LCB

Scale: 1:2,500

0 25 50 100 150 200 Metres

Coordinate System

British National Grid

Client

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

Investigation Supervisor
Atkins

Contractor

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Registered in England No. 1284352
VAT No. GB 133 1704 09
www.fugro.com

Project Title

Sizewell C - Onshore 2020 Ground Investigation Campaign - SZC Green Rail Route

Drawing Title

Exploratory Location Plan (GRR)

Drawing Number

B.2.2

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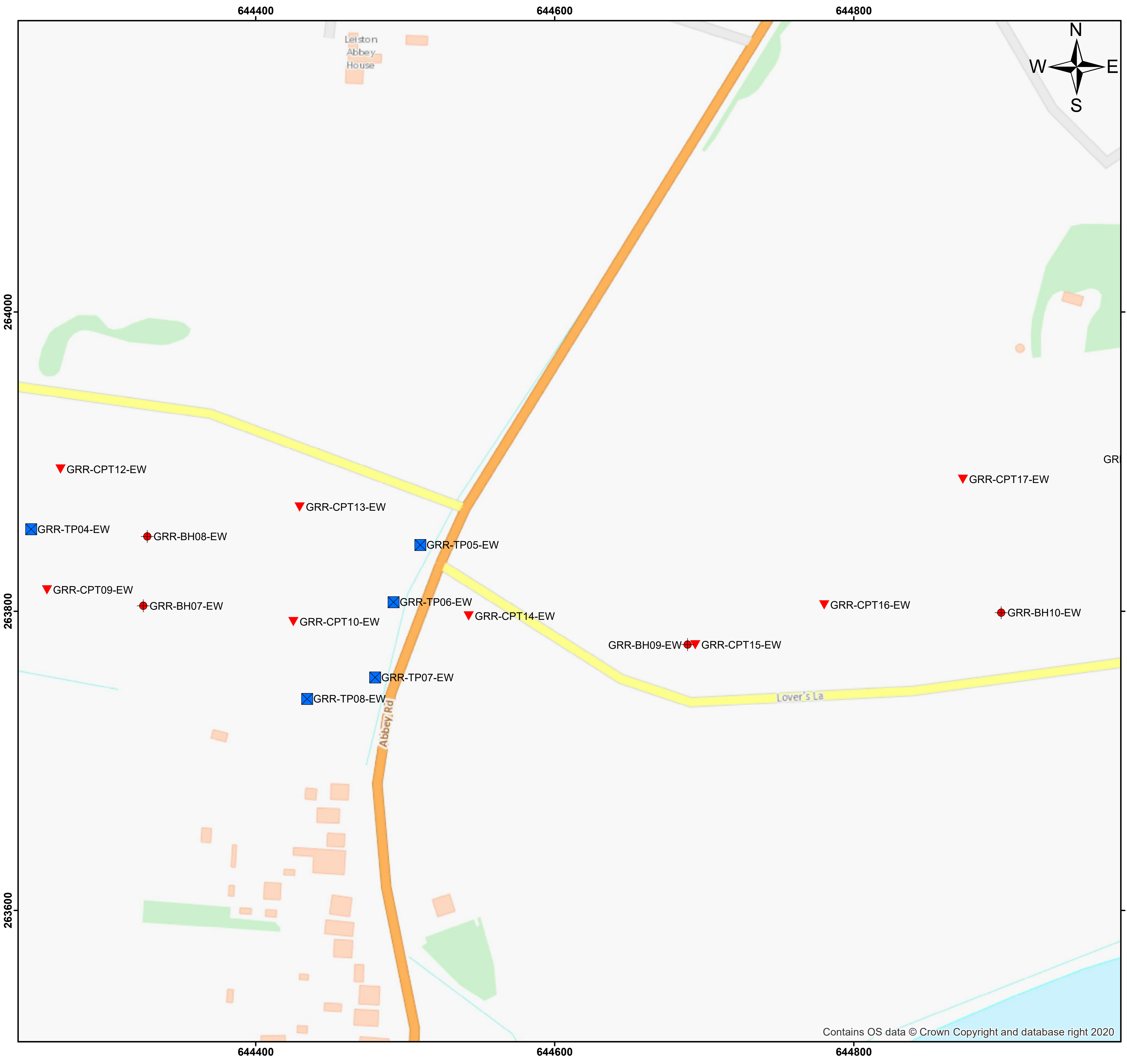
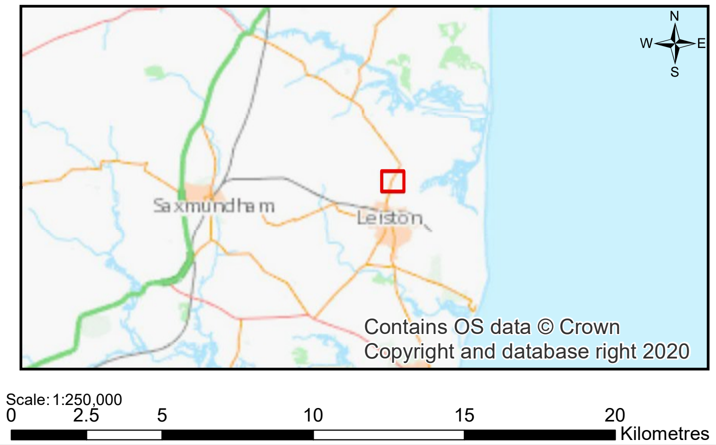


Contains OS data © Crown Copyright and database right 2020

Location



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Legend

Location Type

●	Cable Percussion Borehole
▼	Cone Penetration Test Location (with Inspection Pit)
☒	Trial Pit

Notes

The ground investigation data are presented as two separate reports according to the investigation areas. Please see separate reports for other exploratory locations.

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

Rev.	Date	Description	Initials
1	01/03/2021	Draft Issue	LCB
2	04/05/2021	Final Issue	LCB

Scale: 1:2,500

0 25 50 100 150 200 Metres

Coordinate System

British National Grid

Client

NNB Generation Company (SZC) Limited
EDF-NNB, 90 Whitfield Street,
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Registered in England No. 1284352
VAT No. GB 133 1704 09
www.fugro.com

Project Title

Sizewell C - Onshore 2020 Ground Investigation Campaign - SZC Green Rail Route

Drawing Title

Exploratory Location Plan (GRR)

Drawing Number

B.2.3

Drawn By	Checked By	Issued On	Project No.	Sheet Size	Rev.
LCB	CAY	04/05/2021	G200032U	A3	2

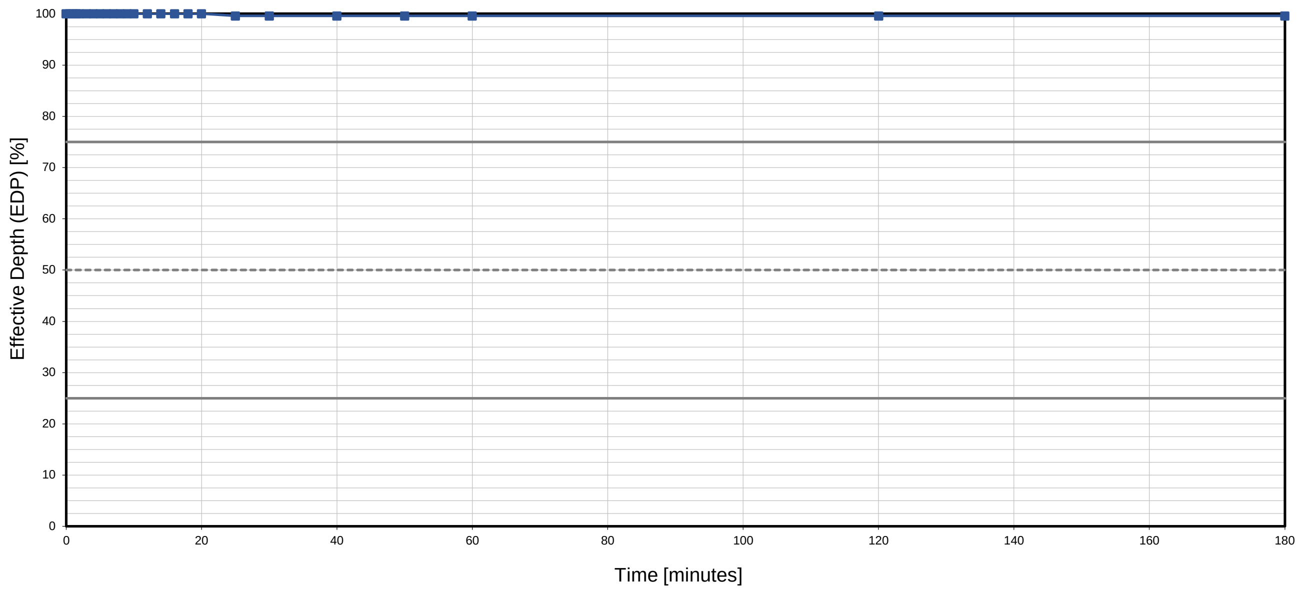
F.3 Infiltration/Soakaway Tests

Title	Reference
Soakaway Test Results	Referenced by Location ID

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	20/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP01-EW



