The Sizewell C Project

6.3/ 10.14 Volume 2 Main Development Site Chapter 2 Description of the Permanent Development Appendix 2A of the Environmental Statement: Drainage Strategy - Clean Version - Part 3 of 3

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ANNEX 2A.5: EXPLANATORY TECHNICAL NOTE

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1. Introduction

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

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

2. Infiltration parameters – selection

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

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

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

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

This approach gave the following figures:

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

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

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

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

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

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

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

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

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

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

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

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

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

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

2021 Results:

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

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

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

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

3. Treatment Indices

3.1. ACA Treatment

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

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

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

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

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

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

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

3.2. Simplified Treatment Indices Approach

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

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

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

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

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

3.2.1. Temporary Construction Area

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

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

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

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

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

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

Table 3-4 - TCA SuDS mitigation indices

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

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

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

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

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

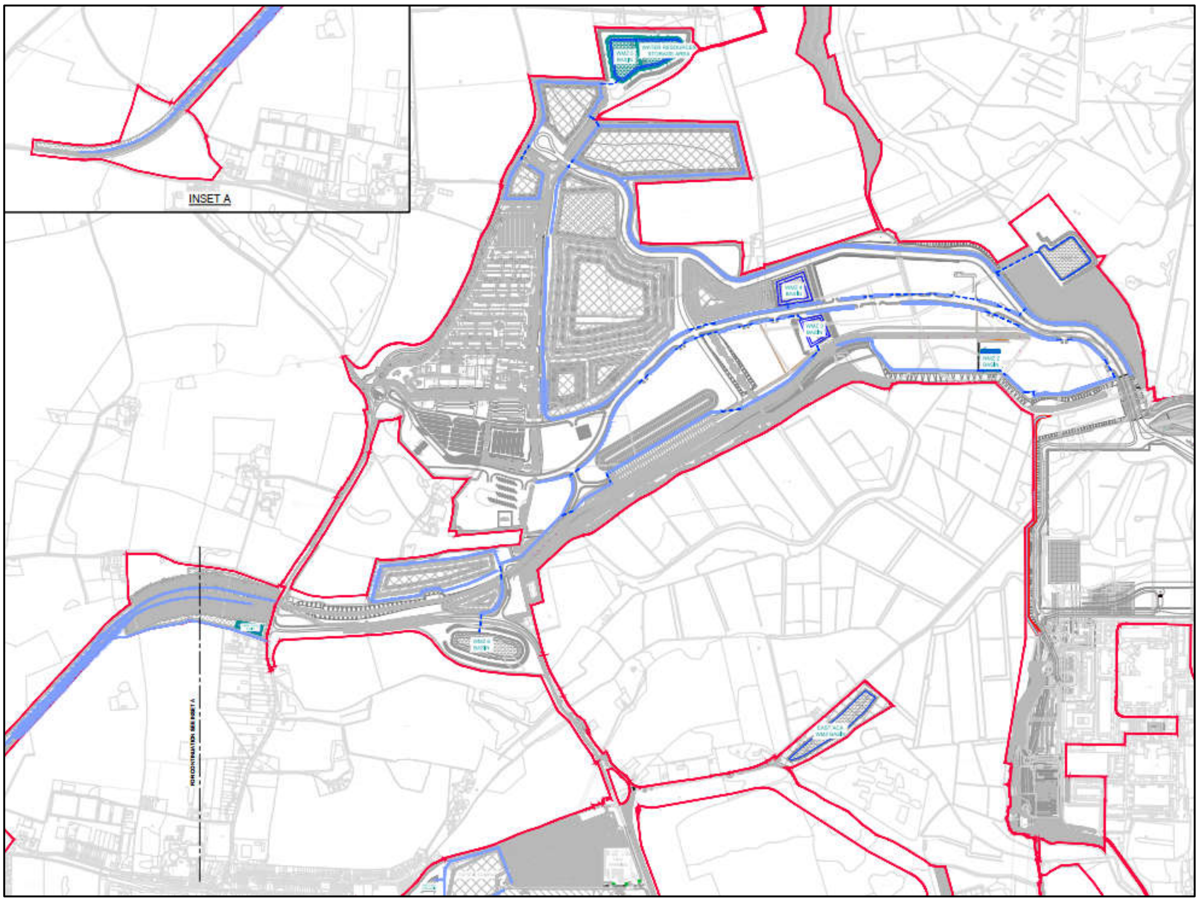
WMZ 8 is currently proposed to drain using filter drains along the verge and attenuated sub surface to restrict it to greenfield runoff rates. From the SIA, filter drains alone do not provide sufficient mitigation (0.4, 0.4, 0.4) and further work will be undertaken at the next design stage to ensure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

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

4. Perimeter Swale

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

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



5. Basin Design for Treatment

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

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

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

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

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

6. Calculation of impermeable / permeable areas

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

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

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

2. Soft areas comprise of stockpile areas only

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

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

7. Hydrological Catchment

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

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

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

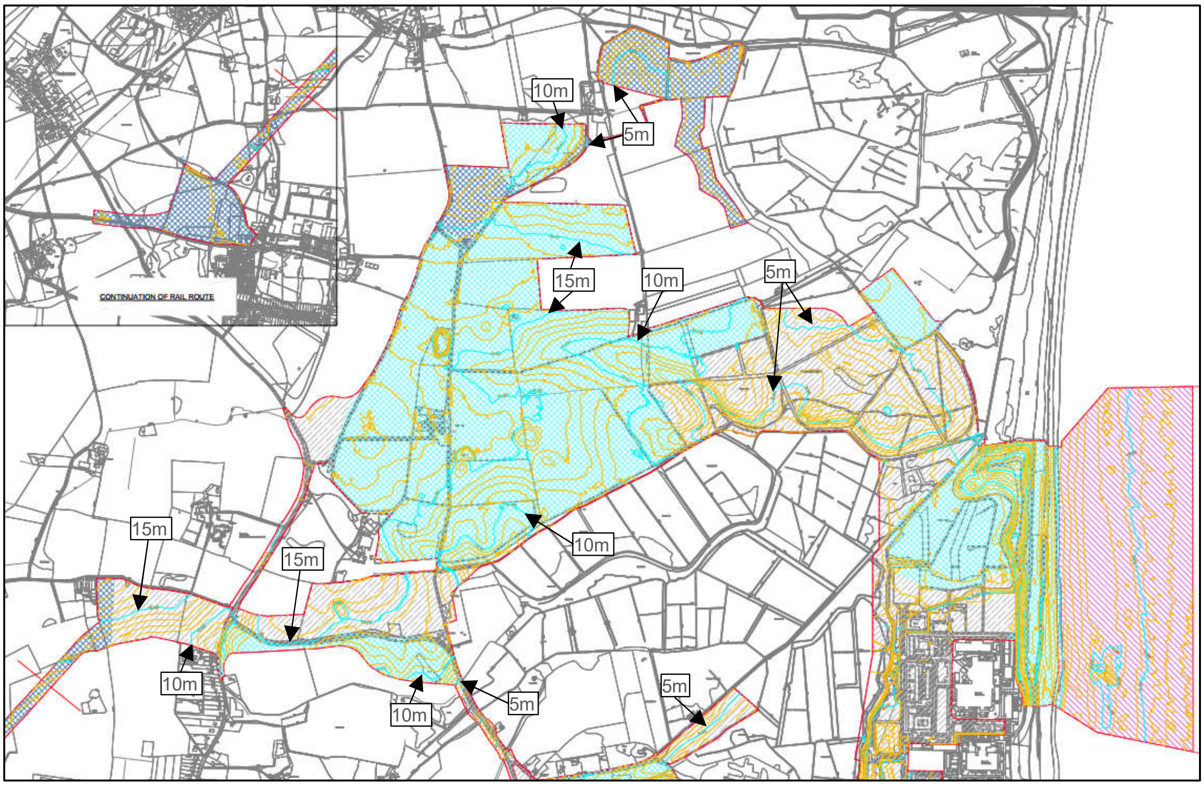


Figure 7-1 - Existing Ground Contours (SZC-EW0000-XX-000-DRW-400008)

8. Basin Design (Size)

8.1. WMZ Basin Parameters

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

Table 8-1 - Allocated WMZ Basin Sizes

WMZ	Area at base (m^2)	Base level of Basin (mAOD)	Area at freeboard level (m^2)	Depth to freeboard level (m)	WMZ Basin Volume (m^3) Base to Freeboard Level	Area at top of basin (m^2) 300mm Freeboard	WMZ Basin Volume (m^3) including freeboard
WMZ1	10579.2	1.200	12618.8	1.500	17398.5	13786.5	**21929.1**
WMZ2	3290.1	3.200	6274.5	3.700	17694.5	6554.8	**19689.8**
WMZ3	3224.3	5.000	6082.1	3.500	16286.2	6368.6	**18226.5**
WMZ4	5357	5.200	8931.8	3.500	25005.4	9279.1	**27808.6**
WMZ5	7051.6	6.000	9193.6	2.000	16245.2	9533.5	**19072.9**
WMZ6	7165.8	8.000	11287.5	2.100	19376.0	11911.5	**22892.8**
ACA East	12968.8	1.600	15431.6	1.100	15620.2	16117.6	**20360.5**
ACA West	659.6	5.700	1510.9	2.000	2170.5	1667.8	**2676.5**
Abbey Road	1268.6	6.742	1964.5	1.158	1872.0	2161	**2500.2**

8.2. Half Drain Times and Follow-on Storms

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

Table 8-2 - Source Control Basin Design Inputs

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

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

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

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

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

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

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

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

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

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

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

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

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

8.3. ACA West Basin Half Drain Time

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

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

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

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

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

8.4. Surface Water Outfalls – Early and Late

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

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

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

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

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

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

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

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

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

Table 8-7 - Summary of Early and Late discharges

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

9. Operational Infrastructure

9.1. Campus

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

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

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

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

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

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

9.2. Nuclear Island

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

9.2.1. Early

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

9.2.2. Construction

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

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

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

9.2.3. Operation

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

9.2.3.1. Permanent Car Park

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

10. Appendices

Appendix A. 2021 GI Site Location Plan

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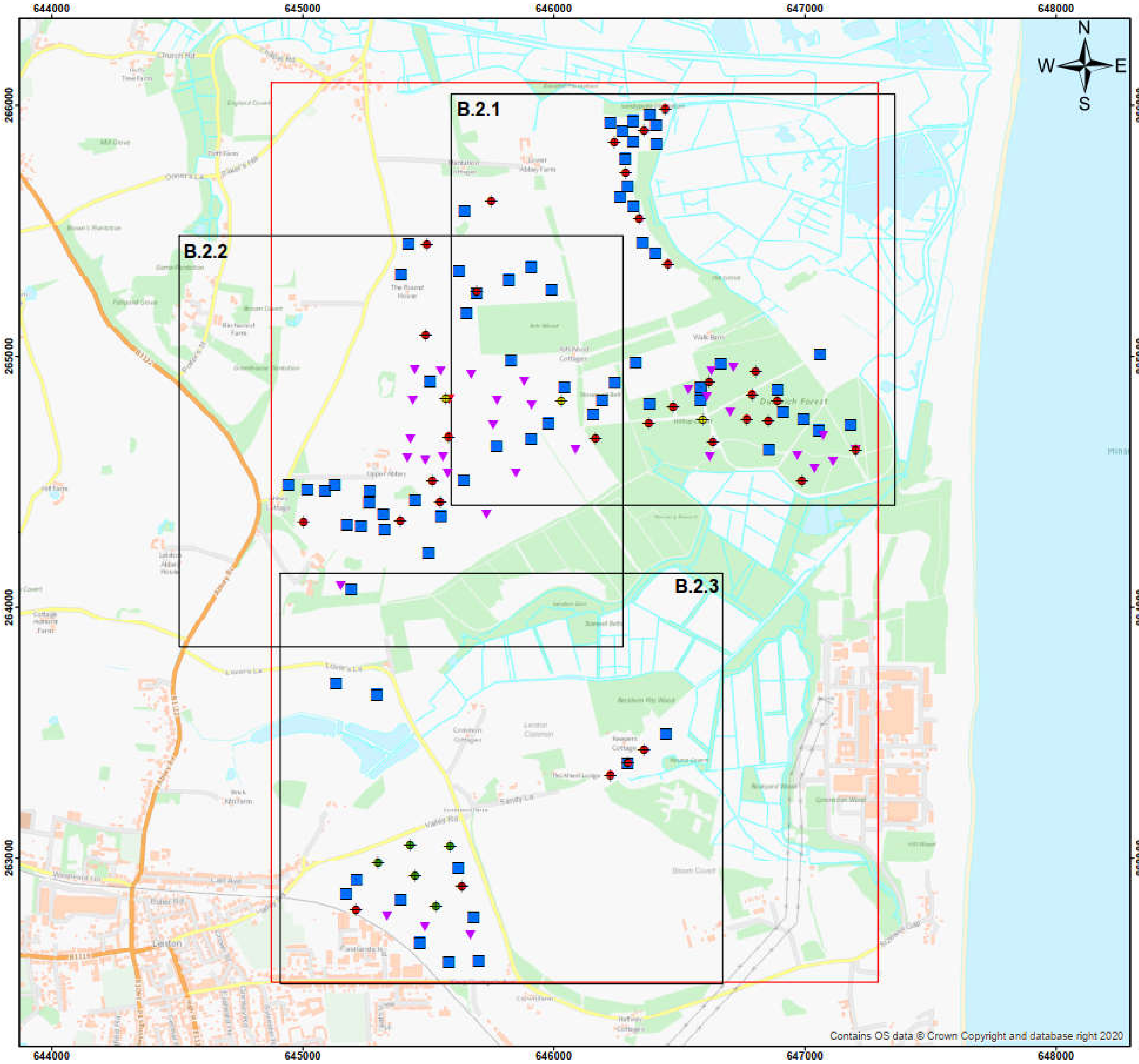

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Legend

- Site Boundary
- Exploratory Location Plan Sheets

Location Type

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

Notes

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

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

Scale: 1:15,000
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Coordinate System

British National Grid

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

Investigation Supervisor
Atkins

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

Project Title

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

Drawing Title

Site Location Plan

Drawing Number

B.1.1

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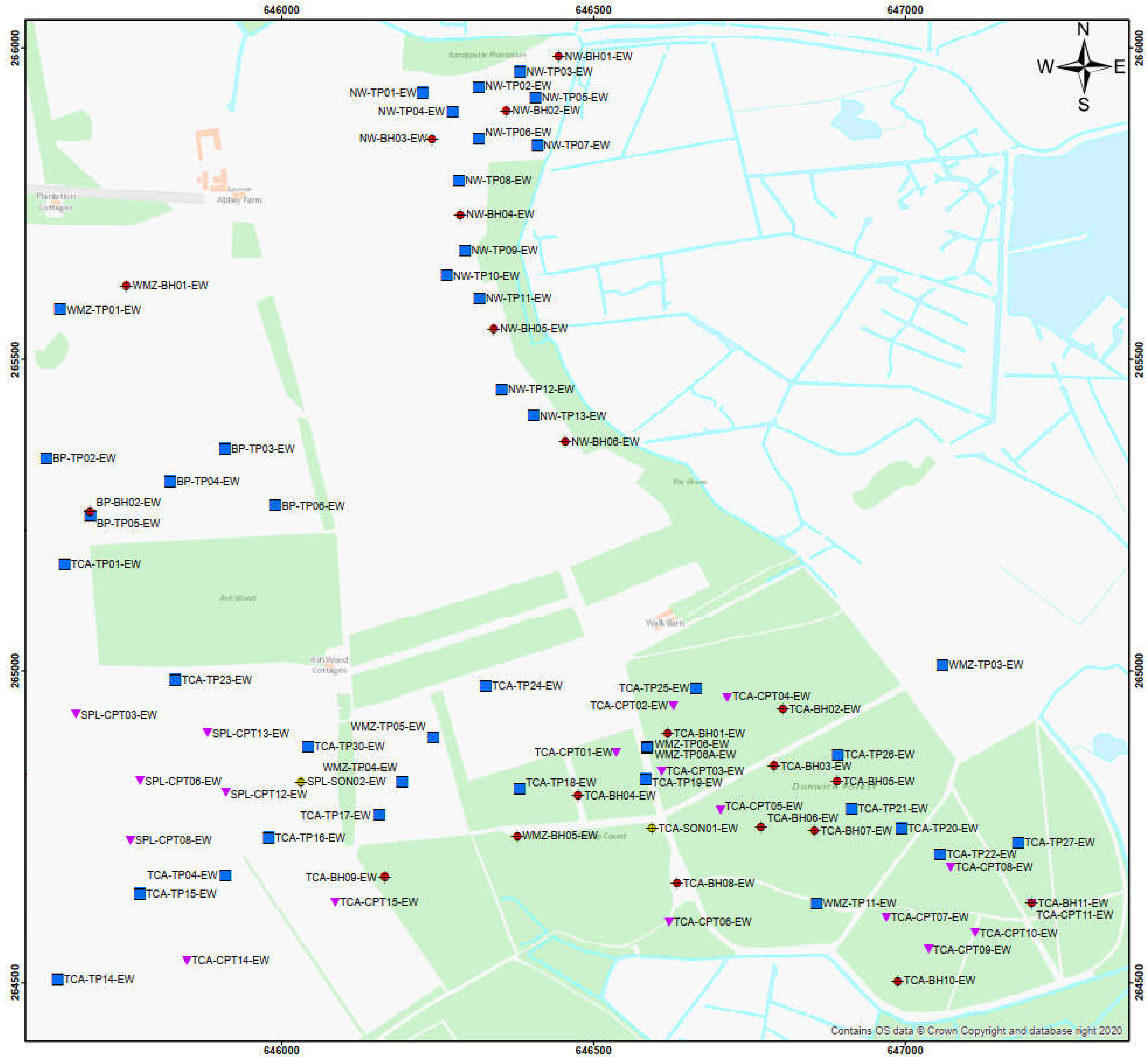

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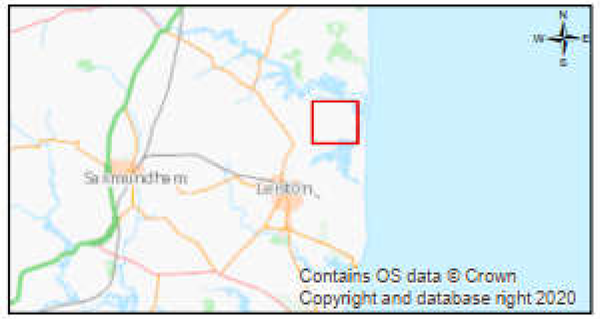

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Legend

Location Type

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- Dynamic Sampling Borehole
- CPT
- Inspection Pit with CPT Follow-on
- Trial Pit

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

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Coordinate System
British National Grid

Client
NNB Generation Company (SZC) Limited
EDF-NNB, 90 Whitfield Street,
London, W1T 4EZ
Tel: 020 321 36311
Website: www.edfenergy.com

SZC

Investigation Supervisor
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Contractor
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VAT No. GB 133 1704 09
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Project Title
Sizewell C - Onshore Ground Investigation for Enabling Works - TCA/ACA/NW/SPL/BP

Drawing Title
Exploratory Location Plan

Drawing Number
B.2.1

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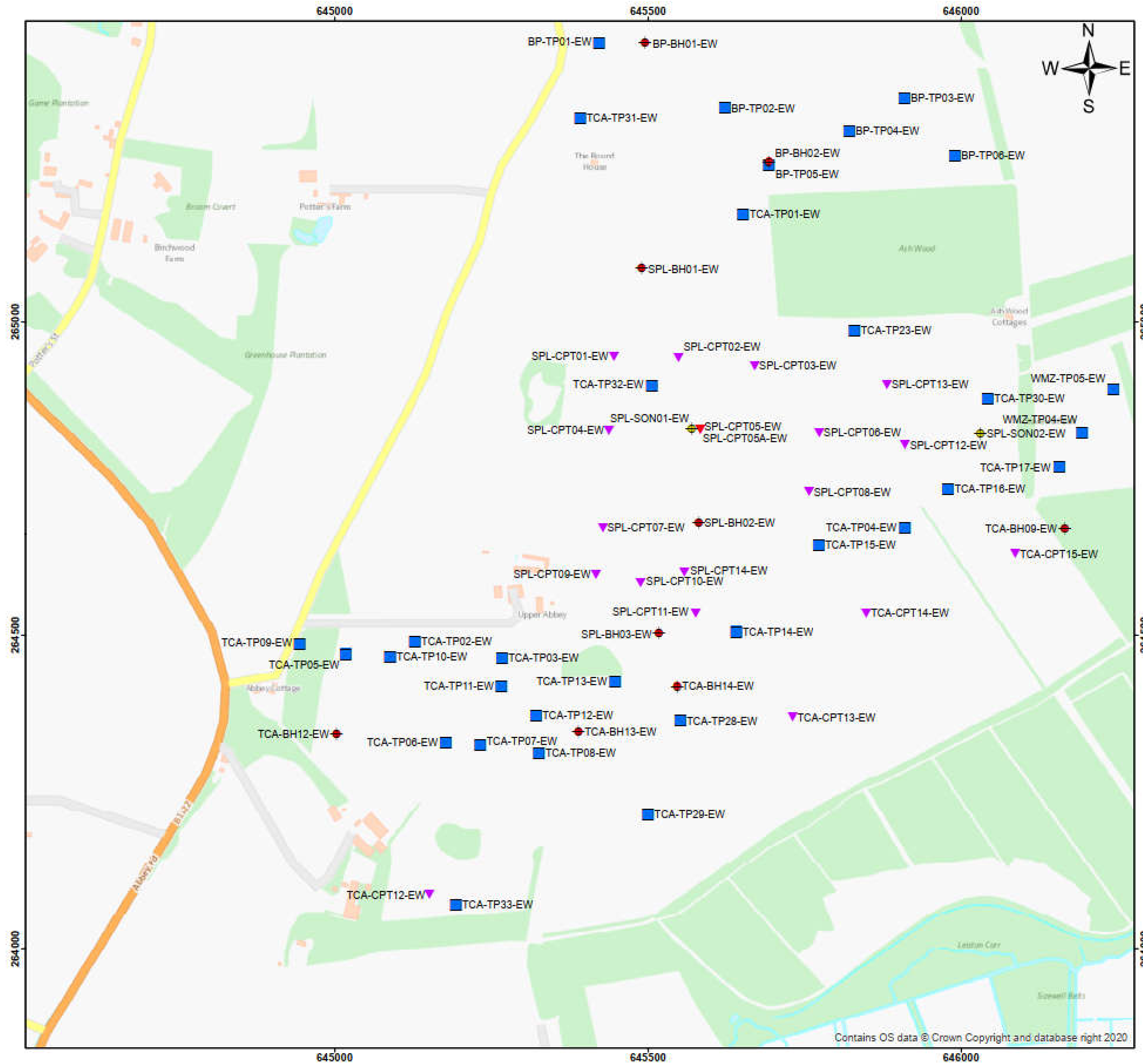

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Legend

Location Type

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

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

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

Client
NNB Generation Company (SZC) Limited
EDF-NNB, 90 Whitfield Street,
London, W1T 4EZ
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Registered in England No. 1284352
VAT No. GB 133 1704 09
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Sizewell C - Onshore Ground Investigation for Enabling Works - TCA/ACA/NW/SPL/BP

Drawing Title
Exploratory Location Plan

Drawing Number
B.2.2

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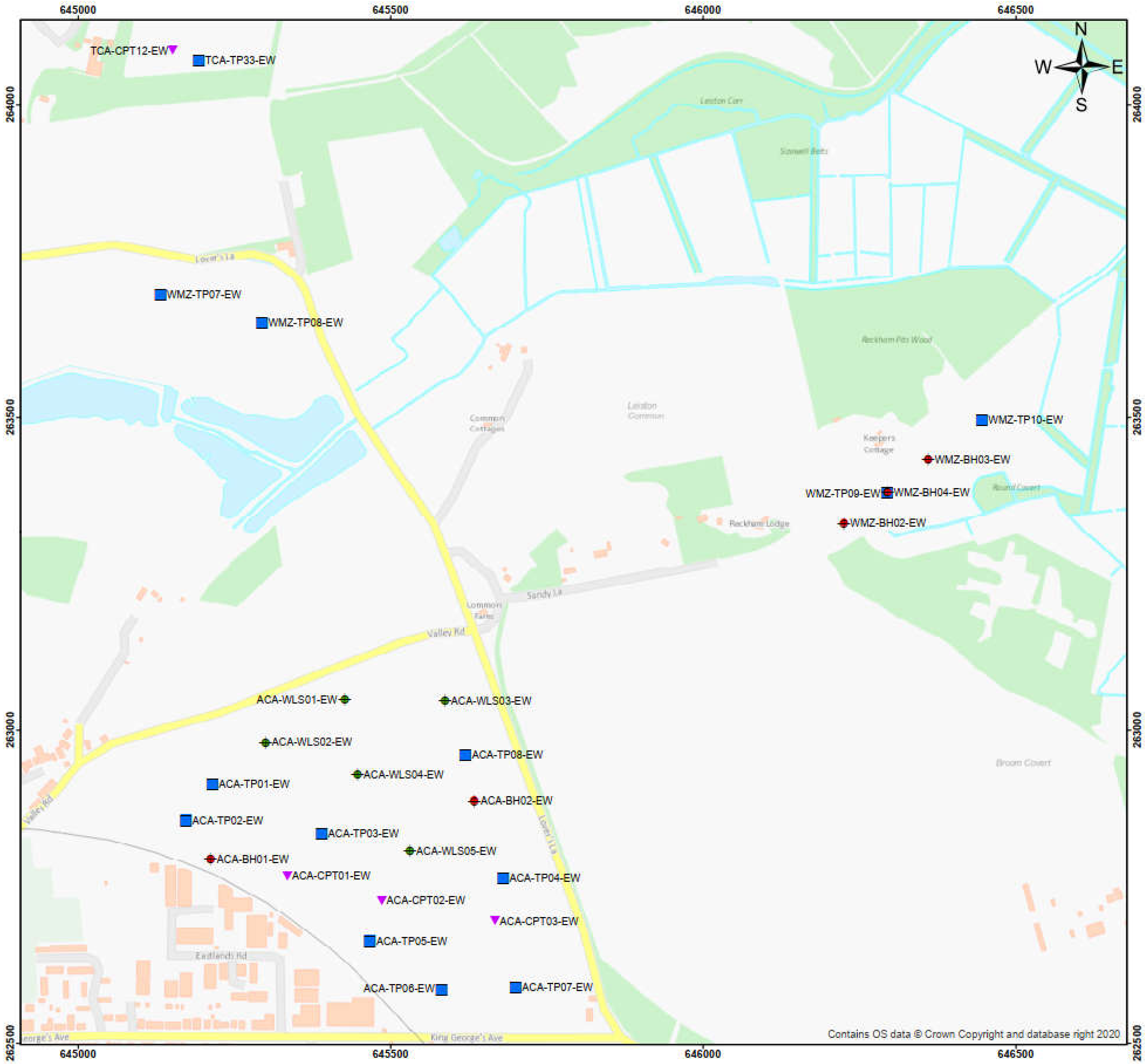

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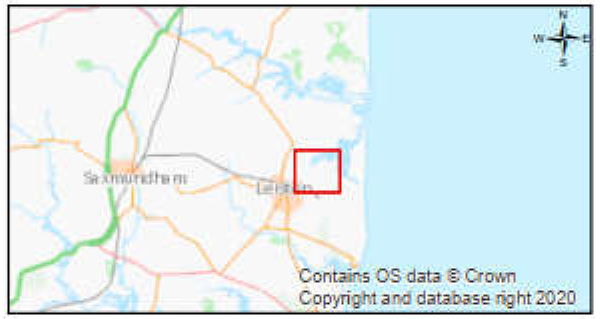

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Location Type

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

Notes

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

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

Scale: 1:6,000
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Coordinate System

British National Grid

Client	Contractor
NNB Generation Company (SZC) Limited EDF-NNB, 90 Whitfield Street, London, W1T 4EZ Tel: 020 321 38311 Website: www.edfenergy.com SZC Investigation Supervisor Atkins	Fugro GeoServices Limited Fugro House, Hithercroft Road, Wallingford, Oxfordshire, OX10 9RB, United Kingdom Registered in England No. 1284352 VAT No. GB 133 1704 09 www.fugro.com FUGRO

Project Title

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

Drawing Title

Exploratory Location Plan

Drawing Number

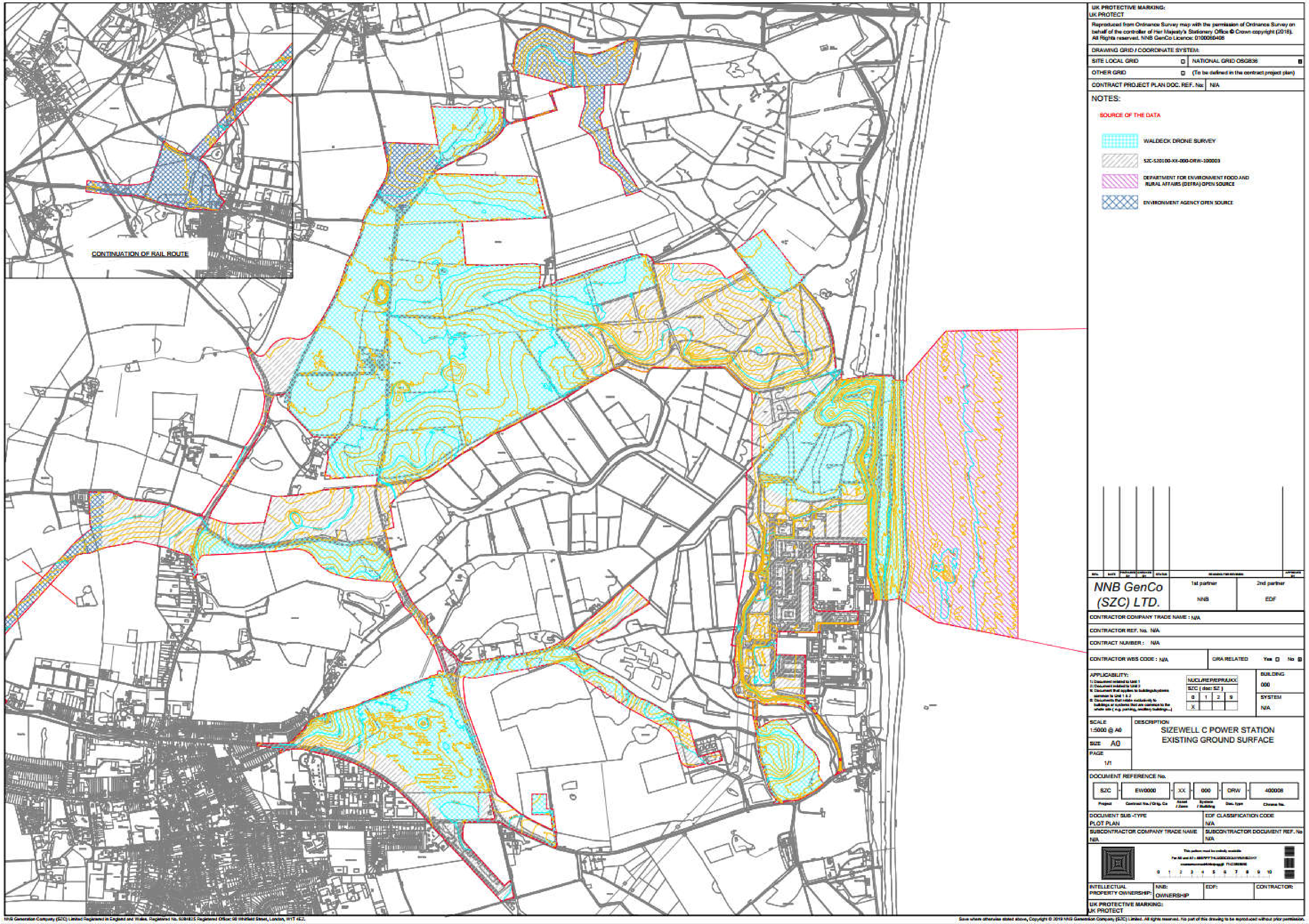
B.2.3

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

Appendix B. Existing Ground Surface

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

NOT PROTECTIVELY MARKED



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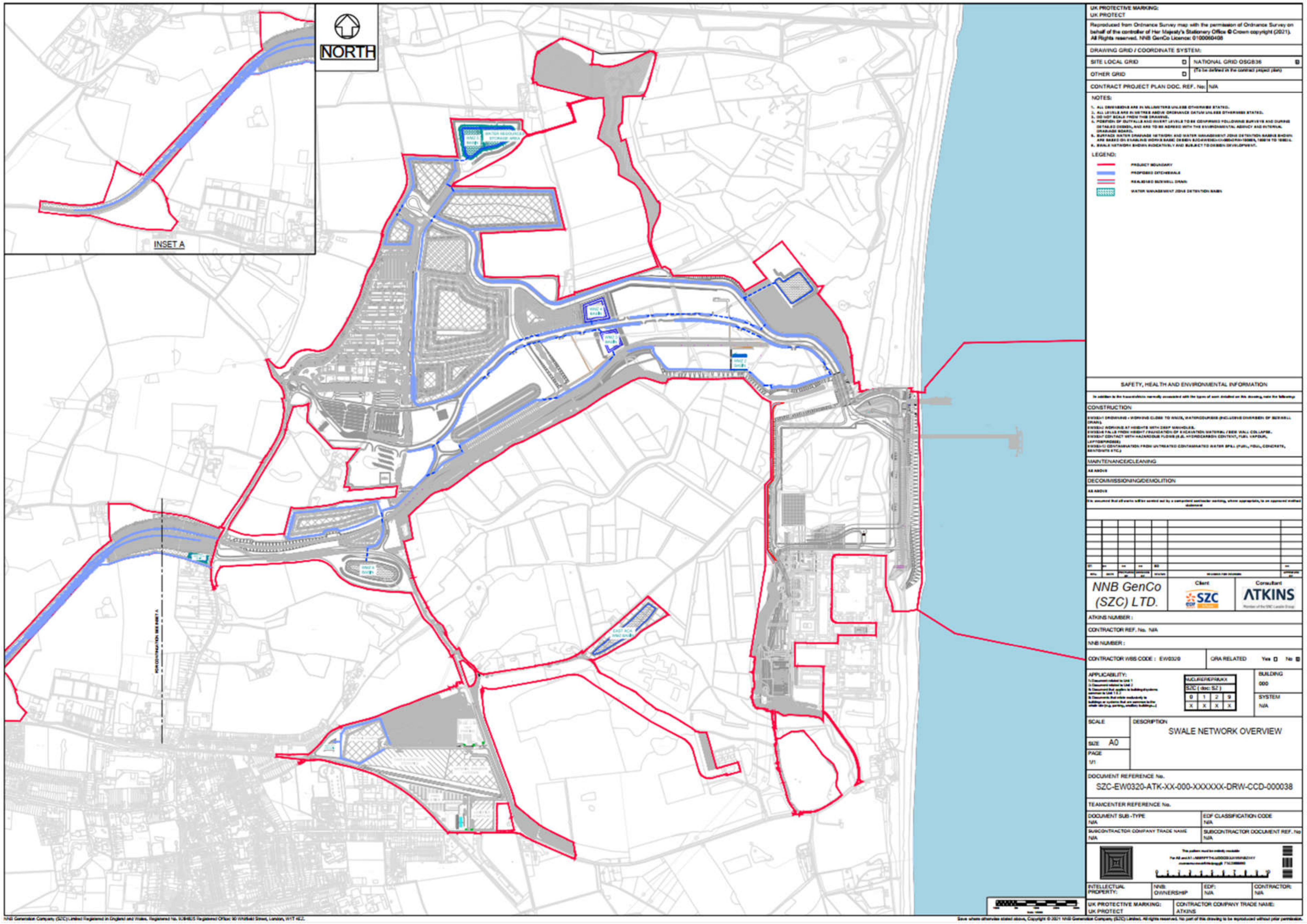
Appendix C. Swale Network Overview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NATIONAL GRID OSGB36
OTHER GRID
CONTRACT PROJECT PLAN DOC. REF. No: N/A
NOTES:
LEGEND:
SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION
CONSTRUCTION
MAINTENANCE/CLEANING
DECOMMISSIONING/DEMOLITION
NNB GenCo (SZC) LTD.
Client
SZC
Consultant
ATKINS
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CONTRACTOR REF. No. N/A
NNB NUMBER :
CONTRACTOR WBS CODE : EW0320
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N/A
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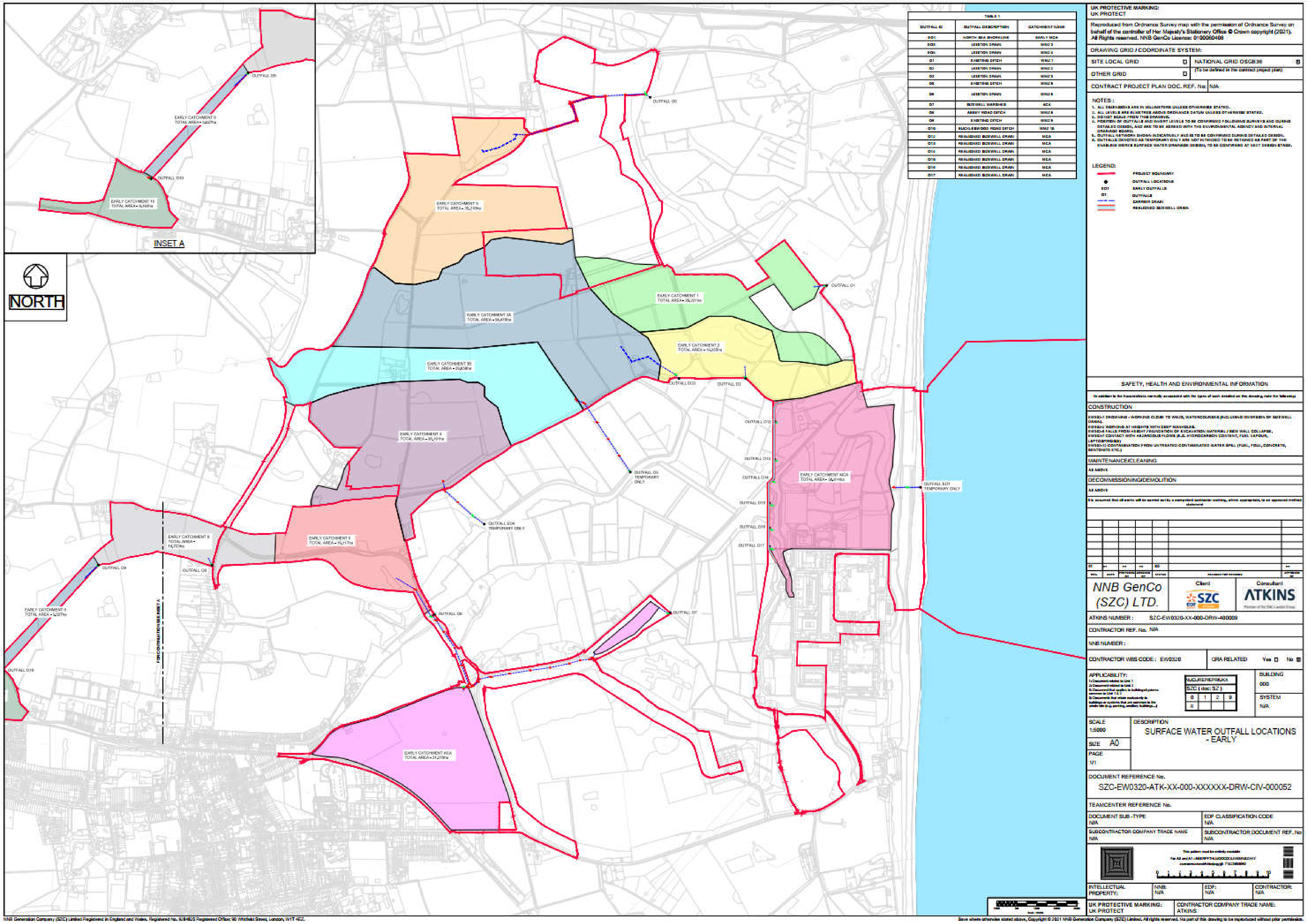
EDF: N/A
CONTRACTOR: N/A
UK PROTECTIVE MARKING:
UK PROTECT
CONTRACTOR COMPANY TRADE NAME:
ATKINS

Appendix D. Early & Late Catchments and Outfalls

D.1. Surface Water Outfall Locations – Early - SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052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NORTH
TABLE 1
OUTFALL ID
OUTFALL DESCRIPTION
CATCHMENT NAME
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EARLY MCA
LEISTON DRAIN
EXISTING DITCH
SIZEWELL MARSHES
ABBEY ROAD DITCH
REALIGNED SIZEWELL DRAIN
MCA
UK PROTECTIVE MARKING:
UK PROTECT
DRAWING GRID / COORDINATE SYSTEM:
SITE LOCAL GRID
NATIONAL GRID OSGB36
OTHER GRID
CONTRACT PROJECT PLAN DOC. REF. No: N/A
NOTES:
LEGEND:
PROJECT BOUNDARY
OUTFALL LOCATIONS
EARLY OUTFALLS
OUTFALLS
CARRIER DRAIN
REALIGNED SIZEWELL DRAIN
SAFETY, HEALTH AND ENVIRONMENTAL INFORMATION
CONSTRUCTION
MAINTENANCE/CLEANING
DECOMMISSIONING/DEMOLITION
NNB GenCo (SZC) LTD.
Client
SZC
Consultant
ATKINS
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CONTRACTOR REF. No. N/A
NNB NUMBER :
CONTRACTOR WBS CODE : EW0320
QRA RELATED Yes No
APPLICABILITY:
SZC (doc: SZ)
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X
BUILDING
000
SYSTEM
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SCALE
1:5000
SIZE A0
PAGE
1/1
DESCRIPTION
SURFACE WATER OUTFALL LOCATIONS - EARLY
DOCUMENT REFERENCE No.
SZC-EW0320-ATK-XX-000-XXXXXX-DRW-CIV-000052
TEAMCENTER REFERENCE No.
DOCUMENT SUB -TYPE
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EDF CLASSIFICATION CODE
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SUBCONTRACTOR COMPANY TRADE NAME
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SUBCONTRACTOR DOCUMENT REF. No
N/A
INTELLECTUAL PROPERTY:
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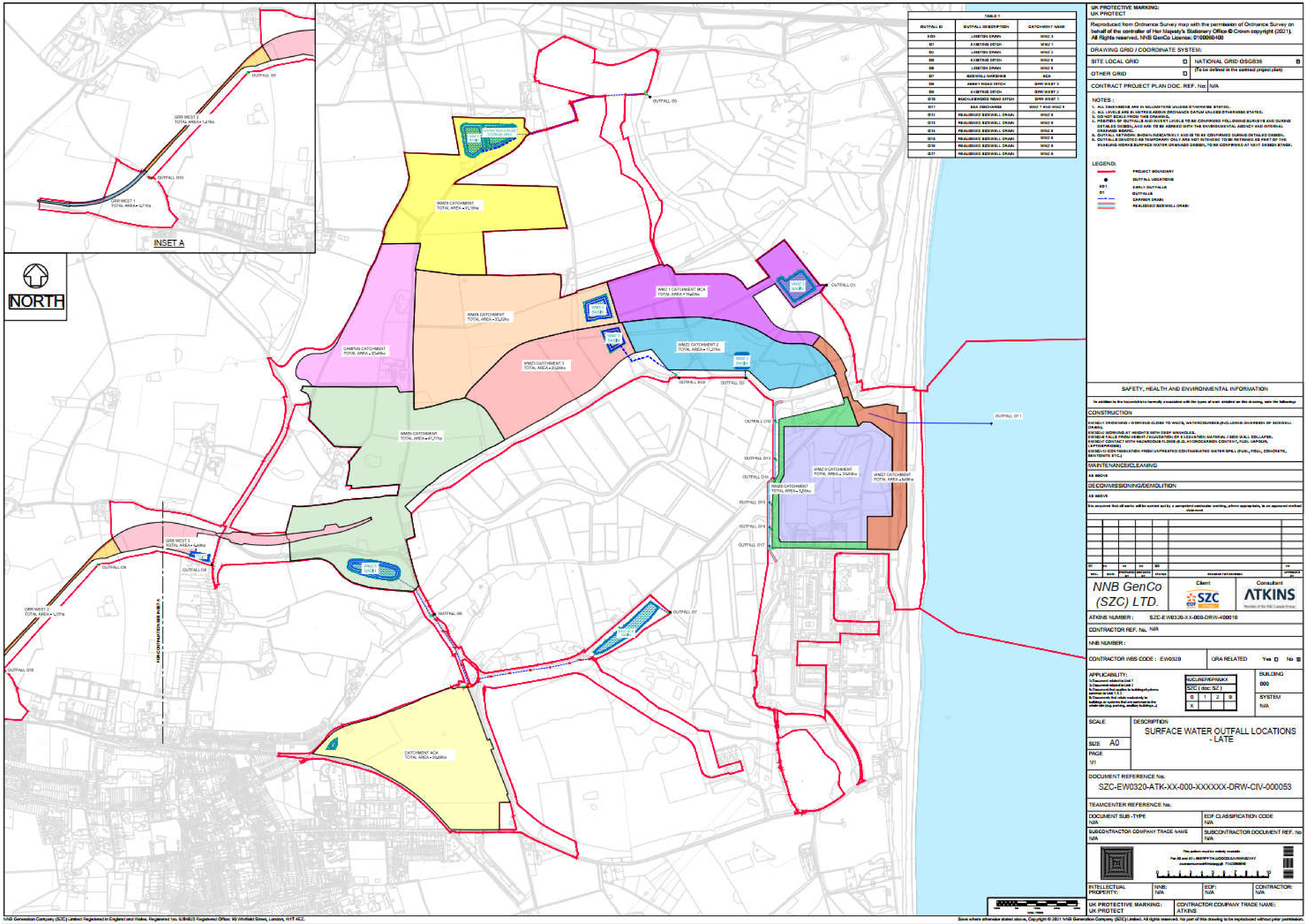
EDF: N/A
CONTRACTOR: N/A
UK PROTECTIVE MARKING:
UK PROTECT
CONTRACTOR COMPANY TRADE NAME:
ATKINS

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

NOT PROTECTIVELY MARKED



NOT PROTECTIVELY MARKED

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

CONTENTS

TABLES

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

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

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

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

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

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

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

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

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

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

2 PURPOSE

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

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

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

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

2.1.4 The purpose of this technical note is to provide details of data which validates the Outline Drainage Strategy, a description of how the proposed concept drainage infrastructure is developing and evolving and to demonstrate that it continues to provide for the effective and satisfactory drainage of the northern park and ride and its associated external road modification, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution.