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	12:12		Start Time =			Start Time =		
Test Top =	0.35	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
75% EDP =	0.64	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	0.93	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.21	m BGL	25% EDP =		m BGL	25% EDP =		m BGL

Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
00:00:00		0.35	100	00:00:00				00:00:00			
00:00:30		0.35	100	00:00:30				00:00:30			
00:01:00		0.35	100	00:01:00				00:01:00			
00:01:30		0.35	100	00:01:30				00:01:30			
00:02:00		0.35	100	00:02:00				00:02:00			
00:03:00		0.35	100	00:03:00				00:03:00			
00:04:00		0.35	100	00:04:00				00:04:00			
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00:06:00		0.35	100	00:06:00				00:06:00			
00:07:00		0.35	100	00:07:00				00:07:00			
00:08:00		0.35	100	00:08:00				00:08:00			
00:09:00		0.35	100	00:09:00				00:09:00			
00:10:00		0.35	100	00:10:00				00:10:00			
00:12:00		0.35	100	00:12:00				00:12:00			
00:14:00		0.35	100	00:14:00				00:14:00			
00:16:00		0.35	100	00:16:00				00:16:00			
00:18:00		0.35	100	00:18:00				00:18:00			
00:20:00		0.35	100	00:20:00				00:20:00			
00:25:00		0.36	100	00:25:00				00:25:00			
00:30:00		0.36	100	00:30:00				00:30:00			
00:40:00		0.36	100	00:40:00				00:40:00			
00:50:00		0.36	100	00:50:00				00:50:00			
01:00:00		0.36	100	01:00:00				01:00:00			
02:00:00		0.36	100								
03:00:00		0.36	100								

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

Input by AH 25/11/2020

Checked by CAY 22/02/2021

Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	20/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP01-EW

Test Details

Datum (-ve denotes AGL) =	0.00 m BGL	<u>Well Screen</u> Well screen not used
Pit Length =	2.00 m	<u>Filter Material</u> Filter not used
Pit Width =	1.40 m	
Pit Depth =	1.50 m BGL	

<u>Weather</u> Cold, dry, light wind, damp ground

<u>Geology</u> CLAY

<u>Remarks</u>

Negligible discharge observed.

Test termination agreed with client representative; Test 2 and Test 3 not required.

Water level did not reach 75% or 25% EDP; infiltration rates cannot be given.

Pit was dry before adding water; water added to the pit to 0.35m BGL (Test 1).

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	12:12		Start Time =			Start Time =		
Test Top =	0.35	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
EDP =	1.15	m	EDP =		m	EDP =		m
75% EDP =	0.64	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	0.93	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.21	m BGL	25% EDP =		m BGL	25% EDP =		m BGL
V =	3.22	m^3	V =		m^3	V =		m^3
Vg =		m^3	Vg =		m^3	Vg =		m^3
Vp =		m^3	Vp =		m^3	Vp =		m^3
Vp75-25 =	1.61	m^3	Vp75-25 =		m^3	Vp75-25 =		m^3
ap =	6.71	m^2	ap =		m^2	ap =		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

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

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

V is the effective storage volume of water in the hole (ESV) when gravel fill not used; Vg is the effective volume taken up by the gravel solid;
Vp is the ESV, less the volume of the gravel fraction.
Vp75-25 is the ESV between 75% and 25% effective depth, less the volume of the gravel fraction; Vp75-50 is used when 25% EDP was not reached.

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

Tp75 is time at 75% EDP; Tp50 is the time at 50% EDP; Tp25 is time at 25% EDP.
Tp75-25 is the assessed time for water level to fall from 75% to 25% EDP; Tp75-50 is used when 25% EDP was not reached.

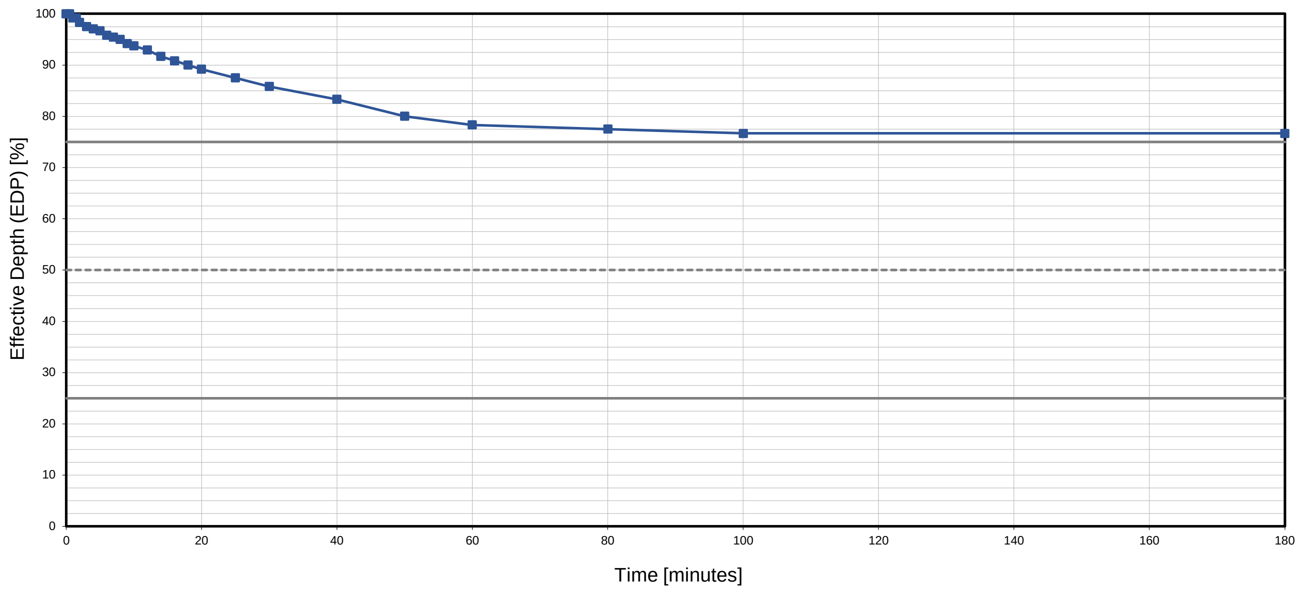
$$Soil\ Infiltration\ rate, f = \frac{Vp_{75-25}}{ap \times Tp_{75-25}} \quad \text{or} \quad Soil\ Infiltration\ rate, f = \frac{Vp_{75-50}}{ap \times Tp_{75-50}}$$

Input by AH 25/11/2020 Checked by CAY 22/02/2021 Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	22/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP02-EW



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	09:27		Start Time =			Start Time =		
Test Top =	0.30	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
75% EDP =	0.60	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	0.90	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.20	m BGL	25% EDP =		m BGL	25% EDP =		m BGL

Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
00:00:00		0.30	100	00:00:00				00:00:00			
00:00:30		0.30	100	00:00:30				00:00:30			
00:01:00		0.31	99	00:01:00				00:01:00			
00:01:30		0.31	99	00:01:30				00:01:30			
00:02:00		0.32	98	00:02:00				00:02:00			
00:03:00		0.33	98	00:03:00				00:03:00			
00:04:00		0.34	97	00:04:00				00:04:00			
00:05:00		0.34	97	00:05:00				00:05:00			
00:06:00		0.35	96	00:06:00				00:06:00			
00:07:00		0.36	95	00:07:00				00:07:00			
00:08:00		0.36	95	00:08:00				00:08:00			
00:09:00		0.37	94	00:09:00				00:09:00			
00:10:00		0.38	94	00:10:00				00:10:00			
00:12:00		0.39	93	00:12:00				00:12:00			
00:14:00		0.40	92	00:14:00				00:14:00			
00:16:00		0.41	91	00:16:00				00:16:00			
00:18:00		0.42	90	00:18:00				00:18:00			
00:20:00		0.43	89	00:20:00				00:20:00			
00:25:00		0.45	88	00:25:00				00:25:00			
00:30:00		0.47	86	00:30:00				00:30:00			
00:40:00		0.50	83	00:40:00				00:40:00			
00:50:00		0.54	80	00:50:00				00:50:00			
01:00:00		0.56	78	01:00:00				01:00:00			
01:20:00		0.57	78								
01:40:00		0.58	77								
03:00:00		0.58	77								

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

Input by AH 25/11/2020 | Checked by CAY 22/02/2021 | Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD							Method BRE 365 2016
Date	22/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP02-EW

Test Details

Datum (-ve denotes AGL) = 0.00 m BGL

Pit Length = 1.80 m
Pit Width = 1.30 m
Pit Depth = 1.50 m BGL

Well Screen
Well screen not used

Filter Material
Filter not used

Weather Cold, dry, calm, damp ground

Geology Clayey SILT.