3 DESCRIPTION OF DCO DRAINAGE CONCEPT DESIGN

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

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

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

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

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

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

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

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

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

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

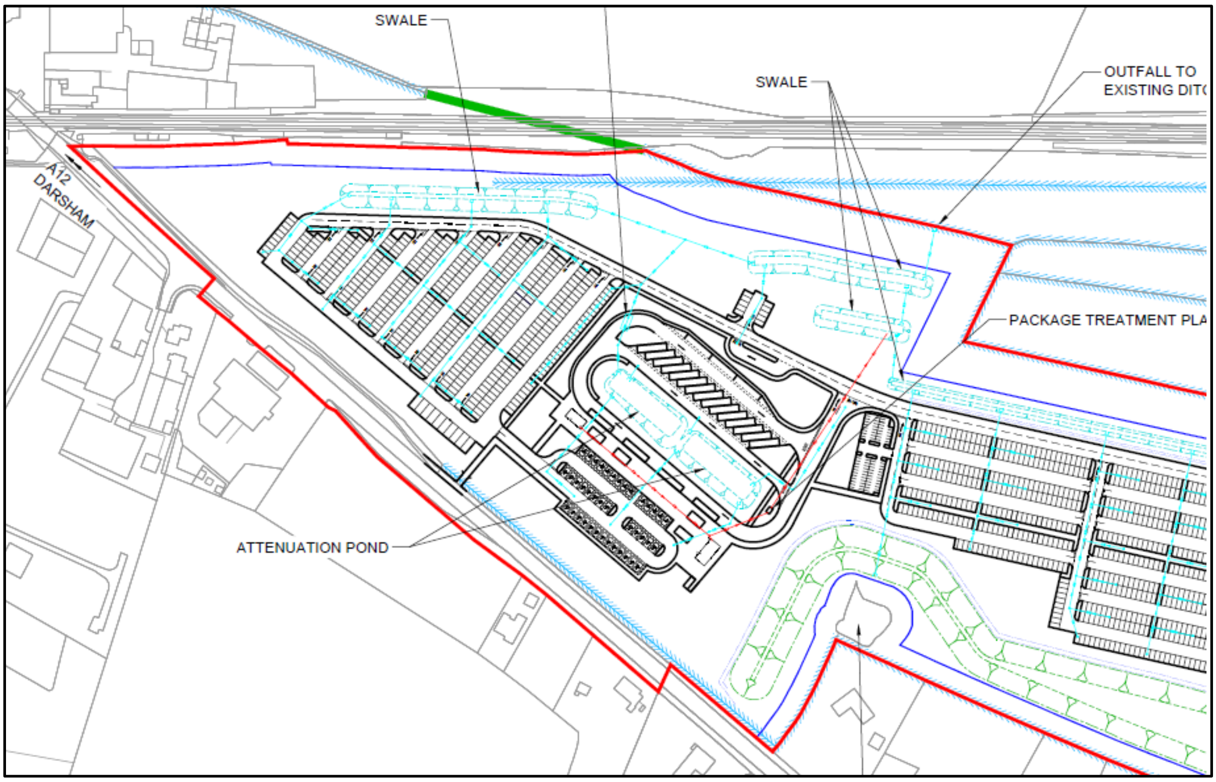
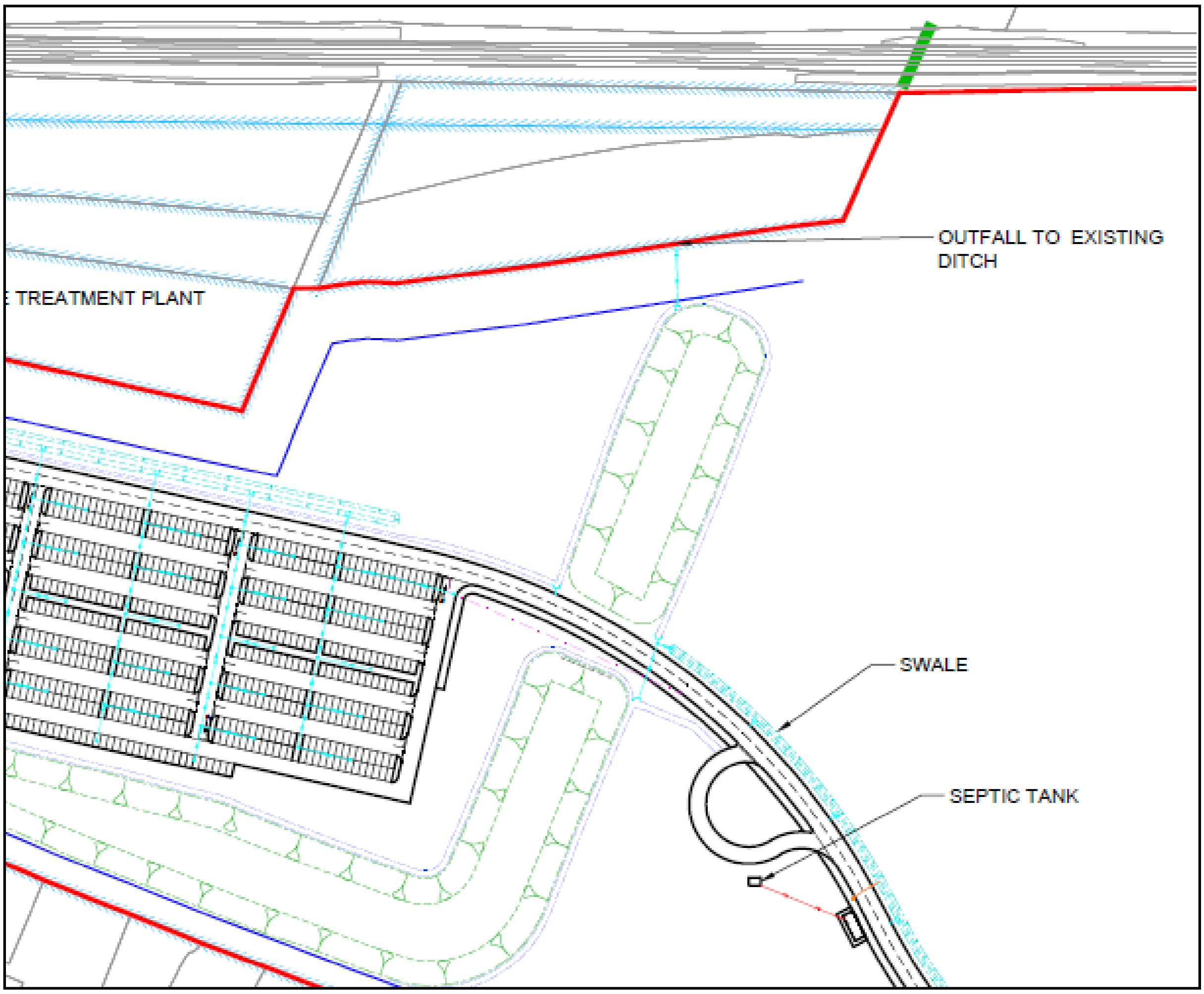
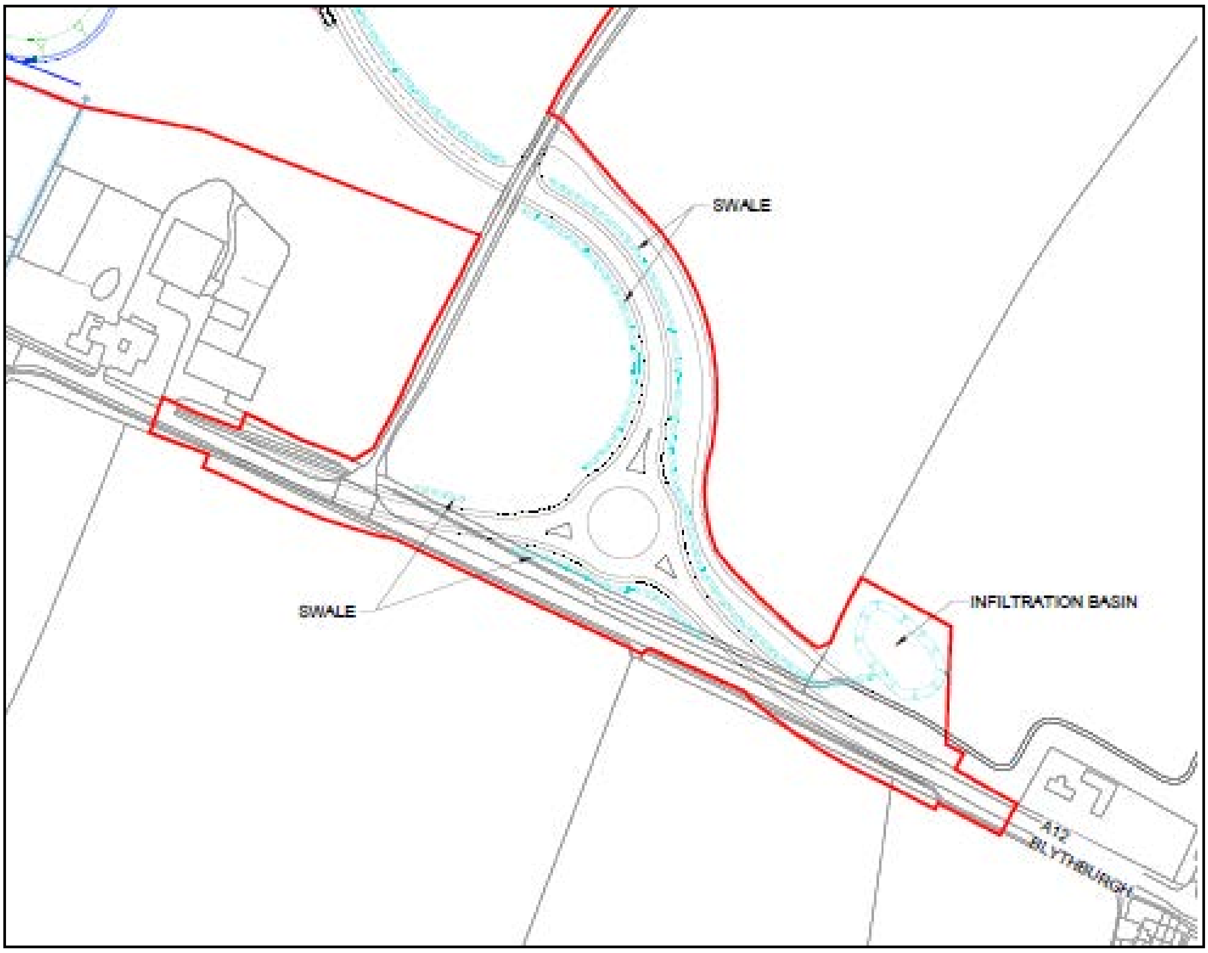


Plate 2: Northern park and ride internal layout showing concept drainage infrastructure to the north



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

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

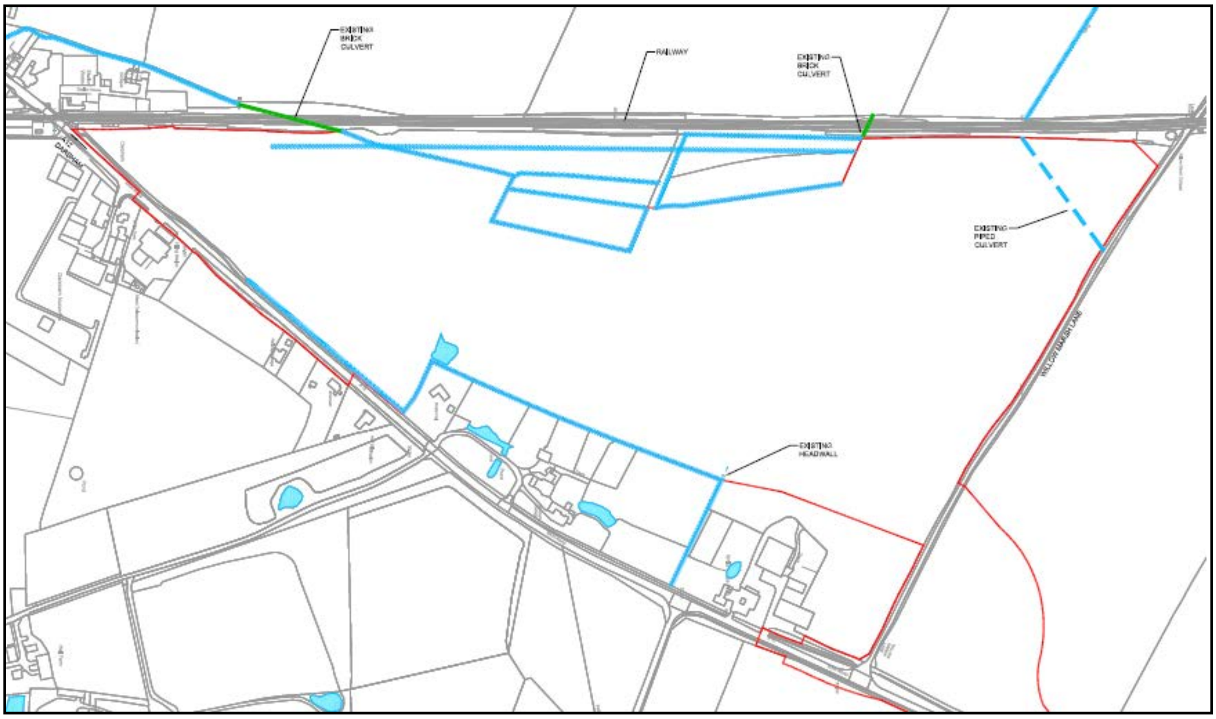


4 EXISTING SITE AND ADJACENT HIGHWAY DRAINAGE ARRANGEMENTS

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

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

Plate 4: Northern park and ride existing drainage infrastructure



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

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

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

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

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

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

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

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

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

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

4.1.11 It appears that the land to the west of the A12 is at a lower level such that the A12 forms a barrier. Overland flow from fields to the west builds up and is predicted to overflow across the road and then follow the field boundary on the east of the A12 before discharging into a watercourse located within 150 m of the A12.

4.1.12 It is possible that there is a field boundary ditch but this needs to be confirmed by site inspection. A site inspection would also confirm if there is a culvert crossing beneath the A12.

5 REVISED DRAINAGE DESIGN STRATEGY INPUT DATA

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

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

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

5.1.3 There is no new data in respect of the highway modifications with site access road and A12 roundabout to the north.

6 GROUND INVESTIGATION AND INFILTRATION TESTING RESULTS

6.1.1 Three trial pits were excavated within the site at locations shown in **Plate 7**.

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

Locations

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

6.1.3 These results clearly demonstrate that infiltration is not viable and therefore surface water runoff from the development site must be disposed to the available watercourse to the west of the site, within the red line boundary.

7 REVISED SURFACE WATER CONCEPT DRAINAGE DESIGN

7.1.1 The surface water arrangements for removal remain as broadly as described in **Volume 3 Northern Park and Ride Chapter 2 Description of the Northern Park and Ride** [APP-350] but are modified to take account of the infiltration test results obtained in May 2020 and the site inspection.

7.1.2 Runoff from roofs would be drained via downpipes and gullies, as appropriate to underground carrier drains and discharge into attenuation basins and swales.

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

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

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

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

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

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

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

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

7.1.9 Hydraulic modelling calculations have been undertaken to determine a required attenuation storage volume if the discharge rate is limited to Qbar. The calculations are shown in **Appendix B**. The required storage is 8,700 m^3 which is an increase of 200% on the concept design. However as shown in a copy of the site layout plan in **Appendix A** this volume represents a very small proportion of the site and would be accommodated within the Order Limits, enabling the appropriate discharge rate to be met. The plan areas shown are for illustrative purposes only and do not represent the fixed or final position of the attenuation storage positions.

7.1.10 The layout drawing shown in **Appendix A** continues to show an infiltration basin within the developed area and swales between the developed area and the watercourse to the west. The infiltration basin would become an attenuation basin. It is intended that the additional required storage would include these features but more swales and basins would be required.

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

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

7.1.13 The position of the attenuation facilities and levels of outfall connections to the watercourse would need to be set to ensure no risk of flooding within the site or increase of flood risk to 3rd party land and to ensure a free outfall with no impact on flow control devices.

8 REVISED FOUL WATER CONCEPT DRAINAGE DESIGN STRATEGY – PARK AND RIDE

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

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

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

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

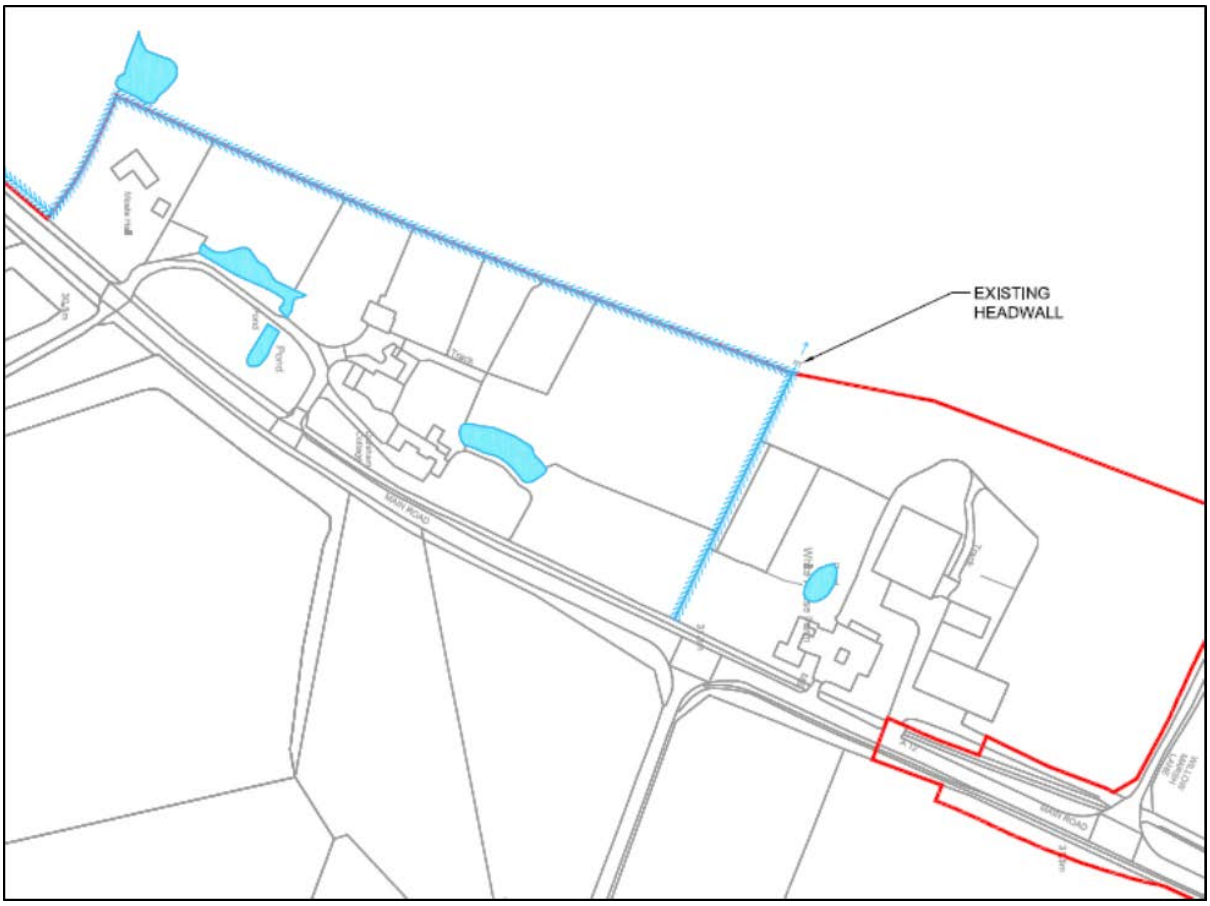
8.1.5 During design development should it be determined that the infiltration rate is insufficient for the provision of a drainfield and therefore creating a flood risk, it would be necessary to collect wastewater and sewage in a cesspit from which it can be collected and regularly be removed by tanker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

9 PROTECTION OF EXISTING DRAINAGE

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

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

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



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

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

10 REVISED SURFACE WATER CONCEPT DRAINAGE DESIGN STRATEGY – A12 ROUNDABOUT AND MAIN SITE ACCESS ROAD

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

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

10.1.3 Although no infiltration testing has been undertaken in vicinity to the infiltration basin, given the results of testing within the development site it is likely that infiltration would not be viable. This will need to be confirmed by testing.

10.1.4 On the basis that infiltration would not be viable, the infiltration basin would change to an attenuation basin with a positive outfall. The basin outfall would pass under the A12 and along the field boundary to the existing watercourse located within 150 m. A culvert beneath the road and boundary ditch may already exist and be capable of being utilised but this will be confirmed by future site visit. Hydraulic calculations have been undertaken to establish the required attenuation basin storage volume and are shown in **Appendix C**. The required footprint for the basin is shown in **Appendix A.**

10.1.5 In summary, based on Qbar calculated as being 4.6 l/s and assuming a tank with a depth of 1.5 m and vertical sides, the storage volume required would be 975 m3 which is less than the footprint for the basin shown at concept design stage. The attenuation would be constructed in the form of an open basin in order to intercept overland flow from adjacent land. This would ensure that the currently predicted surface water flood risk to the A12 due to overland flow from adjacent land is mitigated including allowance for climate change.

11 SUMMARY AND CONCLUSION

11.1.1 The purpose of this technical note is to validate the Outline Drainage Strategy for the northern park and ride. It describes how the concept design has needed to evolve as a result of provision of new information and design development.

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

11.1.3 Subject to the results of DCO examination and acceptance of the drainage design strategy principles contained in this report, the drainage designs would be developed to preliminary design stage.

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

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

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

REFERENCES

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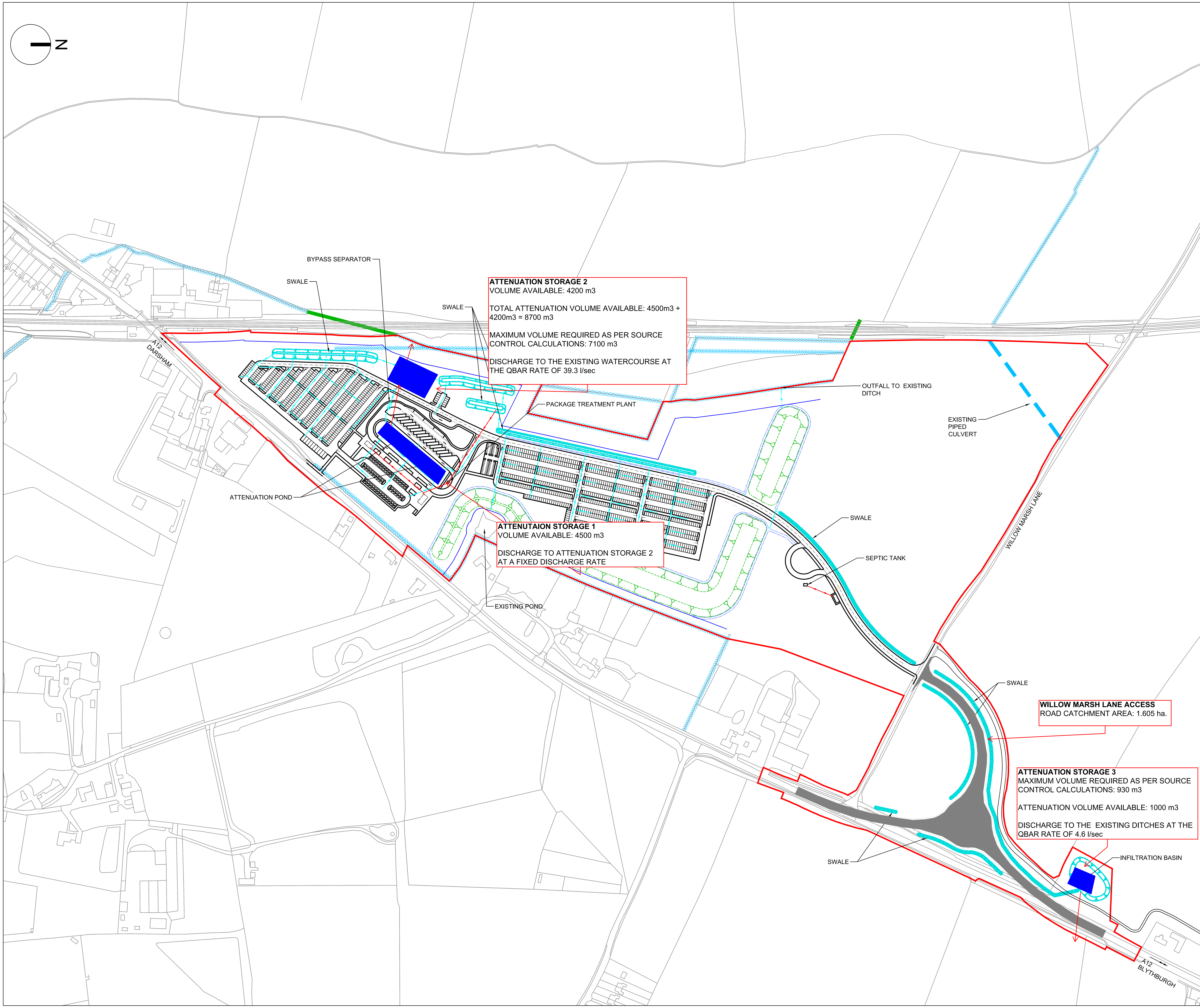
APPENDIX A: LAYOUT PLAN SHOWING ATTENUATION STORAGE REQUIREMENTS



UK PROTECTIVE MARKING:
Not Protectively Marked

NOTES:

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

KEY

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

NOT FOR CONSTRUCTION

NOT FOR APPROVAL

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

NNB GenCo
EDF ENERGY

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

CONTRACTOR COMPANY TRADE NAME : ROYAL HASKONINGDHV

CONTRACTOR REF. No. PB7869

CONTRACT NUMBER : SZ0204

CONTRACTOR WBS CODE : N/A

QRA RELATED Yes ☐ No ☒

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

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

BUILDING
000

SYSTEM
000

SCALE
1:2000

SIZE A1

PAGE
1/1

DESCRIPTION

SIZEWELL C
NORTHERN PARK AND RIDE SITE
DRAINAGE LAYOUT

DOCUMENT REFERENCE No.

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Project	Contract No. / Orig. Co	Asset / Zone	System / Building	Doc. type	Chrono No.

DOCUMENT SUB -TYPE
N/A

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EDF ACCESSIBILITY:
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APPENDIX B: MAIN DEVELOPMENT ATTENUATION STORAGE REQUIREMENTS

ICP SUDS Mean Annual Flood

Input

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

Results	**l/s**
QBAR Rural	39.3
QBAR Urban	39.3
Q2 years	35.2
Q1 year	34.2
Q30 years	94.5
Q100 years	140.1

APPENDIX C: A12 ACCESS ROUNDABOUT ATTENUATION STORAGE REQUIREMENTS

ICP SUDS Mean Annual Flood

Input

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

Results	**l/s**
QBAR Rural	4.6
QBAR Urban	4.6
Q2 years	4.1
Q1 year	4.0
Q30 years	11.0
Q100 years	16.2

Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	25.786	0.436	4.6	283.6	O K
30 min Summer	25.920	0.570	4.6	370.6	O K
60 min Summer	26.057	0.707	4.6	459.4	O K
120 min Summer	26.191	0.841	4.6	546.3	O K
180 min Summer	26.262	0.912	4.6	592.5	O K
240 min Summer	26.306	0.956	4.6	621.3	O K
360 min Summer	26.357	1.007	4.6	654.5	O K
480 min Summer	26.387	1.037	4.6	674.0	O K
600 min Summer	26.403	1.053	4.6	684.6	O K
720 min Summer	26.411	1.061	4.6	689.5	O K
960 min Summer	26.409	1.059	4.6	688.6	O K
1440 min Summer	26.377	1.027	4.6	667.5	O K
2160 min Summer	26.324	0.974	4.6	633.4	O K
2880 min Summer	26.273	0.923	4.6	600.2	O K
4320 min Summer	26.175	0.825	4.6	536.2	O K
5760 min Summer	26.070	0.720	4.6	468.1	O K
7200 min Summer	25.967	0.617	4.6	401.3	O K
8640 min Summer	25.880	0.530	4.6	344.8	O K
10080 min Summer	25.806	0.456	4.6	296.4	O K
15 min Winter	25.839	0.489	4.6	318.1	O K
30 min Winter	25.990	0.640	4.6	416.0	O K
60 min Winter	26.144	0.794	4.6	516.3	O K
120 min Winter	26.295	0.945	4.6	614.2	O K
180 min Winter	26.376	1.026	4.6	667.1	O K
240 min Winter	26.428	1.078	4.6	700.4	O K
360 min Winter	26.489	1.139	4.6	740.0	O K
480 min Winter	26.526	1.176	4.6	764.3	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	95.856	0.0	269.8	26
30 min Summer	62.839	0.0	344.1	41
60 min Summer	39.255	0.0	463.4	70
120 min Summer	23.708	0.0	557.4	130
180 min Summer	17.426	0.0	611.2	190
240 min Summer	13.928	0.0	647.0	248
360 min Summer	10.099	0.0	689.6	368
480 min Summer	8.044	0.0	705.4	486
600 min Summer	6.737	0.0	704.2	604
720 min Summer	5.827	0.0	699.2	724
960 min Summer	4.630	0.0	685.5	962
1440 min Summer	3.344	0.0	654.7	1262
2160 min Summer	2.411	0.0	1036.5	1644
2880 min Summer	1.910	0.0	1092.0	2044
4320 min Summer	1.373	0.0	1156.3	2864
5760 min Summer	1.086	0.0	1252.2	3688
7200 min Summer	0.904	0.0	1303.4	4400
8640 min Summer	0.778	0.0	1345.6	5112
10080 min Summer	0.686	0.0	1380.2	5856
15 min Winter	95.856	0.0	300.4	26
30 min Winter	62.839	0.0	369.7	41
60 min Winter	39.255	0.0	518.0	70
120 min Winter	23.708	0.0	620.5	128
180 min Winter	17.426	0.0	675.4	186
240 min Winter	13.928	0.0	705.1	244
360 min Winter	10.099	0.0	718.5	360
480 min Winter	8.044	0.0	715.9	478

Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
600 min Winter	26.548	1.198	4.6	778.5	O K
720 min Winter	26.560	1.210	4.6	786.5	Flood Risk
960 min Winter	26.566	1.216	4.6	790.4	Flood Risk
1440 min Winter	26.539	1.189	4.6	773.0	O K
2160 min Winter	26.471	1.121	4.6	728.6	O K
2880 min Winter	26.406	1.056	4.6	686.2	O K
4320 min Winter	26.269	0.919	4.6	597.3	O K
5760 min Winter	26.126	0.776	4.6	504.6	O K
7200 min Winter	25.960	0.610	4.6	396.7	O K
8640 min Winter	25.831	0.481	4.6	312.9	O K
10080 min Winter	25.728	0.378	4.6	245.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
600 min Winter	6.737	0.0	710.7	592
720 min Winter	5.827	0.0	704.8	708
960 min Winter	4.630	0.0	693.1	932
1440 min Winter	3.344	0.0	670.9	1364
2160 min Winter	2.411	0.0	1159.2	1716
2880 min Winter	1.910	0.0	1218.7	2172
4320 min Winter	1.373	0.0	1232.6	3116
5760 min Winter	1.086	0.0	1402.5	4040
7200 min Winter	0.904	0.0	1460.1	4752
8640 min Winter	0.778	0.0	1507.7	5448
10080 min Winter	0.686	0.0	1547.1	6064

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.400	Shortest Storm (mins)	15
Ratio R	0.405	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+0

Time Area Diagram

Total Area (ha) 1.605

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

Model Details

Storage is Online Cover Level (m) 26.850

Tank or Pond Structure

Invert Level (m) 25.350

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	650.0	1.500	650.0

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0098-4600-1200-4600
Design Head (m)	1.200
Design Flow (l/s)	4.6
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	98
Invert Level (m)	25.350
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.200	4.6	Kick-Flo®	0.744	3.7
Flush-Flo™	0.357	4.6	Mean Flow over Head Range	-	4.0

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.2	0.800	3.8	2.000	5.8	4.000	8.1	7.000	10.5
0.200	4.3	1.000	4.2	2.200	6.1	4.500	8.5	7.500	10.9
0.300	4.6	1.200	4.6	2.400	6.4	5.000	9.0	8.000	11.2
0.400	4.6	1.400	4.9	2.600	6.6	5.500	9.4	8.500	11.6
0.500	4.5	1.600	5.3	3.000	7.1	6.000	9.8	9.000	11.9
0.600	4.3	1.800	5.6	3.500	7.6	6.500	10.2	9.500	12.2

ANNEX 2A.7: SOUTHERN PARK AND RIDE DRAINAGE DESIGN NOTE

CONTENTS

TABLES

PLATES

1 INTRODUCTION

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

1.1.2 The southern park and ride development forms part of the Application to build and operate a new nuclear power station to the north of Sizewell B.

1.1.3 SZC Co. has undertaken work to validate and develop the design of the southern park and ride that was originally submitted as part of the DCO application. This document forms one of a series of design validation and evolution documents being provided to the Examining Authority in support of the **Outline Drainage Strategy** [REP2-033] and subsequent **Drainage Strategy** submitted at Deadline 7.

1.1.4 The southern park and ride forms one of the Associated Developments (AD) which are required to mitigate traffic impacts arising from the main development site. The southern park and ride is located alongside the A12 at Wickham Market. Its function is to provide a transport hub from which construction workforce are driven to site by coach thus reducing the construction traffic needing to access the main development site. Full details of its facilities are contained in **Volume 4 Southern Park and Ride Chapter 2 Description of the Southern Park and Ride** [APP-380] and are described in summary below.

1.1.5 The site will consist of workforce parking, welfare, security and amenity buildings. The workforce parking includes car parking spaces, accessible spaces, minibus/van spaces, pick up and motorcycle spaces. It also has a Traffic Incident Management Area (TIMA). The TIMA is a holding park to which vehicles can be diverted in the event of an incident on the highway network or at the construction site.

1.1.6 The site access entrance from the B1078/A12 Hacheston slip road will be designed to Suffolk County Council's (SCC) adoptable standards but will remain unadopted.

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

1.1.8 The site access entrance road access from the B1078/A12 Hacheston northbound on slip road will generate surface water highway runoff which will require to be removed, treated as necessary and disposed.

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

1.1.10 The southern park and ride facility and its associated site access entrance will remain in place and use during construction of the SZC power station. Once construction is complete the site will be closed and decommissioned. It will then return to current agricultural use.

2 PURPOSE

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

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

2.1.2 The proposed drainage infrastructure was described in the concept drainage design submitted as part of the Application. This concept design was based on data and information available at that time. The design was supported by the submission of the **Southern Park and Ride Flood Risk Assessment** (FRA) [APP-117].

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

2.1.4 The purpose of this technical note is to provide details of data which validates the **Outline Drainage Strategy** [REP2-033] and subsequent **Drainage Strategy** (Doc. Ref. 6.3 2A (B) submitted at Deadline 7), a description of how the proposed concept drainage infrastructure is developing and evolving and to demonstrate that it continues to provide for the effective and satisfactory drainage of the southern park and ride and its associated external road modification, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution.

3 DESCRIPTION OF DCO DRAINAGE DESIGN STRATEGY

3.1.1 The southern park and ride concept drainage strategy at DCO stage was developed by SZC Co. Proposals were developed for both the southern park and ride development site and associated site access entrance road.

3.1.2 Subject to achievable infiltration rates making infiltration a viable option, all surface water generated within the southern park and ride red line boundary, which includes the site access entrance road from the B1068/A12 slip road, would be contained within the site and discharged to ground by infiltration.

3.1.3 No surface water runoff from the site would be permitted to flow onto the B1078/A12 public highway.

3.1.4 Liaison with Anglian Water took place and it was confirmed that there are no public foul or surface water sewers near to the development site. Accordingly, the proposed infrastructure would be a local private foul water network discharging into a package sewage treatment plant. The treated effluent would discharge to ground by infiltration.

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

3.1.6 A single remote security cabin at the site entrance would drain to a septic tank with infiltration to ground. If infiltration rates are inadequate the septic tank would be replaced by a cess tank.

3.1.7 The internal site layout showing the position of proposed drainage including swales, and infiltration basins is shown in **Plates 1** and **2** which are an extract from Application drawing **"Chapter 2 Description of the Southern Park and Ride** Figure 2.4" [APP-382].

Plate 1: Southern park and ride internal layout showing concept drainage infrastructure to the north

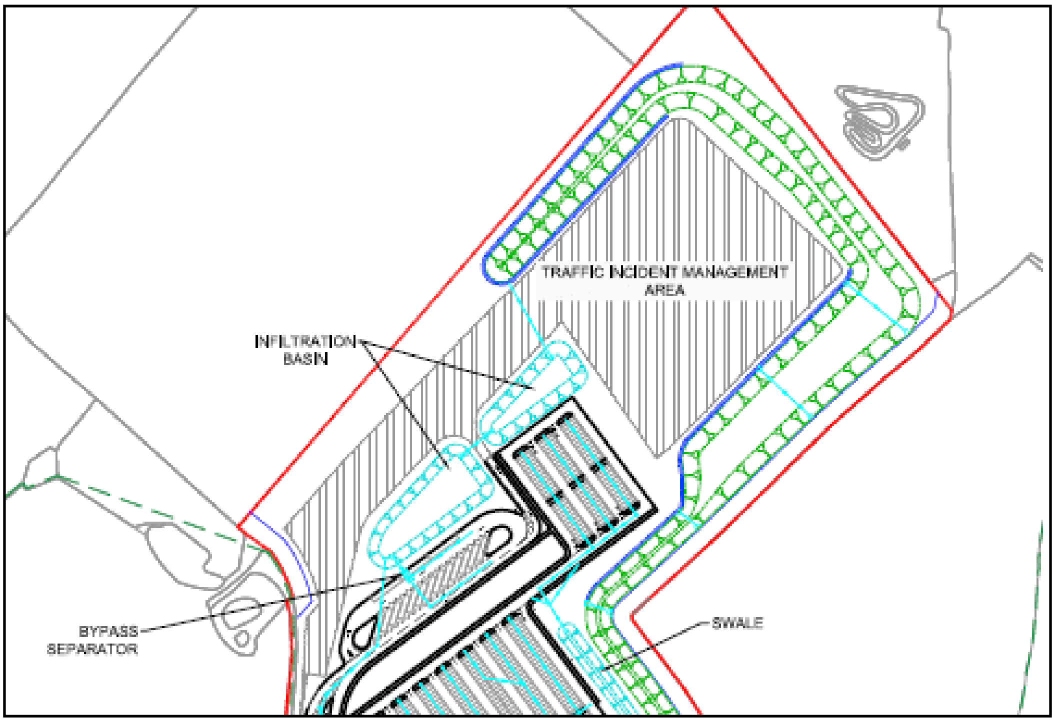
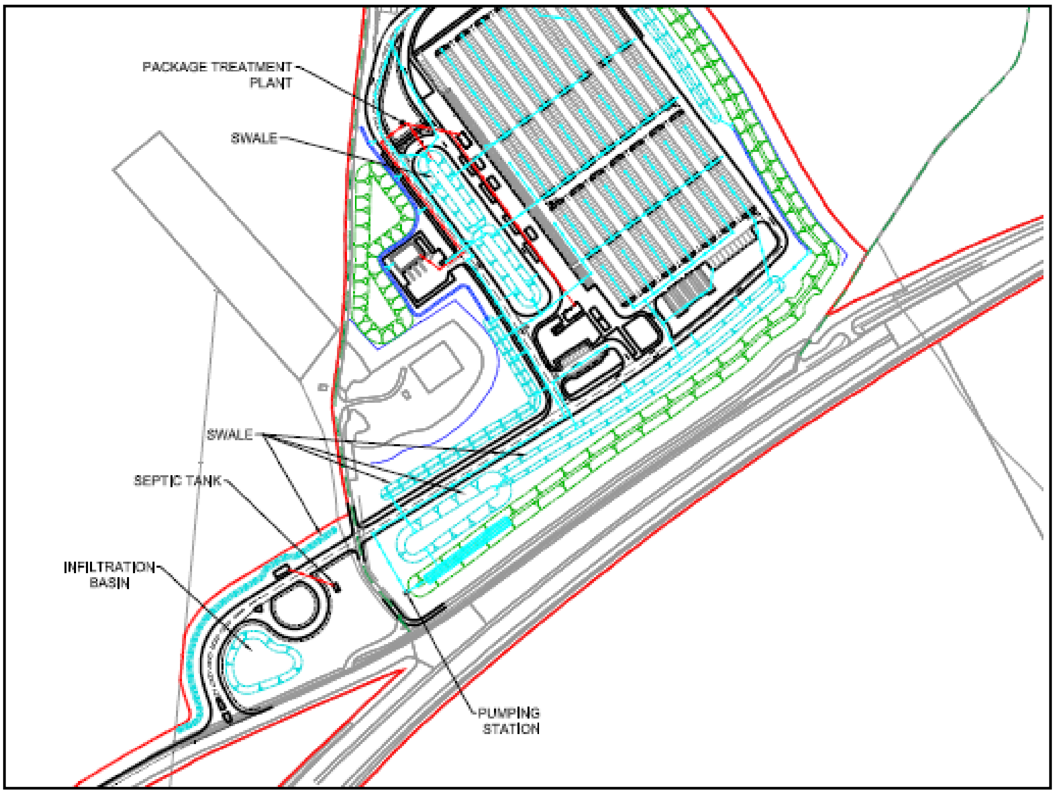
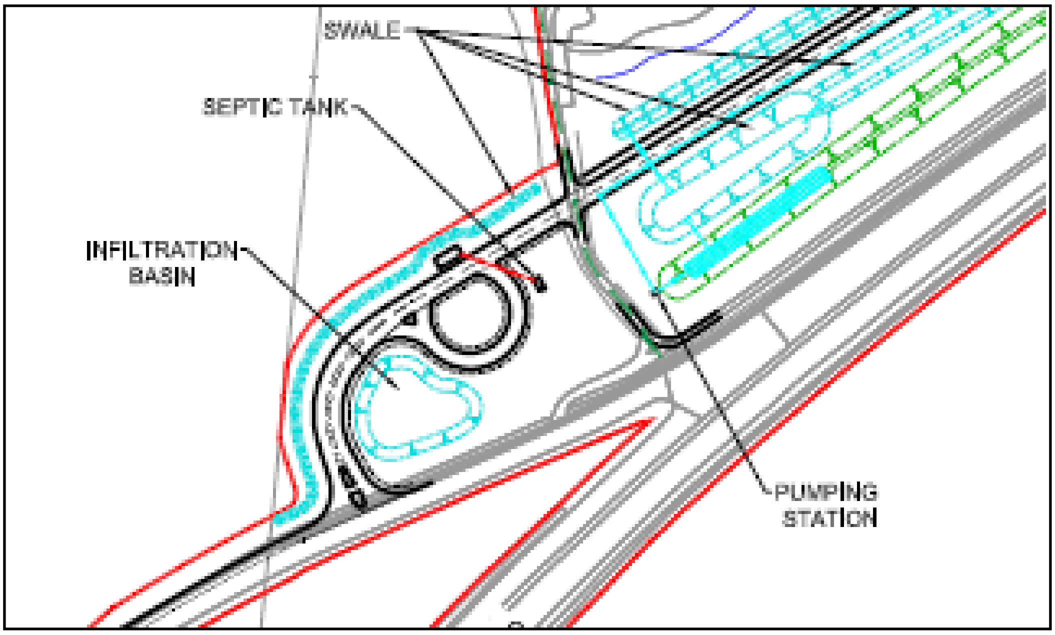


Plate 2: Southern park and ride internal layout showing concept drainage infrastructure to the south



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

Plate 3: Southern park and ride access entrance road



4 EXISTING SITE AND ADJACENT HIGHWAY DRAINAGE ARRANGEMENTS

4.1.1 Subsequent to development of the initial drainage strategy some site investigation has been undertaken within the site red line boundary.

4.1.2 Except for one pond there are no obvious surface drainage features within the proposed site. Given the general topography with a reasonable fall in ground levels approximately 28-29 mAOD at the northern extent of the site to 23 mAOD adjacent to the B1078 A12 slip road and no evidence of ditches or erosion channels etc, it is assumed that surface water overland flow across the site is relatively limited, implying infiltration to ground takes place.

4.1.3 This view, that the site currently infiltrates into the existing soils, is reinforced by desktop study of predicted ground conditions and observation of the surface. Soil Index descriptions from the Institute of Hydrology Flood Studies Report indicate that superficial soil types may be suitable for infiltration. Soil was observed to be sandy in some parts of the site but more cohesive clay closer to the road at lower elevation.

4.1.4 From inspection of the B1078/A12 slip road it is noted that the road is drained by a series of highway gullies and there are manholes located in the footpath. This indicates the presence of highway drainage network. Enquiries have been made with SCC to obtain details of this drainage. Unfortunately, SCC has no asset records or local knowledge of the network. The Wickham Market bypass was constructed by the predecessor body to Highways England in 1976.

4.1.5 The EA Surface Water Flood Map predicts no effective risk of flooding of the site or the slip road and SCC also has no knowledge of flooding issues on the highway.

5 REVISED DRAINAGE DESIGN STRATEGY INPUT DATA

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

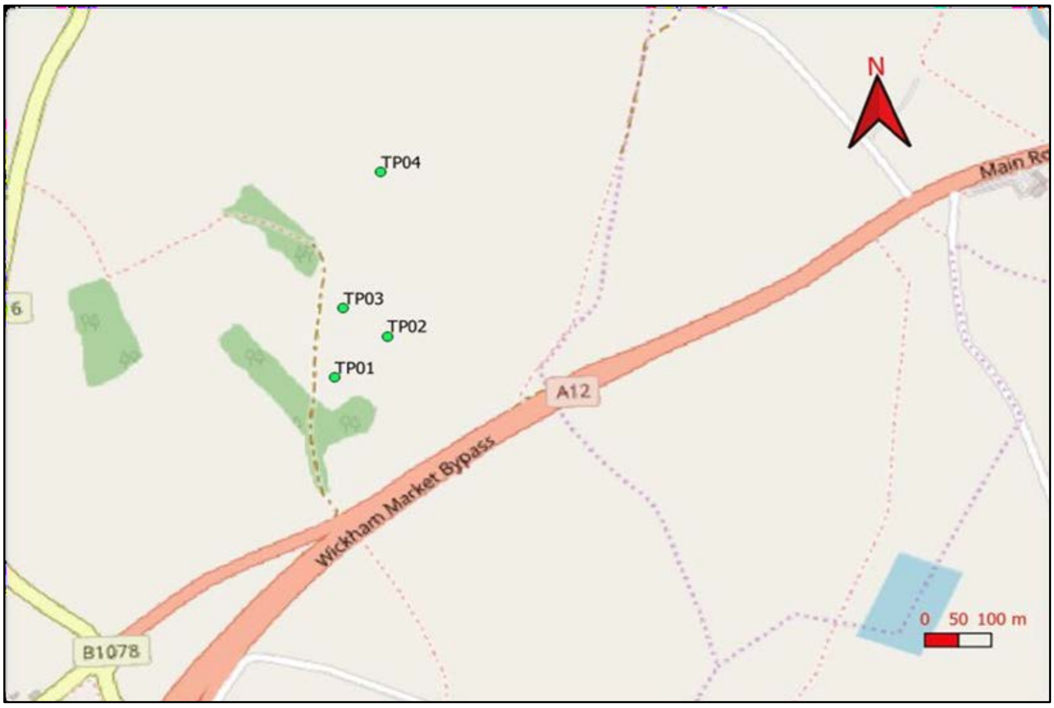
5.1.2 The new data which informs the design development is listed below:

- Ground Investigation and infiltration testing undertaken in November 2019
- Site visit and inspection of southern park and ride extent in 2020
- Site visit and inspection of southern park and ride extent on 3 August 2021

6 GROUND INVESTIGATION AND INFILTRATION TESTING RESULTS

6.1.1 Four trial pits were excavated within the site at locations shown in **Plate 4**.

Plate 4: Southern park and ride site infiltration test trial hole locations



6.1.2 Infiltration testing in accordance with BRE365 (Ref. 1) was undertaken and the results are shown in **Table 1**

Table 1: Southern park and ride site infiltration test trial hole results

Location	Depth (m)	Test 1(m/s)	Test 2(m/s)
TP01	1.25	0	0
TP02	1.30	0	0
TP03	1.32	0	0
TP04	2.1	3.13×10^{-5}	3.01×10^{-5}

6.1.3 In the case of TP01, TP02 and TP03 it was recorded that there was negligible infiltration achieved in 60 hours.

6.1.4 It is not clear as to why TP01, TP02 and TP03 were excavated to a shallower depth.

6.1.5 The nature of the strata in TP01, TP02 and TP03 is stated to be stiff but slightly gravelly clay, Lowestoft Formation Diamicton. At TP04 this changes to a slightly gravely, slightly clayey Lowestoft Formation Sand and Gravel.