Remarks
Slow discharge observed.
Test termination agreed with client representative; Test 2 and Test 3 not required.
Water level did not reach 75% or 25% EDP; infiltration rates cannot be given.

Pit was dry before adding water; water added to the pit to 0.30m BGL (Test 1).

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	09:27		Start Time =			Start Time =		
Test Top =	0.30	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
EDP =	1.20	m	EDP =		m	EDP =		m
75% EDP =	0.60	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	0.90	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.20	m BGL	25% EDP =		m BGL	25% EDP =		m BGL
V =	2.81	m^3	V =		m^3	V =		m^3
Vg =		m^3	Vg =		m^3	Vg =		m^3
Vp =		m^3	Vp =		m^3	Vp =		m^3
Vp75-25 =	1.40	m^3	Vp75-25 =		m^3	Vp75-25 =		m^3
ap =	6.06	m^2	ap =		m^2	ap =		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

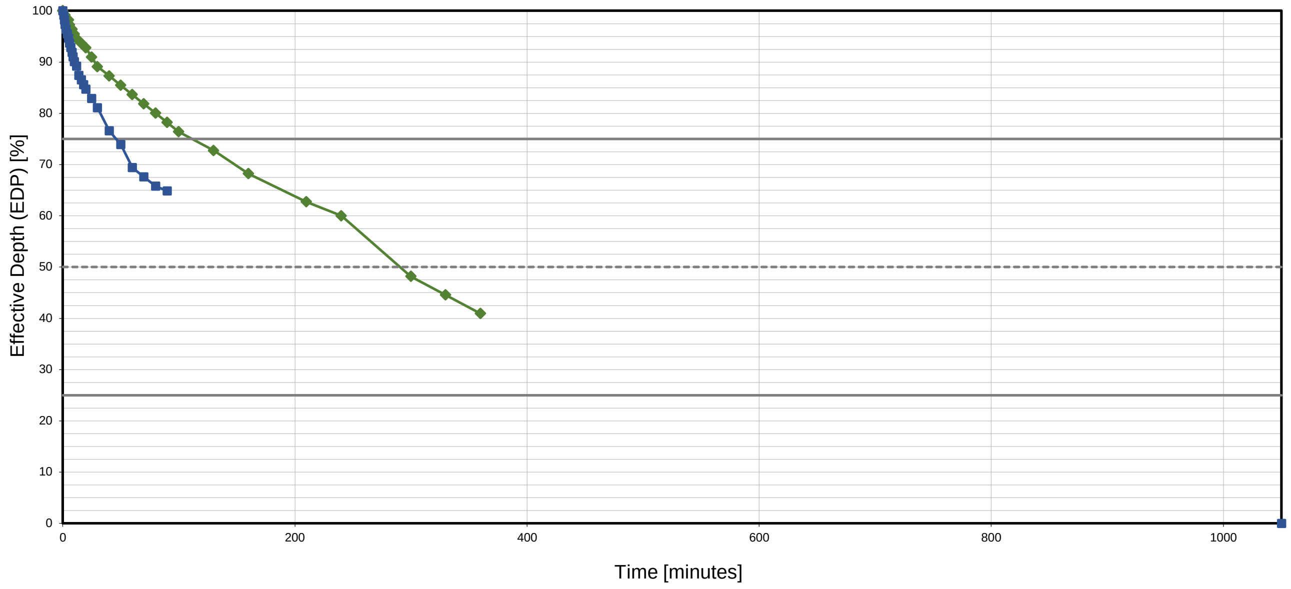
$$Soil\ Infiltration\ rate, f = \frac{Vp_{75-50}}{ap \times Tp_{75-50}}$$

Input by AH 25/11/2020 Checked by CAY 22/02/2021 Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	17/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP05-EW



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	14:25		Start Time =	08:27		Start Time =		
Test Top =	0.39	m BGL	Test Top =	0.40	m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =	1.50	m BGL	Test Base =		m BGL
75% EDP =	0.67	m BGL	75% EDP =	0.68	m BGL	75% EDP =		m BGL
50% EDP =	0.95	m BGL	50% EDP =	0.95	m BGL	50% EDP =		m BGL
25% EDP =	1.22	m BGL	25% EDP =	1.23	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		0.39	100	00:00:00		0.40	100	00:00:00			
00:01:00		0.40	99	00:02:00		0.41	99	00:00:30			
00:01:30		0.41	98	00:04:00		0.42	98	00:01:00			
00:02:00		0.42	97	00:05:00		0.42	98	00:01:30			
00:03:00		0.43	96	00:06:00		0.43	97	00:02:00			
00:04:00		0.44	95	00:08:00		0.44	96	00:03:00			
00:05:00		0.45	95	00:10:00		0.45	95	00:04:00			
00:06:00		0.46	94	00:12:00		0.46	95	00:05:00			
00:07:00		0.47	93	00:16:00		0.47	94	00:06:00			
00:08:00		0.48	92	00:20:00		0.48	93	00:07:00			
00:09:00		0.49	91	00:25:00		0.50	91	00:08:00			
00:10:00		0.50	90	00:30:00		0.52	89	00:09:00			
00:12:00		0.51	89	00:40:00		0.54	87	00:10:00			
00:14:00		0.53	87	00:50:00		0.56	85	00:12:00			
00:16:00		0.54	86	01:00:00		0.58	84	00:14:00			
00:18:00		0.55	86	01:10:00		0.60	82	00:16:00			
00:20:00		0.56	85	01:20:00		0.62	80	00:18:00			
00:25:00		0.58	83	01:30:00		0.64	78	00:20:00			
00:30:00		0.60	81	01:40:00		0.66	76	00:25:00			
00:40:00		0.65	77	02:10:00		0.70	73	00:30:00			
00:50:00		0.68	74	02:40:00		0.75	68	00:40:00			
01:00:00		0.73	69	03:30:00		0.81	63	00:50:00			
01:10:00		0.75	68	04:00:00		0.84	60	01:00:00			
01:20:00		0.77	66	05:00:00		0.97	48				
01:30:00		0.78	65	05:30:00		1.01	45				
17:30:00		1.50	0	06:00:00		1.05	41				

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

Input by AH 25/11/2020

Checked by CAY 22/02/2021

Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	17/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP05-EW

Test Details

Datum (-ve denotes AGL) = 0.00 m BGL

Pit Length = 2.50 m
Pit Width = 1.20 m
Pit Depth = 1.50 m BGL

Well Screen
Well screen not used

Filter Material
Filter not used

Weather Warm, dry, damp ground

Geology SAND

Remarks
Test 1 undertaken on 17/11/2020; Test 2 undertaken on 18/11/2020.
Test termination agreed with client representative; Test 3 not required.
Readings not taken between 1.5 hour and 17.5 hour elapsed time for Test 1; water level variation cannot be discerned around 25% EDP; infiltration rate cannot be given. Water level did not reach 25% EDP for Test 2; infiltration rate cannot be given.
Pit was dry before adding water; water added to the pit to 0.39m BGL (Test 1). Pit was empty on 18/11/2020 before adding water, water added to the pit to 0.40m BGL (Test 2).
Side wall collapsed during Test 2 at about 1 hour 5 minutes; treat results with caution.

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	14:25		Start Time =	08:27		Start Time =		
Test Top =	0.39	m BGL	Test Top =	0.40	m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =	1.50	m BGL	Test Base =		m BGL
EDP =	1.11	m	EDP =	1.10	m	EDP =		m
75% EDP =	0.67	m BGL	75% EDP =	0.68	m BGL	75% EDP =		m BGL
50% EDP =	0.95	m BGL	50% EDP =	0.95	m BGL	50% EDP =		m BGL
25% EDP =	1.22	m BGL	25% EDP =	1.23	m BGL	25% EDP =		m BGL
V =	3.33	m^3	V =	3.30	m^3	V =		m^3
Vg =		m^3	Vg =		m^3	Vg =		m^3
Vp =		m^3	Vp =		m^3	Vp =		m^3
Vp75-25 =	1.67	m^3	Vp75-25 =	1.65	m^3	Vp75-25 =		m^3
ap =	7.11	m^2	ap =	7.07	m^2	ap =		m^2
Tp75 =	2700	s	Tp75 =	6720	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 25/11/2020

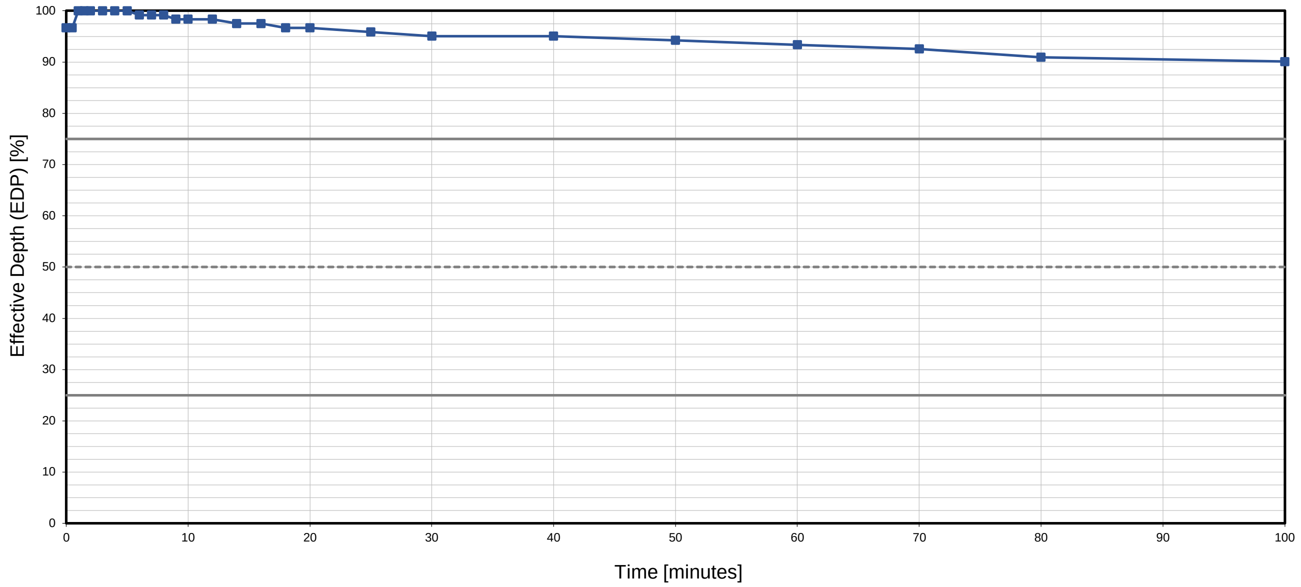
Checked by CAY 22/02/2021

Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	17/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP07-EW



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	11:06		Start Time =			Start Time =		
Test Top =	0.29	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
75% EDP =	0.59	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	0.90	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.20	m BGL	25% EDP =		m BGL	25% EDP =		m BGL

Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
00:00:00		0.33	97	00:00:00				00:00:00			
00:00:30		0.33	97	00:00:30				00:00:30			
00:01:00		0.29	100	00:01:00				00:01:00			
00:01:30		0.29	100	00:01:30				00:01:30			
00:02:00		0.29	100	00:02:00				00:02:00			
00:03:00		0.29	100	00:03:00				00:03:00			
00:04:00		0.29	100	00:04:00				00:04:00			
00:05:00		0.29	100	00:05:00				00:05:00			
00:06:00		0.30	99	00:06:00				00:06:00			
00:07:00		0.30	99	00:07:00				00:07:00			
00:08:00		0.30	99	00:08:00				00:08:00			
00:09:00		0.31	98	00:09:00				00:09:00			
00:10:00		0.31	98	00:10:00				00:10:00			
00:12:00		0.31	98	00:12:00				00:12:00			
00:14:00		0.32	98	00:14:00				00:14:00			
00:16:00		0.32	98	00:16:00				00:16:00			
00:18:00		0.33	97	00:18:00				00:18:00			
00:20:00		0.33	97	00:20:00				00:20:00			
00:25:00		0.34	96	00:25:00				00:25:00			
00:30:00		0.35	95	00:30:00				00:30:00			
00:40:00		0.35	95	00:40:00				00:40:00			
00:50:00		0.36	94	00:50:00				00:50:00			
01:00:00		0.37	93	01:00:00				01:00:00			
01:10:00		0.38	93								
01:20:00		0.40	91								
01:40:00		0.41	90								

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

Input by AH 25/11/2020 Checked by CAY 22/02/2021 Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	17/11/2020	Operator	GK	Pit Depth	1.50 m	Hole No.	GRR-TP07-EW

Test Details

Datum (-ve denotes AGL) = 0.00 m BGL

Pit Length = 2.10 m
Pit Width = 1.40 m
Pit Depth = 1.50 m BGL

Well Screen
Well screen not used

Filter Material
Filter not used

Weather: Warm, dry, light wind, damp ground

Geology: SAND

Remarks
Slow discharge observed.
Test termination agreed with client representative; Test 2 and Test 3 not required.
Water level did not reach 75% or 25% EDP; infiltration rates cannot be given.

Pit was dry before adding water; water added to the pit to 0.33m BGL (Test 1).
Side wall collapsed during Test 1 at about 1 minute (water level displaced); further side wall collapsed at about 34 minutes and 1 hour 8 minutes; treat results with caution.

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	11:06		Start Time =			Start Time =		
Test Top =	0.29	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	1.50	m BGL	Test Base =		m BGL	Test Base =		m BGL
EDP =	1.21	m	EDP =		m	EDP =		m
75% EDP =	0.59	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	0.90	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.20	m BGL	25% EDP =		m BGL	25% EDP =		m BGL
V =	3.56	m^3	V =		m^3	V =		m^3
Vg =		m^3	Vg =		m^3	Vg =		m^3
Vp =		m^3	Vp =		m^3	Vp =		m^3
Vp75-25 =	1.78	m^3	Vp75-25 =		m^3	Vp75-25 =		m^3
ap =	7.18	m^2	ap =		m^2	ap =		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.