6.1.6 The results align with the British Geological Survey data which is noted in the **Southern Park and Ride FRA** [APP-117]. The BGS map records superficial geology for the site to be two types of the Lowestoft Formation; formed of sand and gravel in the south-western and north-eastern sections of the site, with an approximate 500m strip of diamicton running through the site centre. As shown in Figure 4 TP01, TP02 and TP03 are located in the centre of the site and TP04 is to the north east. No trial pits were excavated in the west or south west of the site.

6.1.7 The superficial Lowestoft Formation is underlain by Crag Formation at about 6 m below ground level. Crag Formation is described as shallow-water marine and estuarine sands, gravels, silts and clays. Crag has variable permeability but will have greater potential for infiltration.

6.1.8 In summary these results demonstrate that disposal of surface water runoff by infiltration is achievable but only at TP04 which is to the north and at higher elevation. SCC consider that an infiltration rate in excess of 1.4 x 10-6 m/s is viable for infiltration to ground.

6.1.9 At the time of visit on 3 August 2021 further ground investigation works were in progress and include additional infiltration testing. The results of the further infiltration testing will be taken into account at preliminary design stage. It is hoped that these results will demonstrate that infiltration is viable in other parts of the site but if this is not the case, it is considered that the current concept proposals will provide for suitable and effective drainage of the site.

7 REVISED SURFACE WATER CONCEPT DRAINAGE DESIGN STRATEGY – SOUTHERN PARK AND RIDE SITE

7.1.1 The arrangements for removal of surface water remain as broadly as described in document **“Environmental Statement Volume 4 Chapter 2 Description of the Southern Park and Ride”** [APP-381] but are modified to take account of the site inspections.

7.1.2 It is intended that all surface water runoff is to be contained within the site and removed by infiltration to ground. However, taking account of the proven lack of infiltration in the middle of the site, it is intended that that runoff will be removed and collected in the lowest elevation in the south west and then pumped to the north where infiltration is viable. If the latest infiltration testing demonstrates that infiltration is viable in the south west corner of the site as is suspected, then this would be modified to remove the pumping requirement.

7.1.3 Runoff from roofs will be drained via downpipes and gullies, as appropriate to underground carrier drains and discharge into attenuation basins and swales.

7.1.4 Runoff from the internal roads, the bus/HGV standing areas and the Traffic Incident Management Area, which must have an impermeable surface will be drained via surface outlets, gullies, linear channels and drains etc. These will discharge into underground carrier drains which will convey the runoff to the same attenuation basins and swales or in the north to infiltration basins.

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

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

7.1.7 In the centre and south parts of the site, the underground carrier drains will discharge all surface water into a series of swales and attenuation basins which will provide suitable treatment in accordance with CIRIA C753 The SuDS Manual (Ref. 2). The swale/attenuation basin network will discharge into a pumping station which will pump runoff to the infiltration basins to the north.

7.1.8 In the north part of the site, the underground carrier drains will discharge all surface water into one of two infiltration basins by gravity. The infiltration basins will provide suitable treatment in accordance with CIRIA C753 The SuDS Manual.

7.1.9 At concept design stage, the footprint for each swale and basin was based on indicative calculations using the UK SUDS Storage Estimating Tool (Ref. 3) and assuming an outfall discharge based on a rate of 2 l/s/Ha.

7.1.10 The infiltration basin storage requirements have now been updated with more detailed calculations using MicroDrainage with proven infiltration rates measured at the northern infiltration basin location. They assume discharge of local runoff discharged by gravity to the north plus pumped flows from the centre and south west of the site.

7.1.11 The layout drawing in **Appendix A** shows the existing DCO submitted layout but superimposed with required storage volumes and footprints for infiltration and attenuation basins or underground storage. These have been determined by the hydraulic modelling calculations. The calculations are shown in **Appendix B**.

7.1.12 The attenuation storage for the central and south area is provided using underground storage. The available area and volume has been maximised. A required pump rate has been determined to ensure that the storage capacity is not exceeded.

7.1.13 The calculations allow for Option 1 shown in **Appendix A**, a discharge of 5l/s from the site entrance access road attenuation basin into the pumping station.

7.1.14 The storage requirements for the infiltration basin to the north allow for the pumped flow at 50 l/s.

7.1.15 Hydraulic calculation based requirements are summarised in **Table 2**.

Table 2: Southern park and ride site drainage attenuation and infiltration infrastructure requirements at concept design stage

Infrastructure Location	Dimensions
South central area attenuation storage tank	9,888 m^3
Entrance road Attenuation Basin	338 m^3
Pump Discharge Rate to north Infiltration Basin	50 l/sec
Average Infiltration Rate at north Infiltration Basin (TP04)	104.04 mm/hour
North Infiltration Basin	3209 m^3
North Infiltration Basin Half Drain Time	471 minutes (~8 hours)

7.1.16 It can be seen that the required volumes for the gravity and pumped catchments are linked. If the pumped flow rate is increased required storage volume in the upstream attenuation basins and swales is reduced. However, the higher pumped flow rate will increase the infiltration basin storage volume requirements to the north.

8 REVISED FOUL WATER DRAINAGE CONCEPT DESIGN STRATEGY – SOUTHERN PARK AND RIDE SITE

8.1.1 The foul water drainage strategy remains unchanged with foul water flows collected by an underground gravity pipe drainage network and discharged into a package sewage treatment plant. However, whilst previously the treated effluent would discharge to ground via infiltration through a drainfield network, the current infiltration test results demonstrate that this is not feasible. Therefore, the treated effluent is proposed to discharge into a swale and ultimately having mixed with surface water runoff will be pumped to the north infiltration basin where the treated effluent will infiltrate to ground.

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

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

8.1.4 During design development should it be determined that the infiltration rate is insufficient for the provision of a drainfield and therefore create a flood risk it will be necessary to collect wastewater and sewage in a cesspit from which it can be collected and regularly be removed by tanker for treatment offsite.

9 REVISED SURFACE WATER DRAINAGE CONCEPT DESIGN STRATEGY – B1078/A12 HACHESTON SLIP ROAD AND SITE ENTRANCE ACCESS ROAD

9.1.1 The surface water drainage strategy for the highway drainage remains unchanged being infiltration to ground to the extent that this is achievable. As noted in Section 5 no infiltration testing is currently available for this part of the site. Additional infiltration testing is in progress, but additional results are not currently available.

9.1.2 The level of the site entrance access road will be set to ensure that there is no additional surface water highway runoff that can discharge into the existing B1078 A12 slip road highway drain.

9.1.3 The site entrance access road will remain in SZC Co. private ownership.

9.1.4 Highway surface water runoff will discharge either by "over the edge" or kerb and gullies into a swale. The swale will include for an underlying filter drain. Since infiltration viability is unconfirmed the filter drain will discharge flow that does not infiltrate into an infiltration basin located between the slip road boundary, the access road and the vehicle roundabout.

9.1.5 The roundabout will be drained by gullies which will discharge into the infiltration basin.

9.1.6 If following infiltration testing at the infiltration basin location it is established that infiltration will not be viable, the infiltration basin will change to an attenuation basin. The basin will outfall to the pumping station with discharge to the infiltration basins to the north where viability of infiltration is proven.

9.1.7 SCC do not consider that infiltration is viable where the infiltration rate is proven to be les than 1×10^{-6} m/s. Hydraulic calculations have been undertaken to determine whether for available space and this infiltration rate, infiltration is viable. The results are shown as Option 2 in **Appendices A and C**. They are also summarised in **Table 3.**

Table 3: Southern park and ride site entrance drainage infiltration infrastructure requirements at concept design stage

Infrastructure Location	Dimensions
Entrance Road Infiltration Basin	596 m^3
Minimum Infiltration Rate	1×10^{-6} m/sec
Half Drain Time	More than 7 days

9.1.8 The results demonstrate that infiltration is not viable due to the extended half drain down time.

9.1.9 The alternative Option 1 shown in **Appendices A and C** is for an attenuation basin which will contain the required volume of runoff whilst releasing it at a controlled rate to the pumping station which will discharge flow to the north infiltration basin. This is described in more detail in Section 7.

10 SUMMARY AND CONCLUSION

10.1.1 The purpose of this technical note is to validate the Outline Drainage Strategy and subsequent Drainage Strategy (submitted at Deadline 7) for the southern park and ride. It describes how the concept design has needed to evolve as a result of design development and the lack of certainty as to the viability of removal of surface water runoff by infiltration across the whole site.

10.1.2 Based on the infiltration rates measured at TP04 in the northern part of the site, removal of surface water runoff and treated effluent by infiltration to ground remains viable. It is noted that the alternative options of discharge to local watercourse or sewer are not available.

10.1.3 Subject to the results of DCO examination and acceptance of the drainage design strategy principles contained in this report, the drainage designs will be developed to preliminary design stage.

10.1.4 At this stage subject to the additional infiltration test results particularly in the south west at lowest elevation it is intended that the need to pump flow to the north for removal can be removed. However, if necessary it can be retained. If pumping is required then back up provision in case of pump failure will be incorporated in the design with provision of passive additional storage being the preferred option.

10.1.5 The southern park and ride facility drainage design will be based on CIRIA C753 SuDS Manual, Design and Construction Guidance for Foul and Surface Water Sewers (formerly Sewers for Adoption) (Ref. 4), and PPG4 Treatment and Disposal of Sewage where no Foul Water Sewer is Available (Ref. 5).

10.1.6 The site access entrance road will be based on Design Manual for Roads and Bridges (DMRB) (Ref. 6), Manual of Contract Documents for Highway Works (MCHW) (Ref. 7) and SCC specific guidance (Refs. 8 and 9).

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

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4. SSG Appendix C - Design and construction guidance for foul and surface water sewers offered for adoption under the Code for adoption agreements for water and sewerage companies operating wholly or mainly in England ("the Code"). Approved Version 2.0. 10 March 2020. Water UK. https://www.thenbs.com/PublicationIndex/documents/details?DocID=330727

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7. Manual of Contract Documents for Highway Works (MCHW), Highways Agency. https://www.standardsforhighways.co.uk/ha/standards/mchw/index.htm

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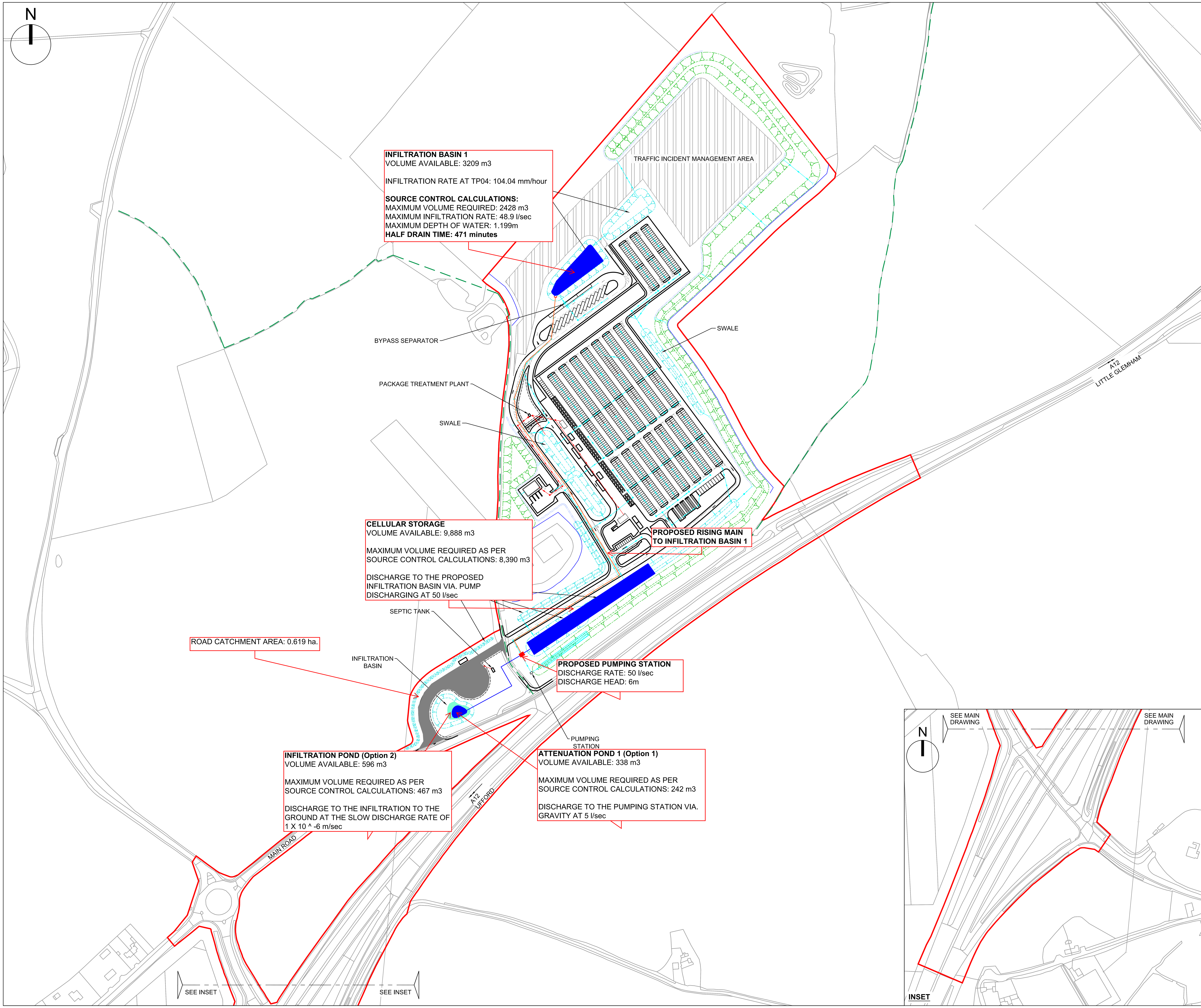
9. Sustainable Drainage Systems (SuDS) a Local Design Guide Appendix A to the Suffolk Flood Risk Management Strategy, Suffolk County Council, May 2018 https://www.greensuffolk.org/app/uploads/2021/05/2018-10-01-SFRMS-SuDS-Guidance-Appendix-A-.pdf

APPENDIX A: LAYOUT PLAN SHOWING ATTENUATION STORAGE REQUIREMENTS



UK PROTECTIVE MARKING:
Not Protectively Marked

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

KEY
- DEVELOPMENT SITE BOUNDARY
- BUFFER ZONE
- SWALE / INFILTRATION BASIN (INDICATIVE)
- LANDSCAPE BUND
- PROPOSED SURFACE WATER DRAINAGE (INDICATIVE)
- PROPOSED FOUL WATER DRAINAGE (INDICATIVE)
- PROPOSED GEOCELLULAR UNDERGROUND STORAGE (64M x 6M)
- PROPOSED DITCH (INDICATIVE)
- PROPOSED BYPASS SEPARATOR (INDICATIVE)

NOT FOR CONSTRUCTION

NOT FOR APPROVAL

REV.	DATE	PREPARED BY	CHECKED BY	STATUS	REASONS FOR REVISION	APPROVED BY
K	21.02.20	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING HSF REVIEW	PJ
J	31.01.20	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING HSF REVIEW	PJ
I	20.01.20	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING HSF REVIEW	PJ
H	12.11.19	NKS	KA	S3	MINOR AMENDMENTS	PJ
G	06.11.19	NKS	KA	S3	BUFFER ZONE SOUTH OF INFILTRATION TRIMMED	PJ
F	31.10.19	NKS	KA	S3	MINOR AMENDMENTS FOLLOWING TEAM REVIEW	PJ
E	25.09.19	NKS	KA	S3	MINOR AMENDMENTS	PJ
D	02.08.19	NKS	KA	S3	MINOR AMENDMENTS	PJ
C	24.07.19	NKS	KA	S3	MINOR AMENDMENTS	PJ
B	09.07.19	NKS	KA	S3	MINOR AMENDMENTS	PJ
A	01.07.19	NKS	KA	S3	FIRST ISSUE	PJ

NNB GenCo
EDF ENERGY

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

CONTRACTOR COMPANY TRADE NAME : ROYAL HASKONINGDHV

CONTRACTOR REF. No. PB7869

CONTRACT NUMBER : SZ0204

CONTRACTOR WBS CODE : N/A

QRA RELATED Yes ☐ No ☒

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

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

BUILDING
000

SYSTEM
000

SCALE
1:2000

SIZE A1

PAGE
1/1

DESCRIPTION

SIZEWELL C
SOUTHERN PARK AND RIDE SITE
DRAINAGE LAYOUT

DOCUMENT REFERENCE No.

SZC	SZ0204	FP	000	DRW	100053
Project	Contract No. / Orig. Co	Asset / Zone	System / Building	Doc. type	Chrono No.

DOCUMENT SUB -TYPE
N/A

EDF CLASSIFICATION CODE
N/A

SUBCONTRACTOR COMPANY TRADE NAME
N/A

SUBCONTRACTOR DOCUMENT REF. No
N/A

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

INTELLECTUAL PROPERTY:
O-Property of Owner

NNB:

EDF:
NNB GenCo © 2018

UK PROTECTIVE MARKING:
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EDF ACCESSIBILITY:
INTERNE ☐ RESTREINT ☐ CONFIDENTIEL ☐

APPENDIX B: MAIN DEVELOPMENT INFILTRATION AND ATTENUATION STORAGE REQUIREMENTS

Cascade Summary of Results for SRC-SPR-CS-Area 1.SRCX

Upstream Structures	Outflow To	Overflow To
(None)	SRC-SPR-Infiltration Basin.SRCX	(None)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	22.870	0.620	50.0	3064.9	O K
30 min Summer	23.060	0.810	50.0	4006.5	O K
60 min Summer	23.252	1.002	50.0	4956.8	O K
120 min Summer	23.436	1.186	50.0	5865.3	O K
180 min Summer	23.532	1.282	50.0	6339.7	O K
240 min Summer	23.590	1.340	50.0	6626.2	O K
360 min Summer	23.659	1.409	50.0	6968.1	O K
480 min Summer	23.697	1.447	50.0	7156.6	O K
600 min Summer	23.716	1.466	50.0	7250.6	O K
720 min Summer	23.723	1.473	50.0	7283.6	O K
960 min Summer	23.713	1.463	50.0	7233.5	O K
1440 min Summer	23.658	1.408	50.0	6961.8	O K
2160 min Summer	23.575	1.325	50.0	6552.3	O K
2880 min Summer	23.495	1.245	50.0	6155.8	O K
4320 min Summer	23.343	1.093	50.0	5403.3	O K
5760 min Summer	23.203	0.953	50.0	4711.9	O K
7200 min Summer	23.075	0.825	50.0	4080.0	O K
8640 min Summer	22.960	0.710	50.0	3509.4	O K
10080 min Summer	22.856	0.606	50.0	2998.0	O K
15 min Winter	22.946	0.696	50.0	3439.6	O K
30 min Winter	23.160	0.910	50.0	4497.6	O K
60 min Winter	23.376	1.126	50.0	5569.4	O K
120 min Winter	23.586	1.336	50.0	6605.3	O K
180 min Winter	23.697	1.447	50.0	7154.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	99.025	0.0	3017.2	30
30 min Summer	64.904	0.0	3798.4	45
60 min Summer	40.510	0.0	5109.6	74
120 min Summer	24.421	0.0	6140.9	134
180 min Summer	17.920	0.0	6731.6	192
240 min Summer	14.300	0.0	7128.3	252
360 min Summer	10.377	0.0	7663.3	370
480 min Summer	8.265	0.0	7997.7	488
600 min Summer	6.922	0.0	8181.6	606
720 min Summer	5.986	0.0	8236.4	724
960 min Summer	4.756	0.0	8121.7	962
1440 min Summer	3.434	0.0	7842.5	1218
2160 min Summer	2.475	0.0	11259.2	1580
2880 min Summer	1.960	0.0	11882.0	1972
4320 min Summer	1.409	0.0	12723.0	2776
5760 min Summer	1.114	0.0	13511.7	3584
7200 min Summer	0.927	0.0	14065.8	4336
8640 min Summer	0.798	0.0	14530.3	5104
10080 min Summer	0.703	0.0	14931.4	5848
15 min Winter	99.025	0.0	3341.2	30
30 min Winter	64.904	0.0	4095.8	44
60 min Winter	40.510	0.0	5714.7	74
120 min Winter	24.421	0.0	6845.4	132
180 min Winter	17.920	0.0	7466.5	188

Cascade Summary of Results for SRC-SPR-CS-Area 1.SRCX

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	23.766	1.516	50.0	7494.3	O K
360 min Winter	23.851	1.601	50.0	7913.8	O K
480 min Winter	23.901	1.651	50.0	8161.0	O K
600 min Winter	23.929	1.679	50.0	8302.1	O K
720 min Winter	23.944	1.694	50.0	8374.6	O K
960 min Winter	23.947	1.697	50.0	8390.2	O K
1440 min Winter	23.897	1.647	50.0	8142.8	O K
2160 min Winter	23.785	1.535	50.0	7588.8	O K
2880 min Winter	23.676	1.426	50.0	7052.4	O K
4320 min Winter	23.454	1.204	50.0	5953.7	O K
5760 min Winter	23.243	0.993	50.0	4910.7	O K
7200 min Winter	23.051	0.801	50.0	3958.4	O K
8640 min Winter	22.879	0.629	50.0	3111.6	O K
10080 min Winter	22.732	0.482	50.0	2381.2	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.300	0.0	7855.1	246
360 min Winter	10.377	0.0	8284.4	362
480 min Winter	8.265	0.0	8387.6	478
600 min Winter	6.922	0.0	8333.6	594
720 min Winter	5.986	0.0	8278.9	708
960 min Winter	4.756	0.0	8166.8	934
1440 min Winter	3.434	0.0	7926.7	1366
2160 min Winter	2.475	0.0	12603.7	1700
2880 min Winter	1.960	0.0	13286.2	2148
4320 min Winter	1.409	0.0	14071.9	3036
5760 min Winter	1.114	0.0	15133.4	3872
7200 min Winter	0.927	0.0	15754.0	4680
8640 min Winter	0.798	0.0	16276.7	5376
10080 min Winter	0.703	0.0	16724.1	6056

Cascade Rainfall Details for SRC-SPR-CS-Area 1.SRCX

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	20.000	Shortest Storm (mins)	15
Ratio R	0.404	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+0

Time Area Diagram

Total Area (ha) 16.854

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	4.214	4	8	4.214	8	12	4.213	12	16	4.213

Cascade Model Details for SRC-SPR-CS-Area 1.SRCX

Storage is Online Cover Level (m) 24.250

Tank or Pond Structure

Invert Level (m) 22.250

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	4944.4	2.000	4944.4

Pump Outflow Control

Invert Level (m) 22.250

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.200	50.0000	1.400	50.0000	2.600	50.0000	3.800	50.0000	5.000	50.0000
0.400	50.0000	1.600	50.0000	2.800	50.0000	4.000	50.0000	5.200	50.0000
0.600	50.0000	1.800	50.0000	3.000	50.0000	4.200	50.0000	5.400	50.0000
0.800	50.0000	2.000	50.0000	3.200	50.0000	4.400	50.0000	5.600	50.0000
1.000	50.0000	2.200	50.0000	3.400	50.0000	4.600	50.0000	5.800	50.0000
1.200	50.0000	2.400	50.0000	3.600	50.0000	4.800	50.0000	6.000	50.0000

Cascade Summary of Results for SRC-SPR-CS-Area 2.SRCX

Upstream Structures	Outflow To	Overflow To
(None)	SRC-SPR-Infiltration Basin.SRCX	(None)

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
15 min Summer	23.441	0.691	4.9	111.2	O K
30 min Summer	23.586	0.836	4.9	143.7	O K
60 min Summer	23.710	0.960	4.9	174.6	O K
120 min Summer	23.803	1.053	4.9	199.5	O K
180 min Summer	23.835	1.085	4.9	208.4	O K
240 min Summer	23.842	1.092	4.9	210.5	O K
360 min Summer	23.830	1.080	4.9	207.0	O K
480 min Summer	23.807	1.057	4.9	200.6	O K
600 min Summer	23.784	1.034	4.9	194.4	O K
720 min Summer	23.762	1.012	4.9	188.3	O K
960 min Summer	23.719	0.969	4.9	176.9	O K
1440 min Summer	23.629	0.879	4.9	154.1	O K
2160 min Summer	23.479	0.729	4.9	119.3	O K
2880 min Summer	23.344	0.594	4.9	91.3	O K
4320 min Summer	23.131	0.381	4.9	52.7	O K
5760 min Summer	22.997	0.247	4.6	32.0	O K
7200 min Summer	22.921	0.171	4.3	21.3	O K
8640 min Summer	22.879	0.129	3.9	15.8	O K
10080 min Summer	22.861	0.111	3.5	13.4	O K
15 min Winter	23.505	0.755	4.9	125.0	O K
30 min Winter	23.661	0.911	4.9	162.0	O K
60 min Winter	23.794	1.044	4.9	197.1	O K
120 min Winter	23.897	1.147	4.9	226.5	O K
180 min Winter	23.936	1.186	4.9	238.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	99.025	0.0	114.8	19
30 min Summer	64.904	0.0	150.5	33
60 min Summer	40.510	0.0	188.2	64
120 min Summer	24.421	0.0	226.9	122
180 min Summer	17.920	0.0	249.8	182
240 min Summer	14.300	0.0	265.8	242
360 min Summer	10.377	0.0	289.3	360
480 min Summer	8.265	0.0	307.2	414
600 min Summer	6.922	0.0	321.6	476
720 min Summer	5.986	0.0	333.8	540
960 min Summer	4.756	0.0	353.5	674
1440 min Summer	3.434	0.0	382.9	952
2160 min Summer	2.475	0.0	414.2	1320
2880 min Summer	1.960	0.0	437.4	1676
4320 min Summer	1.409	0.0	471.5	2376
5760 min Summer	1.114	0.0	497.1	3056
7200 min Summer	0.927	0.0	517.5	3744
8640 min Summer	0.798	0.0	534.5	4408
10080 min Summer	0.703	0.0	549.1	5136
15 min Winter	99.025	0.0	128.6	18
30 min Winter	64.904	0.0	168.6	33
60 min Winter	40.510	0.0	210.8	62
120 min Winter	24.421	0.0	254.2	120
180 min Winter	17.920	0.0	279.8	178

Cascade Summary of Results for SRC-SPR-CS-Area 2.SRCX

Storm Event	Max Level (m)	Max Depth (m)	Max Control (l/s)	Max Volume (m³)	Status
240 min Winter	23.949	1.199	4.9	241.9	O K
360 min Winter	23.945	1.195	4.9	240.9	O K
480 min Winter	23.925	1.175	4.9	234.7	O K
600 min Winter	23.896	1.146	4.9	226.0	O K
720 min Winter	23.870	1.120	4.9	218.5	O K
960 min Winter	23.816	1.066	4.9	203.2	O K
1440 min Winter	23.698	0.948	4.9	171.3	O K
2160 min Winter	23.470	0.720	4.9	117.3	O K
2880 min Winter	23.266	0.516	4.9	76.3	O K
4320 min Winter	22.995	0.245	4.6	31.7	O K
5760 min Winter	22.885	0.135	4.0	16.5	O K
7200 min Winter	22.856	0.106	3.4	12.7	O K
8640 min Winter	22.841	0.091	2.9	10.9	O K
10080 min Winter	22.832	0.082	2.6	9.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
240 min Winter	14.300	0.0	297.7	236
360 min Winter	10.377	0.0	324.0	348
480 min Winter	8.265	0.0	344.1	454
600 min Winter	6.922	0.0	360.2	542
720 min Winter	5.986	0.0	373.8	570
960 min Winter	4.756	0.0	396.0	722
1440 min Winter	3.434	0.0	428.8	1036
2160 min Winter	2.475	0.0	463.9	1424
2880 min Winter	1.960	0.0	489.9	1760
4320 min Winter	1.409	0.0	528.1	2380
5760 min Winter	1.114	0.0	556.7	3000
7200 min Winter	0.927	0.0	579.6	3672
8640 min Winter	0.798	0.0	598.7	4408
10080 min Winter	0.703	0.0	615.1	5136

Cascade Rainfall Details for SRC-SPR-CS-Area 2.SRCX

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	20.000	Shortest Storm (mins)	15
Ratio R	0.404	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+0

Time Area Diagram

Total Area (ha) 0.620

Time (mins) From:	To:	Area (ha)
0	4	0.620

Cascade Model Details for SRC-SPR-CS-Area 2.SRCX

Storage is Online Cover Level (m) 24.250

Tank or Pond Structure

Invert Level (m) 22.750

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	113.8	1.500	367.8

Hydro-Brake® Optimum Outflow Control

Unit Reference	MD-SHE-0098-5000-1500-5000
Design Head (m)	1.500
Design Flow (l/s)	5.0
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	98
Invert Level (m)	22.750
Minimum Outlet Pipe Diameter (mm)	150
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)	Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.500	5.0	Kick-Flo®	0.878	3.9
Flush-Flo™	0.431	4.9	Mean Flow over Head Range	-	4.3

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	3.2	0.800	4.3	2.000	5.7	4.000	7.9	7.000	10.3
0.200	4.4	1.000	4.1	2.200	6.0	4.500	8.4	7.500	10.7
0.300	4.8	1.200	4.5	2.400	6.2	5.000	8.8	8.000	11.0
0.400	4.9	1.400	4.8	2.600	6.5	5.500	9.2	8.500	11.3
0.500	4.9	1.600	5.1	3.000	6.9	6.000	9.6	9.000	11.6
0.600	4.8	1.800	5.4	3.500	7.4	6.500	10.0	9.500	11.9

Cascade Summary of Results for SRC-SPR-Infiltration Basin.SRCX

Upstream Structures	Outflow To	Overflow To
SRC-SPR-CS-Area 1.SRCX	(None)	(None)
SRC-SPR-CS-Area 2.SRCX		

Half Drain Time : 471 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	27.328	0.578	35.3	1040.3	O K
30 min Summer	27.462	0.712	38.2	1315.5	O K
60 min Summer	27.572	0.822	40.5	1551.4	O K
120 min Summer	27.661	0.911	42.5	1748.4	O K
180 min Summer	27.704	0.954	43.4	1845.8	O K
240 min Summer	27.731	0.981	44.0	1906.7	O K
360 min Summer	27.765	1.015	44.8	1986.2	O K
480 min Summer	27.788	1.038	45.3	2039.5	O K
600 min Summer	27.805	1.055	45.7	2078.4	O K
720 min Summer	27.817	1.067	46.0	2108.5	O K
960 min Summer	27.818	1.068	46.0	2111.2	O K
1440 min Summer	27.809	1.059	45.8	2088.8	O K
2160 min Summer	27.876	1.126	47.3	2248.3	O K
2880 min Summer	27.886	1.136	47.5	2272.3	O K
4320 min Summer	27.898	1.148	47.8	2303.2	O K
5760 min Summer	27.908	1.158	48.0	2326.3	O K
7200 min Summer	27.897	1.147	47.7	2299.8	O K
8640 min Summer	27.887	1.137	47.5	2274.5	O K
10080 min Summer	27.875	1.125	47.3	2247.3	O K
15 min Winter	27.384	0.634	36.5	1153.0	O K
30 min Winter	27.519	0.769	39.4	1436.1	O K
60 min Winter	27.627	0.877	41.8	1672.2	O K
120 min Winter	27.713	0.963	43.6	1864.8	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	99.025	0.0	838
30 min Summer	64.904	0.0	1140
60 min Summer	40.510	0.0	1468
120 min Summer	24.421	0.0	1814
180 min Summer	17.920	0.0	2024
240 min Summer	14.300	0.0	2174
360 min Summer	10.377	0.0	2402
480 min Summer	8.265	0.0	2580
600 min Summer	6.922	0.0	2728
720 min Summer	5.986	0.0	2856
960 min Summer	4.756	0.0	2880
1440 min Summer	3.434	0.0	2880
2160 min Summer	2.475	0.0	3872
2880 min Summer	1.960	0.0	4272
4320 min Summer	1.409	0.0	5020
5760 min Summer	1.114	0.0	5736
7200 min Summer	0.927	0.0	6232
8640 min Summer	0.798	0.0	6760
10080 min Summer	0.703	0.0	7304
15 min Winter	99.025	0.0	952
30 min Winter	64.904	0.0	1294
60 min Winter	40.510	0.0	1660
120 min Winter	24.421	0.0	2050

Cascade Summary of Results for SRC-SPR-Infiltration Basin.SRCX

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
180 min Winter	27.753	1.003	44.5	1958.2	O K
240 min Winter	27.778	1.028	45.1	2016.0	O K
360 min Winter	27.810	1.060	45.8	2090.7	O K
480 min Winter	27.830	1.080	46.2	2138.4	O K
600 min Winter	27.830	1.080	46.3	2139.5	O K
720 min Winter	27.830	1.080	46.3	2139.7	O K
960 min Winter	27.830	1.080	46.2	2138.1	O K
1440 min Winter	27.826	1.076	46.2	2128.8	O K
2160 min Winter	27.911	1.161	48.1	2334.8	O K
2880 min Winter	27.921	1.171	48.3	2359.6	O K
4320 min Winter	27.935	1.185	48.6	2393.6	O K
5760 min Winter	27.947	1.197	48.9	2423.1	O K
7200 min Winter	27.949	1.199	48.9	2428.0	O K
8640 min Winter	27.940	1.190	48.7	2405.3	O K
10080 min Winter	27.929	1.179	48.5	2377.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
180 min Winter	17.920	0.0	2286
240 min Winter	14.300	0.0	2452
360 min Winter	10.377	0.0	2704
480 min Winter	8.265	0.0	2880
600 min Winter	6.922	0.0	2880
720 min Winter	5.986	0.0	2880
960 min Winter	4.756	0.0	2880
1440 min Winter	3.434	0.0	2880
2160 min Winter	2.475	0.0	4264
2880 min Winter	1.960	0.0	4652
4320 min Winter	1.409	0.0	5368
5760 min Winter	1.114	0.0	6048
7200 min Winter	0.927	0.0	6592
8640 min Winter	0.798	0.0	7008
10080 min Winter	0.703	0.0	7464

Cascade Rainfall Details for SRC-SPR-Infiltration Basin.SRCX

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	20.000	Shortest Storm (mins)	15
Ratio R	0.404	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+0

Cascade Model Details for SRC-SPR-Infiltration Basin.SRCX

Storage is Online Cover Level (m) 28.250

Infiltration Basin Structure

Invert Level (m) 26.750 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.10404 Porosity 1.00
Infiltration Coefficient Side (m/hr) 0.10404

Depth (m)	**Area (m²)**	**Depth (m)**	**Area (m²)**
0.000	1606.4	1.500	2720.9

APPENDIX C: SITE ENTRANCE INFILTRATION STORAGE REQUIREMENTS

Summary of Results for 100 year Return Period

Half Drain Time exceeds 7 days.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Control Σ (l/s)	Max Outflow (l/s)	Max Volume (m³)	Status
15 min Summer	23.153	0.403	0.2	0.0	0.2	115.0	O K
30 min Summer	23.260	0.510	0.2	0.0	0.2	150.6	O K
60 min Summer	23.365	0.615	0.2	0.0	0.2	187.7	O K
120 min Summer	23.466	0.716	0.3	0.0	0.3	225.7	O K
180 min Summer	23.523	0.773	0.3	0.0	0.3	247.8	O K
240 min Summer	23.561	0.811	0.3	0.0	0.3	263.0	O K
360 min Summer	23.614	0.864	0.3	0.0	0.3	285.0	O K
480 min Summer	23.653	0.903	0.3	0.0	0.3	301.2	O K
600 min Summer	23.683	0.933	0.3	0.0	0.3	313.9	O K
720 min Summer	23.707	0.957	0.3	0.0	0.3	324.4	O K
960 min Summer	23.744	0.994	0.3	0.0	0.3	340.7	O K
1440 min Summer	23.793	1.043	0.3	0.0	0.3	362.9	O K
2160 min Summer	23.837	1.087	0.4	0.0	0.4	383.1	O K
2880 min Summer	23.863	1.113	0.4	0.0	0.4	395.2	O K
4320 min Summer	23.888	1.138	0.4	0.0	0.4	407.1	O K
5760 min Summer	23.894	1.144	0.4	0.0	0.4	410.2	O K
7200 min Summer	23.891	1.141	0.4	0.0	0.4	408.5	O K
8640 min Summer	23.882	1.132	0.4	0.0	0.4	404.3	O K
10080 min Summer	23.873	1.123	0.4	0.0	0.4	400.2	O K
15 min Winter	23.195	0.445	0.2	0.0	0.2	128.8	O K
30 min Winter	23.312	0.562	0.2	0.0	0.2	168.7	O K
60 min Winter	23.426	0.676	0.3	0.0	0.3	210.3	O K
120 min Winter	23.536	0.786	0.3	0.0	0.3	252.9	O K
180 min Winter	23.597	0.847	0.3	0.0	0.3	277.7	O K
240 min Winter	23.638	0.888	0.3	0.0	0.3	294.8	O K
360 min Winter	23.696	0.946	0.3	0.0	0.3	319.5	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
15 min Summer	99.025	0.0	17.2	19
30 min Summer	64.904	0.0	19.1	34
60 min Summer	40.510	0.0	40.9	64
120 min Summer	24.421	0.0	44.3	124
180 min Summer	17.920	0.0	46.1	184
240 min Summer	14.300	0.0	47.2	244
360 min Summer	10.377	0.0	48.6	364
480 min Summer	8.265	0.0	49.5	484
600 min Summer	6.922	0.0	50.0	604
720 min Summer	5.986	0.0	50.2	724
960 min Summer	4.756	0.0	50.3	964
1440 min Summer	3.434	0.0	49.3	1444
2160 min Summer	2.475	0.0	103.6	2164
2880 min Summer	1.960	0.0	101.8	2884
4320 min Summer	1.409	0.0	95.5	4320
5760 min Summer	1.114	0.0	203.4	5760
7200 min Summer	0.927	0.0	198.0	7200
8640 min Summer	0.798	0.0	191.1	8040
10080 min Summer	0.703	0.0	182.9	8664
15 min Winter	99.025	0.0	18.0	19
30 min Winter	64.904	0.0	20.0	34
60 min Winter	40.510	0.0	43.0	64
120 min Winter	24.421	0.0	46.8	124
180 min Winter	17.920	0.0	48.7	182
240 min Winter	14.300	0.0	50.0	242
360 min Winter	10.377	0.0	51.5	362

Summary of Results for 100 year Return Period

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
480 min Winter	23.737	0.987	0.3	0.0	0.3	337.9	O K
600 min Winter	23.770	1.020	0.3	0.0	0.3	352.3	O K
720 min Winter	23.796	1.046	0.3	0.0	0.3	364.1	O K
960 min Winter	23.836	1.086	0.4	0.0	0.4	382.6	O K
1440 min Winter	23.890	1.140	0.4	0.0	0.4	408.2	O K
2160 min Winter	23.938	1.188	0.4	0.0	0.4	431.7	O K
2880 min Winter	23.968	1.218	0.4	0.0	0.4	446.2	Flood Risk
4320 min Winter	23.999	1.249	0.4	0.0	0.4	461.6	Flood Risk
5760 min Winter	24.010	1.260	0.4	0.0	0.4	467.3	Flood Risk
7200 min Winter	24.011	1.261	0.4	0.0	0.4	467.8	Flood Risk
8640 min Winter	24.005	1.255	0.4	0.0	0.4	465.0	Flood Risk
10080 min Winter	23.996	1.246	0.4	0.0	0.4	460.2	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Discharge Volume (m³)	Time-Peak (mins)
480 min Winter	8.265	0.0	52.4	480
600 min Winter	6.922	0.0	52.9	598
720 min Winter	5.986	0.0	53.2	718
960 min Winter	4.756	0.0	53.2	954
1440 min Winter	3.434	0.0	52.1	1428
2160 min Winter	2.475	0.0	109.9	2140
2880 min Winter	1.960	0.0	107.9	2828
4320 min Winter	1.409	0.0	100.9	4232
5760 min Winter	1.114	0.0	216.5	5592
7200 min Winter	0.927	0.0	210.6	6920
8640 min Winter	0.798	0.0	202.9	8216
10080 min Winter	0.703	0.0	193.8	9480

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	20.000	Shortest Storm (mins)	15
Ratio R	0.404	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+0

Time Area Diagram

Total Area (ha) 0.620

Time From:	(mins) To:	Area (ha)
0	4	0.620

Model Details

Storage is Online Cover Level (m) 24.250

Infiltration Basin Structure

Invert Level (m) 22.750 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00360 Porosity 1.00
Infiltration Coefficient Side (m/hr) 0.00360

Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	250.0	1.500	565.8

Weir Outflow Control

Discharge Coef 0.544 Width (m) 0.300 Invert Level (m) 24.250

ANNEX 2A.8: FREIGHT MANAGEMENT FACILITY DRAINAGE DESIGN NOTE

CONTENTS

TABLES

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

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

1.1.4 The freight management facility forms one of the Associated Developments (AD) which are required to mitigate traffic impacts arising from the main development site. The freight management facility is located alongside the A14 near to its interchange with the A12 at Seven Hills near Ipswich. Its function is to provide a hub from which a controlled pattern of deliveries to the main development site can be provided, reducing freight movements during peak and sensitive hours on the road network. It will act as a holding area in the event of problems or congestion on the approaches to the Sizewell C main development site. Full details of its facilities are contained in **Volume 8 Freight Management Facility Chapter 2 Description of the Freight Management Facility** [APP-151] and are described in summary below.

1.1.5 The site will consist of parking for approximately 150 HGVs, workforce parking, welfare, security and amenity buildings. The workforce parking includes car parking spaces, accessible spaces, cycle spaces and motorcycle spaces.

1.1.6 The site access will be from Felixstowe Road where the road will be widened to accommodate a right turn ghost island. The modification of the highway to accommodate the access will be designed to Suffolk County Council's (SCC) adoptable standards.

1.1.7 The freight management facility site will generate surface water runoff from paved areas and roofs which will require to be removed, treated as necessary and disposed.

1.1.8 The site entrance and access from Felixstowe Road will generate highway runoff which will require to be removed, treated as necessary and disposed.

1.1.9 The freight management facility welfare facilities will generate foul water flows which will require to be removed, treated as necessary and disposed.

1.1.10 The freight management facility and its associated access and local road changes will remain in place and use during construction of the Sizewell C power station. Once construction is complete the site will be closed and decommissioned. It will then return to current agricultural use.

1.1.11 It is intended that the proposed access will be removed and Felixstowe Road will be returned to its current alignment.

2 PURPOSE

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

- The effective removal of highway and surface water runoff from the proposed freight management facility and its site access entrance, together with its treatment and disposal; and
- The effective removal and treatment of foul water generated by the workforce from the proposed freight management facility.

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 **Freight Management Facility Flood Risk Assessment** (FRA) [APP-141].

2.1.3 The purpose of this technical note is to provide details of data which validate the **Outline Drainage Strategy** [REP2-033] and subsequent **Drainage Strategy** (submitted at Deadline 7), a description of how the proposed concept drainage infrastructure is developing and evolving and to demonstrate that it continues to provide for the effective and satisfactory drainage of the freight management facility and its associated external road modification, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution.

3 DESCRIPTION OF DCO DRAINAGE CONCEPT DESIGN

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

3.1.2 Given the proven infiltration rates, all surface water generated within the freight management facility red line boundary would be contained within the site and discharged to ground.

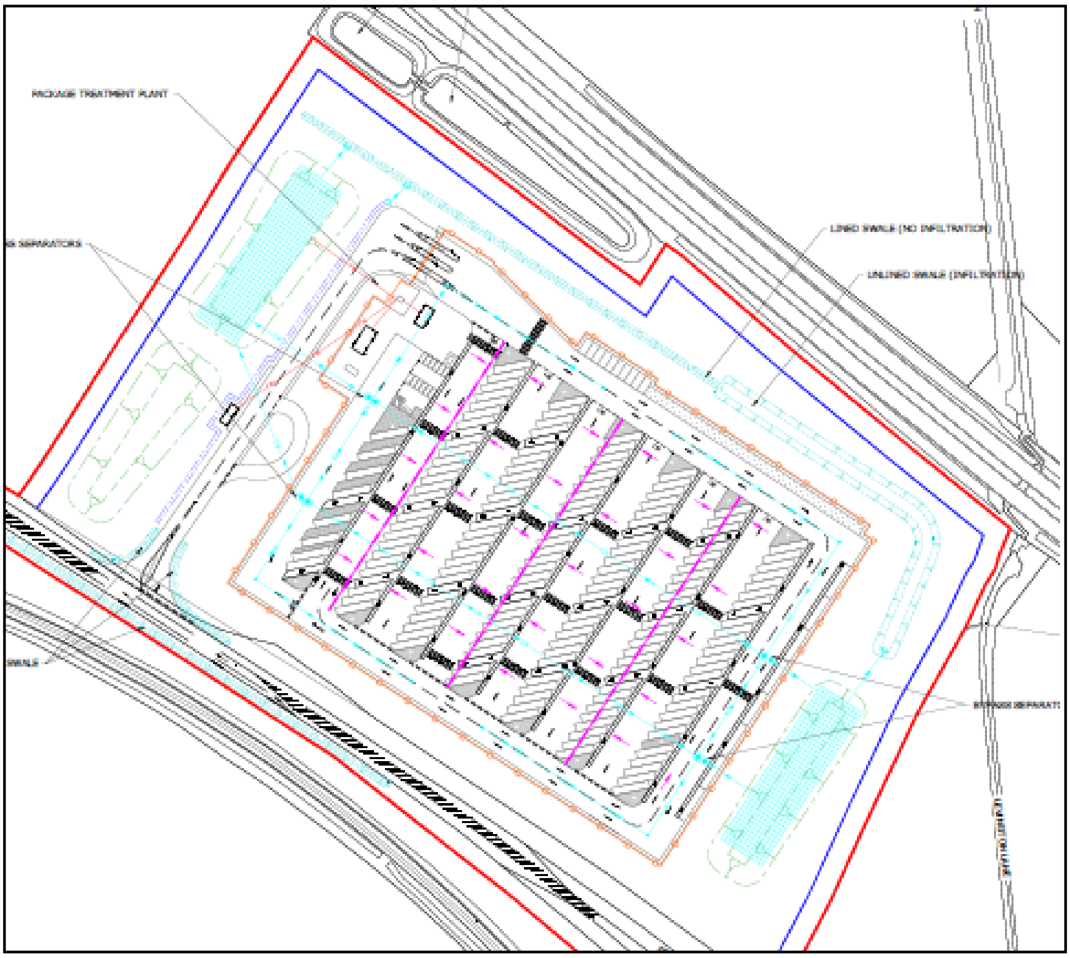
3.1.3 External roads modified to access the site would discharge surface water highway runoff to swales and filter drains where flows will infiltrate to ground.

3.1.4 Liaison took place with Anglian Water to establish whether there are any public foul sewers, in proximity to the freight management facility, to which foul water could be discharged by gravity. Since it was confirmed that there are no foul water sewers in vicinity it would be necessary to pump over long distance offsite to discharge into a public sewer.

3.1.5 Given that freight management facility is a temporary facility and will only operate during construction of Sizewell C the option of treatment on site using a package treatment plant is proposed. The treated effluent would discharge to ground by infiltration.

3.1.6 The internal site layout showing the proposed layout of drainage infrastructure and the sewage treatment plant is shown in **Plate 1**, an extract from the Application drawing ”Chapter 2 Description of the FMF Figure 2.4” [APP-153].