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

Input by AH 25/11/2020

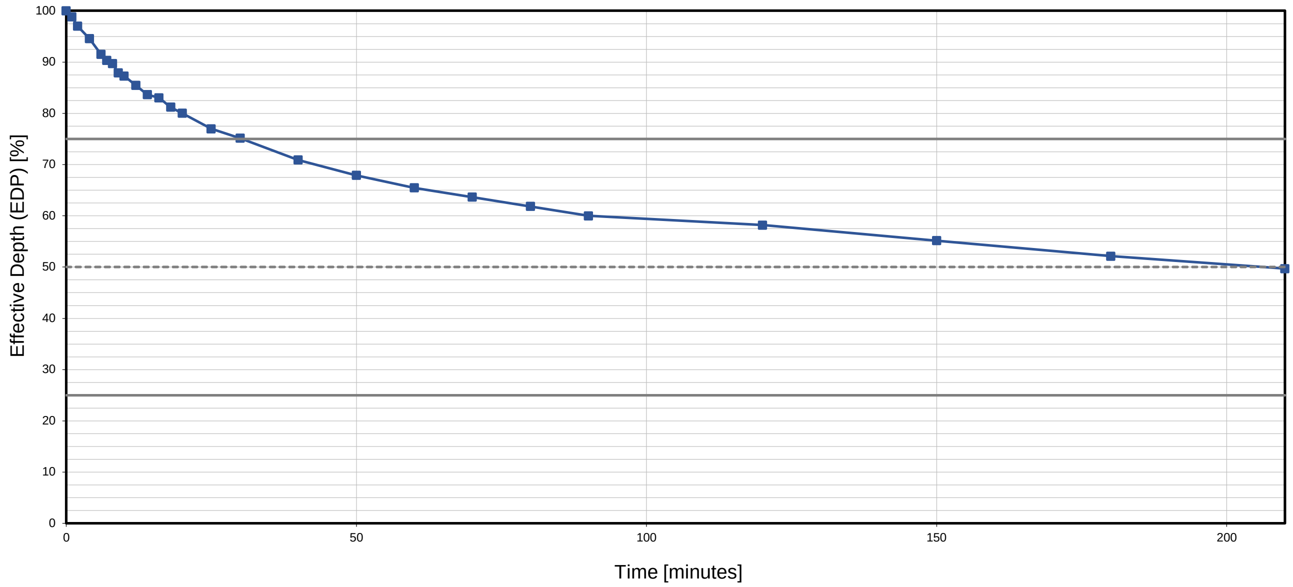
Checked by CAY 22/02/2021

Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	24/11/2020	Operator	GK	Pit Depth	2.00 m	Hole No.	GRR-TP08-EW



Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	12:10		Start Time =			Start Time =		
Test Top =	0.35	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	2.00	m BGL	Test Base =		m BGL	Test Base =		m BGL
75% EDP =	0.76	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	1.18	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.59	m BGL	25% EDP =		m BGL	25% EDP =		m BGL

Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]	Elapsed Time [hr:min:sec]	Water [m BDL]	Water [m BGL]	EDP [%]
00:00:00	1.54	0.35	100	00:00:00				00:00:00			
00:01:00	1.56	0.37	99	00:00:30				00:00:30			
00:02:00	1.59	0.40	97	00:01:00				00:01:00			
00:04:00	1.63	0.44	95	00:01:30				00:01:30			
00:06:00	1.68	0.49	92	00:02:00				00:02:00			
00:07:00	1.70	0.51	90	00:03:00				00:03:00			
00:08:00	1.71	0.52	90	00:04:00				00:04:00			
00:09:00	1.74	0.55	88	00:05:00				00:05:00			
00:10:00	1.75	0.56	87	00:06:00				00:06:00			
00:12:00	1.78	0.59	85	00:07:00				00:07:00			
00:14:00	1.81	0.62	84	00:08:00				00:08:00			
00:16:00	1.82	0.63	83	00:09:00				00:09:00			
00:18:00	1.85	0.66	81	00:10:00				00:10:00			
00:20:00	1.87	0.68	80	00:12:00				00:12:00			
00:25:00	1.92	0.73	77	00:14:00				00:14:00			
00:30:00	1.95	0.76	75	00:16:00				00:16:00			
00:40:00	2.02	0.83	71	00:18:00				00:18:00			
00:50:00	2.07	0.88	68	00:20:00				00:20:00			
01:00:00	2.11	0.92	65	00:25:00				00:25:00			
01:10:00	2.14	0.95	64	00:30:00				00:30:00			
01:20:00	2.17	0.98	62	00:40:00				00:40:00			
01:30:00	2.20	1.01	60	00:50:00				00:50:00			
02:00:00	2.23	1.04	58	01:00:00				01:00:00			
02:30:00	2.28	1.09	55								
03:00:00	2.33	1.14	52								
03:30:00	2.37	1.18	50								

Datum (-ve denotes AGL) = -1.19 m BGL.

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

Input by AH 25/11/2020 Checked by CAY 22/02/2021 Approved by NHA 07/05/2021

SOAKAWAY TEST RECORD

Method BRE 365 2016

Date	24/11/2020	Operator	GK	Pit Depth	2.00 m	Hole No.	GRR-TP08-EW

Test Details

Datum (-ve denotes AGL) = -1.19 m BGL

Pit Length = 2.00 m
Pit Width = 1.20 m
Pit Depth = 2.00 m BGL

Well Screen
Well screen not used

Filter Material
Assumed Solid Fraction = 63.13 %
Assumed Porosity = 36.87 %

Weather Warm, dry, light wind, dry ground

Geology SAND over CLAY

Remarks
Slow discharge observed.
Test termination agreed with client representative; Test 2 and Test 3 not required.
Volume of gravel fraction assumed to be 63.13% of the total volume of gravel filled space, giving an estimated porosity of 36.87%.
Water level did not reach 25% EDP; infiltration rate cannot be given.
Gravel fill up to 0.40m BGL to support unstable pit. Water depth measurements were taken from top of pipe 1.19m AGL.
Pit was dry before adding water; water added to the pit to 0.35m BGL (Test 1).
Side wall collapsed on addition of water; treat results with caution.

Calculation

Test 1 ■			Test 2 ◆			Test 3 ▲		
Start Time =	12:10		Start Time =			Start Time =		
Test Top =	0.35	m BGL	Test Top =		m BGL	Test Top =		m BGL
Test Base =	2.00	m BGL	Test Base =		m BGL	Test Base =		m BGL
EDP =	1.65	m	EDP =		m	EDP =		m
75% EDP =	0.76	m BGL	75% EDP =		m BGL	75% EDP =		m BGL
50% EDP =	1.18	m BGL	50% EDP =		m BGL	50% EDP =		m BGL
25% EDP =	1.59	m BGL	25% EDP =		m BGL	25% EDP =		m BGL
V =	3.96	m^3	V =		m^3	V =		m^3
Vg =	2.50	m^3	Vg =		m^3	Vg =		m^3
Vp =	1.46	m^3	Vp =		m^3	Vp =		m^3
Vp75-25 =	0.73	m^3	Vp75-25 =		m^3	Vp75-25 =		m^3
ap =	7.68	m^2	ap =		m^2	ap =		m^2
Tp75 =	1800	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 25/11/2020 Checked by CAY 22/02/2021 Approved by NHA 07/05/2021

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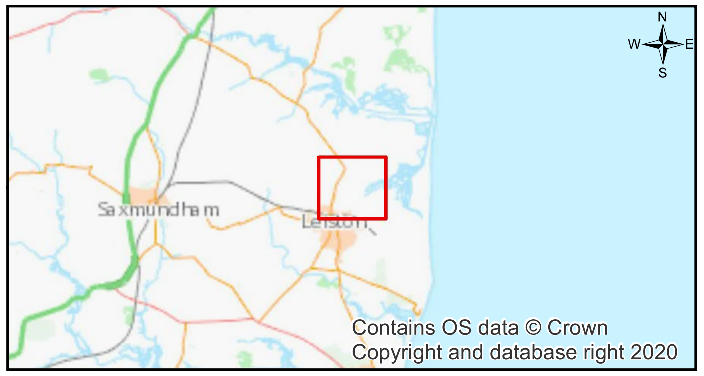
Location



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Scale: 1:250,000

0 2.5 5 10 15 20 Kilometres

Legend

Location Type

- ● Cable Percussion Borehole
- ☒ Trial Pit

Notes

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

Rev.	Date	Description	Initials
1	07/05/2021	Draft Issue	LCB

Scale: 1:7,500

0 50 100 200 300 400 500 600 Metres

Coordinate System

British National Grid

Client

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

Investigation Supervisor
WSP

Contractor

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

Project Title

AD6 Road Schemes

Drawing Title

Exploratory Location Plan

Drawing Number

B.2.1

Drawn By	Checked By	Issued On	Project No.	Sheet Size	Rev.
LCB		07/05/2021	F181386	A3	1

Contract Name	AD6 Road Schemes			Location ID
Client	NNB Generation Company (SZC) Limited			**AD6-TH305**
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
Template: FGSL/HBSI/FGSL BH Summary.hbt/Config Fugro Rev5/26/06/2019/TS+AW				Print Date	14/05/2021

Contract Name	AD6 Road Schemes			Location ID
Client	NNB Generation Company (SZC) Limited			**AD6-TH305**
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

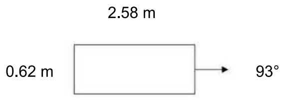
Depth (m)	Type	No.	Test Results
0.00 - 0.30	B	1	
0.10 - 0.20	D	2	
0.20 - 0.30	ES	3	
0.20	PID		< 0.1 ppm
0.30 - 0.80	B	4	
0.40 - 0.50	D	5	
0.50 - 0.60	ES	6	
0.50	PID		< 0.1 ppm
0.80 - 1.10	B	7	
0.90 - 1.00	ES	9	
0.90	PID		< 0.1 ppm
1.00 - 1.10	D	8	
1.10 - 2.20	B	10	
1.50 - 1.60	ES	12	
1.50	PID		< 0.1 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	



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

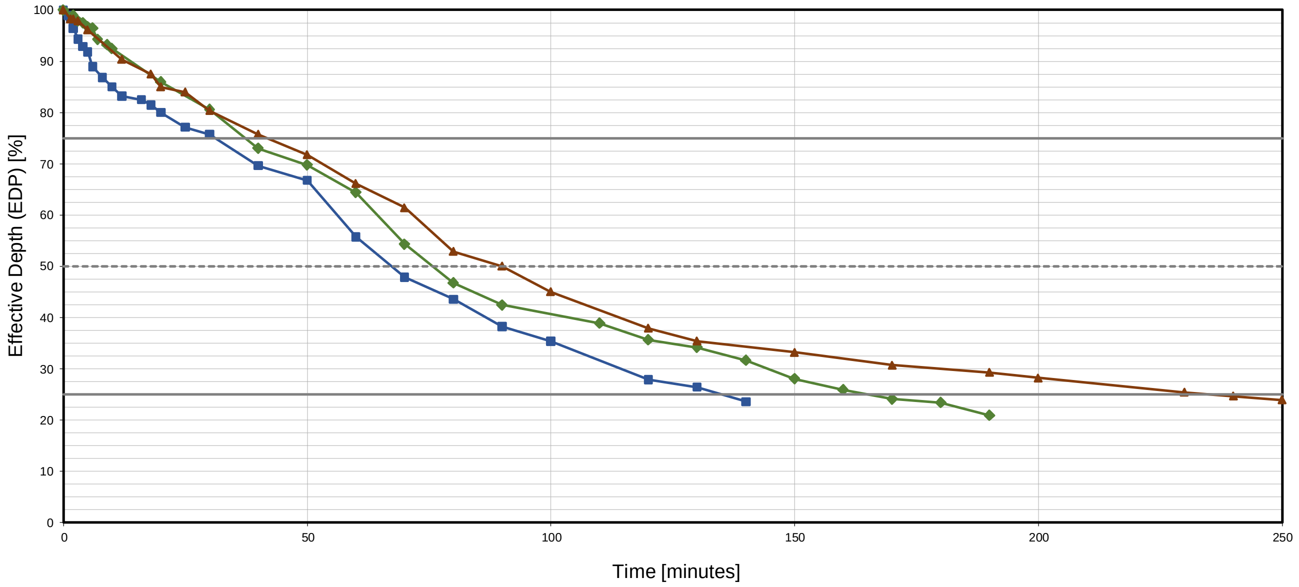
Print Date 13/05/2021

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

FUGRO

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

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

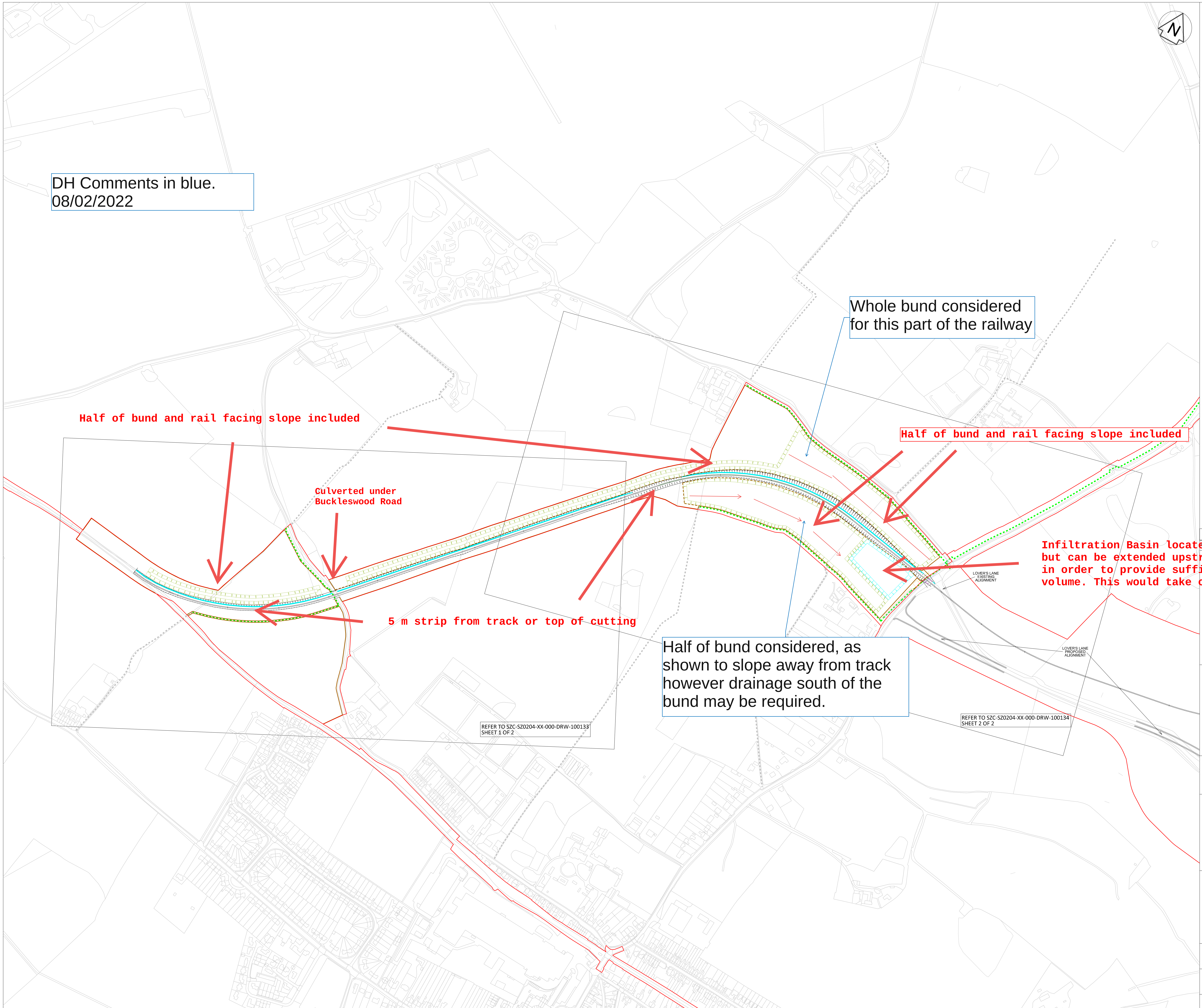
Input by AH 24/03/2021

Checked by CAY 13/05/2021

Contract No. F181386

APPENDIX C

GREEN RAIL ROUTE WEST OF ABBEY ROAD CONTRIBUTING RUNOFF AREA



DH Comments in blue.
08/02/2022

Whole bund considered for this part of the railway

Half of bund and rail facing slope included

Culverted under Buckleswood Road

5 m strip from track or top of cutting

Half of bund considered, as shown to slope away from track however drainage south of the bund may be required.

Half of bund and rail facing slope included

Infiltration Basin located her but can be extended upstream in order to provide sufficient volume. This would take out some bund

NOTES:

THE PROPOSED GREEN RAIL ROUTE WOULD BE CONSTRUCTED, OPERATED AND MAINTAINED ANYWHERE WITHIN THE AREA SHOWN ON WORK PLAN SZ/WP/09 REV.01 (SHOWING LATERAL LIMITS OF DEVIATION) AND TO A MAXIMUM OF +/- 1 METRE VERTICALLY

KEY:

INFRASTRUCTURE FEATURES
LANDSCAPE BUND
DEVELOPMENT SITE BOUNDARY
SWALE OR INFILTRATION BASIN
EXISTING GROUND LEVELS
PROPOSED PUBLIC RIGHT OF WAY
EXISTING PUBLIC RIGHT OF WAY
SECURITY FENCING
ECOLOGICAL FENCING
EARTHWORKS

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION / COMMENTS	APPROVED
01	FEB 2020	ES	PF	DCO SUBMISSION	SZC Co

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PROJECT:
SIZEWELL C

DOCUMENT:
APPLICATION DRAWING
- FOR APPROVAL REGULATION 5(2)(o)

DRAWING TITLE:
GREEN RAIL ROUTE
PROPOSED GENERAL ARRANGEMENT PLAN
KEY PLAN

DRAWING NO: SZC-SZ0204-XX-000-DRW-100473
REVISION: 01
DATE: FEB 2020
DRAWN: E.S.
SCALE: 1:2500 @ A0

SCALE BAR:
0 50 100 150 200 250m
SCALE 1:2500

APPENDIX D

GREEN RAIL ROUTE WEST OF ABBEY ROAD HYDRAULIC CALCULATIONS

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

Half Drain Time exceeds 7 days.