Plate 1: Freight management facility internal layout showing concept drainage infrastructure

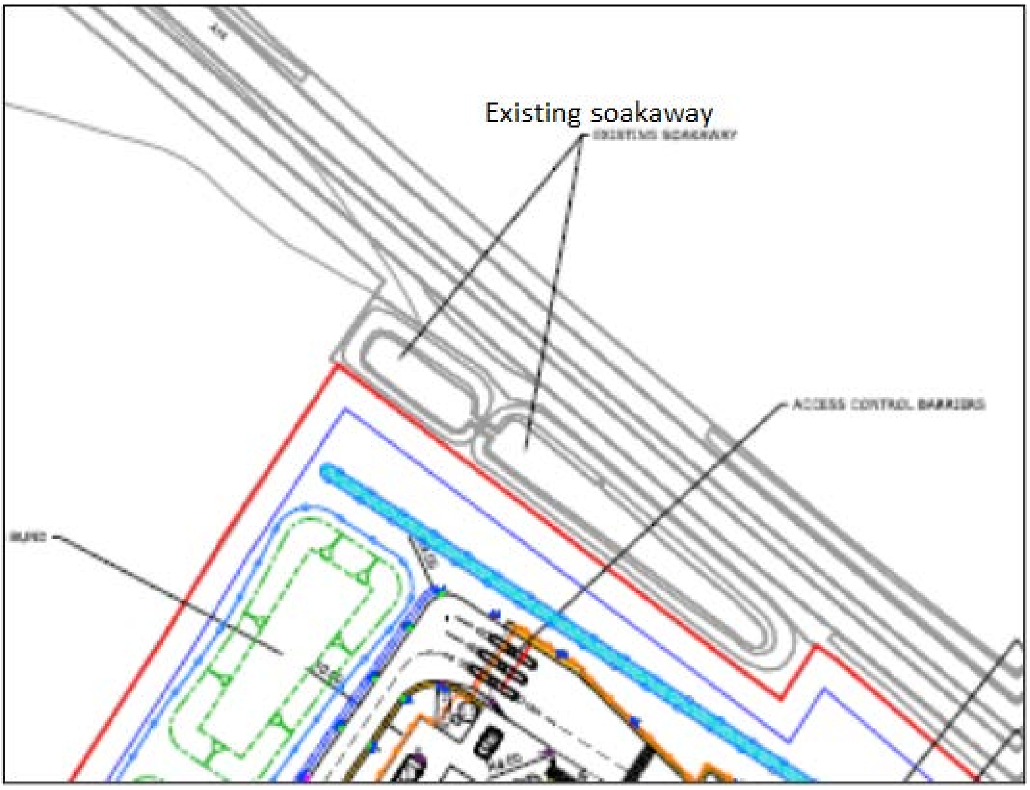


4 EXISTING SITE AND ADJACENT HIGHWAY DRAINAGE ARRANGEMENTS

4.1.1 The extent of the freight management facility within the red line boundary forms agricultural land and has no obvious sign of drainage infrastructure.

4.1.2 The A14 located to the north of the red line boundary appears to have highway drainage infrastructure which outfalls to an infiltration basin facility. This is shown in **Plate 2** and abuts the red line boundary.

Plate 2: Existing A14 infiltration basin location



4.1.3 Given the close proximity of the existing A14 infiltration basin adjacent to the site, the proposed freight management facility site drainage infrastructure must not provide for infiltration to ground in this area as this could compromise the absorption capacity of the ground for A14 highway runoff.

4.1.4 No detailed site inspection of Felixstowe Road has been undertaken. However, based on remote inspection of the road using Google Streetview there is no sign of obvious highway drainage infrastructure. It is assumed that currently highway runoff is removed “over the edge” with infiltration into the verge.

4.1.5 The Environment Agency Surface Water Flood Map shows a predicted overland flow path with minor flooding passing through the A14 infiltration basins and through the north west corner of the freight management facility. This is shown in **Plate 3**.

Plate 3: A14 predicted surface water flood risk at the freight management facility

4.1.6 If flooding does occur, it would be captured by the lined swale and would then be infiltrated to ground.

5 GROUND INVESTIGATION AND INFILTRATION TESTING RESULTS

5.1.1 Three trial pits were excavated within the site at locations shown in **Plate 4**.

Plate 4: Freight management facility site infiltration test trial hole locations

5.1.2 Infiltration testing in accordance with BRE365 (Ref. 1) was undertaken and the results are shown in **Table 1**.

Table 1: Freight management facility site infiltration test trial hole results

Location	Test 1(m/s)	Test 2(m/s)	Test 3 (m/s)
TP01	3.53×10^{-6}	1.73×10^{-6}	9.89×10^{-7}
TP02	4.72×10^{-5}	4.66×10^{-5}	3.32×10^{-5}
TP03	5.80×10^{-7}	5.36×10^{-7}	5.70×10^{-7}

5.1.3 These results demonstrate that disposal of surface water runoff by infiltration is achievable. SCC consider that an infiltration rate in excess of 1.4×10^{-6} m/s is viable for infiltration to ground. However, the variation in infiltration rate is noted and has been taken into consideration as part of developing the concept layout as described in this technical note in Section 6.

6 UPDATED SURFACE WATER DRAINAGE DESIGN STRATEGY

6.1.1 The surface water arrangements for removal currently remain, in principle, as described in document “Environmental Statement Volume 8 Chapter 2 Description of the Freight Management Facility” dated July 2020 and shown in DCO Figure 2.4. An extract of this Figure is shown in **Plate 1** of this report. The Environmental Statement takes account of the infiltration test results obtained in October 2019.

6.1.2 Surface water runoff from roofs will be drained via downpipes and gullies, as appropriate to underground carrier drains.

6.1.3 All of the internal roads and the HGV parking areas will have an impermeable surface. Surface water runoff will be drained via surface outlets, gullies, linear channels and drains, etc. These will discharge into underground carrier drains.

6.1.4 Bypass interceptors will be installed on the carrier drains downstream of the bus/HGV standing areas in order to remove hydrocarbon and silt contaminants which will improve the water quality of the runoff before discharge to ground.

6.1.5 The concept design submitted for DCO and shown in **Plate 1** provided for underground carrier drains which will discharge all surface water runoff into two underground attenuation storage tanks from where it will infiltrate to ground. The tanks are proposed to be located beneath the landscape bunds located on the east and west sides of the site.

6.1.6 The size of the tanks calculated for concept design stage was 88 m long x 22 m wide x 0.6 m deep. The surface water drainage network capacity was assessed by hydraulic calculation. The calculation was based on the average of measured infiltration rates at TP01, TP02 and TP03 and a requirement for the tanks to drain down by half their storage volume in 24 hours. For a 1 in 30 year return period rainfall event it was found that there was insufficient storage and as a result it is proposed that additional storage volume be provided by swales.

6.1.7 The swales were located over the full length of the northern side of the site and the lowest part of the eastern side of the site. Since ground levels fall from south to north the swales will also intercept runoff from surface water overland flow which does not drain into the underground drainage network.

6.1.8 The swales will also remove surface water runoff by infiltration to ground. However due to the proximity of the western portion of the swale to the A14 infiltration basin facility, this length of the swale is lined making it impermeable. This will avoid any risk of infiltration causing adverse impact on the performance of the A14 infiltration basin.

6.1.9 Whilst the concept design provided sufficient evidence and confidence that removal of surface water runoff by infiltration is viable, as part of development of the concept drainage design the location and performance of the two storage tanks has been reviewed.

6.1.10 The position of the west storage tank is noted to be in proximity to TP01 infiltration test trial hole whilst the east storage tank is noted to be in proximity to TP03. These tanks are located clear of the paved area and beneath the landscaping bunds. It was considered desirable to avoid locating tanks beneath the paved area in order to minimise loading issues on the tank.

6.1.11 In review of the storage tank sizes it has been considered more appropriate to use infiltration rates obtained in proximity to the tank location rather than an average value. This is because of the variation in infiltration rate, as shown in **Table 1.**

6.1.12 In using individual infiltration rates, it is apparent that the east storage basin is unfavourably located because the infiltration rate stated in Table 1 is less than the 1.4×10^{-6} m/s considered by SCC as the minimum viable value for infiltration to ground. Accordingly, the location of a storage tank at this location is discounted.

6.1.13 Calculations have been undertaken for two alternative options. Option 1 provides for a single tank in the west and Option 2 provides for a single tank in the centre of the site in proximity to the TP02 location. The approximate location and footprint of the tanks is shown in Appendix A. Hydraulic calculations which validate the tank sizes are provided in Appendices B and C.

6.1.14 The Option 1 tank size has been determined by a requirement for it to be located within the unpaved area to the west. The available size has been used in hydraulic modelling. A summary of predicted hydraulic performance is shown in Table 2 with full results in Appendix B.

Table 2: Freight management facility option 1 storage tank parameters

Parameters	Values
Cellular Soakaway Storage Dimension	135m (L) x 22m (B) x 1.2m (D)
Volume Available	3564 m^3
Average Infiltration Rate at TP01	7.5 mm/hour
Half Drain Time	8004 minutes (~5.5 days)

6.1.15 The results demonstrate that infiltration is viable in that the stored volume will eventually be removed by infiltration. However, the half drain time is excessive. In the event of follow on rainfall events within days of the design event, there may not be sufficient storage volume which could result in surface flooding. For this reason, Option 1 is not acceptable.

6.1.16 The Option 2 tank size is not constrained since it can be located anywhere within the central paved area. As a result, the tank size has been determined by the hydraulic modelling. A summary of predicted hydraulic performance is shown in Table 3 with full results in Appendix C.

Table 3: Freight management facility option 2 storage tank parameters

Parameters	Values
Cellular Soakaway Storage Dimension	56m (L) x 50m (B) x 0.6m (D)
Volume Available	1680 m^3
Average Infiltration Rate at TP02	152.4 mm/hour
Half Drain Time	212 minutes (~3.5 hours)

6.1.17 The infiltration rate at TP02 is significantly greater that that at TP01, and thus the required storage tank volume is substantially less. Accordingly, it is proposed that the site be drained to a storage tank for infiltration to ground located within the central paved area. The shape of the tank whether square or rectangular will be developed as design progresses. This will also need to take account of the structural design of the tank and the required depth of cover to accommodate surface loading.

6.1.18 Although the storage tank can accommodate all surface water runoff within the site, it is intended to retain the swale at the northern and eastern sides of the site in order to intercept and capture exceedance overland flow from adjacent 3rd party land.

7 UPDATED FOUL WATER DRAINAGE DESIGN STRATEGY

7.1.1 The foul water drainage strategy remains unchanged with foul water flows collected by an underground drainage network and discharged into a package sewage treatment plant. Treated effluent is drained into an attenuation tank from where it will infiltrate to ground. The question as to whether it is more appropriate to provide a separate treated effluent attenuation tank or to discharge into the surface water storage tank, as currently proposed will be determined as design progresses and in accordance with environmental permit requirements.

7.1.2 It is noted that foul water flow rates generated will be low and intermittent with a range of flow. This makes the delivery of a consistent treated effluent more challenging. Once the environmental permit requirements - which will set quality standards - have been determined, it will be necessary to ensure that a suitable package plant and associated treatment infrastructure can reliably produce a compliant treated effluent.

7.1.3 In the event of any doubt regarding the ability of a package treatment plant being able to produce the required quality of treated effluent, the alternative will be to collect the foul water sewage in an underground sealed cess tank from which it can be collected and removed by tanker for treatment offsite.

8 UPDATED SURFACE WATER DRAINAGE DESIGN STRATEGY – MODIFIED LOWESTOFT ROAD SITE ACCESS ENTRANCE

8.1.1 The surface water drainage strategy for the highway drainage subject to adoption by SCC remains unchanged being infiltration to ground.

8.1.2 Surface water highway runoff will be removed by “over the edge” flow and collected in swales for disposal by infiltration to ground. The proven infiltration rates in the locale demonstrate that this is feasible. When the swales dimensions are determined at detailed design, if necessary, an underlying filter drain will be provided to increase the efficiency of infiltration.

9 SUMMARY AND CONCLUSION

9.1.1 The purpose of this technical note is to validate the **Outline Drainage Strategy** [REP2-033] and subsequent **Drainage Strategy** (submitted at Deadline 7) for the freight management facility. It describes how the concept design is evolving to provide for the effective drainage of the freight management facility.

9.1.2 The drainage design for both the internal freight management facility and modification to Lowestoft Road and site entrance has been developed to a level of detail to provide sufficient evidence of an achievable drainage strategy that is compliant with national planning and environmental regulatory requirements.

9.1.3 Subject to the results of DCO examination and acceptance of the drainage design strategy principles contained in this report, the drainage designs will be developed to preliminary design stage.

9.1.4 The freight management facility drainage design will be based on CIRIA C753 SuDS Manual (Ref. 2), Design and Construction Guidance for Foul and Surface Water Sewers (formerly Sewers for Adoption) (Ref. 3), and PPG4 Treatment and Disposal of Sewage where no Foul Water Sewer is Available (Ref. 4).

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

9.1.6 As preliminary design progresses, SZC Co. will liaise with SCC and the Environment Agency through design review meetings to build acceptance of the drainage infrastructure and to enable compliance with regulatory requirements and environmental permits.

REFERENCES

1. BRE Digest Soakaway design: DG 365 – 2016, BRE, 2016 https://www.brebookshop.com/details.jsp?id=327592

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

3. SSG Appendix C - Design and construction guidance for foul and surface water sewers offered for adoption under the Code for adoption agreements for water and sewerage companies operating wholly or mainly in England ("the Code"). Approved Version 2.0. 10 March 2020. Water UK. https://www.thenbs.com/PublicationIndex/documents/details?DocID=330727

4. Pollution Prevention Guidelines PPG4: Treatment and disposal of sewage where no foul sewer is available, Environment and Heritage Service / Scottish Environment Protection Agency / Environment Agency, July 2006. PMHO0706BJGL-E-E. https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/485181/pmho0706bjgl-e-e.pdf

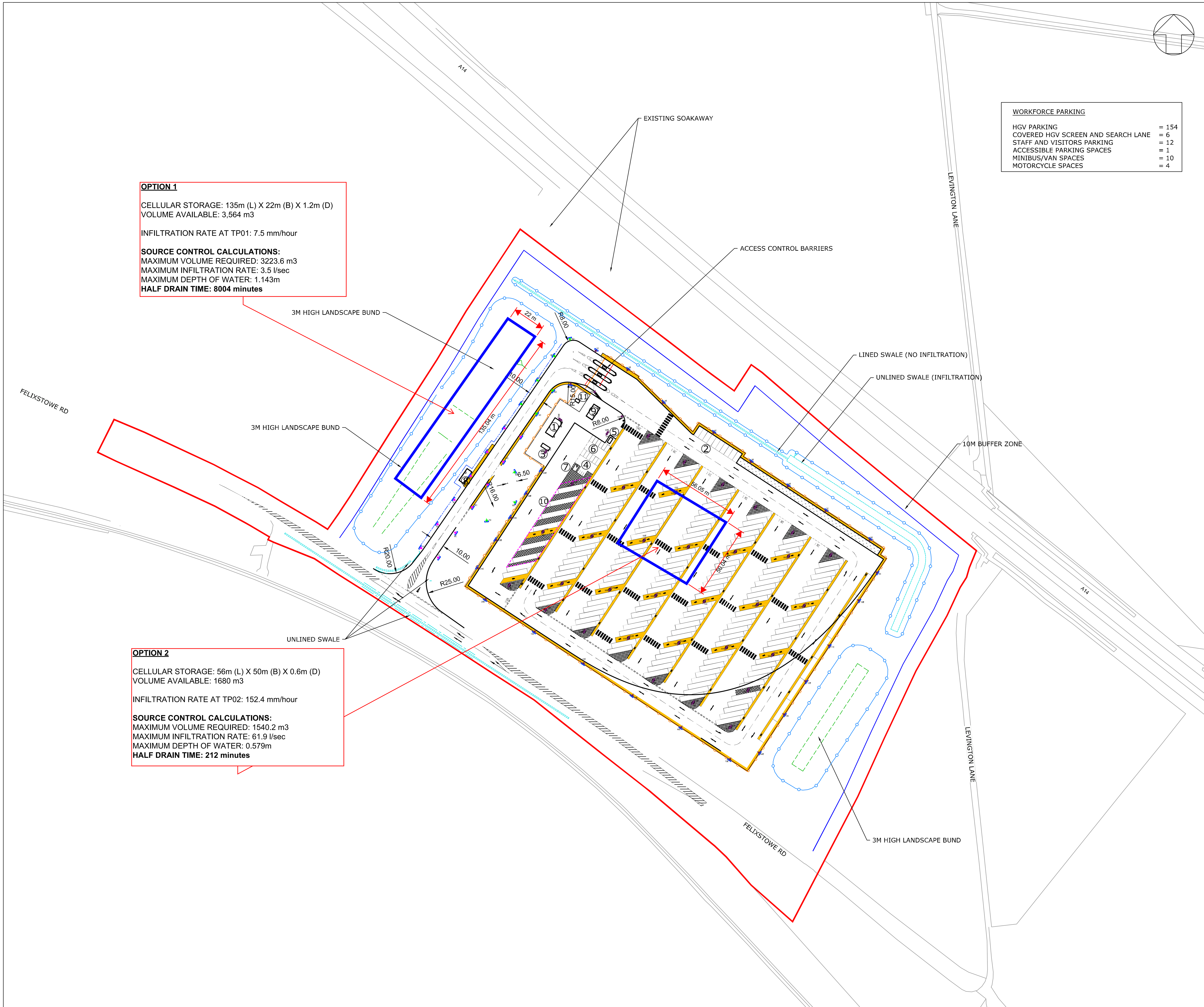
5. Highways Agency et al. (2009). Volume 11, Section 3, Part 10: Road Drainage and the Water Environment, HD45/09. http://www.standardsforhighways.co.uk/ha/standards/dmrb/vol11/section3/hd4509.pdf

6. Manual of Contract Documents for Highway Works (MCHW), Highways Agency. https://www.standardsforhighways.co.uk/ha/standards/mchw/index.htm

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

8. Sustainable Drainage Systems (SuDS) a Local Design Guide Appendix A to the Suffolk Flood Risk Management Strategy, Suffolk County Council, May 2018 https://www.greensuffolk.org/app/uploads/2021/05/2018-10-01-SFRMS-SuDS-Guidance-Appendix-A-.pdf

APPENDIX A: OPTIONS 1 AND 2 STORAGE TANK LOCATIONS



OPTION 1

CELLULAR STORAGE: 135m (L) X 22m (B) X 1.2m (D)
VOLUME AVAILABLE: 3,564 m3

INFILTRATION RATE AT TP01: 7.5 mm/hour

SOURCE CONTROL CALCULATIONS:
MAXIMUM VOLUME REQUIRED: 3223.6 m3
MAXIMUM INFILTRATION RATE: 3.5 l/sec
MAXIMUM DEPTH OF WATER: 1.143m
HALF DRAIN TIME: 8004 minutes

OPTION 2

CELLULAR STORAGE: 56m (L) X 50m (B) X 0.6m (D)
VOLUME AVAILABLE: 1680 m3

INFILTRATION RATE AT TP02: 152.4 mm/hour

SOURCE CONTROL CALCULATIONS:
MAXIMUM VOLUME REQUIRED: 1540.2 m3
MAXIMUM INFILTRATION RATE: 61.9 l/sec
MAXIMUM DEPTH OF WATER: 0.579m
HALF DRAIN TIME: 212 minutes

WORKFORCE PARKING

HGV PARKING	= 154
COVERED HGV SCREEN AND SEARCH LANE	= 6
STAFF AND VISITORS PARKING	= 12
ACCESSIBLE PARKING SPACES	= 1
MINIBUS/VAN SPACES	= 10
MOTORCYCLE SPACES	= 4

NOTES:

1. Do not scale from this drawing. All dimensions are in metres unless noted otherwise.
2. For Lighting Column specifications see drawings SZC-SZ0204-FP-000-DRW-100031-32.
3. For HGV Search and Screen canopy see drawing SZC-SZ0204-FP-000-DRW-100104.

KEY:

- DEVELOPMENT SITE BOUNDARY
- 1 AMENITY AND WELFARE BUILDING (12M X 6M)
- 2 VAN / MINIBUS PARKING
- 3 SMOKING SHELTER (6M X 2.5M)
- 4 MOTORCYCLE PARKING
- 5 CYCLE SHELTER (4M X 2M)
- 6 STAFF AND VISITORS PARKING
- 7 ACCESSIBLE PARKING
- 8 SECURITY BOOTH (10M X 4M)
- 9 SECURITY BUILDING (10M X 5M)
- 10 COVERED HGV SCREEN & SEARCH LANE
- 11 PACKAGE TREATMENT PLANT
- BUFFER ZONE
- SECURITY FENCE (1.8M HIGH)
- ECOLOGICAL FENCING
- GATE
- SWALE (INDICATIVE)
- LANDSCAPE BUND
- PROPOSED DITCH (INDICATIVE)
- LANDSCAPING
- PEDESTRIAN WALKWAY
- MANUAL RISING ARM BARRIERS
- BOLLARD
- TACTILE PAVING
- PROPOSED LIGHTING COLUMNS (INDICATIVE)
- LIGHTING LANTERN DEMOUNTABLE SHIELD
- COVERED HGV SCREEEN AND SEARCH LANE

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION / COMMENTS	APPROVED
01	FEB 2020	NKS	KA	DCO SUBMISSION	PJ

NOT PROTECTIVELY MARKED

edF ENERGY SZC CGN

PROJECT:
SIZEWELL C

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

DRAWING TITLE:
FREIGHT MANAGEMENT FACILITY
PROPOSED GENERAL ARRANGEMENT

DRAWING NO:	REVISION:
SZC-SZ0204-FP-000-DRW-100026	01

DATE:	DRAWN:	SCALE:
FEB 2020	NKS	1:1250@A1

SCALE BAR:
25m 0 25 50 75 100m
SCALE IN METRES 1:1250

APPENDIX B: OPTION 1 STORAGE TANK HYDRAULIC CALCULATIONS

Summary of Results for 100 year Return Period

Half Drain Time : 8004 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	24.803	0.303	3.2	854.0	O K
30 min Summer	24.897	0.397	3.2	1120.8	O K
60 min Summer	24.996	0.496	3.3	1398.9	O K
120 min Summer	25.096	0.596	3.3	1682.8	O K
180 min Summer	25.154	0.654	3.3	1846.5	O K
240 min Summer	25.194	0.694	3.3	1958.3	O K
360 min Summer	25.250	0.750	3.3	2115.6	O K
480 min Summer	25.291	0.791	3.4	2230.5	O K
600 min Summer	25.322	0.822	3.4	2318.7	O K
720 min Summer	25.347	0.847	3.4	2389.5	O K
960 min Summer	25.385	0.885	3.4	2497.0	O K
1440 min Summer	25.433	0.933	3.4	2633.6	Flood Risk
2160 min Summer	25.471	0.971	3.4	2739.8	Flood Risk
2880 min Summer	25.487	0.987	3.4	2785.5	Flood Risk
4320 min Summer	25.488	0.988	3.4	2786.5	Flood Risk
5760 min Summer	25.466	0.966	3.4	2724.2	Flood Risk
7200 min Summer	25.436	0.936	3.4	2640.6	Flood Risk
8640 min Summer	25.409	0.909	3.4	2563.4	Flood Risk
10080 min Summer	25.383	0.883	3.4	2490.9	O K
15 min Winter	24.839	0.339	3.2	956.9	O K
30 min Winter	24.945	0.445	3.2	1256.0	O K
60 min Winter	25.056	0.556	3.3	1568.0	O K
120 min Winter	25.169	0.669	3.3	1887.4	O K
180 min Winter	25.234	0.734	3.3	2072.3	O K
240 min Winter	25.279	0.779	3.3	2198.8	O K
360 min Winter	25.343	0.843	3.4	2377.7	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	97.600	0.0	31
30 min Summer	64.093	0.0	46
60 min Summer	40.092	0.0	76
120 min Summer	24.228	0.0	136
180 min Summer	17.804	0.0	196
240 min Summer	14.222	0.0	254
360 min Summer	10.328	0.0	374
480 min Summer	8.231	0.0	494
600 min Summer	6.897	0.0	614
720 min Summer	5.967	0.0	734
960 min Summer	4.744	0.0	972
1440 min Summer	3.428	0.0	1450
2160 min Summer	2.473	0.0	2168
2880 min Summer	1.960	0.0	2888
4320 min Summer	1.410	0.0	4324
5760 min Summer	1.115	0.0	5760
7200 min Summer	0.929	0.0	6416
8640 min Summer	0.800	0.0	7096
10080 min Summer	0.705	0.0	7776
15 min Winter	97.600	0.0	31
30 min Winter	64.093	0.0	45
60 min Winter	40.092	0.0	76
120 min Winter	24.228	0.0	134
180 min Winter	17.804	0.0	192
240 min Winter	14.222	0.0	252
360 min Winter	10.328	0.0	370

Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
480 min Winter	25.389	0.889	3.4	2509.1	O K
600 min Winter	25.425	0.925	3.4	2610.8	Flood Risk
720 min Winter	25.454	0.954	3.4	2692.8	Flood Risk
960 min Winter	25.499	0.999	3.4	2818.9	Flood Risk
1440 min Winter	25.557	1.057	3.4	2983.2	Flood Risk
2160 min Winter	25.605	1.105	3.5	3119.1	Flood Risk
2880 min Winter	25.630	1.130	3.5	3187.7	Flood Risk
4320 min Winter	25.643	1.143	3.5	3223.6	Flood Risk
5760 min Winter	25.630	1.130	3.5	3188.7	Flood Risk
7200 min Winter	25.604	1.104	3.5	3116.2	Flood Risk
8640 min Winter	25.571	1.071	3.4	3022.9	Flood Risk
10080 min Winter	25.535	1.035	3.4	2921.4	Flood Risk

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
480 min Winter	8.231	0.0	488
600 min Winter	6.897	0.0	606
720 min Winter	5.967	0.0	724
960 min Winter	4.744	0.0	960
1440 min Winter	3.428	0.0	1432
2160 min Winter	2.473	0.0	2136
2880 min Winter	1.960	0.0	2832
4320 min Winter	1.410	0.0	4204
5760 min Winter	1.115	0.0	5544
7200 min Winter	0.929	0.0	6848
8640 min Winter	0.800	0.0	8056
10080 min Winter	0.705	0.0	8976

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.800	Shortest Storm (mins)	15
Ratio R	0.400	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+0

Time Area Diagram

Total Area (ha) 4.691

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	1.172	4	8	1.173	8	12	1.173	12	16	1.173

Model Details

Storage is Online Cover Level (m) 25.700

Cellular Storage Structure

Invert Level (m) 24.500 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.00750 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.00750

Depth (m)	**Area (m²)**	**Inf. Area (m²)**	**Depth (m)**	**Area (m²)**	**Inf. Area (m²)**
0.000	2970.0	2970.0	1.200	2970.0	3346.8

APPENDIX C: OPTION 2 STORAGE TANK HYDRAULIC CALCULATIONS

Summary of Results for 100 year Return Period

Half Drain Time : 212 minutes.

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
15 min Summer	24.793	0.293	60.6	778.6	O K
30 min Summer	24.879	0.379	61.0	1008.8	O K
60 min Summer	24.953	0.453	61.3	1205.0	O K
120 min Summer	24.996	0.496	61.5	1319.7	O K
180 min Summer	24.995	0.495	61.5	1316.2	O K
240 min Summer	24.982	0.482	61.4	1282.4	O K
360 min Summer	24.954	0.454	61.3	1208.4	O K
480 min Summer	24.928	0.428	61.2	1138.3	O K
600 min Summer	24.902	0.402	61.1	1069.4	O K
720 min Summer	24.877	0.377	61.0	1002.4	O K
960 min Summer	24.829	0.329	60.7	874.3	O K
1440 min Summer	24.743	0.243	60.4	647.0	O K
2160 min Summer	24.645	0.145	59.9	385.8	O K
2880 min Summer	24.583	0.083	59.6	220.4	O K
4320 min Summer	24.545	0.045	53.2	118.9	O K
5760 min Summer	24.536	0.036	42.5	94.7	O K
7200 min Summer	24.530	0.030	35.4	79.4	O K
8640 min Summer	24.526	0.026	30.6	67.9	O K
10080 min Summer	24.523	0.023	27.1	60.0	O K
15 min Winter	24.831	0.331	60.8	880.1	O K
30 min Winter	24.930	0.430	61.2	1142.5	O K
60 min Winter	25.016	0.516	61.6	1373.0	O K
120 min Winter	25.073	0.573	61.8	1523.9	O K
180 min Winter	25.079	0.579	61.9	1540.2	O K
240 min Winter	25.065	0.565	61.8	1502.8	O K
360 min Winter	25.028	0.528	61.6	1403.9	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
15 min Summer	97.600	0.0	28
30 min Summer	64.093	0.0	42
60 min Summer	40.092	0.0	68
120 min Summer	24.228	0.0	124
180 min Summer	17.804	0.0	176
240 min Summer	14.222	0.0	204
360 min Summer	10.328	0.0	266
480 min Summer	8.231	0.0	334
600 min Summer	6.897	0.0	402
720 min Summer	5.967	0.0	468
960 min Summer	4.744	0.0	602
1440 min Summer	3.428	0.0	856
2160 min Summer	2.473	0.0	1212
2880 min Summer	1.960	0.0	1536
4320 min Summer	1.410	0.0	2204
5760 min Summer	1.115	0.0	2936
7200 min Summer	0.929	0.0	3672
8640 min Summer	0.800	0.0	4400
10080 min Summer	0.705	0.0	5136
15 min Winter	97.600	0.0	28
30 min Winter	64.093	0.0	42
60 min Winter	40.092	0.0	70
120 min Winter	24.228	0.0	124
180 min Winter	17.804	0.0	178
240 min Winter	14.222	0.0	230
360 min Winter	10.328	0.0	286

Summary of Results for 100 year Return Period

Storm Event	Max Level (m)	Max Depth (m)	Max Infiltration (l/s)	Max Volume (m³)	Status
480 min Winter	24.991	0.491	61.5	1305.6	O K
600 min Winter	24.952	0.452	61.3	1203.4	O K
720 min Winter	24.914	0.414	61.1	1101.8	O K
960 min Winter	24.841	0.341	60.8	906.1	O K
1440 min Winter	24.712	0.212	60.2	564.6	O K
2160 min Winter	24.580	0.080	59.6	213.5	O K
2880 min Winter	24.545	0.045	53.8	120.2	O K
4320 min Winter	24.533	0.033	39.0	86.6	O K
5760 min Winter	24.526	0.026	31.2	69.2	O K
7200 min Winter	24.522	0.022	25.9	57.3	O K
8640 min Winter	24.519	0.019	22.3	49.3	O K
10080 min Winter	24.517	0.017	19.9	44.0	O K

Storm Event	Rain (mm/hr)	Flooded Volume (m³)	Time-Peak (mins)
480 min Winter	8.231	0.0	362
600 min Winter	6.897	0.0	438
720 min Winter	5.967	0.0	510
960 min Winter	4.744	0.0	650
1440 min Winter	3.428	0.0	904
2160 min Winter	2.473	0.0	1216
2880 min Winter	1.960	0.0	1476
4320 min Winter	1.410	0.0	2204
5760 min Winter	1.115	0.0	2928
7200 min Winter	0.929	0.0	3608
8640 min Winter	0.800	0.0	4320
10080 min Winter	0.705	0.0	5144

Rainfall Details

Rainfall Model	FSR	Winter Storms	Yes
Return Period (years)	100	Cv (Summer)	0.750
Region	England and Wales	Cv (Winter)	0.840
M5-60 (mm)	19.800	Shortest Storm (mins)	15
Ratio R	0.400	Longest Storm (mins)	10080
Summer Storms	Yes	Climate Change %	+0

Time Area Diagram

Total Area (ha) 4.691

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	1.172	4	8	1.173	8	12	1.173	12	16	1.173

Model Details

Storage is Online Cover Level (m) 25.700

Cellular Storage Structure

Invert Level (m) 24.500 Safety Factor 2.0
Infiltration Coefficient Base (m/hr) 0.15240 Porosity 0.95
Infiltration Coefficient Side (m/hr) 0.15240

Depth (m)	**Area (m^2)**	**Inf. Area (m^2)**	**Depth (m)**	**Area (m^2)**	**Inf. Area (m^2)**
0.000	2800.0	2800.0	0.600	2800.0	2927.2

ANNEX 2A.9: SIZEWELL LINK ROAD PRELIMINARY DRAINAGE DESIGN NOTE

CONTENTS

TABLES

PLATES

FIGURES

None provided.

APPENDICES

None provided.

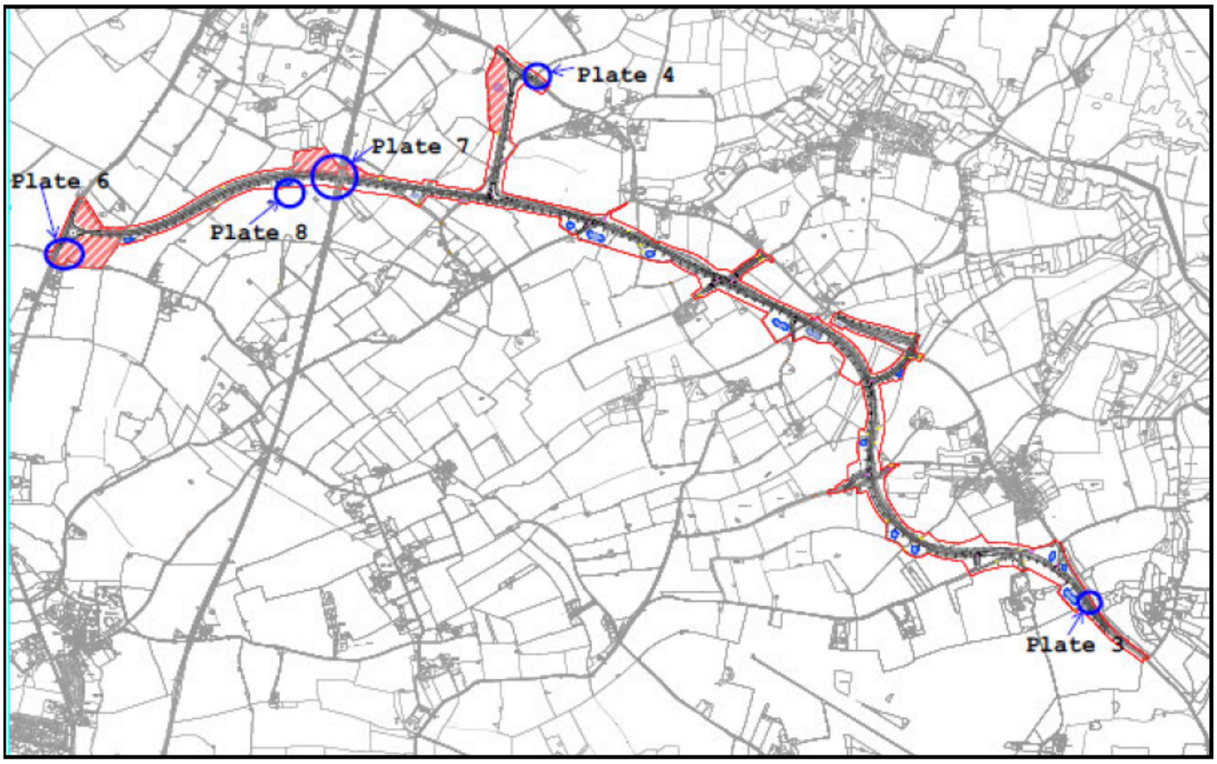
1 INTRODUCTION

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 undertaken work to validate and develop the design of the Sizewell link road that was originally submitted as part of the DCO application. This document forms one of a series of design validation and evolution documents being provided to the Examining Authority in support of the **Outline Drainage Strategy** [REP2-033].

1.1.3 The proposed development is one of the Sizewell C Project's associated development sites; a permanent single carriageway road that would run 6.8km from the A12 just south of Yoxford in an easterly direction, joining the B1122 south of the town of Theberton. A large-scale plan showing the route of Sizewell link road is shown in **Plate 1.** The specific locations shown in subsequent plates are identified for convenience.

Plate 1: Sizewell link road location and route



1.1.4 The Sizewell link road would create a new route around the south of the villages of Yoxford, Middleton Moor and Theberton, helping to reduce the amount of traffic on the B1122 during the peak construction phase of the Sizewell C Project.

1.1.5 The Sizewell link road will be designed to Suffolk County Council's (SCC) adoptable standards (Ref. 1).

1.1.6 The Sizewell link road will generate highway surface water runoff which will require to be removed, treated as necessary and disposed at a controlled rate of discharge.

1.1.7 The Sizewell link road will cross six watercourses at seven locations. Three local field ditch crossings have also been identified following a site visit in January 2021.

2 PURPOSE

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

- The effective removal of runoff from the proposed Sizewell link road highway and its disposal;
- The crossing of watercourses along the line of the Sizewell link road.

2.1.2 This strategy was developed in consultation with drainage regulators and local authorities, including SCC and the Environment Agency (EA). A number of workshops were held and the observations/requirements of drainage regulators were incorporated in the strategy.

2.1.3 The proposed drainage infrastructure was described in the concept drainage design submitted as part of the DCO application. This concept design was based on data and information available at that time. The design was supported by the submission of the **Sizewell Link Road Flood Risk Assessment** (FRA) [APP-136].

2.1.4 SZC Co. has subsequently developed the concept design to preliminary design stage. SZC Co. has also developed and updated the FRA with the submission of the **Sizewell Link Road FRA Addendum** [REP2-026].

2.1.5 The purpose of this technical note is to provide details of how the concept design has been modified in response to the new data, such that it continues to provide for the effective and satisfactory drainage of Sizewell link road, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution.

2.1.6 The content of this technical note summarises the design details and approach already shared in a series of design review meetings held with key stakeholders, including the EA and SCC.

3 DESCRIPTION OF DCO DRAINAGE CONCEPT DESIGN

3.1.1 The basic proposals for watercourse crossings were tested by hydraulic modelling as part of the Sizewell link road FRA and modified to mitigate any increase in flood risk due to the construction of the Sizewell link road and its associated side roads.

3.1.2 Based on available data at that stage, the concept design for the disposal of highway runoff was by infiltration to ground. However, it was not possible to undertake geotechnical investigation to confirm actual infiltration rates at that stage.

3.1.3 The concept design provided for traditional drainage at the A12 and B1122 Middleton Link roundabouts with a combination of highway gullies and combined kerb drains (CKDs) collecting runoff and discharging via carrier drains to infiltration basins where runoff would infiltrate to ground.

3.1.4 The required size of infiltration basins required for the roundabout runoff could not be accurately determined without validated infiltration rates. As a result, they were shown schematically and sufficient space within the red line boundary was provided.

3.1.5 Elsewhere on the main line of Sizewell link road and side roads drainage would be by “over the edge” with runoff flowing from the carriageway to be collected in swales. The swales were proposed to be 1 m wide, 0.5 m deep and have side slopes of 1 in 3.

3.1.6 Given the lack of validated infiltration rates the design included for the potential need for filter trenches in the base of the swale.

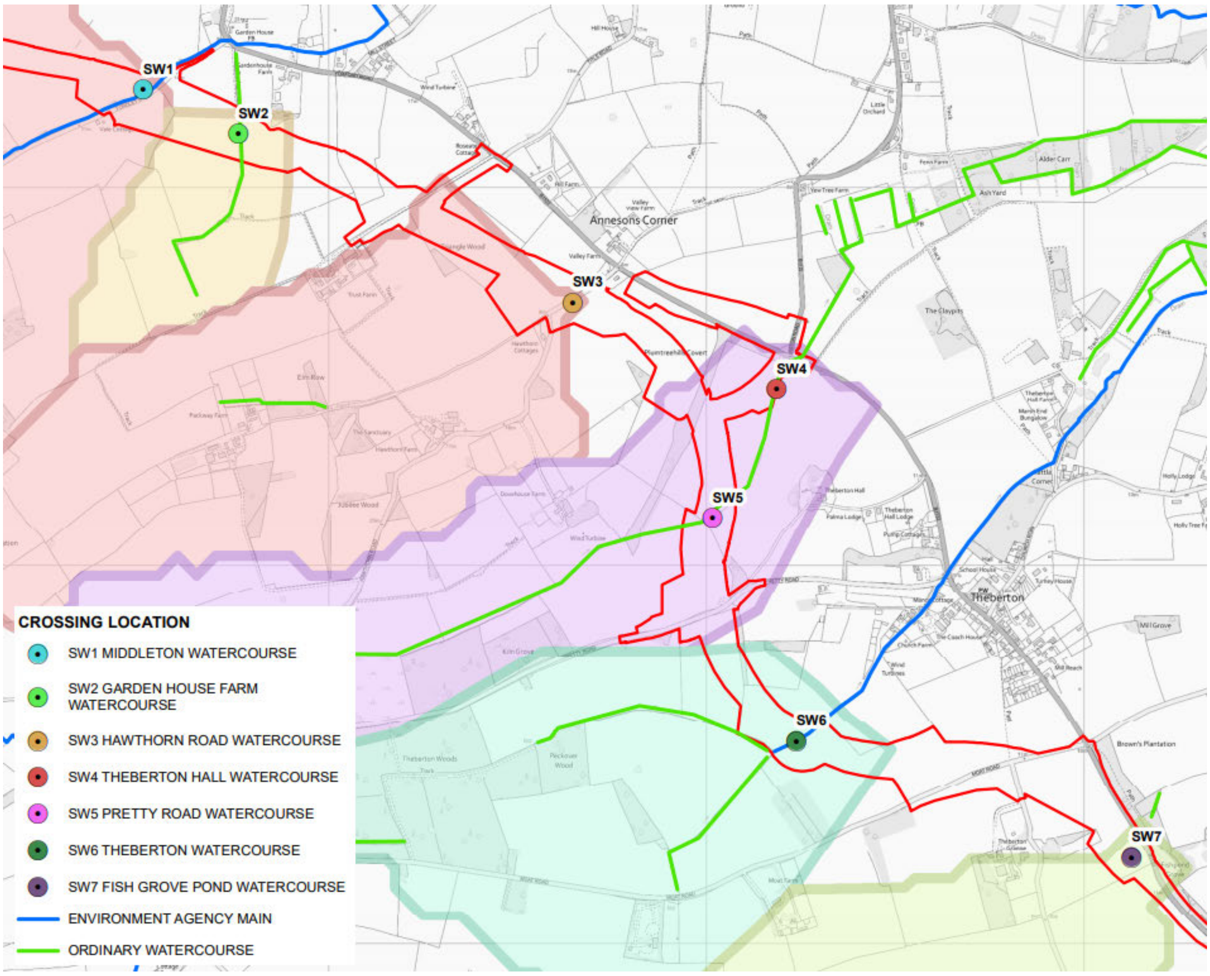
3.1.7 In addition, as back up to the swale/filter drain arrangement, a further allowance was made for 15 infiltration basins located along the line of Sizewell link road which would collect runoff not removed by the swale/filter drains.

3.1.8 Seven watercourse crossings were identified along the line of the Sizewell link road, numbered and named in the FRA as shown in **Table 1**. The location of these watercourses is shown in **Plate 2**.

Table 1: FRA referenced DCO Concept Drainage Watercourse Crossings

Crossing Number	Watercourse Number	Watercourse Name	Legal Status/Regulator
1	1	Middleton Drain	Main River EA
2	2	Garden House Farm Drain	Ordinary Watercourse SCC
3	3	Hawthorn Road Drain	Ordinary Watercourse SCC
4	5	Pretty Road Drain Leiston Road Crossing	Ordinary Watercourse SCC
5	5	Pretty Road Drain	Ordinary Watercourse SCC
6	6	Theberton Watercourse	Main River EA
7	7	Fishpond Grove Drain	Ordinary Watercourse SCC

Plate 2: Watercourse crossing locations (from Sizewell Link Road FRA [APP-136])



3.1.9 During development of concept design, it was established that there would be no change in road level or width required at crossing 4, the Pretty Road Drain beneath the B1122 near to the junction with the B1125 and link to Sizewell link road.

3.1.10 It was also determined that side road diversions for Fordley Road and Hawthorn Road would require to cross watercourse 1 – Middleton Drain and watercourse 3 Hawthorn Road Drain.

3.1.11 At its eastern end the Sizewell link road ties into the existing B1122 at the point where watercourse 7 – Fishpond Grove Drain crosses beneath the road in a 450 mm culvert. Since there was no open space between the existing road and Sizewell link road the need to extend the existing culvert rather than construct a new culvert was confirmed.

3.1.12 With the exception of watercourse 7 crossing, all other crossings were proposed to be formed by use of portal culverts. Portal culverts have no base and can be installed such that the watercourse channel and immediate banks can be left in a natural state.

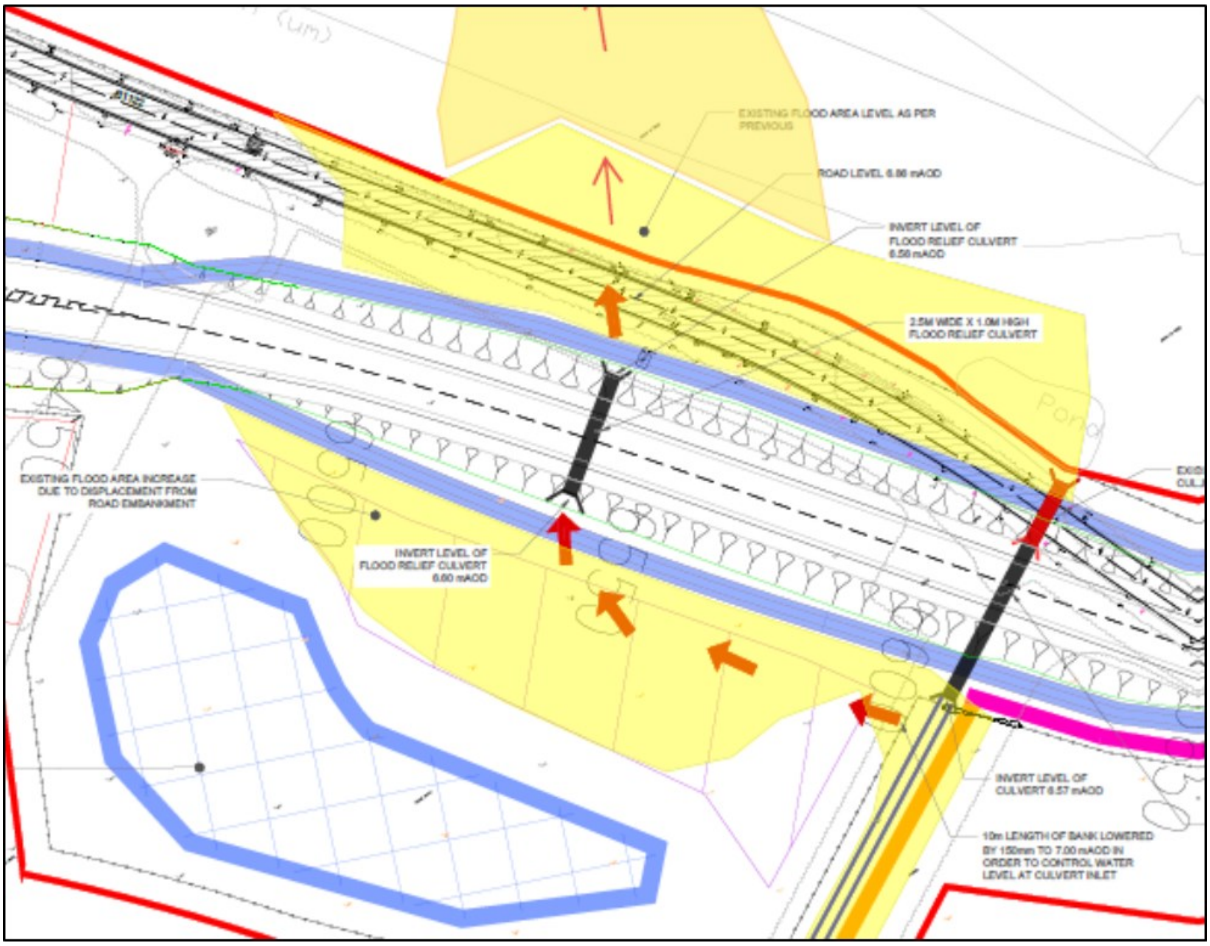
3.1.13 Portal culverts of maximum available width of 5.5 m were tested for watercourse crossings 1, 2, 3, 5 and 6 by hydraulic modelling to confirm whether their size would be sufficient to ensure no unacceptable increase in flood risk to the watercourses and any adjacent property.