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

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	6.972	0.230	0.0	1439.3	O K
30 min Summer	7.054	0.312	0.0	1961.9	O K
60 min Summer	7.141	0.399	0.0	2523.9	O K
120 min Summer	7.249	0.507	0.0	3230.1	O K
180 min Summer	7.326	0.584	0.0	3737.5	O K
240 min Summer	7.387	0.645	0.0	4147.5	O K
360 min Summer	7.484	0.742	0.0	4799.9	O K
480 min Summer	7.557	0.815	0.0	5298.3	O K
600 min Summer	7.613	0.871	0.0	5686.3	O K
720 min Summer	7.658	0.916	0.0	5998.3	O K
960 min Summer	7.724	0.982	0.0	6460.1	O K
1440 min Summer	7.805	1.063	0.0	7027.6	O K
2160 min Summer	7.868	1.126	0.0	7477.0	O K
2880 min Summer	7.904	1.162	0.0	7733.1	Flood Risk
4320 min Summer	7.942	1.200	0.0	8004.4	Flood Risk
5760 min Summer	7.968	1.226	0.0	8189.4	Flood Risk
15 min Winter	6.990	0.248	0.0	1554.7	O K
30 min Winter	7.078	0.336	0.0	2119.1	O K
60 min Winter	7.172	0.430	0.0	2726.1	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	100.080	0.0	31
30 min Summer	68.208	0.0	46
60 min Summer	43.872	0.0	76
120 min Summer	28.074	0.0	136
180 min Summer	21.656	0.0	196
240 min Summer	18.024	0.0	256
360 min Summer	13.906	0.0	376
480 min Summer	11.513	0.0	496
600 min Summer	9.884	0.0	616
720 min Summer	8.689	0.0	736
960 min Summer	7.018	0.0	976
1440 min Summer	5.090	0.0	1456
2160 min Summer	3.610	0.0	2176
2880 min Summer	2.801	0.0	2896
4320 min Summer	1.933	0.0	4336
5760 min Summer	1.483	0.0	5776
15 min Winter	100.080	0.0	31
30 min Winter	68.208	0.0	46
60 min Winter	43.872	0.0	76

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

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
120 min Winter	7.288	0.546	0.0	3488.8	O K
180 min Winter	7.371	0.629	0.0	4036.9	O K
240 min Winter	7.436	0.694	0.0	4479.8	O K
360 min Winter	7.540	0.798	0.0	5184.4	O K
480 min Winter	7.618	0.876	0.0	5722.8	O K
600 min Winter	7.679	0.937	0.0	6141.8	O K
720 min Winter	7.727	0.985	0.0	6478.8	O K
960 min Winter	7.798	1.056	0.0	6977.7	O K
1440 min Winter	7.884	1.142	0.0	7590.6	O K
2160 min Winter	7.952	1.210	0.0	8076.0	Flood Risk
2880 min Winter	7.990	1.248	0.0	8352.6	Flood Risk
4320 min Winter	8.031	1.289	0.0	8645.7	Flood Risk
5760 min Winter	8.058	1.316	0.0	8845.5	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
120 min Winter	28.074	0.0	136
180 min Winter	21.656	0.0	196
240 min Winter	18.024	0.0	256
360 min Winter	13.906	0.0	376
480 min Winter	11.513	0.0	496
600 min Winter	9.884	0.0	616
720 min Winter	8.689	0.0	736
960 min Winter	7.019	0.0	976
1440 min Winter	5.090	0.0	1456
2160 min Winter	3.610	0.0	2176
2880 min Winter	2.801	0.0	2896
4320 min Winter	1.932	0.0	4336
5760 min Winter	1.483	0.0	5776

Rainfall Details

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

Time Area Diagram

Total Area (ha) 8.230

Time From:	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**	**Time From:**	**(mins) To:**	**Area (ha)**
0	4	2.057	4	8	2.057	8	12	2.058	12	16	2.058

Model Details

Storage is Online Cover Level (m) 8.200

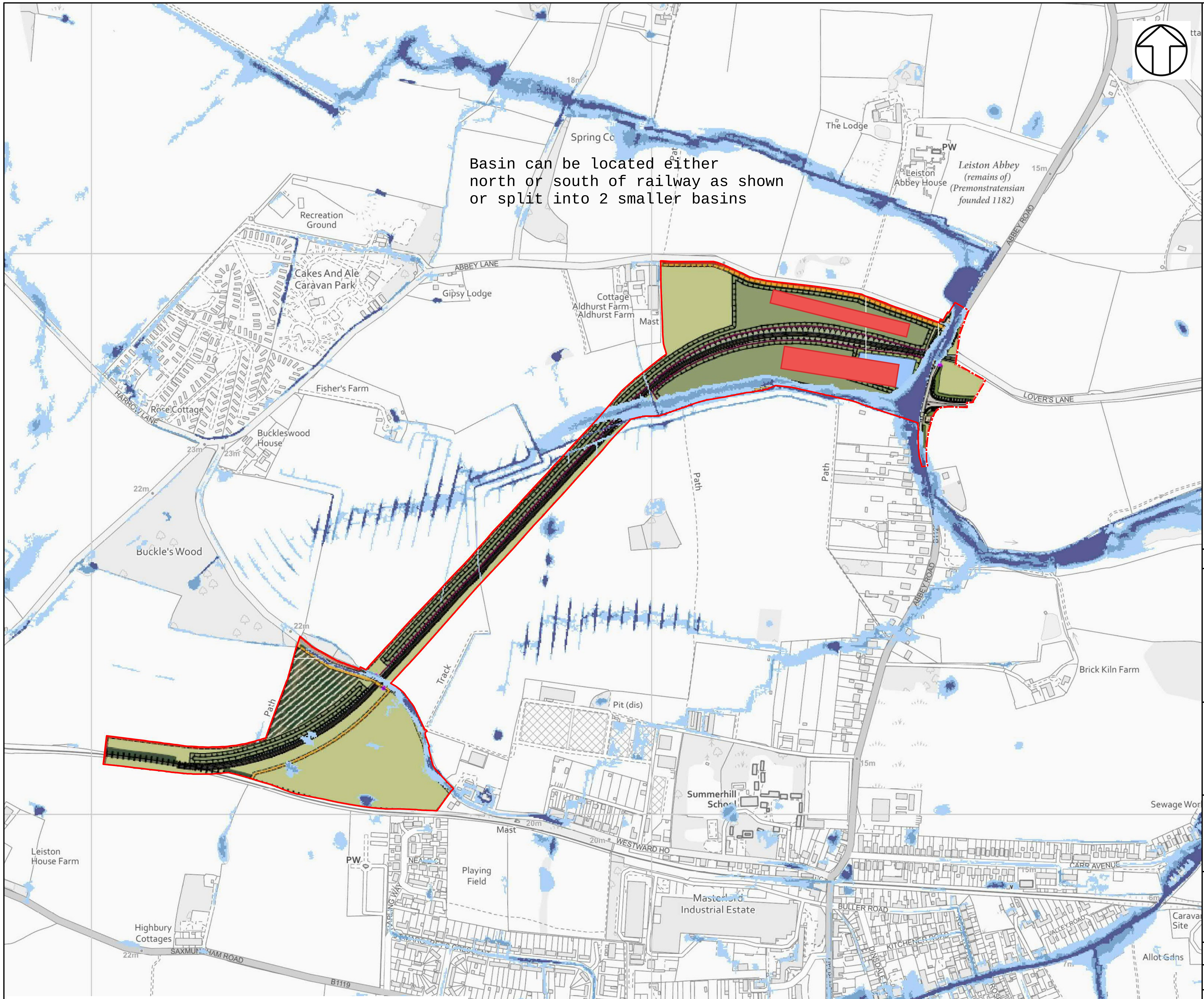
Infiltration Basin Structure

Invert Level (m) 6.742 Safety Factor 1.5
Infiltration Coefficient Base (m/hr) 0.00000 Porosity 1.00
Infiltration Coefficient Side (m/hr) 0.00000

Depth (m)	**Area (m²)**	**Depth (m)**	**Area (m²)**	**Depth (m)**	**Area (m²)**
0.000	6156.4	1.158	7160.6	1.458	7433.1

APPENDIX E

PROPOSED LOCATION OF INFILTRATION BASIN WEST OF ABBEY ROAD



Basin can be located either
north or south of railway as shown
or split into 2 smaller basins

NOTES

KEY

RAIL EXTENSION ROUTE

—	RAIL EXTENSION ROUTE DEVELOPMENT SITE BOUNDARY
-·-	VOLUME 9 ASSESSMENT AREA

ENVIRONMENT AGENCY RISK OF SURFACE WATER FLOODING

🟦	HIGH RISK (GREATER THAN 1 IN 30 ANNUAL PROBABILITY OF FLOODING)
🟦	MEDIUM RISK (BETWEEN 1 IN 100 AND 1 IN 30 ANNUAL PROBABILITY OF FLOODING)
🟦	LOW RISK (BETWEEN 1 IN 1,000 AND 1 IN 100 ANNUAL PROBABILITY OF FLOODING)
☐	VERY LOW RISK (LESS THAN 1 IN 1,000 ANNUAL PROBABILITY OF FLOODING)
🟥	Approximate basin footprint for 7,200 m3 storage

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DOCUMENT:
SIZEWELL C
RAIL
FLOOD RISK ASSESSMENT

DRAWING TITLE:
RAIL EXTENSION ROUTE
ENVIRONMENT AGENCY
RISK OF SURFACE WATER FLOODING MAP
WITH SITE LAYOUT

DRAWING NO:
FIGURE 4

DATE:	DRAWN:	SCALE :
FEB 2020	E.H.	1:6,500 @A3

SCALE BAR
0 60 120 180 240 300
M

APPENDIX F

GREEN RAIL ROUTE EAST OF ABBEY ROAD HYDRAULIC CALCULATIONS

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

Half Drain Time : 226 minutes.

Storm Event	**Max Level (m)**	**Max Depth (m)**	**Max Infiltration (l/s)**	**Max Control (l/s)**	**Max Σ Outflow (l/s)**	**Max Volume (m³)**	**Status**
15 min Summer	7.205	1.005	13.4	1.9	15.3	286.7	O K
30 min Summer	7.301	1.101	14.9	2.0	16.9	323.9	O K
60 min Summer	7.384	1.184	16.2	2.1	18.3	358.0	O K
120 min Summer	7.434	1.234	17.0	2.1	19.1	379.2	O K
180 min Summer	7.439	1.239	17.1	2.1	19.2	381.2	O K
240 min Summer	7.438	1.238	17.1	2.1	19.2	380.8	O K
360 min Summer	7.428	1.228	16.9	2.1	19.0	376.4	O K
480 min Summer	7.409	1.209	16.6	2.1	18.7	368.6	O K
600 min Summer	7.387	1.187	16.3	2.1	18.3	359.2	O K
720 min Summer	7.363	1.163	15.9	2.0	17.9	349.2	O K
960 min Summer	7.307	1.107	15.0	2.0	17.0	326.3	O K
1440 min Summer	7.206	1.006	13.4	1.9	15.3	286.9	O K
15 min Winter	7.350	1.150	15.7	2.0	17.7	344.0	O K
30 min Winter	7.458	1.258	17.4	2.1	19.5	389.2	O K
60 min Winter	7.553	1.353	19.0	2.2	21.2	431.6	O K
120 min Winter	7.615	1.415	20.1	2.2	22.3	460.2	O K
180 min Winter	7.621	1.421	20.2	2.2	22.4	463.1	O K
240 min Winter	7.615	1.415	20.1	2.2	22.3	460.5	O K
360 min Winter	7.597	1.397	19.8	2.2	22.0	451.7	O K
480 min Winter	7.566	1.366	19.2	2.2	21.4	437.7	O K

Storm Event	**Rain (mm/hr)**	**Flooded Volume (m³)**	**Discharge Volume (m³)**	**Time-Peak (mins)**
15 min Summer	184.621	0.0	299.4	25
30 min Summer	106.552	0.0	345.5	39
60 min Summer	61.496	0.0	399.2	66
120 min Summer	35.492	0.0	460.8	122
180 min Summer	25.733	0.0	501.2	158
240 min Summer	20.484	0.0	531.9	188
360 min Summer	14.851	0.0	578.5	254
480 min Summer	11.822	0.0	614.0	322
600 min Summer	9.905	0.0	643.0	392
720 min Summer	8.571	0.0	667.8	460
960 min Summer	6.770	0.0	703.2	594
1440 min Summer	4.855	0.0	756.4	858
15 min Winter	184.621	0.0	358.4	25
30 min Winter	106.552	0.0	413.6	39
60 min Winter	61.496	0.0	478.0	66
120 min Winter	35.492	0.0	551.7	120
180 min Winter	25.733	0.0	600.0	172
240 min Winter	20.484	0.0	636.8	196
360 min Winter	14.851	0.0	692.6	272
480 min Winter	11.822	0.0	735.1	348

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

Storm Event	**Max Level (m)**	**Max Depth (m)**	**Max Infiltration (l/s)**	**Max Control (l/s)**	**Max Σ Outflow (l/s)**	**Max Volume (m³)**	**Status**
600 min Winter	7.531	1.331	18.6	2.2	20.8	421.8	O K
720 min Winter	7.495	1.295	18.0	2.1	20.2	405.6	O K
960 min Winter	7.415	1.215	16.7	2.1	18.8	371.0	O K
1440 min Winter	7.276	1.076	14.5	2.0	16.5	313.9	O K

Storm Event	**Rain (mm/hr)**	**Flooded Volume (m³)**	**Discharge Volume (m³)**	**Time-Peak (mins)**
600 min Winter	9.905	0.0	769.8	422
720 min Winter	8.571	0.0	799.4	494
960 min Winter	6.770	0.0	841.9	636
1440 min Winter	4.855	0.0	905.6	908

Rainfall Details

Rainfall Model	FEH
Return Period (years)	100
FEH Rainfall Version	1999
Site Location	GB 647450 264900 TM 47450 64900
C (1km)	-0.020
D1 (1km)	0.299
D2 (1km)	0.272
D3 (1km)	0.215
E (1km)	0.311
F (1km)	2.506
Summer Storms	Yes
Winter Storms	Yes
Cv (Summer)	0.568
Cv (Winter)	0.680
Shortest Storm (mins)	15
Longest Storm (mins)	1440
Climate Change %	+20

Time Area Diagram

Total Area (ha) 1.143

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

Model Details

Storage is Online Cover Level (m) 8.000

Infiltration Basin Structure

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

Depth (m)	**Area (m²)**	**Depth (m)**	**Area (m²)**
0.000	200.0	1.500	489.2

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0066-2300-1500-2300
Design Head (m)	1.500
Design Flow (l/s)	2.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	66
Invert Level (m)	6.200
Minimum Outlet Pipe Diameter (mm)	100
Suggested Manhole Diameter (mm)	1200

Control Points	**Head (m)**	**Flow (l/s)**
Design Point (Calculated)	1.500	2.3
Flush-Flo™	0.287	1.9
Kick-Flo®	0.587	1.5
Mean Flow over Head Range	-	1.8

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

Depth (m)	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**	**Depth (m)**	**Flow (l/s)**
0.100	1.6	1.200	2.1	3.000	3.2	7.000	4.7
0.200	1.8	1.400	2.2	3.500	3.4	7.500	4.9
0.300	1.9	1.600	2.4	4.000	3.6	8.000	5.0
0.400	1.8	1.800	2.5	4.500	3.8	8.500	5.1
0.500	1.7	2.000	2.6	5.000	4.0	9.000	5.3
0.600	1.5	2.200	2.7	5.500	4.2	9.500	5.4
0.800	1.7	2.400	2.9	6.000	4.4		
1.000	1.9	2.600	3.0	6.500	4.5		