3.1.14 Watercourse crossings 2, 5 and 6 were predicted to create no adverse impact but watercourse crossings 1 and 3 were predicted to cause unacceptable increase in flood levels. Since the portal culvert width could not be increased, additional adjacent flood relief box culverts were proposed. These would be 2.4 m wide and 1.0 m high.

3.1.15 In the case of watercourse crossing 1 it was not possible to install a portal culvert under the side road due to levels and so following discussion with regulators, it was proposed that the watercourse be diverted to avoid the side road thus eliminating the need for the crossing.

3.1.16 Due to lack for level data it was not possible to undertake hydraulic modelling of the watercourse crossing 7 existing baseline situation or with the extended length of culvert. However, it is apparent from the EA surface water flood map that an overland flow path routes water which overflows the bank to the west and then across the B1122. This is shown in **Plate 3**.

Plate 3: Crossing 7 overland flow path



3.1.17 The flow path is show by red arrows with predicted extent shaded yellow. Since the line of Sizewell link road would block this flow path a flood relief culvert, 2.4 m wide and 1.0 m high, was proposed to pass through the Sizewell link road in order to allow the flow path to remain.

3.1.18 Based on available desktop information it was noted that local field boundary ditches may exist at three locations. These were shown on DCO drawings as requiring possible ditch culverts.

4 ADDITIONAL INPUT DATA

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

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

- Drone topographic survey of Sizewell link road route
- Topographic survey of watercourses within and adjacent to red line boundary
- Aerial view from drone flyover
- Ground investigation and infiltration testing
- Ground penetrating radar (GPR) survey
- Additional traditional topographic survey of critical locations
- Site visit and inspection of parts of Sizewell link road route where land access was available on 13 January 2021
- Site visit and inspection of parts of Sizewell link road route where land access was available on 24 February 2021
- Sizewell Link Road FRA Addendum (Draft)
- Highways England Water Risk Assessment Tool (HEWRAT)
- Hawthorn Road side road design change

4.1.3 The site visit covering the Sizewell link road extent between the A12 and the East Suffolk railway line was undertaken jointly with SCC.

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

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

5 PRELIMINARY DRAINAGE DESIGN – HIGHWAY DRAINAGE EAST OF THE RAILWAY

5.1.1 The results of geotechnical investigation demonstrate that it is not possible to remove highway runoff by infiltration to ground. The infiltration rate data has been shared with SCC who have agreed that infiltration is not achievable. Accordingly, it is proposed that highway runoff is removed and disposed by discharge to existing watercourses.

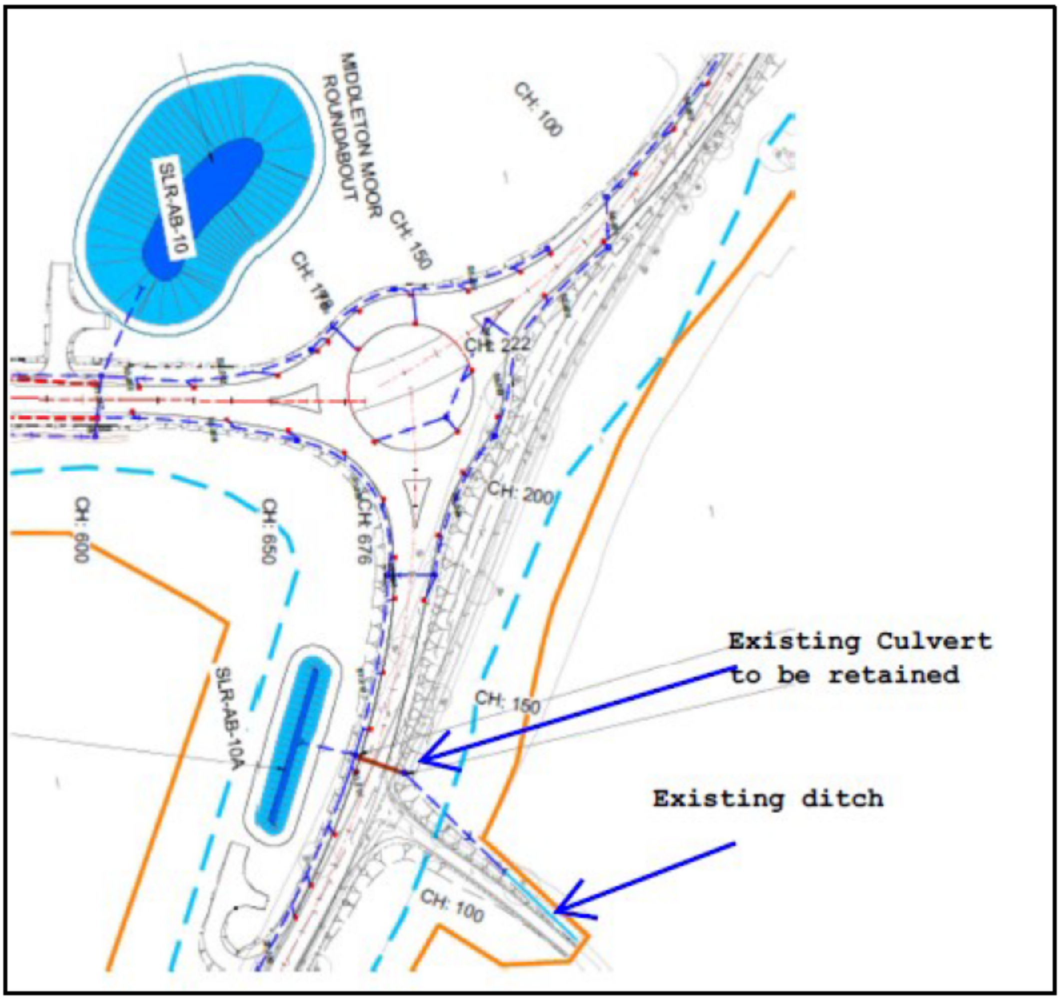
5.1.2 For the Sizewell link road and its side roads located to the east of the East Suffolk railway line there are watercourses to which discharge by gravity can be made. The watercourses are identified in **Table 1** / **Plate 2**.

5.1.3 In the case of the Middleton Link roundabout on the B1122, there is no watercourse shown on available OS based plans. However, the EA Surface Water Flood Map shows a flow path across the B1122 in proximity to the proposed roundabout. A 750 mm culvert was found at this location crossing below the B1122 road during a site visit. This culvert discharges into a deep ditch to the north which discharges into a tributary of the Minsmere River. The culvert is at a low point on the B1122 and drains the carriageway via a grip. A significant field land drainage network also discharges upstream of the culvert.

5.1.4 Given the extent of flows that currently discharge into the culvert, it is assumed that since the Sizewell link road drainage will replace the existing respective part of the B1122 runoff and the land drainage network, discharge to the existing culvert and ditch at an attenuated flow rate is acceptable.

5.1.5 The proposed drainage arrangement at the Middleton Link roundabout is shown in **Plate 4.** The proposed highway drainage for Middleton Link roundabout will remain unchanged with discharge to attenuation basin SLR-AB-10. The basin will also receive highway runoff from swales located on either side of the road, to the north of the link road crest point.

Plate 4: Middleton link roundabout drainage outfall



5.1.6 For Sizewell link road and all other side roads, the swale drainage and filter drains proposed will remain broadly as shown in DCO drawings. However, these will now provide a continuous outfall route to a watercourse.

5.1.7 Although in reality some removal of runoff will occur through limited infiltration, adsorption by vegetation and evaporation, as agreed with SCC, all drainage networks are designed and sized on the basis of no loss such that all highway runoff reaches and discharges into a watercourse.

5.1.8 In accordance with the hierarchy for disposal of highway runoff and to limit discharge so as not to increase flood risk, the discharge rate needs to be limited by flow control. This results in a requirement to provide temporary storage of runoff in the form of an attenuation basin adjacent to the watercourse.

5.1.9 In order to limit the size of these outfall attenuation basins and their inflow rate, upstream flow control points and offline attenuation basins are proposed along the line of the swales.

5.1.10 The number of basins required to attenuate the surface water runoff from the Sizewell link road will be reviewed further at the detail design stage and rationalised where possible.

5.1.11 Since the swales are no longer designed to infiltrate runoff, taking into account highway safety issues, their dimensions have been altered. When located in cutting or at level grade the depth of swale is reduced to 200 mm and the side slopes slackened to 1 in 5. This enables the requirement for the provision of vehicle restraint systems (VRS) to be avoided.

5.1.12 Swales at the toe of embankment will remain as proposed in the DCO design, at 0.5 m deep.

5.1.13 SCC guidance document “Sustainable Drainage Systems (SuDS) Appendix A” (Ref. 2) states that where discharge to watercourse is accepted, flow rate should be limited to either QBar or 2 l/s/ha whichever is the greater. However, since some Sizewell link road catchments are small this would result in a low permitted flow rate. Since the SCC guidance also requires a minimum flow control device opening of 100 mm, there is potential conflict between these requirements. Following liaison, SCC has agreed that a minimum controlled flow rate of 5 l/s could be accepted subject to evidence that it does not cause unacceptable increase in flood risk from the receiving watercourse.

5.1.14 Details of individual catchments, their gross areas and their flow rates set at 5 l/s flow rates for rainfall return periods of up to and including 1 in 100 years plus climate change are shown in Table 2 below.

Table 2: Highway discharge points to watercourse

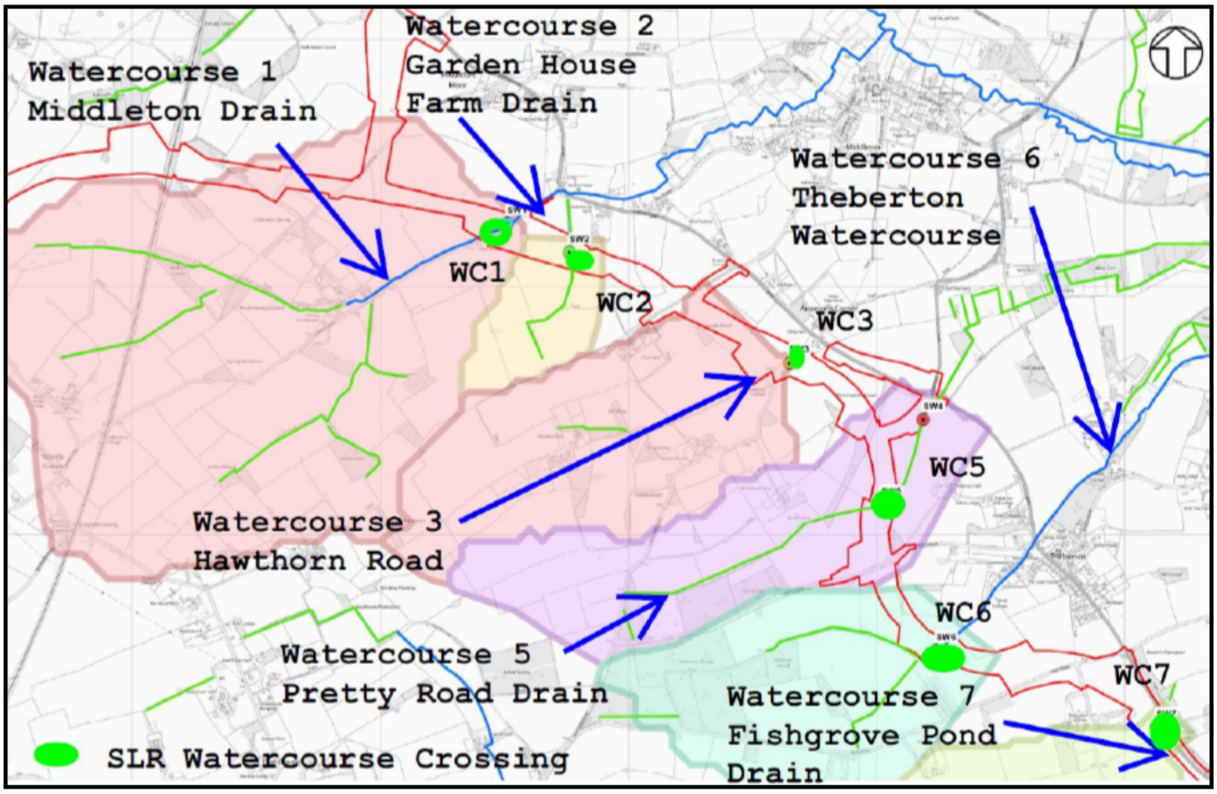
Receiving Watercourse	Outfall Basin ref	Catchment area gross (Ha)	Flow Rate @ 2 l/s/Ha	Proposed Flow Rate (l/s)
Unnamed watercourse west of A12 road	None	0.184	0.4	5.0
Unnamed watercourse north of B1122	SLR-AB-10	1.629	3.3	5.0

Middleton Moor				
Watercourse 1 Middleton Drain	AB-13 west	14.809	29.6	5.0
Watercourse 1 Middleton Drain	SLR-AB-15 east	0.947	1.9	5.0
Watercourse 2 Garden House Farm Drain	SLR-AB-16 east	3.075	6.2	5.0
Watercourse 3Hawthorn Road Drain SLR west	SLR-AB-20	2.888	5.8	5.0
Watercourse3 Hawthorn Road Drain B1122 east	SLR-AB-21	3.885	7.8	5.0
Watercourse 5 Pretty Road Drain B1125 west	SLR-AB-25	2.299	4.6	5.0
Watercourse 5 Pretty Road Drain B1125 east	SLR-AB-26	1.027	2.1	5.0
Watercourse 5 Pretty Road Drain SLR west	SLR-AB-27	0.558	1.1	5.0
Watercourse 5 Pretty Road Drain SLR east	SLR-AB-30	1.834	3.7	5.0
Watercourse 6 Theberton Watercourse west	SLR-AB-32	2.605	5.2	5.0

Watercourse 6 Theberton Watercourse east	SLR-AB-33	1.772	3.5	5.0
Watercourse 7 Fish Grove Pond Drain	SLR-AB-37	4.651	9.3	5.0

5.1.15 The catchment extents determined as part of the FRA study are shown in **Plate 5**.

Plate 5: Sizewell link road impacted local watercourse catchment extents [APP-138 Figure 4]



5.1.16 Details of flow rates derived from the 2 l/s/ha criteria are included for information. Of the 13 outfalls, 7 would be required to attenuate to less than the 5 l/s. Although 6 outfalls could discharge at greater rate with the exception of the catchment discharging from the west into Middleton Drain, the difference is limited. Accordingly, the design provides for a controlled flow rate of 5 l/s at each outfall.

5.1.17 In the case of the Middleton Drain catchment west, it is noted that the catchment includes for a pumped discharge for the Sizewell link road A12 roundabout and mainline across the railway at a flow rate of 5 l/s and thus the calculated 2 l/s/ha flow rate of 29.6 l/s is not applicable. As described in Section 7 below, subject to further investigation the transfer of flow from the west of the railway may not be taken forward to detailed design if a subsequent gravity solution can be substantiated.

5.1.18 The impact of discharging each catchment into its respective watercourse has been accessed using the FRA Addendum individual watercourse hydraulic models, including for watercourse 7 (not previously modelled as part of the Sizewell link road FRA). As confirmed in the Addendum report there is no adverse impact of these flows on the watercourse's capacity nor unacceptable increase in flood risk.

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

5.1.20 To undertake an assessment of the hydraulic or environmental impact of discharge into the deep ditch north of the B1122 at Middleton Link further survey work would be required during the detailed design stage.

6 PRELIMINARY DRAINAGE DESIGN – HIGHWAY DRAINAGE WEST OF THE RAILWAY

6.1.1 Desktop study identified no watercourse crossings by the Sizewell link road between the A12 roundabout and the East Suffolk railway line. As a result, the FRA does not include any hydraulic models for watercourse crossings between the A12 roundabout and East Suffolk railway line. The FRA does confirm that this area forms part of the Middleton Drain catchment.

6.1.2 Following confirmation that infiltration is not possible the following options for removal of highway runoff have been considered:

- Swales and filter drains with additional oversized basins to contain the 1 in 100 year return period runoff volume plus follow on rainfall events

- Discharge to remote watercourses shown on OS maps either by gravity or pumping
- Discharge over the East Suffolk railway line to the catchment discharging to Middleton Drain
- Discharge to ground via deep boreholes into permeable strata

6.1.3 The options have been discussed with SCC as Highway Authority and LLFA.

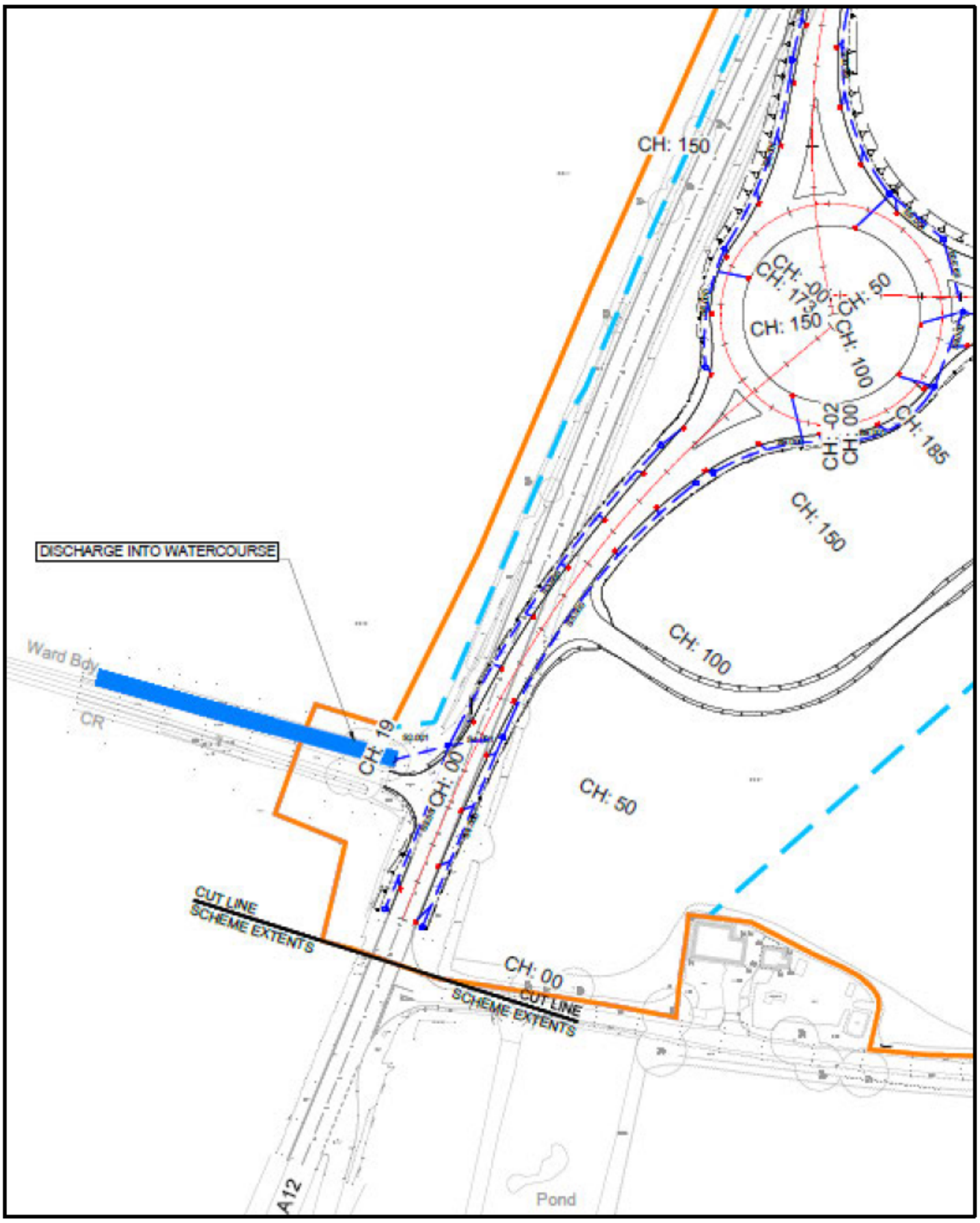
6.1.4 After consideration of the alternatives SCC has advised that the provision of oversized basins is not acceptable and that all drainage networks must have a positive outfall, to limit flood risk.

6.1.5 Following the joint SCC/WSP site visit on 13 January 2021 it is agreed that discharge west of the railway to the remote watercourses (within the red line boundary) to the north or south by gravity is not achievable. Since the alternative would be pumping, it is considered to be less disruptive to pump to the east of the railway and into the Middleton Drain west catchment compared to pumping to the remote watercourses.

6.1.6 SCC guidance document "Sustainable Drainage Systems (SuDS) Appendix A" states that deep borehole soakaways with depth greater than 2 m are considered not viable and will only be accepted as a last resort. Based on available geotechnical information the required depth of boreholes would be around 13 m and with uncertainty that suitable infiltration rates can be achieved this option has been discounted.

6.1.7 During the site visit and subsequently confirmed by the GPR survey, a small section of the existing A12 to the south of the roundabout was found to discharge to a deep ditch to the west of the road. It is proposed that the southern arm of the roundabout will be drained to this ditch using the same outfall point. The arrangement is shown in **Plate 6**. As shown in Table 2, the discharge rate will be controlled to 5 l/s.

Plate 6: Proposed highway drainage A12 roundabout southern arm



6.1.8 Based on the above considerations, the drainage of the A12 roundabout, its northern arm and the first portion of Sizewell link road arm will remain as a combination of highway gullies and combined kerb drains (CKDs) collecting runoff and discharging via carrier drains but the receiving infiltration basin changes to an attenuation basin SLR-AB-01.

6.1.9 In order to provide basin SLR-AB-01 an outfall as required by SCC, a small pumping station will be provided. This will pump at a rate of 5 l/s via a rising main which will discharge to a gravity network to the east of the Sizewell link road crest point at chainage 488 m.

6.1.10 The length of Sizewell link road from the A12 to chainage 488 m will drain in the same way as for the east of the railway with swale/filter drains and 2 supporting attenuation basins.

6.1.11 The length of Sizewell link road from chainage 488 m to the East Suffolk railway bridge will drain via swale/filter drains and discharge into a single attenuation basin SLR-AB-05 at chainage 1200 m. The attenuation basin SLR-AB-05 will discharge into a pumping station which will pump at a rate of 5 l/s via a rising main discharging into a gravity network to the east of the railway bridge and ultimately to Middleton Drain.

6.1.12 The rising main will be attached to the bridge structure when crossing the railway.

6.1.13 The Sizewell link road will have longfall and crossfall gradients as it crosses the railway bridge from east to west. Deck CKDs will be provided to remove runoff and they will outfall into the swales to the west.

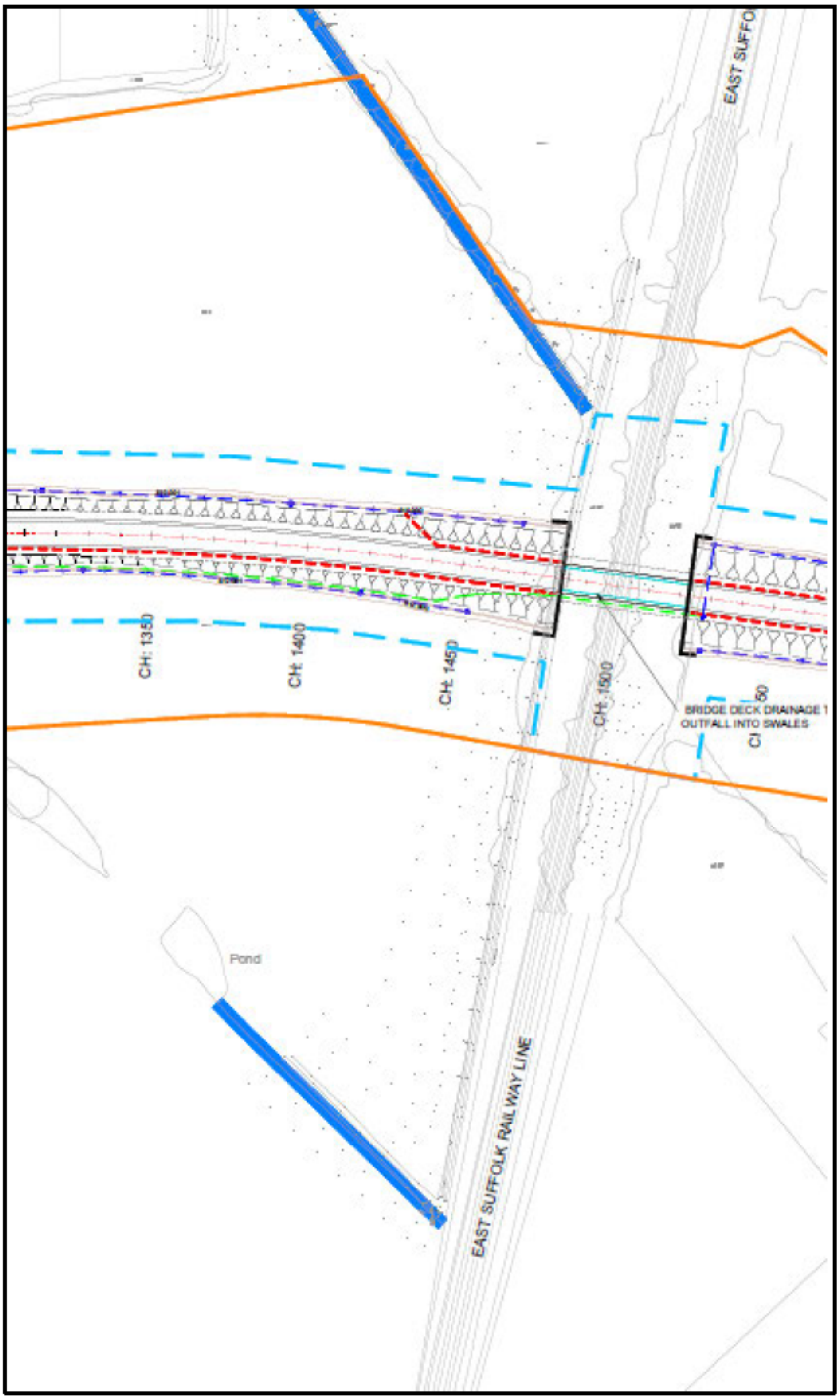
6.1.14 During the site visit the presence of two field boundary ditches was confirmed. These ditches are heavily silted and in poor state of maintenance. It is not clear as to whether they drain effectively or if they have a formal outfall. However, as confirmed with SCC where such ditches exist, they will be culverted beneath Sizewell link road such that their function remains intact. The ditches LDC1 and LDC2 are located at chainage 250 m and 750 m.

7 PRELIMINARY DRAINAGE DESIGN – WEST OF THE RAILWAY (GRAVITY OPTION ALTERNATIVE)

7.1.1 During the site visit on 13 January 2021, the presence of a shallow local ditch was discovered approximately 40 m to the north of Sizewell link road, adjacent to the East Suffolk railway boundary. This ditch follows a field boundary running north west and is shown blue in **Plate 7**. It is in a state of dereliction and would not be suitable as an outfall for Sizewell link road drainage. However, it does discharge into a watercourse, not shown on OS mapping, approximately 250 m north of Sizewell link road which runs north and which subject to confirmation of levels could be a suitable gravity outfall.

7.1.2 Similarly, to the south of the proposed Sizewell link road route a deep excavated field boundary ditch was discovered, approximately 200 m south, continuing in a southeast direction to a headwall with a 600mm outfall pipe. This ditch is also shown blue in **Plate 7**. Unverified plans provided by the landowner, show the 600mm pipe continues southwards within the railway at the base of the cutting before discharging into the Middleton Drain.

Plate 7: Local watercourses to north and south of Sizewell link road adjacent to East Suffolk Railway



7.1.3 Based on site observation it appeared that there are general falls in land from Sizewell link road towards these watercourses. It was agreed that subject to confirmation of ground levels that it may be possible to connect to either the watercourse or ditch and obtain a gravity outfall.

7.1.4 As can be seen in **Plate 7** both the watercourses and ditch are located outside of the DCO boundary limits shown orange. Although connection to the watercourses would not be within the DCO limits, in accordance with the SCC required hierarchical approach, before SCC acceptance of a pumped outfall, it is to be established first whether a gravity connection is achievable.

7.1.5 A topographic survey of the area along the potential outfall route has been undertaken. This confirms that the bed level of the watercourse to the north is at a level of 33.66 mAOD. The invert level of the 750 mm outfall drain to the south is at a level of 34.84 mAOD. Sizewell link road is on an embankment above existing ground level which is at level of 37.8 mAOD in proximity to the railway boundary. As observed during the site visit these levels do confirm a fall in ground level both to the north and to the south.

7.1.6 Given that infiltration of highway runoff to ground is not possible, it is necessary for the filter drain pipe to have a falling gradient in order to remove runoff to an outfall point for removal, whether by gravity or pumping. The distance from the crest point at chainage 488 m to the railway boundary at chainage 1480 m is 992 m. Attenuation basin SLR-AB-05 which receives runoff from this section of Sizewell link road is located at chainage 1200 and collects runoff from both east and west. It is kept clear of the railway which is in cutting.

7.1.7 In order to provide the required continuous falling gradient, the basin bed level is fixed at 36.329 mAOD.

7.1.8 If an outfall is provided to the northern watercourse following Sizewell link road and field boundaries, its distance would be approximately 510 m. Given a need for a falling gradient and for the drain to have self-cleansing velocity a gradient of 1 in 150 is assumed. This would result in a fall of 3.4 m with an outfall level of 32.929 mAOD approximately 0.73 m lower than the watercourse.

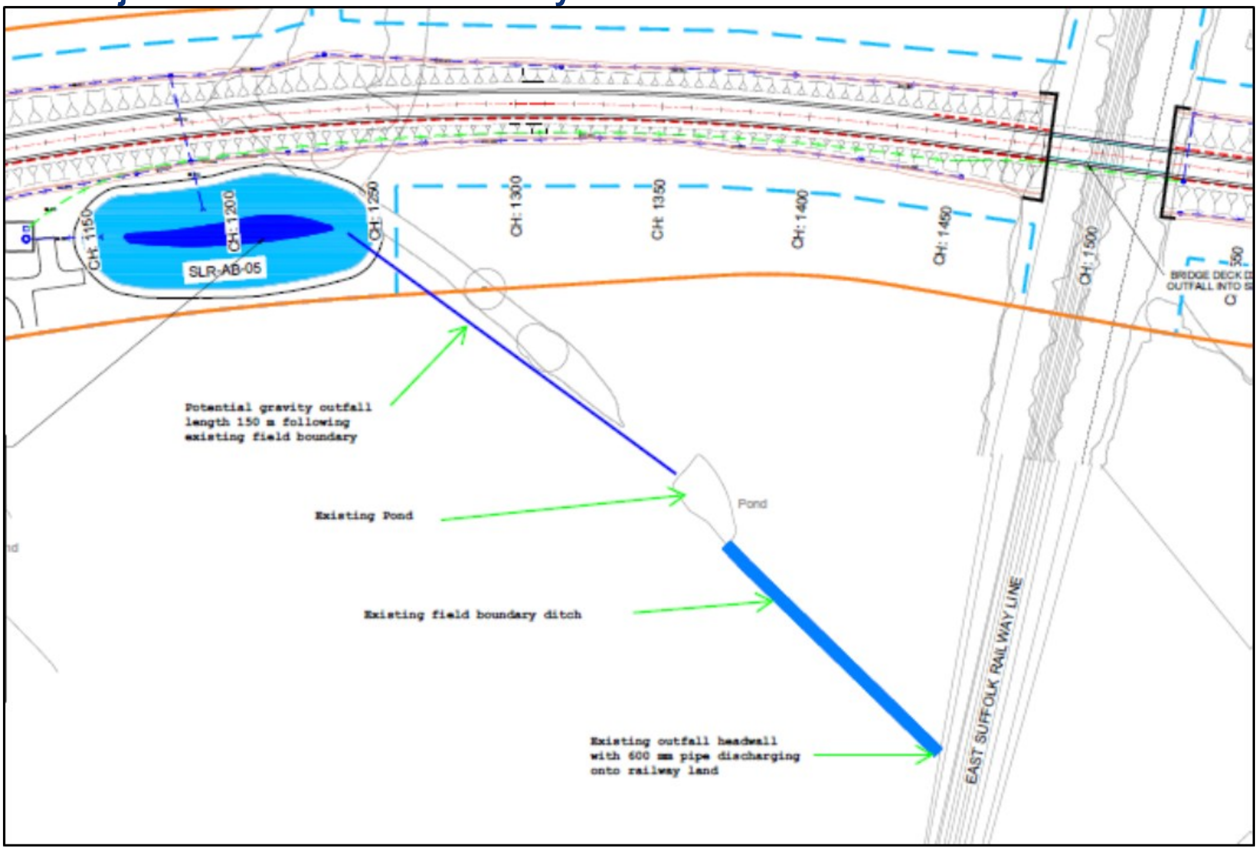
7.1.9 If an outfall is provided directly from the basin to the northern watercourse, the distance reduces to 300 m. This would result in an outfall level of 34.329 mAOD which is 0.67 m above the watercourse.

7.1.10 If an outfall is provided to the southern ditch following Sizewell link road and field boundaries, its distance would be approximately 460 m. This would result in a fall of 3.4 m with an outfall level of 33.262 mAOD approximately 1.58 m lower than the ditch.

7.1.11 If an outfall is provided directly from the basin to the southern ditch outfall, the distance reduces to 300 m. This would result in an outfall level of 34.329 mAOD which is 0.51 m below the ditch.

7.1.12 However, in reviewing a direct route, it is noted that the southern outfall would be able to discharge into an existing pond rather than extend to the 600 mm outfall drain. With discharge into the pond the invert level at outfall would be 35.329 mAOD, approximately 0.49 m above the outfall drain. The pond is at the upstream end of the ditch. This arrangement is shown in **Plate 8.**

Plate 8: Proposed gravity outfall to local pond and ditch south of Sizewell link road adjacent to East Suffolk Railway.



7.1.13 Of all of the alternatives, discharge via an outfall from the basin to the existing pond would have the minimum impact on land use since it would follow a field boundary and be the shortest distance. Whilst no levels are available for the pond, it was noted to be fairly deep during the site visit.

7.1.14 During liaison, SCC has confirmed that subject to the Sizewell link road being adopted as public highway, they would if necessary, accept the use of pumping stations provided that it can be demonstrated that no practical gravity solution is available. On the basis that all attenuation basis and highway networks must have an outfall, it is necessary to provide a pumping station at the A12 roundabout from basin SLR-AB-01. However, if an outfall can be provided from attenuation basin SLR-AB-05 discharging via the pond and ditch, this would eliminate the need for a second pumping station and rising main across the railway discharging into the Middleton Drain west catchment.

7.1.15 SCC has advised that for any discharge to watercourse to be accepted, it is necessary to demonstrate that:

- the proposed rate of discharge to the watercourse will not result in increased flood risk
- the proposed discharge will not adversely impact on water quality
- there is continuity of downstream discharge route providing a guaranteed outfall

7.1.16 On the basis of currently available data, it is believed that that the SCC stated requirements can be met, whilst acknowledging that further data will be required to confirm this position.

7.1.17 The preliminary design solution proposed was completed subsequent to the finalisation of the **Sizewell Link Road FRA Addendum** [REP2-026]. The flood risk impact of the proposed discharge rate at this location requires to be confirmed formally in a future update to the FRA, but sensitivity testing in the hydraulic model has demonstrated that there is no resulting impact on properties.

7.1.18 Accordingly, whilst continuing to propose a drainage solution with a pumping station for the catchment to the west of the railway to ensure a reliable solution for the DCO application, with discharge to the east of the railway, the solutions to connect to the local watercourses will continue to be progressed through the design stages following further surveys such that if it can be demonstrated a gravity outfall option can be substantiated it will be substituted in preference of a pumped solution in accordance with the hierarchical approach required by SCC.

7.1.19 The proposed outfall is not located within the DCO red line boundary. Accordingly, SZC Co. has submitted a DCO red line boundary amendment into consultation which incorporates the land required for this outfall.

8 VALIDATION OF OUTLINE DRAINAGE STRATEGY

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

8.1.2 The approach in the **Outline Drainage Strategy** [REP2-033] is validated by the completed preliminary design, which has demonstrated that infiltration is not applicable and proposes the attenuated discharge of water to watercourses.

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

9 SUMMARY AND CONCLUSION

9.1.1 The purpose of this technical note is to provide details of how the design has needed to evolve and develop as a result of provision of new information. The primary cause of change has been that following the completion of ground investigation works, the assumption that the primary means of removal and disposal of highway surface water runoff by infiltration to ground has been discounted. The alternative of discharge at controlled rates to watercourse, as an alternative has been discussed with SCC and the EA in liaison and through design review meetings.

9.1.2 The primary reason for updating the original DCO submitted FRA was to address the concerns expressed by the EA with regard to the extent of hydraulic modelling of the Sizewell link road culvert crossings. The extent and accuracy of modelling has been enhanced following the obtainment of watercourse surveys both upstream and downstream of crossing points.

9.1.3 Following the change with discharge of highway runoff to watercourse at a flow rate of 5 l/s per outfall, the FRA hydraulic modelling was used to test and demonstrate the insignificant impact of these direct flows on flood risk in the watercourses.

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

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

REFERENCES

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2. Sustainable Drainage Systems (SuDS) a Local Design Guide Appendix A to the Suffolk Flood Risk Management Strategy, Suffolk County Council, May 2018 https://www.greensuffolk.org/app/uploads/2021/05/2018-10-01-SFRMS-SuDS-Guidance-Appendix-A-.pdf

3. Highways Agency et al. (2009). Volume 11, Section 3, Part 10: Road Drainage and the Water Environment, HD45/09. http://www.standardsforhighways.co.uk/ha/standards/dmrb/vol11/section3/hd4509.pdf

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

ANNEX 2A.10: YOXFORD ROUNDABOUT UPDATED DRAINAGE STRATEGY

CONTENTS

TABLES

None provided.

PLATES

FIGURES

None provided.

APPENDICES

None provided.

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 undertaken work to validate and develop the design of the Yoxford roundabout that was originally submitted as part of the DCO application. This document forms one of a series of design validation and evolution documents being provided to the Examining Authority in support of the **Outline Drainage Strategy** [REP2-033].

1.1.3 The Yoxford roundabout forms one of the Associated Developments (AD) which are required to mitigate traffic impacts arising from the main development site. The Yoxford roundabout consists of a new three arm roundabout, which includes the realignment of the existing A12 and B1122 Middleton Road, and the removal of the existing A12 and B1122 ghost island junction.

1.1.4 Yoxford roundabout modifies the existing public highway and as such will continue to form part of the highway network maintained by Suffolk County Council (SCC). As a result, it will require to be designed to meet SCC adoptable standards (Ref. 1).

1.1.5 Yoxford roundabout highway modifications will continue to generate surface water highway runoff which will require to be removed, treated as necessary and disposed.

2 PURPOSE

2.1.1 The **Outline Drainage Strategy** [REP2-033] identified at concept level the proposed drainage approach for the effective removal of runoff from the proposed Yoxford roundabout highway and its disposal.

2.1.2 This drainage strategy was developed in consultation with drainage regulators and local authorities, including SCC and the Environment Agency (EA). A number of workshops were held and the observations/requirements of drainage regulators were incorporated in the strategy.

2.1.3 The proposed drainage infrastructure was included in the concept design submitted as part of the DCO application. This concept design was based on data and information available at that time.

2.1.4 Following the provision of new data, and subsequent to the DCO submission, SZC Co. has developed the concept level design to preliminary design stage.

2.1.5 The purpose of this technical note is to provide details of how the concept design has been modified in response to the new data, such that it continues to provide for the effective and satisfactory drainage of Yoxford roundabout, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution.

2.1.6 The content of this technical note summarises the design details and approach already shared in a series of design review meetings held with key stakeholders, including the EA and SCC.

3 DESCRIPTION OF DCO DRAINAGE DESIGN STRATEGY

3.1.1 Based on available data at that stage, the concept design for the disposal of highway runoff was by infiltration to ground. However, it was not possible to undertake geotechnical investigation to confirm actual infiltration rates at that stage.

3.1.2 The drainage strategy provided for traditional drainage at the A12/B1122 roundabout and its east, north and southwest arms with a combination of highway gullies and combined kerb drains (CKDs) collecting runoff and discharging via carrier drains to an infiltration basin where runoff would infiltrate to ground.

3.1.3 The required size of the infiltration basin required for the roundabout runoff could not be accurately determined without validated infiltration rates. As a result, it was shown schematically and sufficient space within the red line boundary was provided. The position of the infiltration basin is to the south of the roundabout as shown in **Plate 1**, shaded blue. It is intended that the basin be integrated within a landscaped area shaded green.

Plate 1: Proposed location of infiltration basin at concept design stage

3.1.4 At concept design stage it was not clear as to whether the full extent of the modified highway B1122 Eastern Arm and A12 Northern Arm would be able to drain by gravity to the infiltration basin. However, the extent of carriageway discharging to the north and east would be reduced.

3.1.5 Similarly, and in accordance with SCC aspirations, the extent of carriageway draining via the highway drainage network into Yoxford would be substantially reduced.

4 ADDITIONAL INPUT DATA

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

4.1.2 The new data which informs the design is listed below

- Drone topographic survey
- Aerial view from drone flyover
- Ground investigation and infiltration testing
- Ground penetrating radar (GPR) survey
- Additional traditional topographic survey of critical locations
- Site visits and inspection of Yoxford roundabout on 12 January 2021 and 24 February 2021
- Highways England Water Risk Assessment Tool (HEWRAT) (Ref. 2)

4.1.3 The design development has also evolved through the design review meetings held with SCC and between SZC Co. and SCC's Lead Local Flood Authority (LLFA) officer. Comments and requirements confirmed by SCC have been recorded in minutes of the review meetings and taken into account.

4.1.4 The final draft preliminary design has been submitted to SCC as the Highway Authority, to SCC as LLFA, and the EA. Any final comments can be addressed in the preliminary design drawings and reports, prior to issue as final design.

5 EXISTING HIGHWAY DRAINAGE ARRANGEMENTS

5.1.1 Since Yoxford roundabout replaces the existing T-junction of the A12 and B1122 highways and ties back into them to the east, north and southwest, it is necessary to take account of existing highway drainage and catchment areas in the design of drainage required for Yoxford roundabout.

5.1.2 The existing highway drainage is owned and maintained by SCC as Highway Authority. Liaison has taken place with SCC and although there are currently no formal plans showing the existing highway drainage infrastructure, SCC has been able to describe the drainage from the junction of the A12/B1122 into Yoxford village and separately the B1122 to the east. The accuracy of these descriptions has been validated by site inspection.

5.1.3 During liaison SCC confirmed that the downstream drainage network in Yoxford has no spare capacity and there are existing flooding problems. As a result, no additional highway paved area can be connected to the highway drainage network that outfalls into Yoxford.

5.1.4 SCC has no knowledge of any flooding issues with the local highway drainage network which drains the B1122 to the east of Yoxford and outfalls via the track leading to the sewage treatment works.

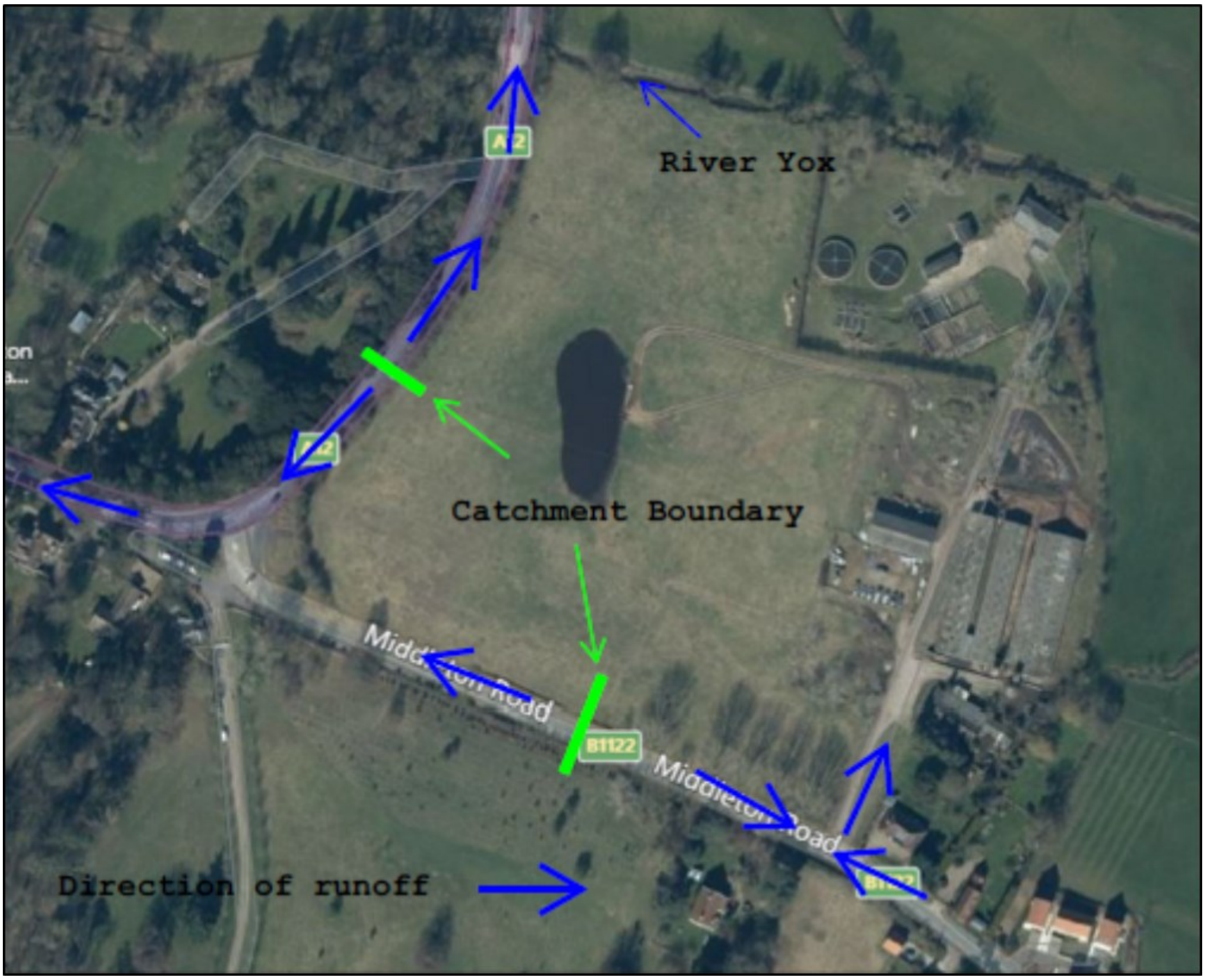
5.1.5 SCC was not able to confirm details of highway drainage on the A12 to the north of the road high point which falls towards the River Yox. Following site inspection, the presence of 2 gullies was noted on the northbound carriageway next to the Satis House entrance. However, there is no evidence of any highway drain or outfall into the River Yox for either northbound or southbound carriageways. Highway runoff is removed by overland flow across the River Yox bridge and then by a series of gullies which appear to outfall into drainage ditches which in turn discharge into the River Yox.

5.1.6 In summary it can be confirmed that there are 3 existing catchments within the extent of the Yoxford roundabout scheme:

- A12/B1122 outfalling southwest into Yoxford
- A12 outfalling north to the River Yox
- B1122 outfalling north

5.1.7 The extent of these catchments is shown in **Plate 2**.

Plate 2: Existing highway layout showing catchment boundaries and overland flow paths



6 PRELIMINARY DRAINAGE DESIGN

6.1.1 The results of Geotechnical Investigation with infiltration rate testing at the site of the infiltration basin demonstrate that it is possible to remove highway runoff by infiltration to ground. The infiltration rate data has been shared with SCC who has agreed that infiltration is achievable. Accordingly, it is proposed that highway runoff is removed and disposed by infiltration to ground to the extent possible.

6.1.2 In order to drain the maximum area into the infiltration basin located south of the roundabout the base level for the basin has been set at 10.5 mAOD. Side slopes for the basin have been set at 1 in 4 which is in accordance with SCC guidance. Hydraulic modelling demonstrates that the basin does fit within available space as shown in **Plate 3** with a maximum depth of water predicted to be 0.857 m during a 1 in 100 year return period rainfall event plus 40% allowance for climate change. The calculated maximum depth of attenuated water is on the basis that the base of the basin is lined and that infiltration is through the basin sides. This arrangement is required following completion of the HEWRAT assessment.

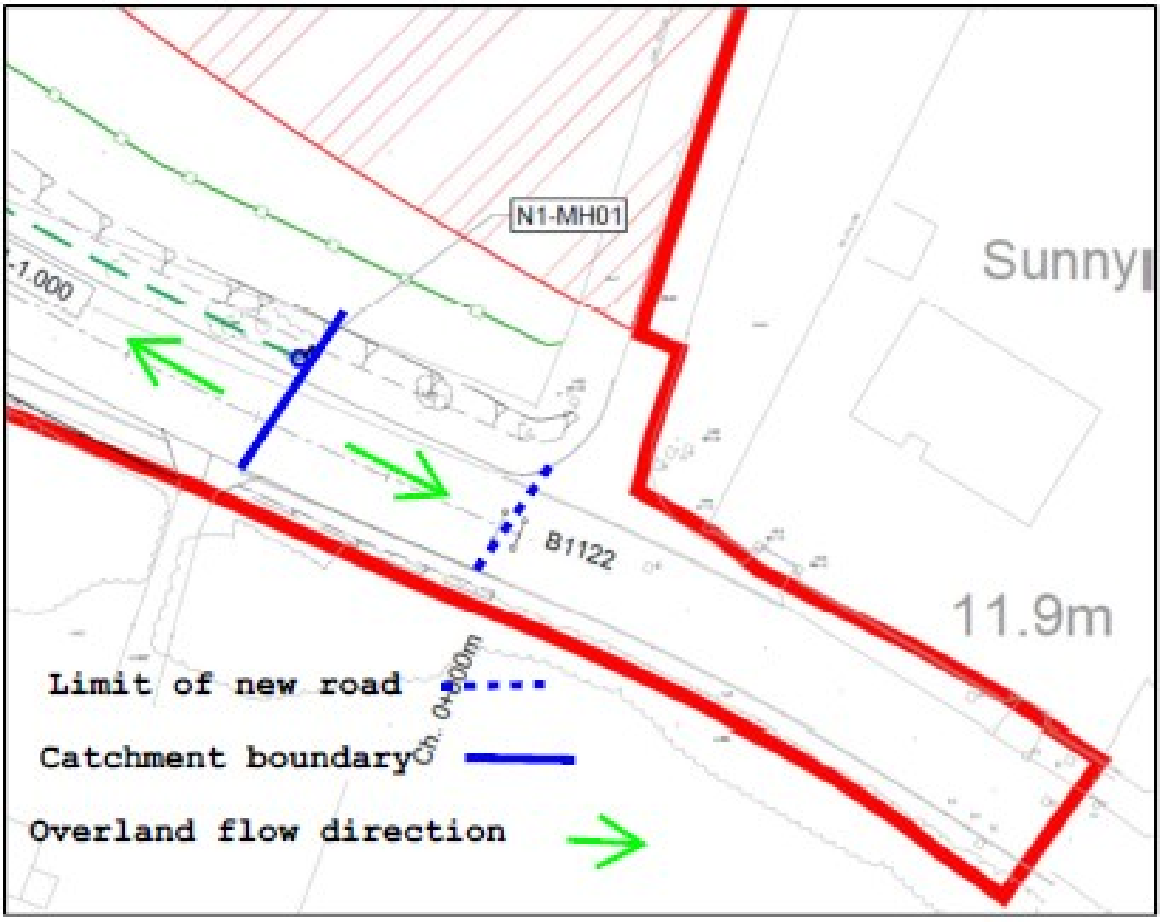
Plate 3: Infiltration basin footprint

6.1.3 The DCO drainage strategy allowed for removal of surface water highway runoff by use of gullies and combined kerb drains (CKDs). This strategy continues to apply for the roundabout and A12 southern arm. However, as part of design development, clear of the roundabout the southbound carriageway of the A12 northern arm and the eastbound carriageway of the

B1122 will discharge “over the edge” and into filter drains. The filter drain arrangement will also ensure that any runoff from the cutting adjacent the road is collected and does not flow onto the road.

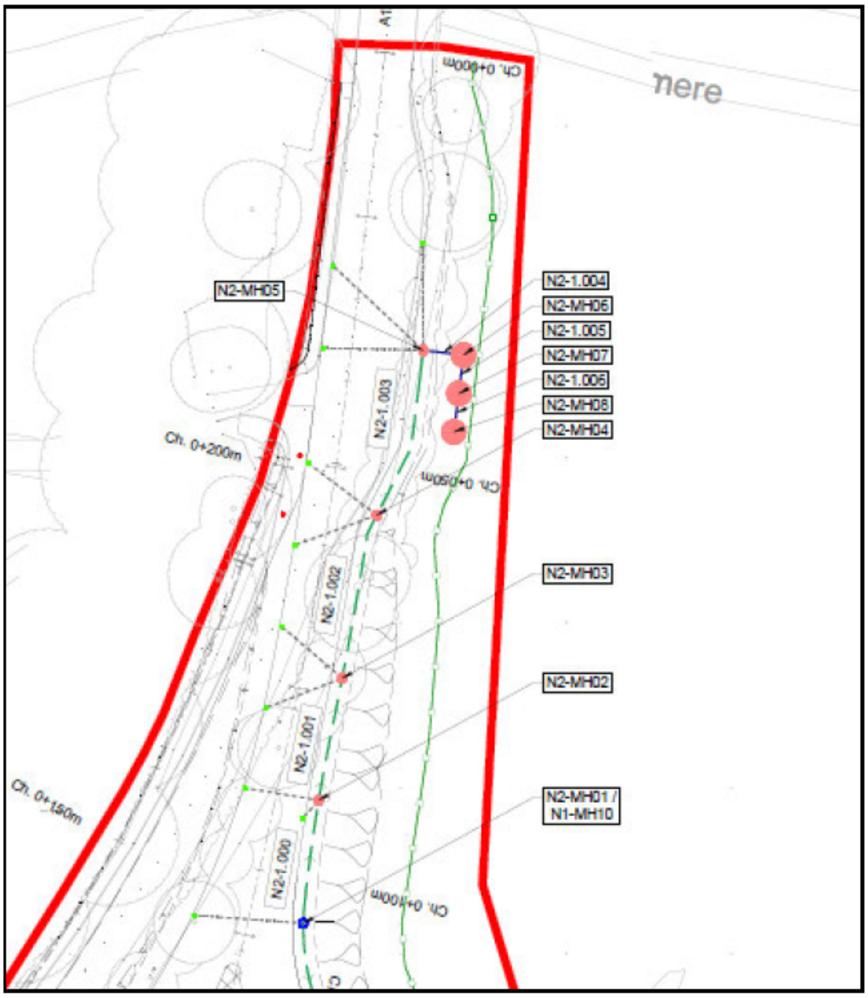
6.1.4 A short 20 m section of the B1122 at the tie into existing road is not able to drain back against the road longfall and to the infiltration basin. The existing road has no formal drainage and overland flow follows the falling gradient to the east of the junction with the sewage treatment works access track at which point it is collected in existing gullies. The area is shown in **Plate 4** and as can be seen by reference to **Plate 2** the length of road discharging runoff by overland flow is reduced by approximately 80 m. Given this reduction and no evidence of flooding issues, it is not intended to provide any new formal surface drainage for removal of runoff on this section.

Plate 4: 20 m length of B1122 with no formal drainage



6.1.5 Most of the northern arm of the roundabout cannot drain back against the road longfall and to the infiltration basin by gravity. The length of this section of the A12 is approximately 100 m to the crossing of the River Yox. Given its length, road modification, and long overland flow path to the north of the river, a local drainage network is proposed, separate to the infiltration basin catchment. As the northbound carriageway has a footpath and kerb, a series of gullies are proposed. The proposed gullies discharge across the road to a filter drain/carrier drain which in turn discharges north to proposed soakaways at the rear of the layby adjacent to the River Yox bridge. The southbound carriageway drains "over the edge" into the filter drain which also collects runoff from the adjacent cutting side. The arrangement is shown in **Plate 5** with the proposed soakaways shown by the three larger red circles.

Plate 5: A12/B1122 roundabout northern arm drainage



6.1.6 The HEWRAT assessment was completed following the design of northern arm with soakaway outfall. This indicated that due to assumed groundwater levels there would be a medium risk of pollution to groundwaters due to the short transit time for highway runoff to percolate through the strata. In the case of the main infiltration basin this issue can be addressed by installing an impermeable base such that infiltration takes place through the sides of the basin, increasing transit time. An impermeable base soakaway option is not available for the northern arm soakaways.

6.1.7 The northern arm drainage strategy was discussed with SCC and draining the northern arm with discharge to the River Yox has been proposed as an alternative. The location of the required discharge point was inspected at the second site visit confirming the feasibility of an outfall route to the rear of the bridge parapet wall. SCC has confirmed that given the relative pollution risk to groundwater and the fact that existing highway runoff does discharge indirectly to the river, a discharge to the river from northern arm will be acceptable, subject to provision of effective treatment trains. However, since the River Yox is classified as main river the discharge and outfall headwall will require to be consented by the EA. Given that this section of A12 currently drains indirectly via a local ditch to the River Yox, with no provision of treatment, the proposed outfall will provide an improvement to the existing situation.

6.1.8 It is agreed that the design of the northern arm drainage outfall will be changed from soakaway to discharge to the River Yox at detailed design stage.

7 VALIDATION OF OUTLINE DRAINAGE STRATEGY

7.1.1 In accordance with the drainage hierarchy, the **Outline Drainage Strategy** [REP2-033] proposed the primary use of infiltration, with additional use of attenuation techniques (e.g. ponds and swales) to manage water quality and to further promote infiltration, and disposal to watercourse where necessary. The **Outline Drainage Strategy** is validated by the completed preliminary design.

7.1.2 The preliminary design documents will be made available for review and acceptance by SCC and the EA with respect to obtaining SCC consent to construct the road works and EA consent to discharge to main river.

8 SUMMARY AND CONCLUSION

8.1.1 The purpose of this technical note is to provide details of how the concept design has needed to evolve and develop as a result of provision of new information. The only significant change is that the A12 northern arm between Yoxford roundabout and the River Yox bridge will now discharge to the River Yox instead of to ground by infiltration.

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

8.1.3 At this preliminary design stage, it is considered that the design provides for the effective removal, treatment and disposal of highway runoff. It reduces current level of flood risk within Yoxford and to the B1122 to the east providing a flood risk legacy benefit. The flood risk performance of the highway is as specified in DMRB and SCC guidance.

REFERENCES

1. Design Manual for Roads and Bridges and Manual of Contract Documents for Highway Works Series 500 Highways England 2000, http://www.standardsforhighways.co.uk/ha/standards/dmrb

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

3. Sustainable Drainage Systems (SuDS) a Local Design Guide Appendix A to the Suffolk Flood Risk Management Strategy, Suffolk County Council, May 2018 https://www.greensuffolk.org/app/uploads/2021/05/2018-10-01-SFRMS-SuDS-Guidance-Appendix-A-.pdf

4. Highways Agency et al. (2009). Volume 11, Section 3, Part 10: Road Drainage and the Water Environment, LA 113 http://www.standardsforhighways.co.uk/ha/standards/dmrb/vol11/section3/LA 113.pdf

ANNEX 2A.11: TWO VILLAGE BYPASS PRELIMINARY DRAINAGE DESIGN NOTE

CONTENTS

TABLES

None provided.

PLATES

FIGURES

None provided.

APPENDICES

None provided.

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 undertaken work to validate and develop the design of the Sizewell link road that was originally submitted as part of the DCO application. This document forms one of a series of design validation and evolution documents being provided to the Examining Authority in support of the **Outline Drainage Strategy** [REP2-033].

1.1.3 The two village bypass forms one of the Associated Developments (AD) which are required to mitigate traffic impacts arising from the main development site. The two village bypass consists of a new 2.4 km long single carriageway road bypassing the villages of Stratford St Andrew and Farnham. The new bypass will connect to the existing A12 via at grade roundabouts at both the western and eastern ends of the scheme. The roundabout at the western end ties in with the existing A12 Main Road and the roundabout at the eastern end ties in with Friday Street.

1.1.4 The two village bypass will be designed to Suffolk County Council's (SCC) adoptable standards (Ref. 1).

1.1.5 The two village bypass will generate highway surface water runoff which will require to be removed, treated as necessary and disposed by infiltration to ground.

1.1.6 The two village bypass will cross the River Alde, its associated flood plain and two minor watercourses which need to be accommodated.

2 PURPOSE

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

- The effective removal of runoff from the proposed two village bypass highway and its disposal
- The crossing of the River Alde and its associated floodplain and watercourses along the line of the two village bypass

2.1.2 This strategy was developed in consultation with drainage regulators and local authorities, including SCC and the Environment Agency (EA). A number of workshops were held and the observations/requirements of drainage regulators were incorporated in the strategy.

2.1.3 The proposed drainage infrastructure was described in the concept drainage design submitted as part of the DCO application. This concept design was based on data and information available at that time. The design was supported by the submission of the **Two Village Bypass Flood Risk Assessment** (FRA) [APP-119].

2.1.4 Following the provision of new data, and subsequent to the DCO submission, SZC Co. has developed the concept level design to preliminary design stage.

2.1.5 The purpose of this technical note is to provide details of how the concept design has been modified in response to the new data, such that it continues to provide for the effective and satisfactory drainage of the two village bypass, without unacceptable adverse impact on the water environment, both in terms of flood risk and pollution.

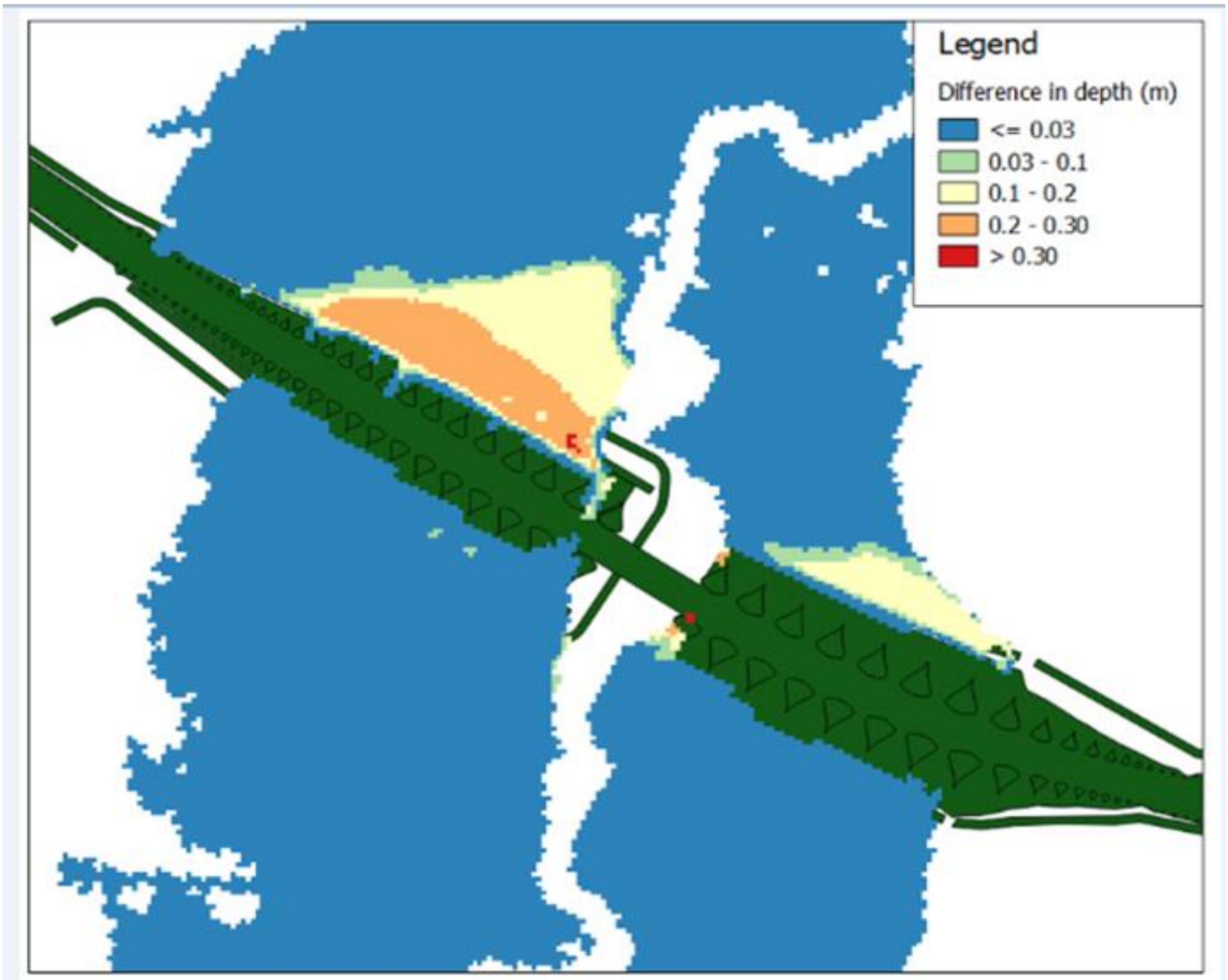
2.1.6 The content of this technical note summarises the design details and approach already shared in a series of design review meetings held with key stakeholders, including the EA and SCC.

3 DESCRIPTION OF DCO DRAINAGE CONCEPT DESIGN

3.1.1 The concept design proposal for the road crossing of the River Alde, its floodplain, and two adjacent local watercourses is with an embankment, wide bridge and flood relief culverts. The proposals were tested by hydraulic modelling undertaken as part of the **Two Village Bypass FRA** [APP-119]. This enabled the arrangements to be modified to optimise hydraulic performance and mitigate any increase in flood risk due to the construction of the two village bypass.

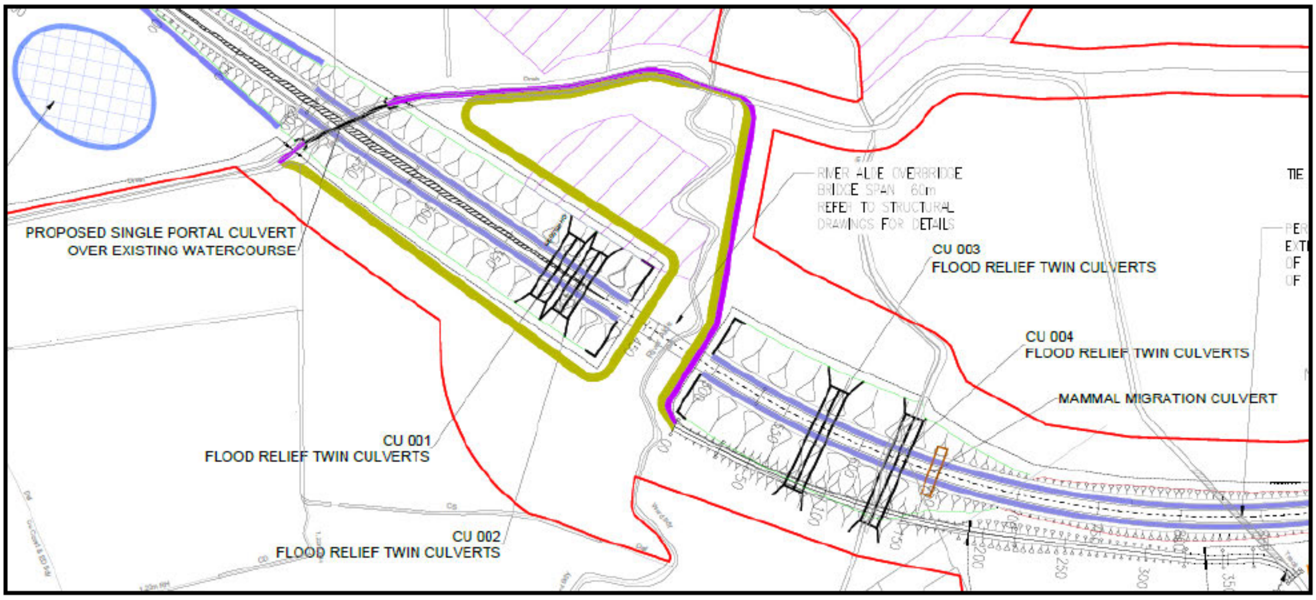
3.1.2 The final layout hydraulic modelling results predicted a limited increase in flood levels at the bridge of approximately 14 mm due to a 1 in 100 year rainfall event plus 35% climate change. Higher increases of up to 320 mm were predicted immediately upstream of the embankment but over a limited area and for a limited period. The results are shown in **Plate 1**.

Plate 1: River Alde crossing - predicted increase in flood depth for proposed bridge and embankment



3.1.3 The proposed final layout submitted for DCO is shown in **Plate 2**.

Plate 2: River Alde crossing - ridge and flood relief culvert locations



3.1.4 The concept drainage design provided for traditional drainage at the A12 roundabouts located at either end of the bypass with a combination of highway gullies and combined kerb drains (CKDs) collecting runoff and discharging via carrier drains to infiltration basins where runoff would infiltrate to ground.

3.1.5 The required size of infiltration basins required for the roundabout runoff could not, at that time, be accurately determined without validated infiltration rates. As a result, the basins were shown schematically and sufficient space within the red line boundary was provided.

3.1.6 Elsewhere on the main line of two village bypass and Hill Farm Lane side road, drainage would be by “over the edge” with runoff flowing from the carriageway to be collected in the adjacent swales. The swales were proposed to be 1 m wide, 0.5 m deep and have side slopes of 1 in 3. The location and extent of swales are shown indicatively on the DCO drawings.

3.1.7 Given the lack of validated infiltration rates the design included for the potential need for filter trenches in the base of the swale

3.1.8 In addition, as back up to the swale/filter drain arrangement, a further allowance was made for an infiltration basin located immediately east of the proposed embankment crossing of the River Alde floodplain. This basin has the twin purpose of collecting runoff not removed by the swale/filter drains but also prevents runoff flowing onto the embankment.

3.1.9 During liaison, SCC confirmed that removal of highway runoff from the River Alde embankment via infiltration within the floodplain would not be acceptable.

3.1.10 The DCO proposed the section of road between the eastern end of the embankment and the River Alde bridge would be drained either by underground drainage or drainage channel towards the bridge and then outfall with discharge into the river. Both the EA and SCC did not want highway runoff discharging directly to the river therefore in response to their concerns it was proposed that the highway on the embankment would therefore be drained either by underground drainage or a drainage channel, over the bridge, and discharge to the west and into the infiltration basin adjacent to the A12 roundabout.

3.1.11 A pipe would be provided within the bridge structure to pass forward the runoff to the west of the bridge.

3.1.12 The section of road between the River Alde bridge and the western end of the embankment would be drained either by underground drainage or drainage channel to the west and then discharge into the infiltration basin adjacent the A12 roundabout.

4 ADDITIONAL INPUT DATA

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

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

- Drone topographic survey of the two village bypass route
- Topographic survey of watercourses within and adjacent to red line boundary
- Aerial view from drone flyover
- Ground investigation and infiltration testing
- Ground penetrating radar (GPR) survey
- Additional traditional topographic survey of critical locations
- Site visit and inspection of the full length of the two village bypass route on 12 January 2021

- Highways England Water Risk Assessment Tool (HEWRAT) (Ref. 2)

4.1.3 The January 2021 site visit was undertaken jointly with SCC.

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

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

5 EXISTING HIGHWAY DRAINAGE ARRANGEMENTS

5.1.1 Following the site visit and review of survey data, details of existing highway drainage arrangements have been determined and are described below. The existing arrangements have been taken into account as part of preliminary design.

5.1.2 The existing section of the A12 south of Stratford St Andrew which will be modified with the construction of the two village bypass western roundabout and shown in **Plate 3** does have some limited formal surface drainage outlets in the form of 5 gullies and 4 kerb gullies.

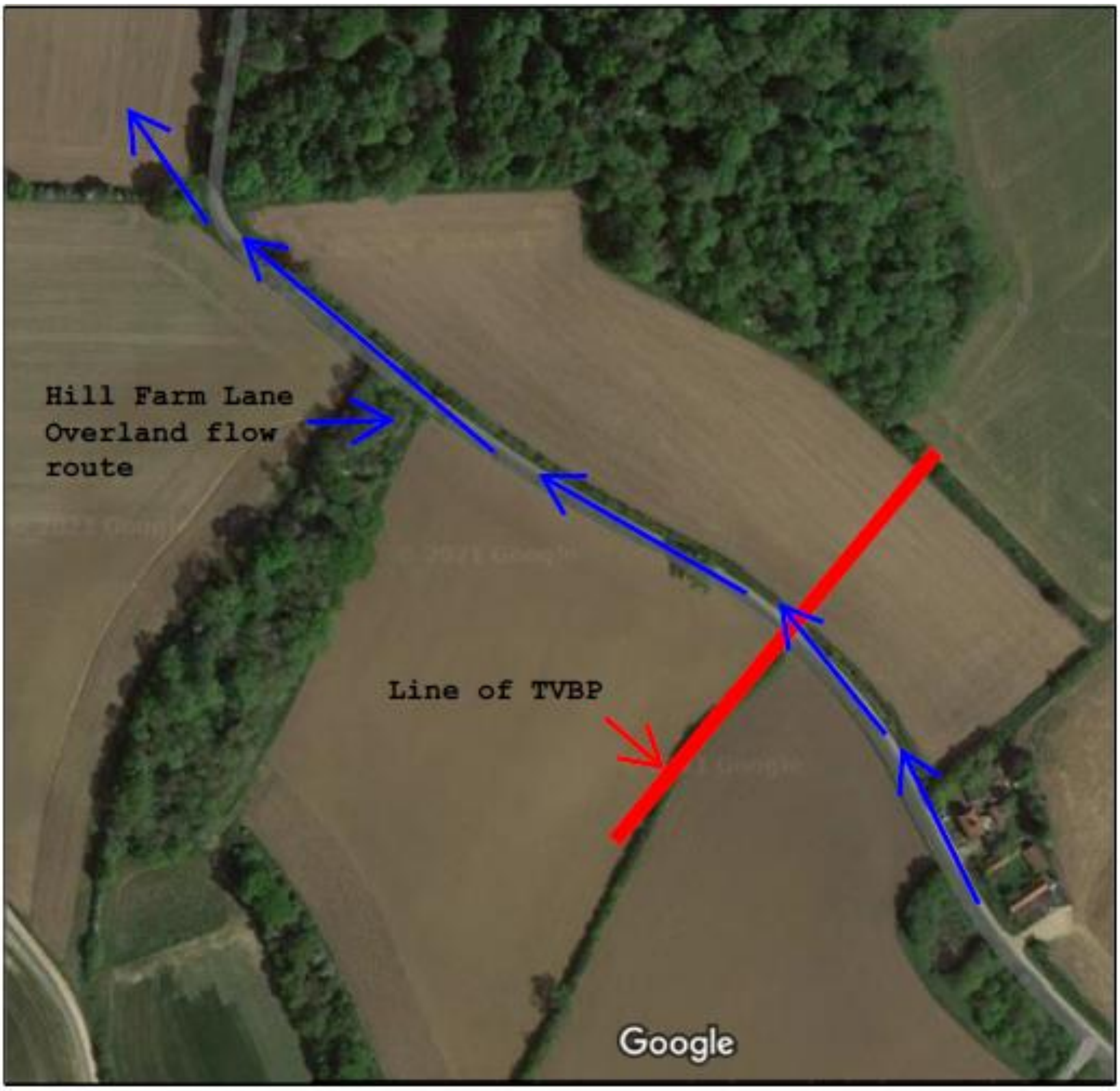
Plate 3: Existing A12 at the location of two village bypass western roundabout

5.1.3 For part of the southbound carriageway where there is no kerb, runoff is over the edge and onto the verge.

5.1.4 During the GPR utility survey the gullies and kerb gullies were inspected and all found to be silted up to an extent which prevented survey to establish the outfall routes. No surface water manholes were identified. As a result, it is assumed that all of the gullies and kerb gullies discharge to soakaway.

5.1.5 Further to the east the proposed two village bypass route cuts across the line of Hill Farm Lane and connectivity is retained through the provision of offset staggered T junctions. The current layout of Hill Farm Lane is shown in **Plate 4**.

Plate 4: Hill Farm Lane showing line of two village bypass and overland flow path



5.1.6 From survey data it can be seen that there is steep fall in level along Hill Farm Lane from Pond Barn Cottages, south of the line of two village bypass, to the north. This fall was noted during the site visit. No surface drainage outlets or formal infrastructure exists. As shown in **Plate 5** there are high banks on either side of the road. These contain runoff within the road and this flows downhill to a bend in the road where it runs off into a field to the north west.

Plate 5: Hill Farm Lane showing high banks

5.1.7 The existing section of the A12 to the east of Farnham will be modified with the construction of the two village bypass eastern roundabout is shown in **Plate 6**. It does have a formal drainage network which has been identified and recorded as part of the GPR utility survey. The GRP survey traced the line of some highway drains but no outfall has been identified.

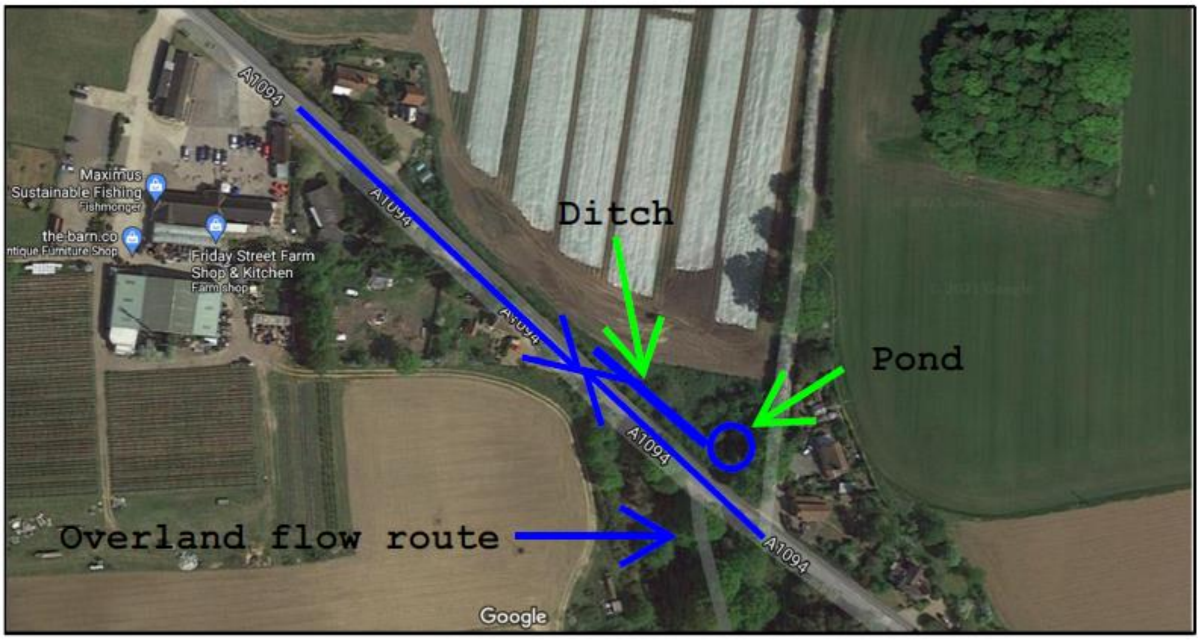
Plate 6: Existing A12 and A1094 junction at the location of two village bypass eastern roundabout

5.1.8 During the GPR utility survey a large number of gullies and kerb gullies were inspected and all found to be silted up to an extent which prevented survey to establish their outfall routes.

5.1.9 The junction of A12 and A1094 forms a local low point. The eastbound carriageway is lower than the westbound carriageway. The westbound carriageway has a crossfall to the centre reserve. Any runoff that does not get removed via gullies and kerb gullies flows through the gap created by the junction turning lanes and ultimately reaches the eastbound carriageway channel line.

5.1.10 The A1094 from its junction with the A12 crest point, has a steep fall to the south east to a low point. Surface water overland flow passes from the crest point to the local low point where it is removed by a highway grip and discharges into an existing ditch. The ditch runs south east behind the highway boundary hedge and terminates in an existing pond as shown in **Plate 7**.

Plate 7: Existing A12 and A1094 junction at the location of two village bypass eastern roundabout



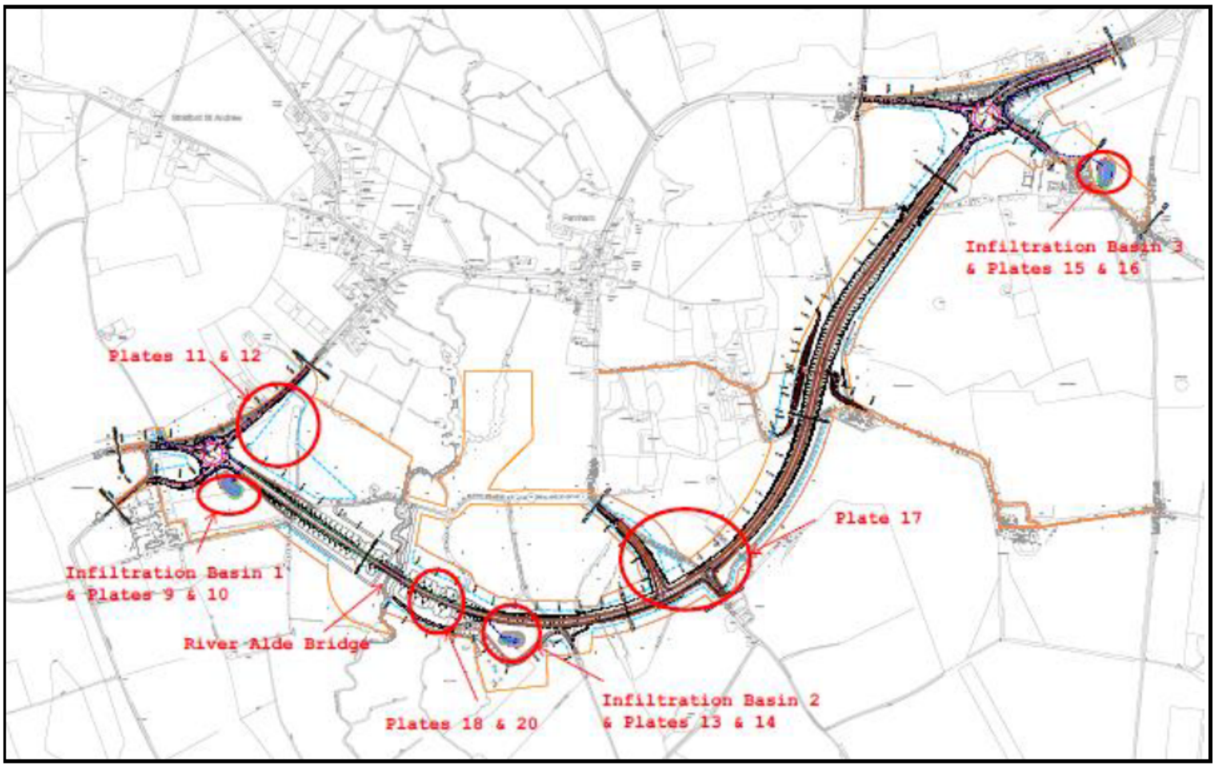
6 PRELIMINARY DRAINAGE DESIGN – HIGHWAY DRAINAGE

6.1.1 The results of geotechnical investigation infiltration testing undertaken at the proposed location of the three infiltration basins and at locations along the line of two village bypass demonstrate that it is possible to remove highway runoff by infiltration to ground. The infiltration rate data has been shared with SCC who have agreed that infiltration is achievable. Accordingly, the concept design proposal that highway runoff is removed and disposed by infiltration to ground remains broadly unchanged.

6.1.2 As agreed with SCC and the EA prior to DCO submission, the environmental impact of discharging highway runoff is to be assessed using the Highways England Water Risk Assessment Tool (HEWRAT) (Ref. 4) methodology. The assessment results confirm that a SuDS management train with the combination of swales, filter drains and infiltration basins, for the two village bypass discharges are low risk and therefore acceptable. SCC has indicated that they will wish to review the management train at detailed design stage and may wish to see provision of additional treatment stages.

6.1.3 A large-scale plan showing the route of the two village bypass is shown in **Plate 8.** The specific locations shown in subsequent Plates are identified for convenience.

Plate 8: Two village bypass location, route and features



6.1.4 The 3 proposed infiltration basins were shown schematically in the DCO drawings. As part of preliminary design, the highway drainage network has been developed using hydraulic modelling. This has enabled the required size of the basins to be determined and space has been allocated.

6.1.5 The location and performance of the A12 West Roundabout infiltration basin 1 is shown in **Plate 9** and the current dimensions hydraulic performance is shown in **Plate 10**.

Plate 9: A12 west roundabout with infiltration basin 1

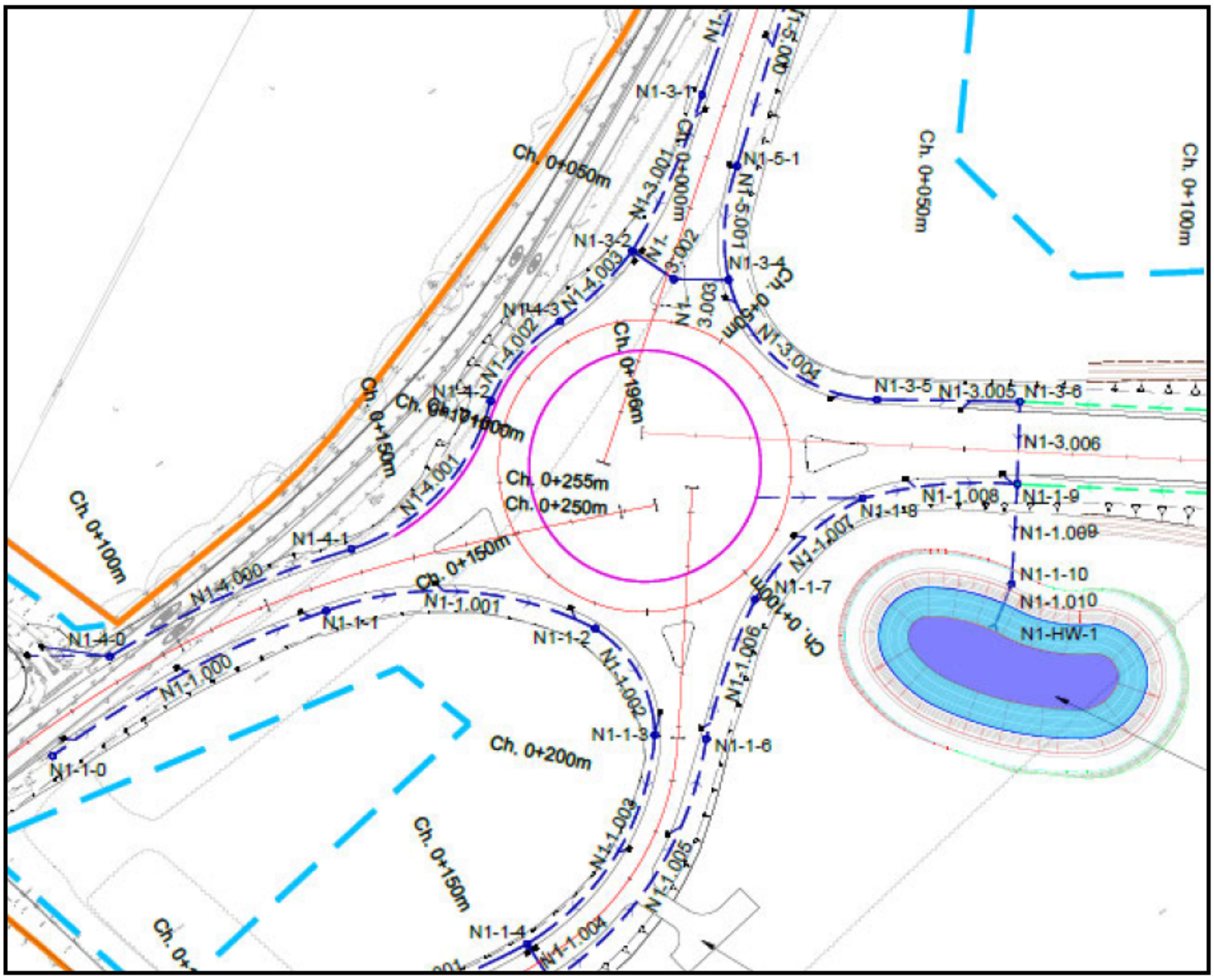


Plate 10: Two village bypass infiltration basin 1 approximate size and hydraulic performance

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

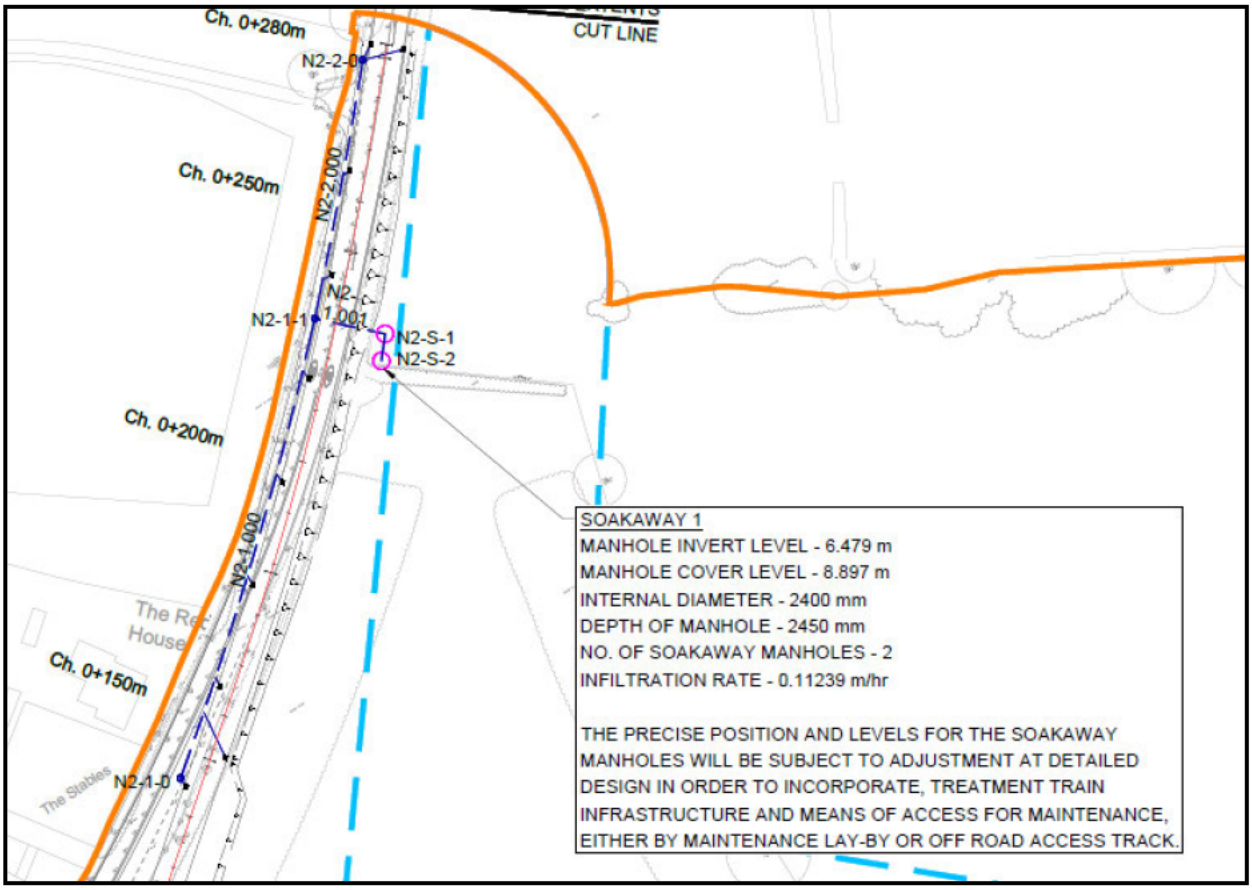
PREDICTED MAXIMUM WATER LEVEL IN 100 YEAR +40 % CC RETURN PERIOD - 8.326 m AOD

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.

6.1.6 In addition to the majority of the A12 Roundabout, the infiltration basin will also provide an outfall for highway runoff from the two village bypass embankment crossing of the River Alde floodplain.

6.1.7 As a result of receiving detailed topographic levels and development of highway gradients, it has become apparent that it is not practical to drain the full length of the roundabout northern arm, back to the infiltration basin as it would require the basin to be approximately 4 m deep and with potential interaction with groundwater. Accordingly, whilst the first 150 m of the northern arm does drain back to the infiltration basin, the remaining 130 m where the proposed road alignment ties into the existing A12 drains to the north and would discharge into proposed soakaway manholes which enable infiltration. The northern roundabout tie-in drainage arrangement, based on assumed performance using the same infiltration rate as infiltration basin 1, is shown in **Plate 11**.

Plate 11: A12 west roundabout northern arm with soakaway manhole location



6.1.8 Given its proximity to the infiltration basin and the fact that the existing highway appears to drain via local kerb drains or over the edge, infiltrating runoff to ground, it was reasonable to assume that the soakaways would work. The hydraulic modelling of the catchment used the infiltration rates applicable to the infiltration basin. However, it was agreed with SCC and as a reasonable precaution, that infiltration testing at the soakaway manhole location should be undertaken prior to commencing detailed design.

6.1.9 Field log sheets for infiltration testing have been received in May 2021. They indicate that infiltration rates are not sufficient at the proposed soakaway manhole location for the western roundabout northern arm. In consequence the layout contained in the preliminary design will not be acceptable. Further site investigation and consideration of alternative outfall arrangements will be required either in advance of or at the start of detailed design stage.

6.1.10 Potential options have been identified which provide for an outfall arrangement located entirely within the permanent land take areas. These include:

i. possible connection to the proposed infiltration basin with increased depth;

ii. moving the soakaway closer to the roundabout; or

iii. an outfall route along the line of the two village bypass embankment discharging into the local watercourse.

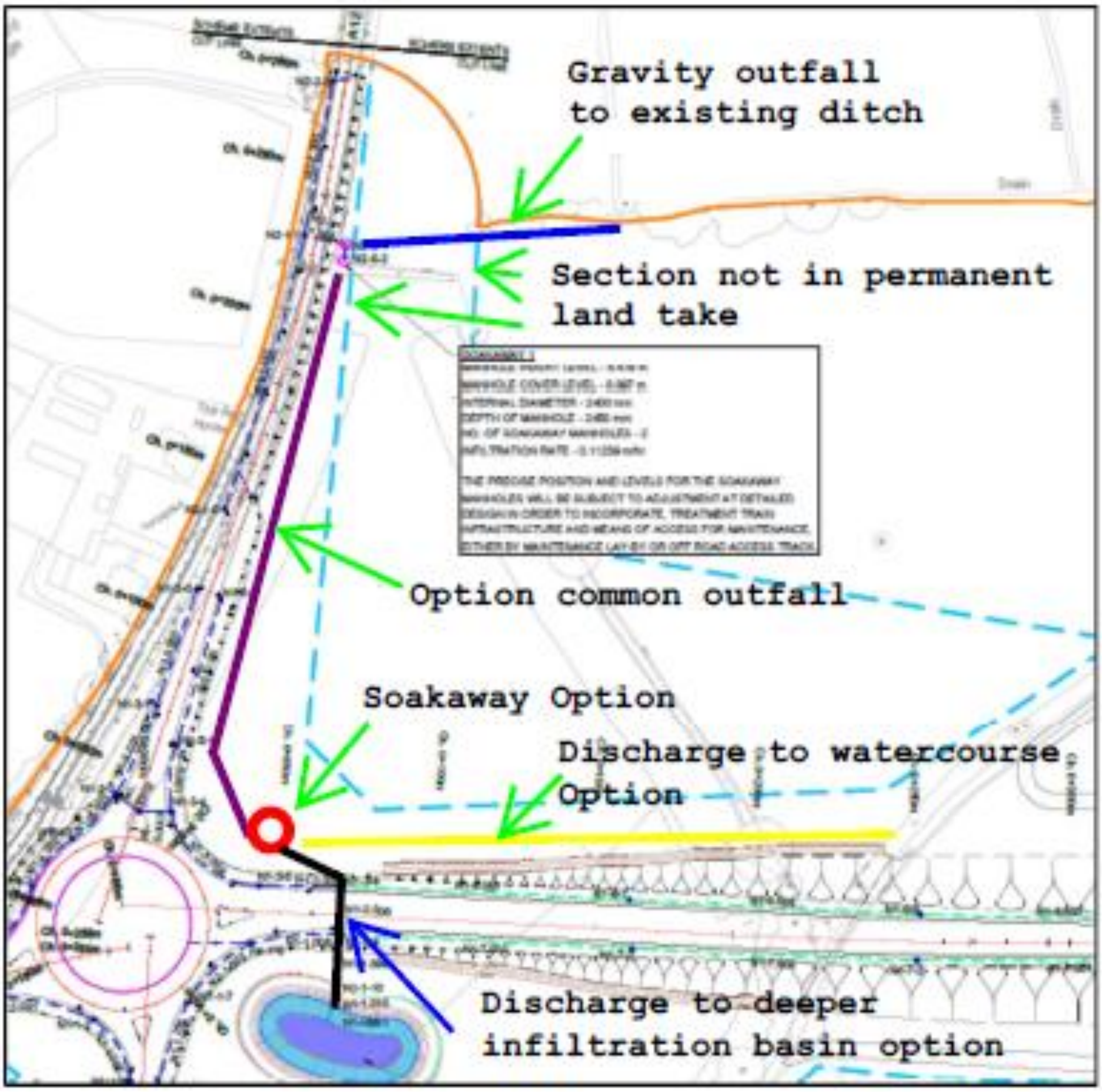
6.1.11 A further alternative option would be to

iv. discharge into a local ditch located within 100 m of the soakaway manhole.

6.1.12 The route would be located entirely within the red line boundary. However, it crosses land that is not retained and which will be transferred back to the landowner. SCC has confirmed that as a general position they do not require outfall highway drains to be located in land under their ownership. If this option was pursued, SCC would require a legal agreement in the form of an easement or wayleave providing a right to have the drain located in the land (not within their ownership) and a right of access for maintenance

6.1.13 The potential options are shown in **Plate 12**.

Plate 12: A12 west roundabout northern arm showing soakaway alternative options



6.1.14 Infiltration basin 2, east of the River Alde, remains in the same area indicated at DCO. It receives runoff from the two village bypass swales to the east. All runoff flowing along the swales and associated filter drains which does not infiltrate to ground is intercepted and diverted to the basin. This prevents the runoff overrunning onto the River Alde embankment.

6.1.15 Hydraulic modelling has been undertaken to determine the required location, size, depth and volume of the infiltration basin. Details are provided in the first issue of the preliminary design drawings. Subsequent to issue it has been necessary to move the position of the basin to coordinate with the revised position and levels of the accommodation access track. The revised location of the infiltration basin will be shown either on an updated preliminary design drawing or in first revision of detailed design.

6.1.16 The DCO schematic location of infiltration basin 2 is shown in **Plate 13** and the current dimensions hydraulic performance is shown in **Plate 14**. The movement of the basin will nominally change the data in **Plate 14,** but it is included to demonstrate the technical feasibility of the design.

Plate 13: Two village bypass infiltration basin 2 location

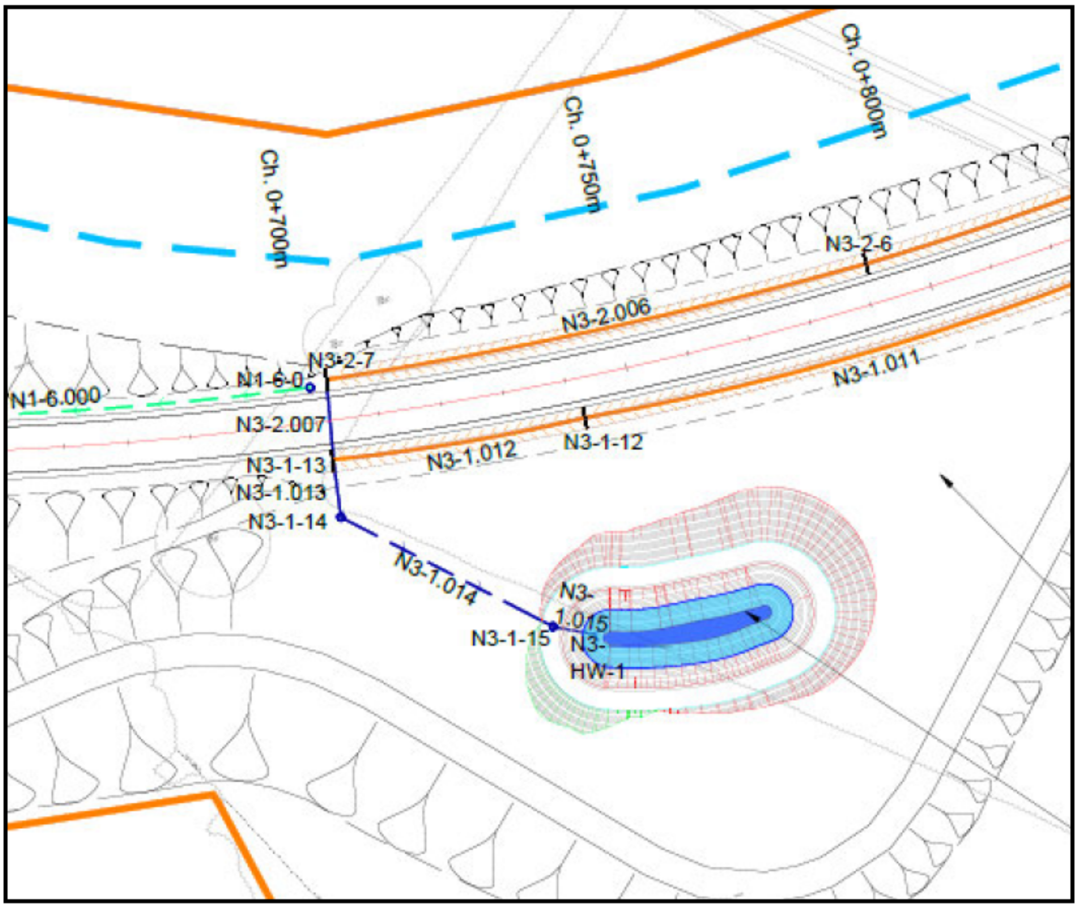


Plate 14: Two village bypass infiltration basin 2 approximate size and hydraulic performance

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.

6.1.17 The location and performance of the A12 East Roundabout infiltration basin 3 is shown in **Plate 15**. and the current dimensions hydraulic performance is shown in **Plate 16.**

Plate 15: A12 east roundabout infiltration basin 3 location

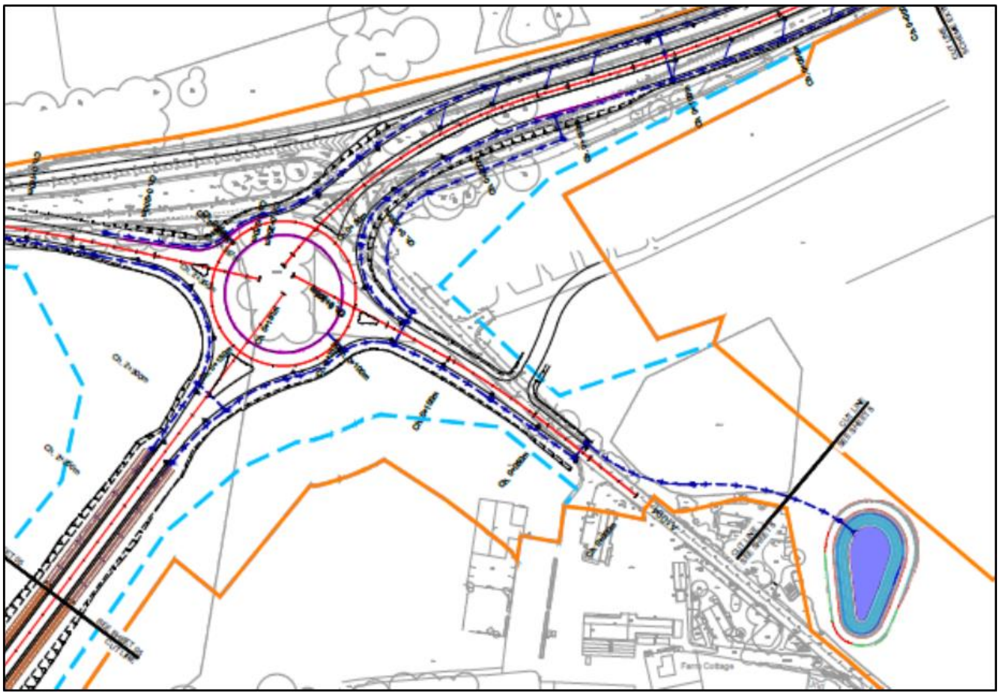


Plate 16: Two village bypass infiltration basin 3 approximate size and hydraulic performance

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

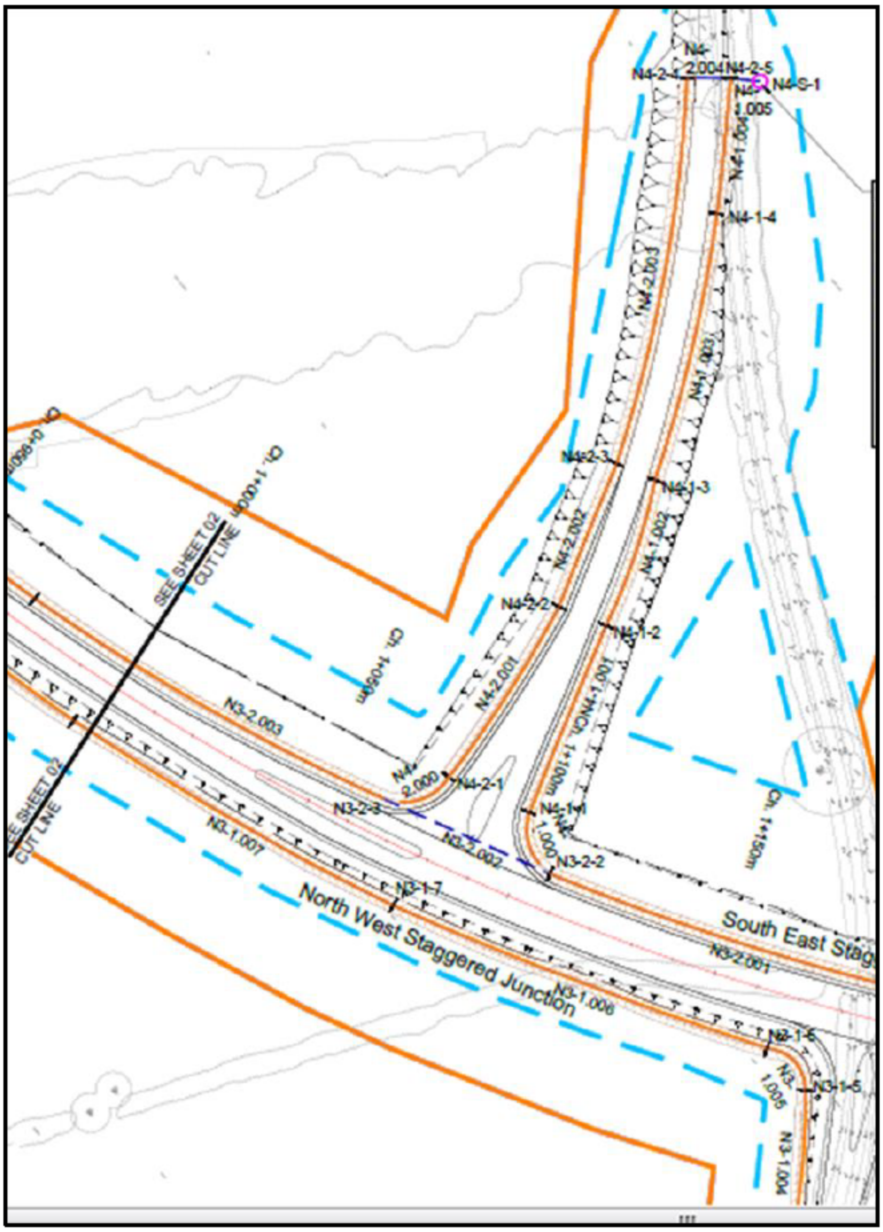
THE PRECISE POSITION, SHAPE AND LEVELS FOR THE BASIN WILL BE SUBJECT TO ADJUSTMENT AT DETAILED DESIGN IN ORDER TO INCORPORATE, TREATMENT TRAIN INFRASTRUCTURE, POTENTIAL ECOLOGICAL ENHANCEMENT AND OPTIMISE EARTHWORKS.

6.1.18 The infiltration basin receives all the runoff from the A12 East Roundabout and its arms. It also receives some runoff from the northern part of the two village bypass that has not fully infiltrated via the swales and filter drains.

6.1.19 The infiltration basin remains located in the same field adjacent to the A1094 but has been moved further north. This change has been made following the discovery of the ditch and pond described in section 5.1.10 and shown in **Plate 7**. If the infiltration were to be located in proximity to the pond the efficiency of infiltration could be reduced because the additional infiltration could cause local rising of groundwater.

6.1.20 As described in section 5.1.5, and shown in **Plate 17**, Hill Farm Lane is severed by the proposed two village bypass route with a proposed staggered junction arrangement being provided for connectivity.

Plate 17: Two village bypass Hill Farm Lane junction with soakaway manhole location



6.1.21 The DCO Drainage Strategy assumed that runoff from Hill Farm Lane would discharge to swales and filter drains allowing infiltration to ground.

6.1.22 The section of Hill Farm Lane to the south, which is at a higher level than two village bypass, has swale/filter drains which discharge into those proposed alongside the two village bypass, and for which Infiltration testing achieves satisfactory results.

6.1.23 The section of Hill Farm Lane to the north is at a lower level that two village bypass. A borehole excavated next to this side road found cohesive material which typically has poor infiltration rates and therefore would suggest that infiltration would not be viable. Based on available data it was considered that granular material would be found at a deeper depth below the cohesive material at a depth of around 5 m.

6.1.24 Discussions have taken place with SCC. Since it is agreed that there are no watercourses to which discharge can be made, it has been agreed that subject to detail, in principle a deep borehole soakaway could be permitted, providing evidence of the underlying granular material at reasonable depth is proven.

6.1.25 Results of a borehole excavation have been received in May 2021 and confirm the presence of the granular material at depth of 3.2 m bgl, adjacent to Hill Farm Lane. Infiltration testing results demonstrate that flow passes through into the permeable strata and that hence infiltration will be suitable. As agreed with SCC, when construction takes place, excavation will be made down into the granular material and an infiltration test to an appropriate standard (e.g. BRE Digest Soakaway design: DG 365 – 2016 (Ref. 3)) will be undertaken. The required size of soakaway manholes will be determined by the results.

7 REVISED DRAINAGE DESIGN STRATEGY – RIVER ALDE CROSSING

7.1.1 The position of the embankment with its flood relief culverts and the 60 m wide bridge remain as shown in the DCO drawings and as hydraulically modelled, with only minor changes which do not change hydraulic performance as reported in the **Two Village Bypass FRA**.

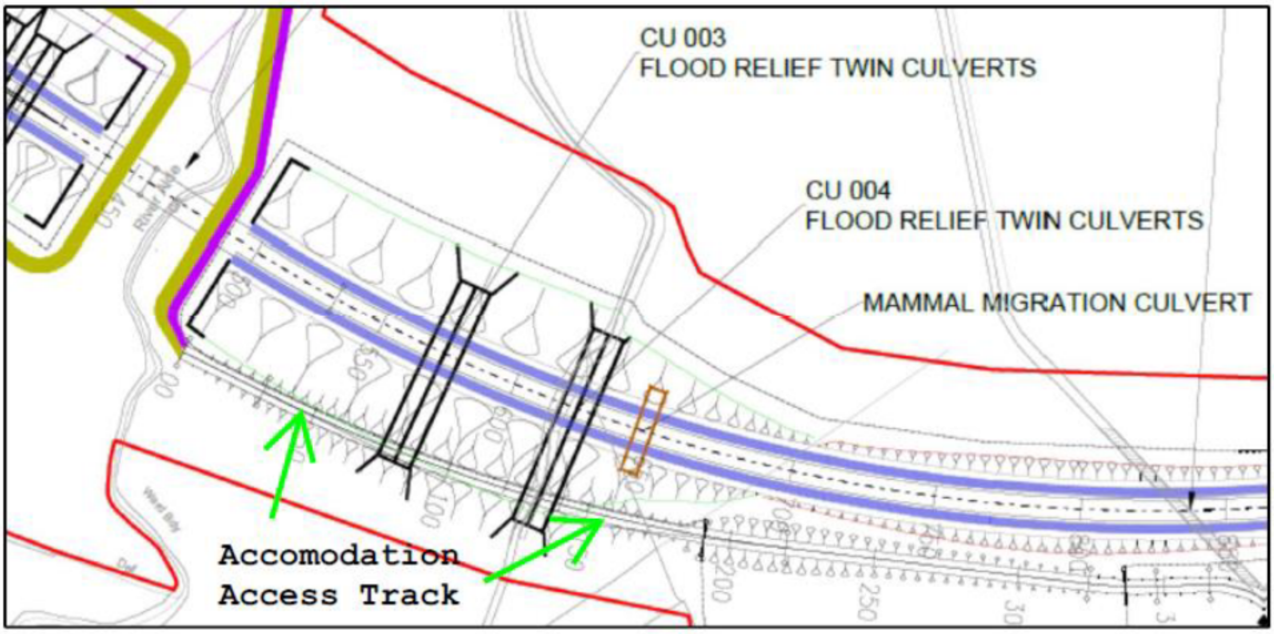
7.1.2 Following review of the topographic survey the central piers of the bridge have been moved to create greater distance from the proven position of the River Alde and its banks. The central span is increased to a width of 30 m and the side spans reduced to 15 m. The change is made to reduce the risk of scour should the river move its position and to provide a better width between top of bank and pier for maintenance. The reduction in side span widths does not adversely impact on the access track which is provided.

7.1.3 It is confirmed that this modification of piers does not cause any change to the hydraulic modelling results.

7.1.4 The change to the bridge pier positions has been notified to and discussed with SCC and the EA in design review meetings and no objection was raised.

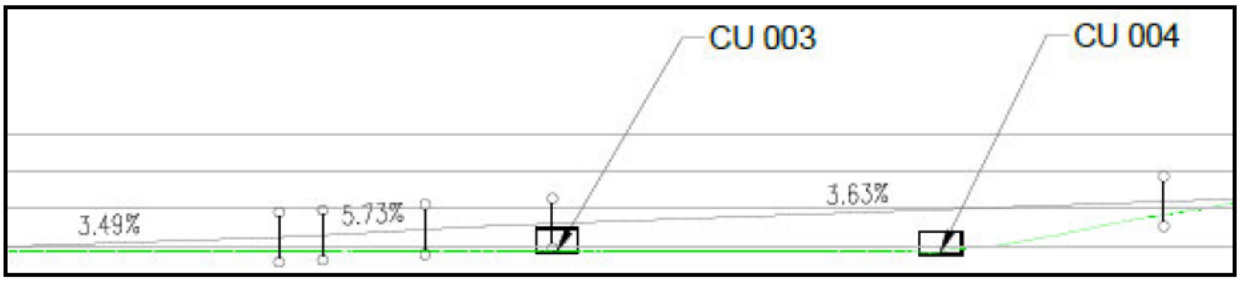
7.1.5 An accommodation access track is shown on the DCO drawing extract in **Plate 18**. This passes under the bridge on the east side of the River Alde and then follows in parallel alongside the downstream side of the proposed embankment and over the flood relief culverts as shown in **Plate 19**.

Plate 18: Two village bypass accommodation track east of the River Alde at DCO Submission



7.1.6 The culvert lengths were reviewed to meet an EA request to achieve a maximum length of 50m. The flood relief culverts CU003 and CU004 have a length of 65 m, and the flood relief culverts to the west have a length of 40m.

Plate 19: two village bypass accommodation track east of the River Alde – long section

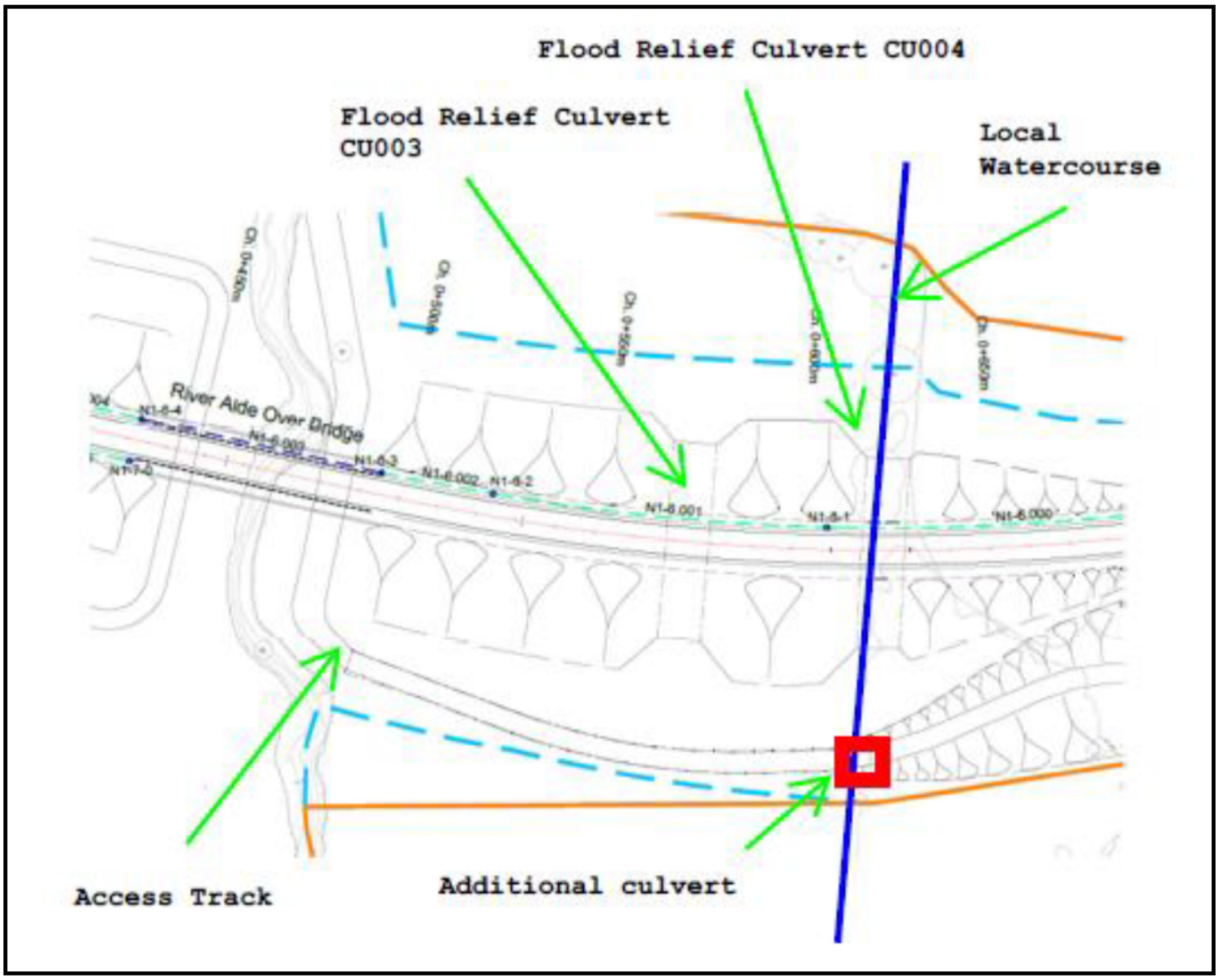


7.1.7 In order to reduce the length of culverts CU003 and CU004, it has been proposed that the accommodation track be laid at existing ground levels in front of the toe of the embankment but with a steeper section to the east to achieve the higher tie-in levels at Hill Farm Road.

7.1.8 A residual impact of the accommodation track amendment is that, for the watercourse which passes through flood relief culvert CU004, there is a requirement for a new downstream culvert where the proposed track crosses.

7.1.9 The accommodation track amendment and associated alterations has been notified to and discussed with SCC and the EA in Design Review Meetings and no objection in principle was raised. The shortening for the length of the flood relief culverts albeit with the new downstream culvert crossing is the preferred solution. The revised arrangement is shown in **Plate 20.** Accordingly, SZC Co. has submitted a design amendment into consultation.

Plate 20: Two village bypass accommodation track and flood relief culverts east of the River Alde – revised arrangement



7.1.10 As part of structural design, the form of the flood relief culverts has been considered. Whilst the cross-sectional dimensions of the culverts remain unchanged, it has been decided that portal culvert structures will be proposed. The advantage of this is that the portal culvert better allows for the watercourse to pass through culvert CU004.

7.1.11 The same portal form of culvert is proposed for the watercourse and track which passes under the embankment to the west of the River Alde at chainage 210 m. This culvert is located just clear of the River Alde floodplain extent also provides a mammal crossing point beneath the embankment.

7.1.12 The DCO design included for the provision of a dedicated mammal crossing culvert to the east of culvert CU004 at chainage 650 m but this is some distance clear of the flood plain. As discussed with the EA it is acceptable to delete this culvert and replace it with a mammal ledge in culvert CU004 to better follow the flood plain extent and existing watercourse route. The mammal ledge is to be installed above the 1 in 100 year plus 40% climate change flood level.

7.1.13 During the joint site inspection with SCC which took place on 12 January 2021 it was confirmed that swales at the toe of embankment within the River Alde flood plain would not serve useful purpose as they will be regularly flooded.

7.1.14 The EA confirmed that the predicted limited increase in flood levels adjacent to the upstream side of the embankment is acceptable subject to agreement with the landowner.

7.1.15 SZC Co. has reached agreement with the landowner in a form that satisfies the EA. Accordingly, the provision of flood compensation areas (within the concept design) is deleted from the preliminary design.

7.1.16 During liaison SCC confirmed that there must be no infiltration directly adjacent the carriageway for highway runoff where drainage is located on top of the road embankment as SCC were concerned of erosion within the embankment layers. It was suggested to SCC that the section highway on the embankment could drain to either a linear channel drain or surface V channel and be removed by an underground drain discharging to the infiltration basin to the west and adjacent to the A12 roundabout. SCC stated that a linear channel drain would not be acceptable due to maintenance concerns. For the preliminary design the indicative proposed swales are replaced by options of either: surface V channel and gully; or kerb and gully infrastructure. The drainage options have been discussed with SCC in design review meetings and the preliminary design currently proposes the use of concrete surface V channels and gullies. However, following recent review and consideration of integration with landscaping, SZC Co. is exploring a design alternative which provides for a vegetated surface with filtration layer and impermeable membrane below to ensure no infiltration to ground and this will be reviewed at the next design stage. Any proposed alternative would be discussed with and approved by SCC.

8 VALIDATION OF OUTLINE DRAINAGE STRATEGY

8.1.1 In accordance with the drainage hierarchy, the **Outline Drainage Strategy** [REP2-033] proposed the primary use of infiltration, with additional use of attenuation techniques (e.g. basins and swales) to manage water quality and to further promote infiltration. The **Outline Drainage Strategy** is validated by the completed preliminary design.

8.1.2 The preliminary design documents will be made available for review and acceptance by SCC and the EA with respect to potential adoption of the two village bypass by SCC and for required regulatory consents.

9 SUMMARY AND CONCLUSION

9.1.1 The purpose of this technical note is to provide details of how the concept design has needed to evolve and develop as a result of provision of new information. The proposed developments have been discussed with SCC and the EA in liaison and through design review meetings.

9.1.2 The highway drainage has been designed in accordance with Design Manual for Roads and Bridges, the CIRIA SUDs Manual C753 (Ref. 4) and to comply with stated requirements of SCC contained in their SUDs Local Design Guide Appendix A (Ref. 5).

9.1.3 At this preliminary design stage, it is considered that the design provides for the effective removal, treatment and disposal of highway runoff without adversely increasing flood risk to or from watercourses, impacting on third parties. The flood risk performance of the highway is as specified in DMRB and SCC guidance.

REFERENCES

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

2. Highways Agency et al. (2009). Volume 11, Section 3, Part 10: Road Drainage and the Water Environment, HD45/09. http://www.standardsforhighways.co.uk/ha/standards/dmrb/vol11/section3/hd4509.pdf

3. BRE Digest Soakaway design: DG 365 – 2016, BRE, 2016 https://www.brebookshop.com/details.jsp?id=327592

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

5. Sustainable Drainage Systems (SuDS) a Local Design Guide Appendix A to the Suffolk Flood Risk Management Strategy, Suffolk County Council, May 2018 https://www.greensuffolk.org/app/uploads/2021/05/2018-10-01-SFRMS-SuDS-Guidance-Appendix-A-.pdf

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 epage 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.

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.

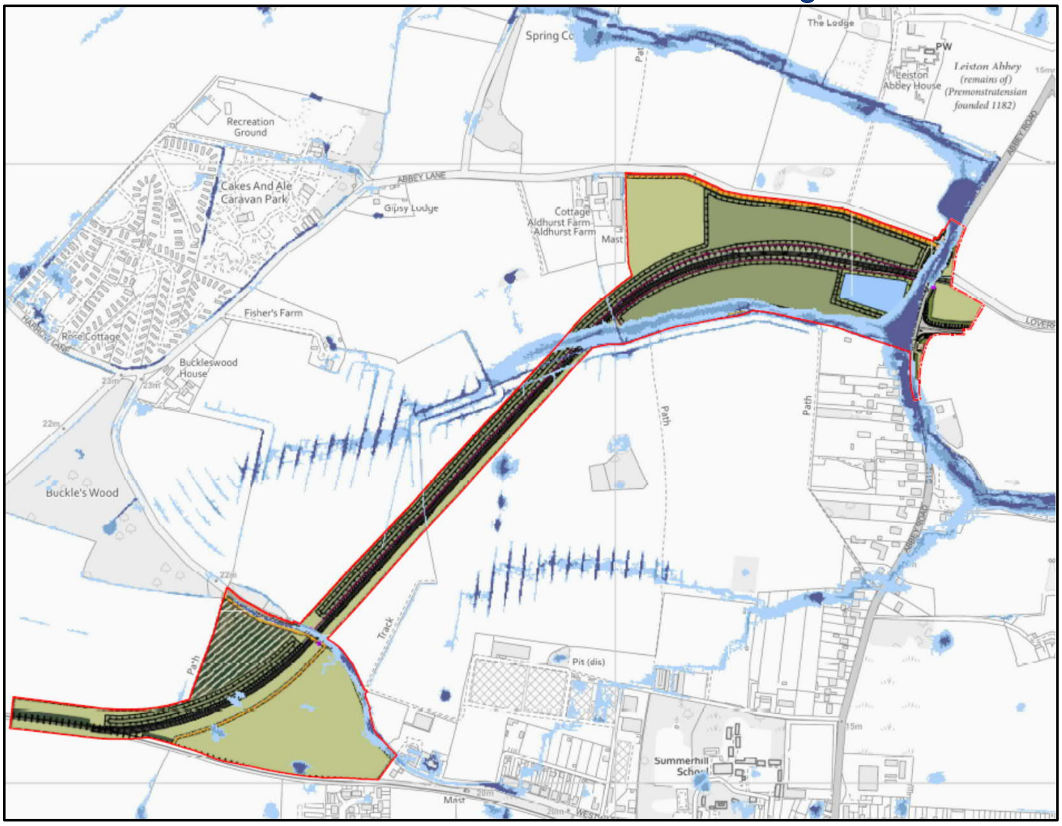
3.1.2 Local ditches are located on either side of Buckleswood Road. The ditches drain the road and, to an extent, the adjacent land. The green rail route level crossing will be required to accommodate these ditches which will need to be culverted.

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 SZC Co. is continuing to undertake ground investigation with infiltration testing. The latest infiltration rates will be used to inform railway drainage design development. However, in the interim, the drainage design strategy has been updated to reflect the outputs of these initial ground investigation and infiltration testing results.

5 DESCRIPTION OF DCO DRAINAGE CONCEPT DESIGN

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

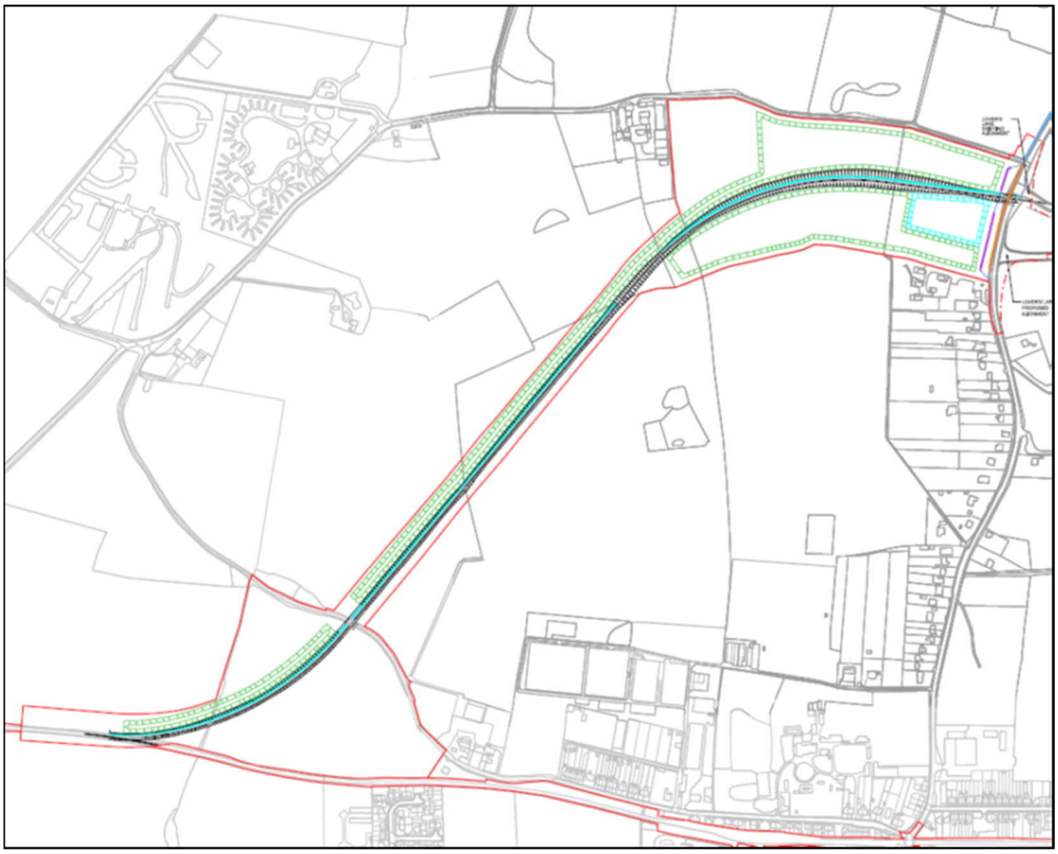
5.1.2 The design approach was conservative to provide assurance on land take and accommodating drainage infrastructure.

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 epage 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**.

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 B**.

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.

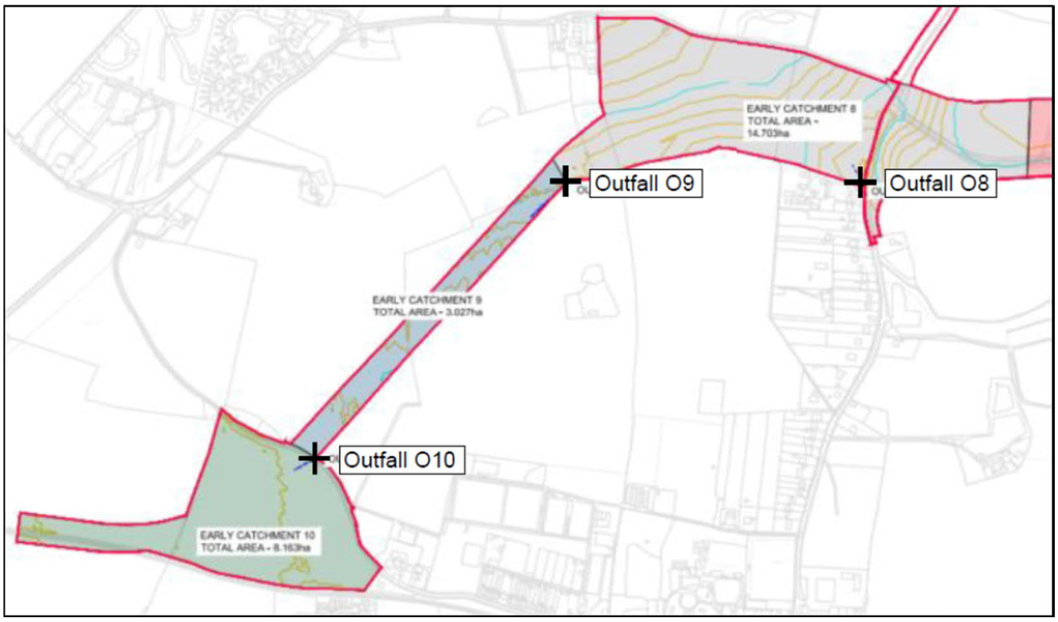
6.1.3 The design provides for a swale with filter drain which removes runoff from the track and adjacent strip together with cutting sides. In addition, cut off drains are provided at the top of cuttings to limit overland flow from adjacent land. The cut off drains outfall into the trackside swale where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

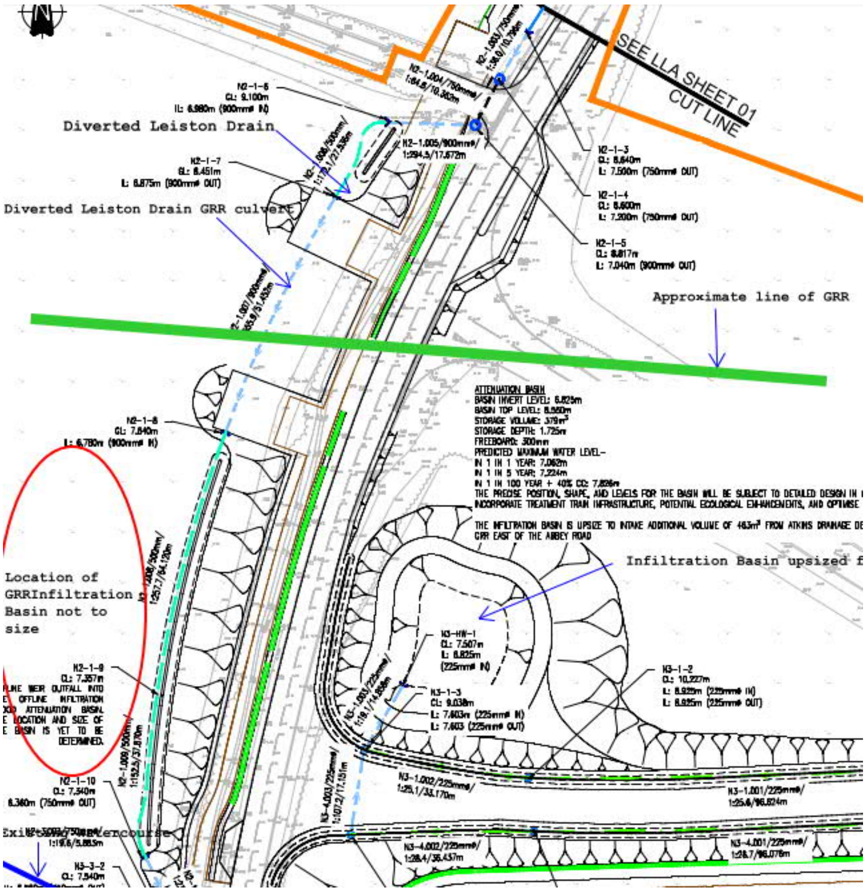
6.1.13 At outfall 09, the green rail route will cross the line of the local watercourse or the upstream flow path that feeds the watercourse. The green rail route is in cutting at this location and may be at a lower level than the watercourse. If this is the case then then the watercourse will have to 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 shown in full as **Appendix C.**

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 full as **Appendix C.** 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.

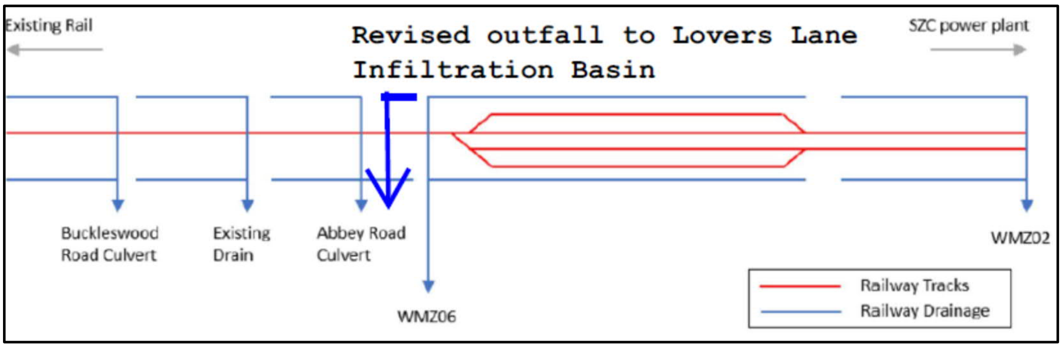
7.1.13 Following design development and hydraulic modelling, WMZ2 is proposed to be an infiltration basin with discharge to watercourse. However, a high-level overflow is also proposed for exceedance rainfall events. This will discharge to the Combined Drain Outfall which is proposed to discharge to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

8 SUMMARY AND CONCLUSION

8.1.1 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.

8.1.2 The currently proposed green rail route catchments and outfalls are shown schematically in **Plate 6.**

Plate 6: Green rail route outfalls schematic



8.1.3 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.

8.1.4 Based on latest infiltration test data the need for any outfall from the green rail route, west of Abbey Road into local watercourses at outfalls 08, 09 and 010 will be refined and developed subject to Requirement 5.

8.1.5 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).

8.1.6 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.

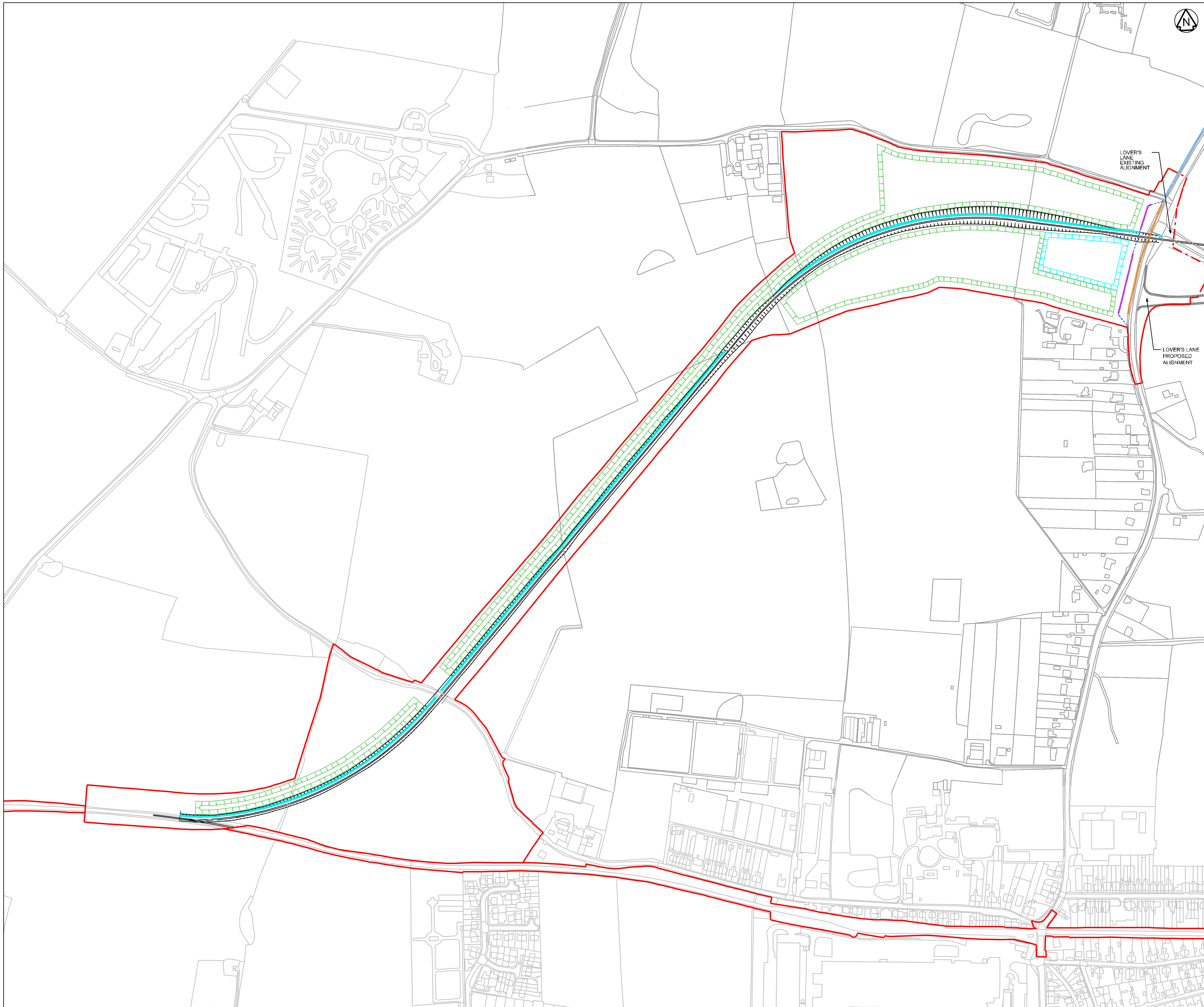
REFERENCES

1. BRE Digest Soakaway design: DG 365 – 2016, BRE, 2016
https://www.brebookshop.com/details.jsp?id=327592

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: GREEN RAIL ROUTE WEST OF ABBEY ROAD



NOTES:

KEY:

- INFRASTRUCTURE FEATURES
- LANDSCAPE BUND
- GREEN RAIL ROUTE DEVELOPMENT SITE BOUNDARY
- VOLUME 9 ASSESSMENT AREA
- SWALE OR INFILTRATION BASIN
- PROPOSED DITCHES
- PROPOSED CULVERTS
- EXISTING CULVERTS (TO REMAIN)
- EXISTING DITCHES (TO BE DIVERTED)
- EXISTING DITCHES (TO REMAIN)
- EARTHWORKS

NOT PROTECTIVELY MARKED

DOCUMENT:
SIZEWELL C
ENVIRONMENTAL STATEMENT
VOLUME 9
CHAPTER 2
DESCRIPTION OF RAIL

DRAWING TITLE:
GREEN RAIL ROUTE
ILLUSTRATIVE DRAINAGE PLAN FOR THE GREEN RAIL ROUTE

DRAWING NO:
FIGURE 2.6

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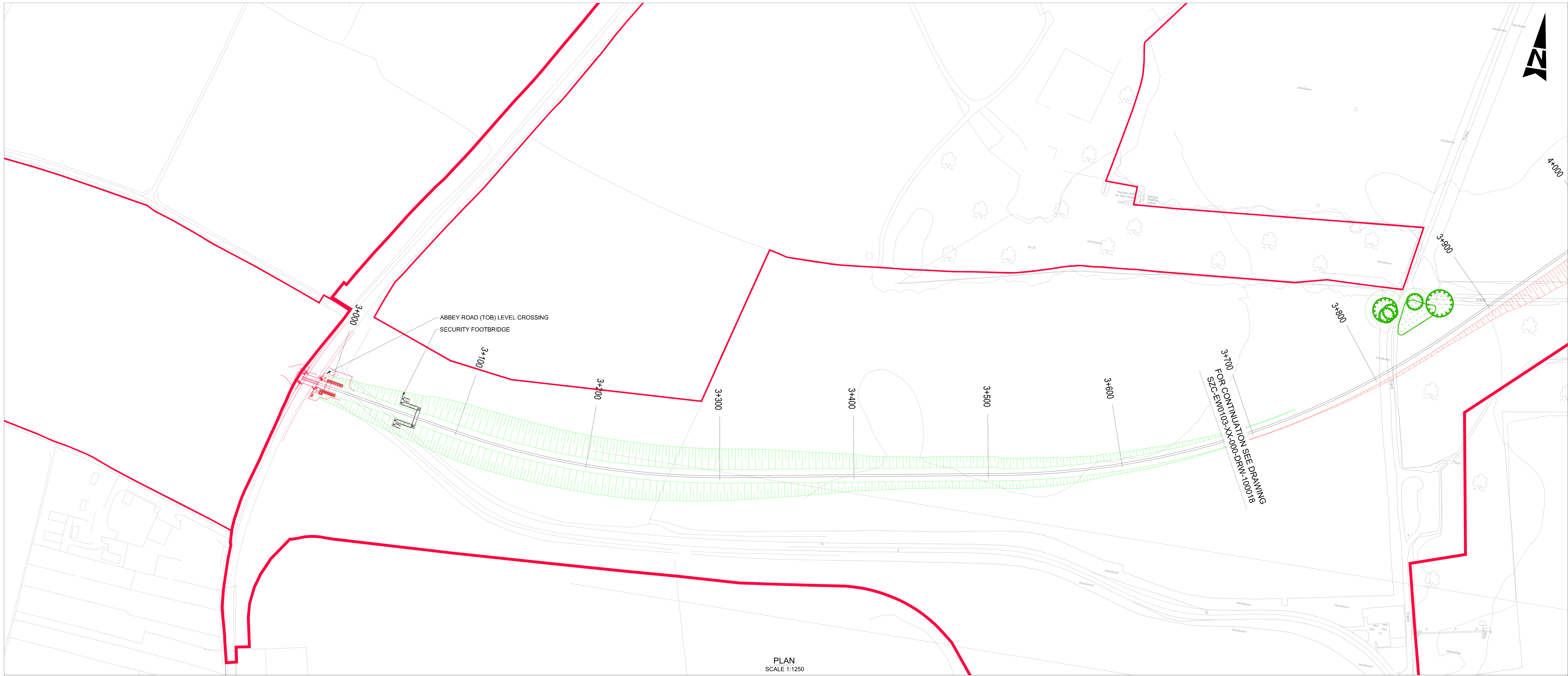
APPENDIX B: GREEN RAIL ROUTE ROUTE EAST OF ABBEY ROAD WITHIN MAIN DEVELOPMENT SITE



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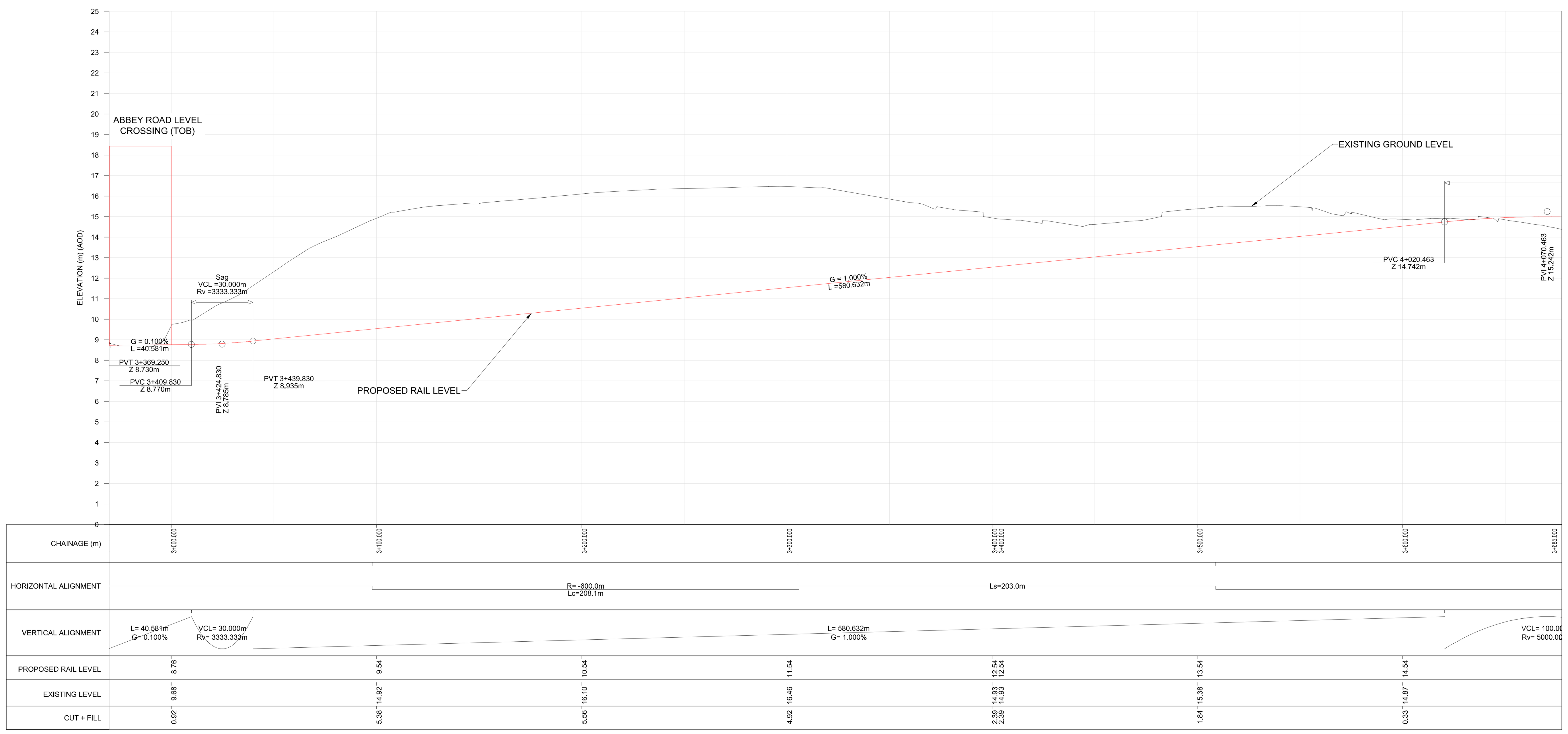

VERTICAL PROFILE
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NOTES:

1. ALL PROPOSED LEVELS ARE INDICATED TO A TOLERANCE OF 0.5m
2. ABBREVIATIONS:
   - PVI = POINT OF VERTICAL INTERSECTION
   - PVT = POINT OF VERTICAL TANGENT
   - PVC = POINT OF VERTICAL COMMENCEMENT
   - VCL = LENGTH OF VERTICAL CURVE
   - TOB = TRAIN CREW OPERATED BARRIER
   - R_C = RADIUS OF CURVE
   - R_V = VERTICAL CURVE RADIUS
   - L_S = LENGTH OF STRAIGHT
   - L = LENGTH
   - G = GRADIENT
   - R = RADIUS
   - Z = ELEVATION

KEY:

- PROPOSED RAIL TRACKS
- RETAINED VEGETATION
- EARTHWORKS - IN CUTTING
- EARTHWORKS - EMBANKMENT
- DEVELOPMENT SITE BOUNDARY

REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION / COMMENTS	APPROVED
01	FEB 2020	PB	DW	DCO SUBMISSION	SZC CO

NOT PROTECTIVELY MARKED

PROJECT:
SIZEWELL C

DOCUMENT:
APPLICATION DRAWING FOR APPROVAL
REGULATION 5(2)(O)

DRAWING TITLE:
MAIN DEVELOPMENT SITE
TEMPORARY CONSTRUCTION AREA
PROPOSED GENERAL ARRANGEMENT
SHEET 1 OF 4

DRAWING NO:	REVISION:
SZC-EW0103-XX-000-DRW-100017	01

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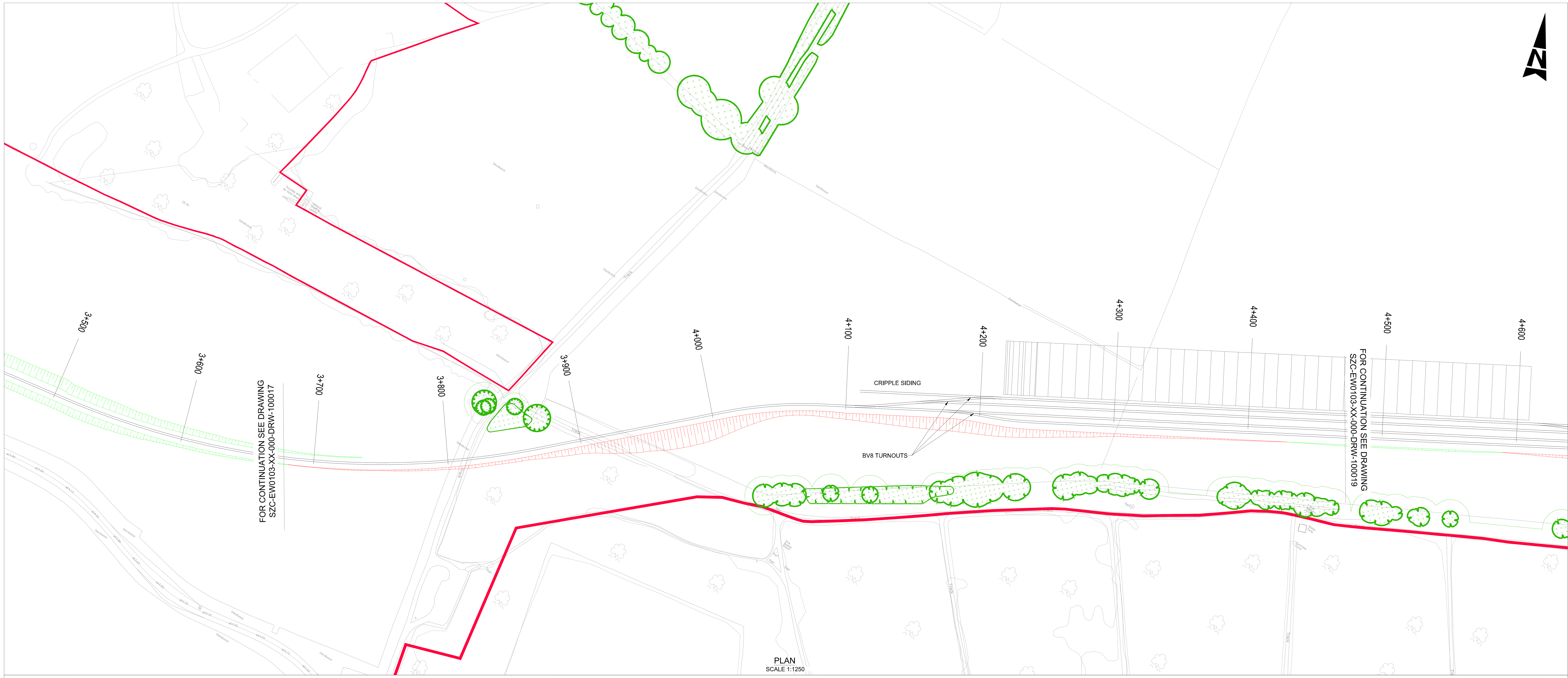

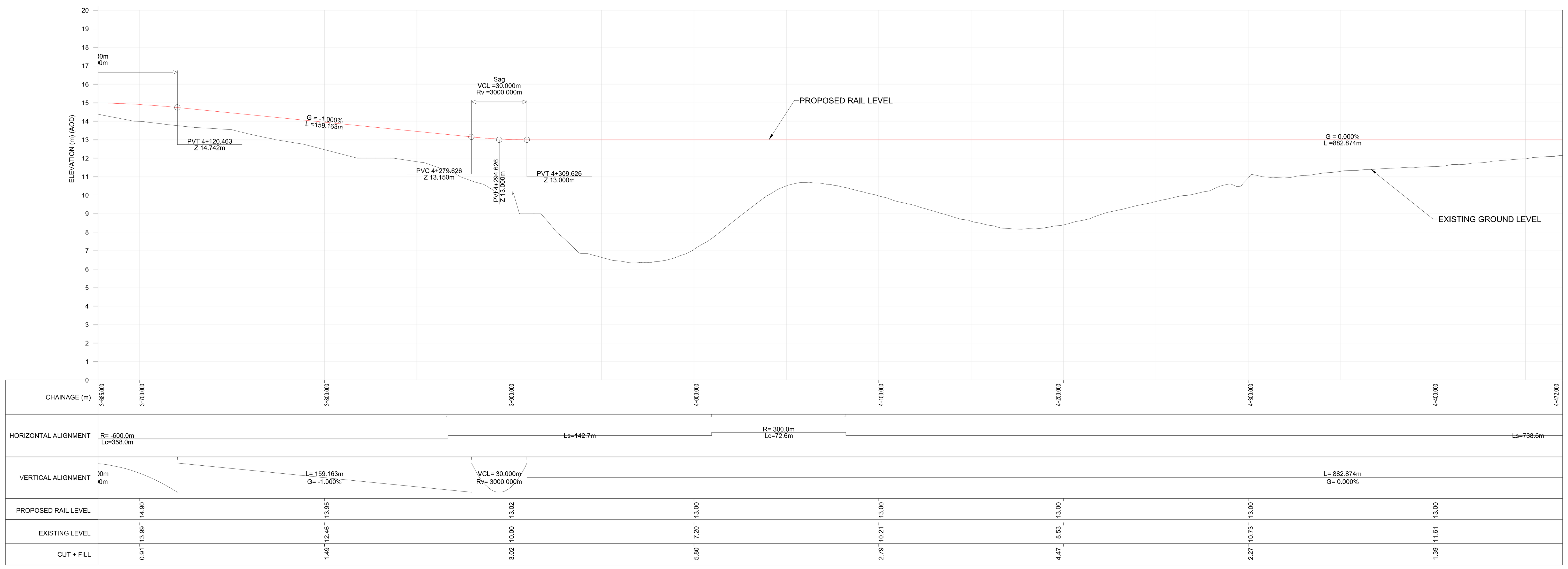

VERTICAL PROFILE
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KEY:

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- RETAINED VEGETATION
- EARTHWORKS - IN CUTTING
- EARTHWORKS - EMBANKMENT
- DEVELOPMENT SITE BOUNDARY

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SIZEWELL C

DOCUMENT:
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REVISION: 01

DATE: JAN 2020
DRAWN: PB
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   - R = RADIUS
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KEY:

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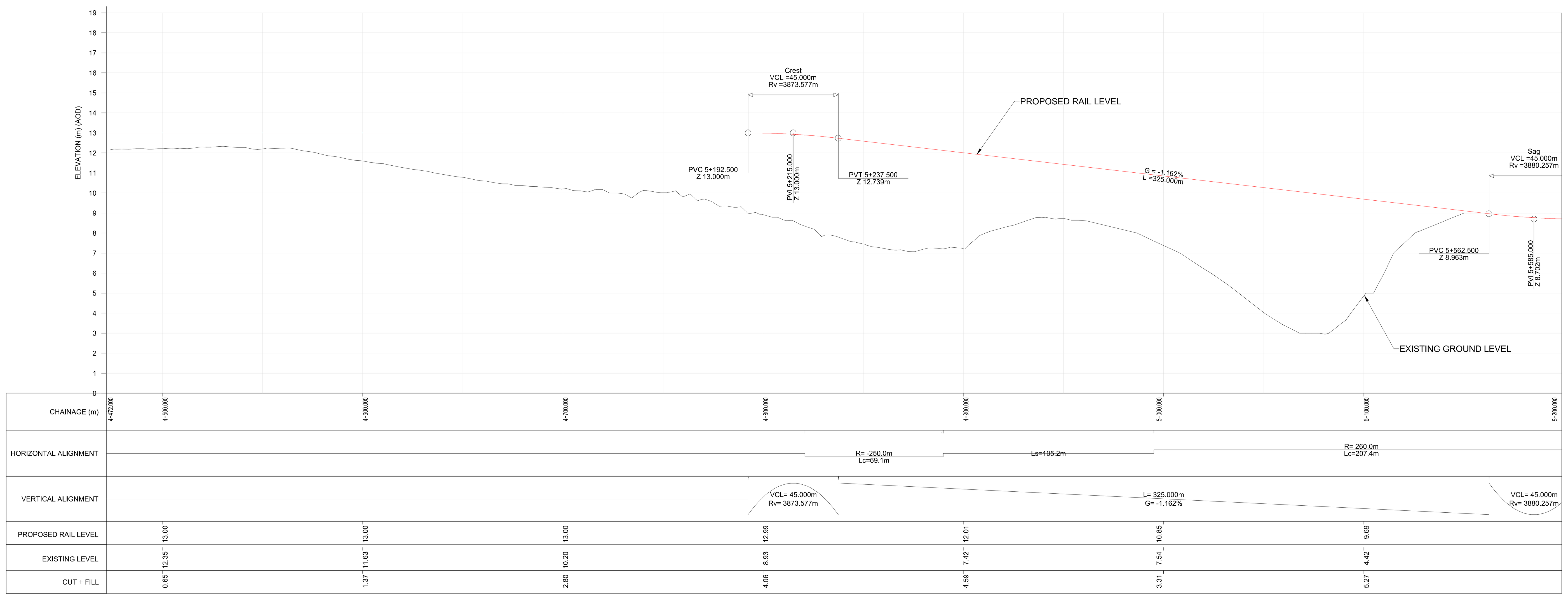
- RETAINED VEGETATION
- EARTHWORKS - IN CUTTING
- EARTHWORKS - EMBANKMENT
- DEVELOPMENT SITE BOUNDARY



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PROJECT:
SIZEWELL C

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

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TEMPORARY CONSTRUCTION AREA
PROPOSED GENERAL ARRANGEMENT
SHEET 3 OF 4

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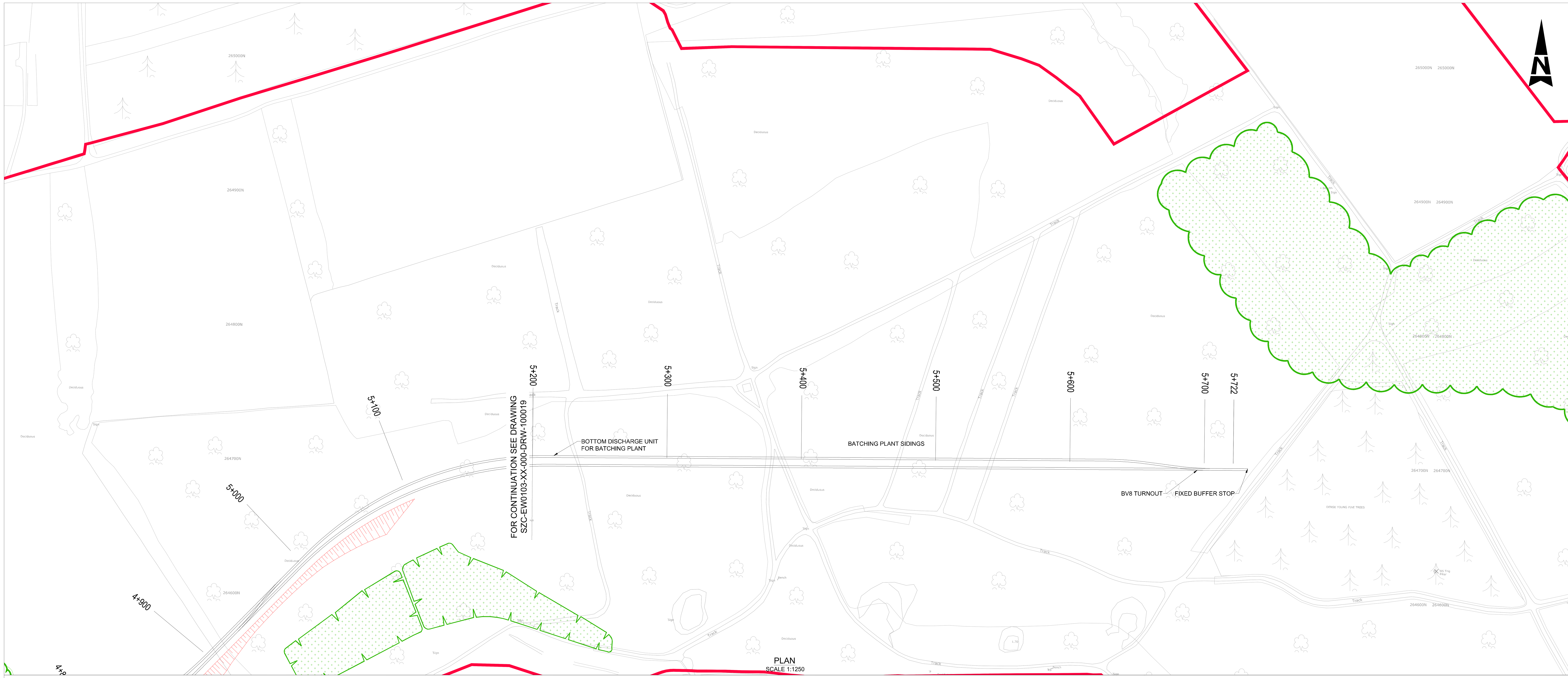

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NOTES:

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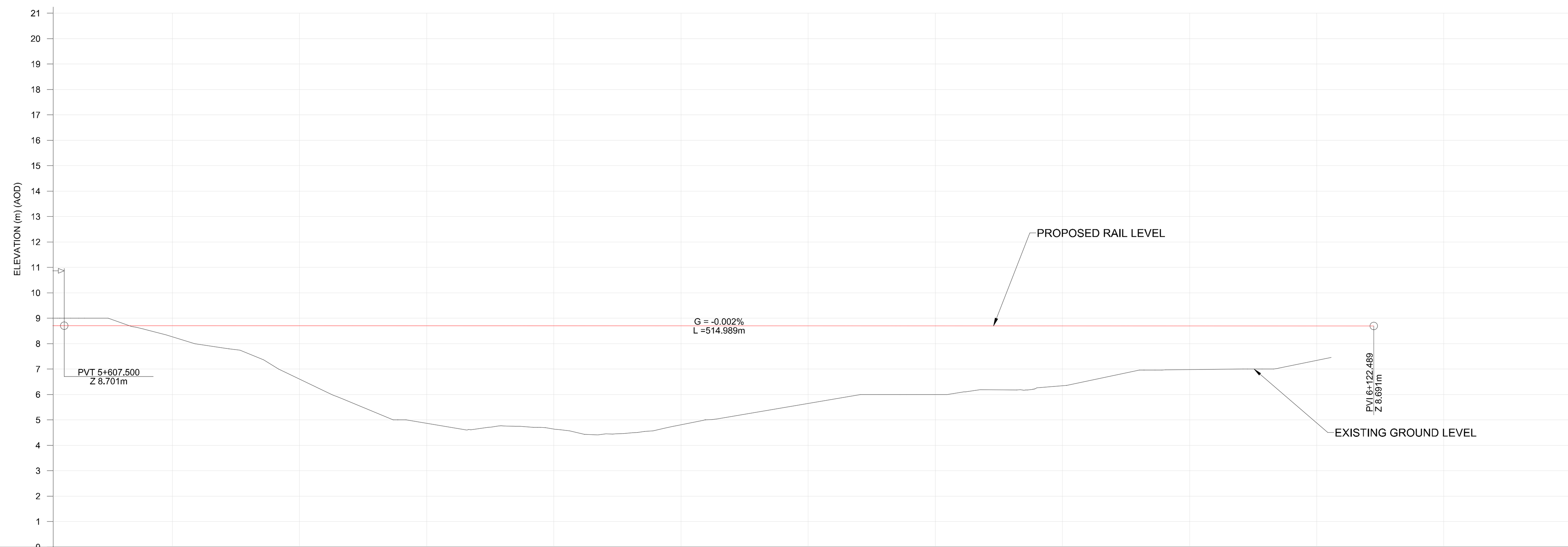
   - R = RADIUS
   - Z = ELEVATION

KEY:

- PROPOSED RAIL TRACKS
- RETAINED VEGETATION
- EARTHWORKS - IN CUTTING
- EARTHWORKS - EMBANKMENT
- DEVELOPMENT SITE BOUNDARY



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VERTICAL ALIGNMENT	L= 514.989m G= -0.002%						
PROPOSED RAIL LEVEL		8.70	8.70	8.70	8.69	8.69	
EXISTING LEVEL		6.66	4.38	5.66	6.61	7.31	
CUT + FILL		2.04	4.32	3.04	2.08	1.38	

VERTICAL PROFILE
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01	FEB 2020	PB	DW	DCO SUBMISSION	SZC CO
REVISION	DATE	DRAWN	CHECKED	REASONS FOR REVISION / COMMENTS	APPROVED

NOT PROTECTIVELY MARKED

EDF ENERGY SZC CGN

PROJECT:
SIZEWELL C

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

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JAN 2020	PB	1:1250 @ A0

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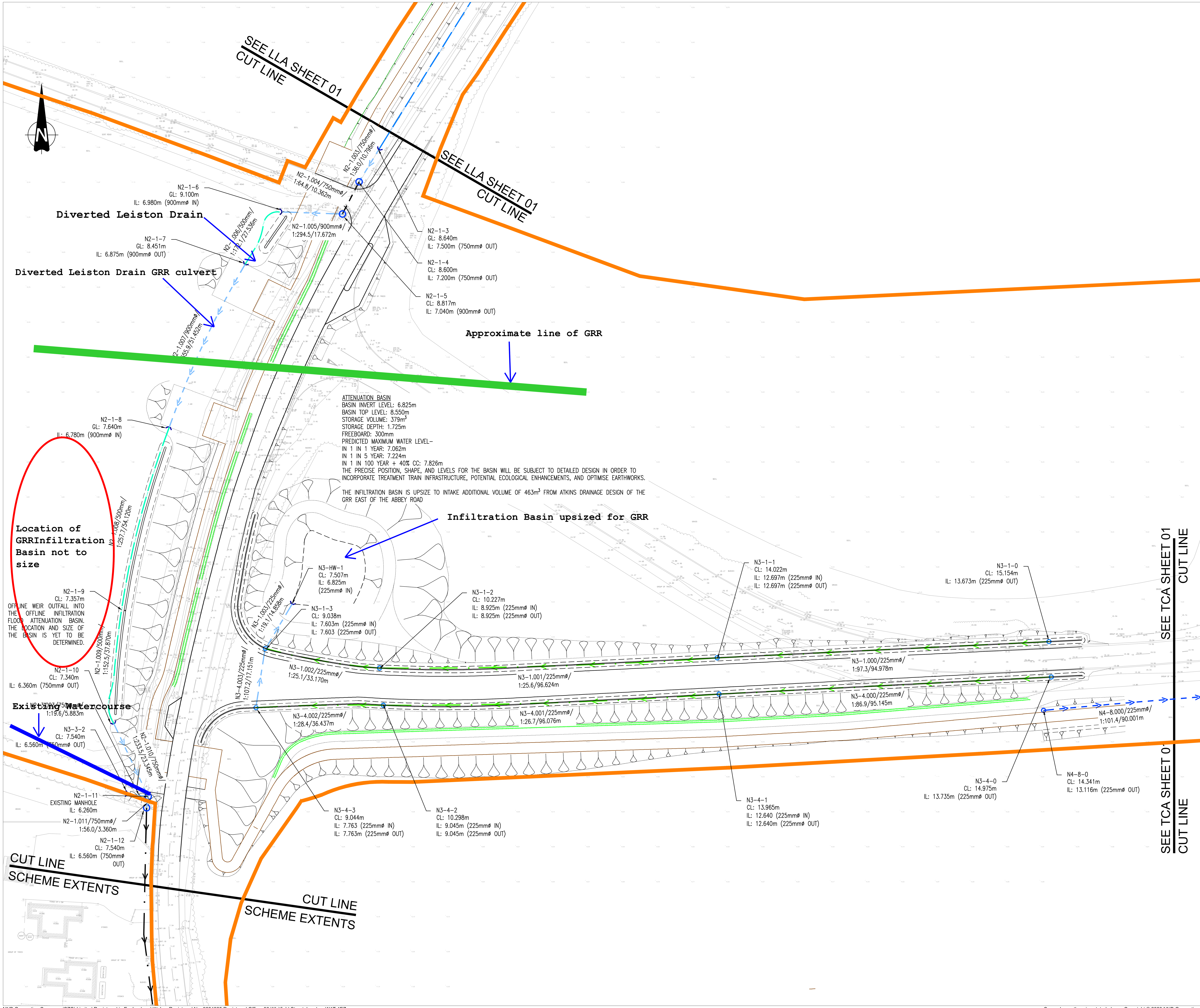
APPENDIX C: GREEN RAIL ROUTE LEISTON DRAIN CROSSING



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DRAWING GRID / COORDINATE SYSTEM:

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

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

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. THE PROPOSED ROADSIDE SWALES IS TO BE 200 mm DEEP, 200 mm BASE WIDTH WITH 1:4 SIDE SLOPES. THE FILTER PIPES OF ATLEAST 225 mm DIA. SHALL BE PROPOSED AT THE DEPTH OF AT LEAST 1 m DEPTH UNDER THE ROADSIDE SWALES
8. THE PROPOSED EARTHWORKS DITCHES IS TO BE 500 mm DEEP, 500 mm BASE WIDTH WITH 1:3 SIDE SLOPES.
9. POLLUTION CONTROL MEASURES TO BE ADDED AT THE DETAILED DESIGN STAGE FOLLOWING RECOMMENDATIONS OF HEWRAT ASSESSMENT.
10. DRAINAGE PIPE AND MANHOLE REFERENCE RELATE TO THOSE CONTAINED IN HYDRAULIC MODEL.
11. 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 FILTER DRAIN
- PROPOSED EARTHWORKS
- DITCH
- EXISTING UNCHANGED WATERCOURSE
- EXISTING DIVERTED WATERCOURSE
- PROPOSED UNDERDRAIN PIPE OF ROADSIDE SWALES
- EXISTING CULVERT
- EXISTING SW NETWORK
- PROPOSED CATCHPIT/MANHOLES
- PROPOSED TYPE 10 GRATING CHAMBERS
- PROPOSED HYDROBRAKE
- PROPOSED SOAKAWAY MANHOLE
- PROPOSED COMBINED KERB DRAINAGE
- PROPOSED MAMMAL CHANNELS
- PROPOSED GULLY
- PROPOSED HEADWALL
- PROPOSED CELLULAR STORAGE
- PROPOSED INFILTRATION/ATTENUATION BASIN

REV.	DATE	PREPARED BY	CHECKED BY	STATUS	REASONS FOR REVISION	APPROVED BY
P01	16/03/21	JS	DL	S2	First Revision	BS

NNB GenCo (SZC) LTD.	1st partner: NNBGEN	2nd partner: EDF ENERGY

CONTRACTOR COMPANY TRADE NAME: WSP

CONTRACTOR REF. No: SZC-AD0600-WSP-LLAHDG-ZZ0000-DRW-HCD-305002

CONTRACT NUMBER: SZC-AD0600

CONTRACTOR WBS CODE: N/A — QRA RELATED Yes ☐ No X

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

NUCL/REP/EPR/UKX
SZC (doc: SZ)

0	1	2	9
X			

BUILDING: 000
SYSTEM: N/A

SCALE: 1:500
SIZE: A1
PAGE

DESCRIPTION:
PRELIMINARY DESIGN
DRAINAGE WORKS
GENERAL ARRANGEMENT SHEET 2

TEAMCENTER DOCUMENT REFERENCE No.
TBC — P01

DOCUMENT SUB -TYPE: TEMPLATE	EDF CLASSIFICATION CODE: N/A
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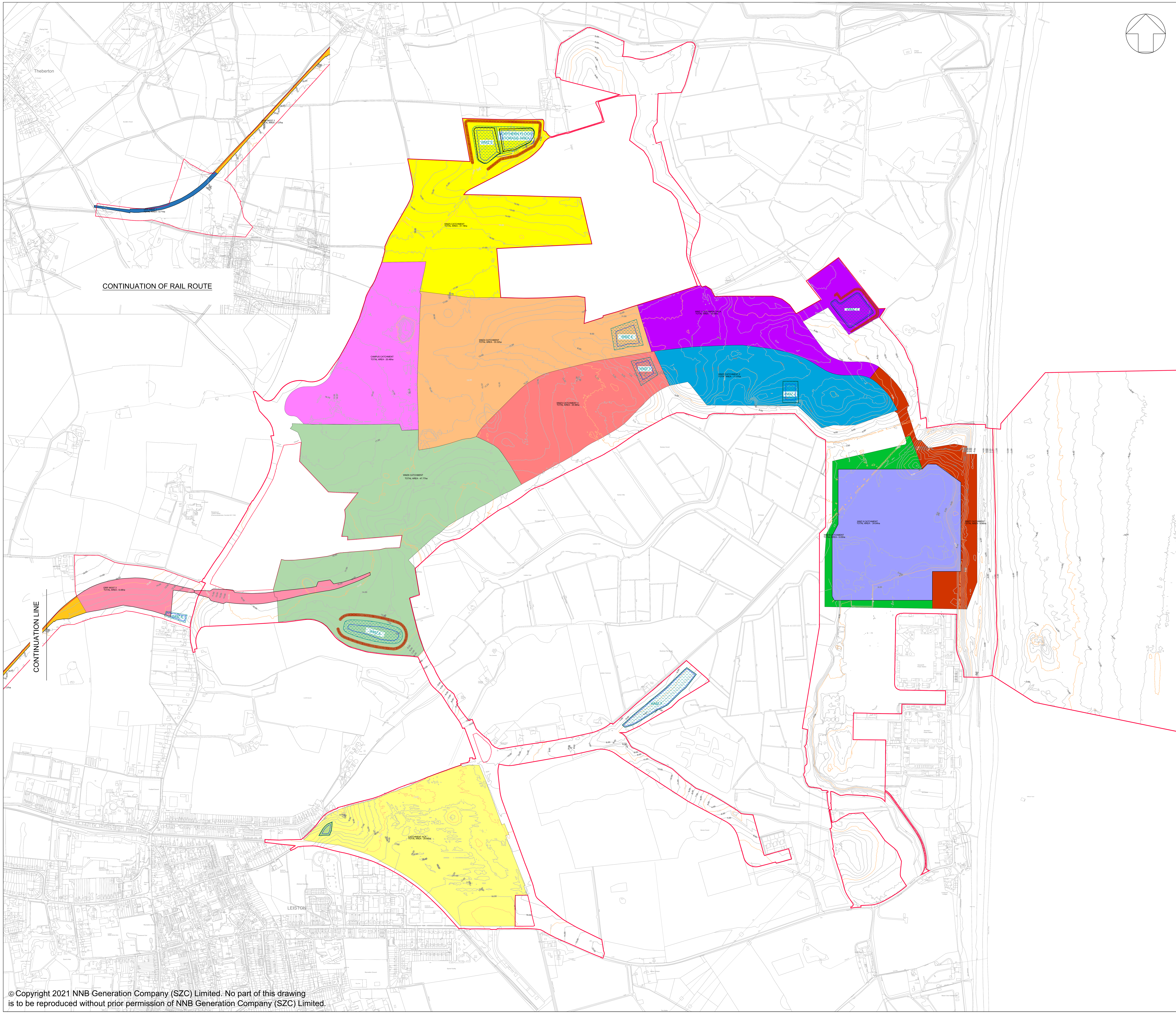
ANNEX 2A.13: COMPARISON OF MDS BASELINE TOPOGRAPHY AND WMZ CATCHMENTS



NOTES:

KEY:

NOT PROTECTIVELY MARKED

SizewellC
Doing the power of good for Britain

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

DRAWING TITLE:
SIZEWELL C - TOPOGRAPHICAL PLAN WITH WMZ OVERLAY

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ANNEX 2A.14: TEMPORARY MARINE OUTFALL OPERATION SUMMARY

CONTENTS

FIGURES

1 INTRODUCTION

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

1.2 Background

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

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

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

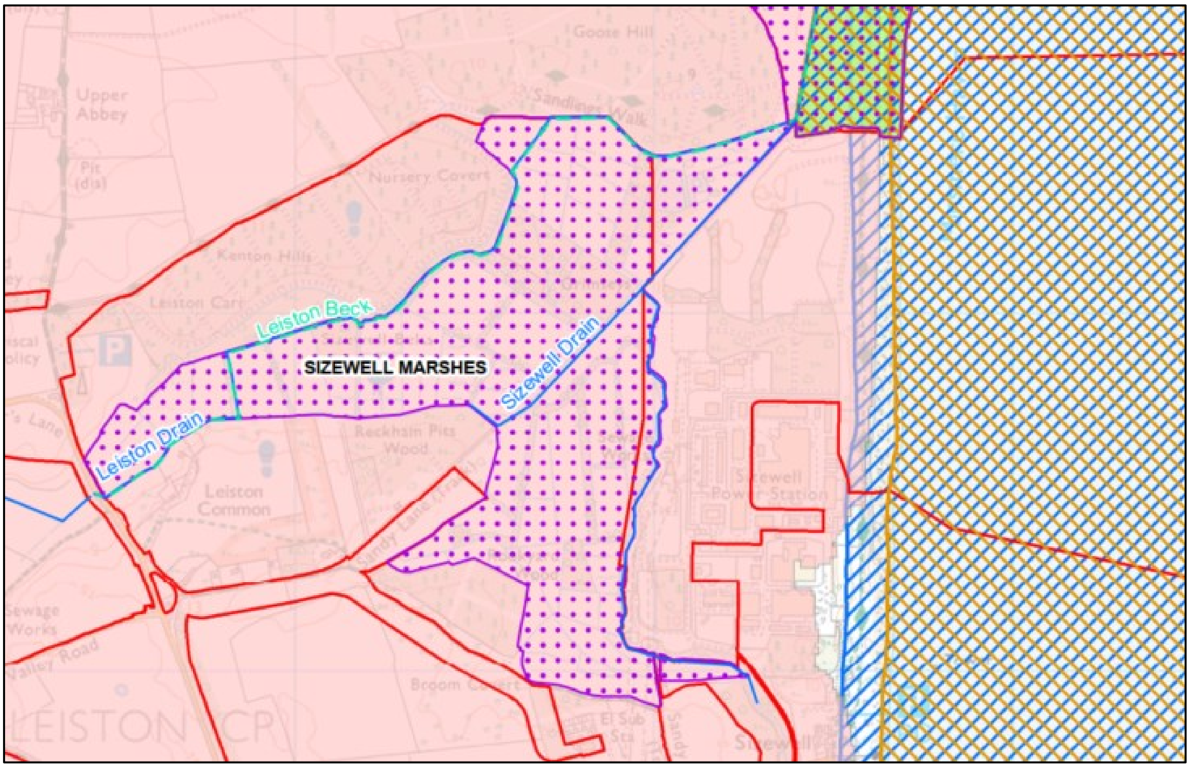
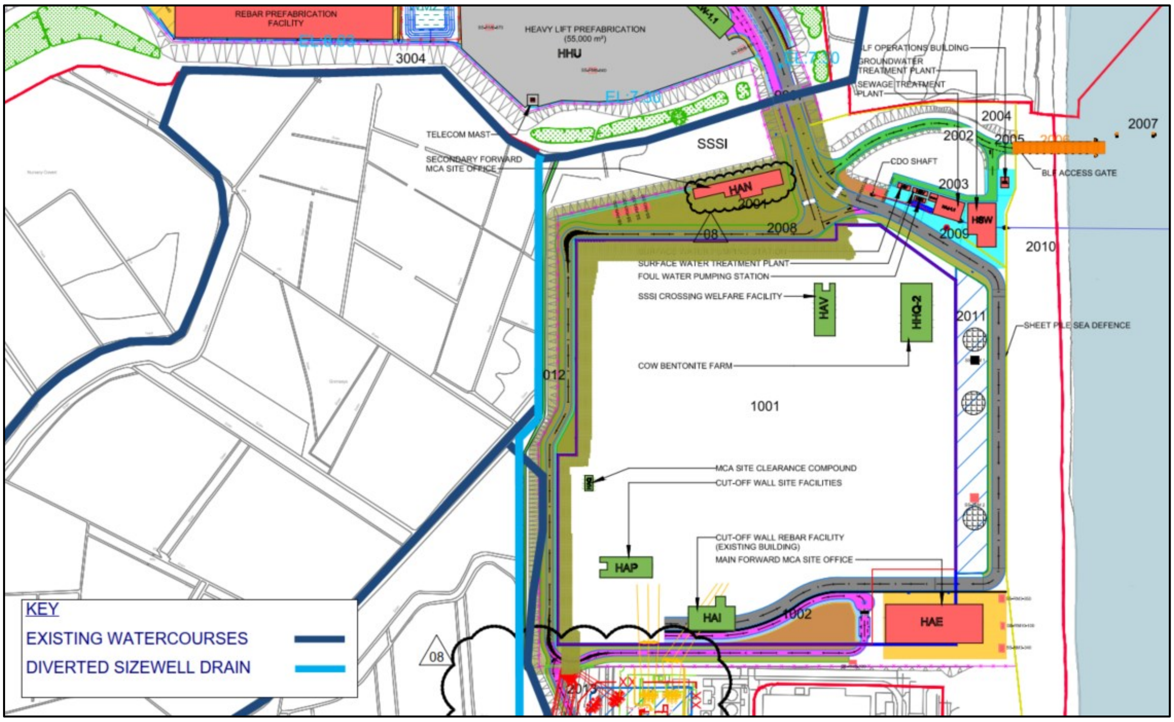


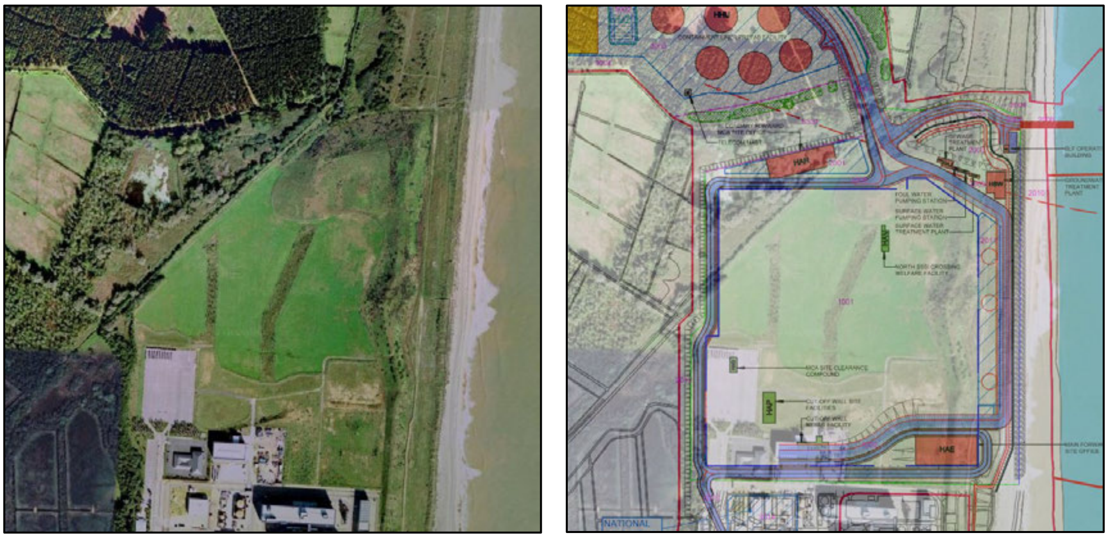
Figure 1-2 - Indicative Surface Watercourses – Existing and Diverted Sizewell Drain



1.3 Existing Site

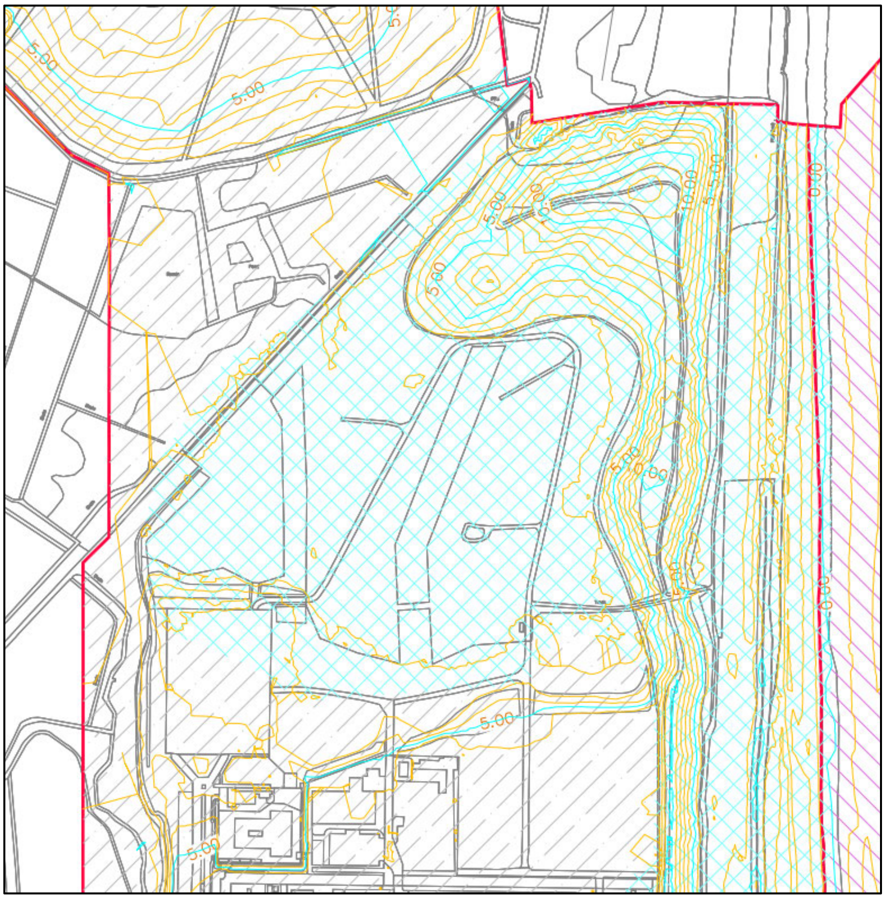
1.3.1 The existing site is largely grassland. The section of the MCA to the north-west of the Sizewell Drain makes up part of the SSSI. The area to the south and east of the Sizewell Drain includes grassland as well as some buildings and hardstanding. Figure 1-3 shows an aerial photograph of the existing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

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



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

Figure 1-4 - Existing Site Contours



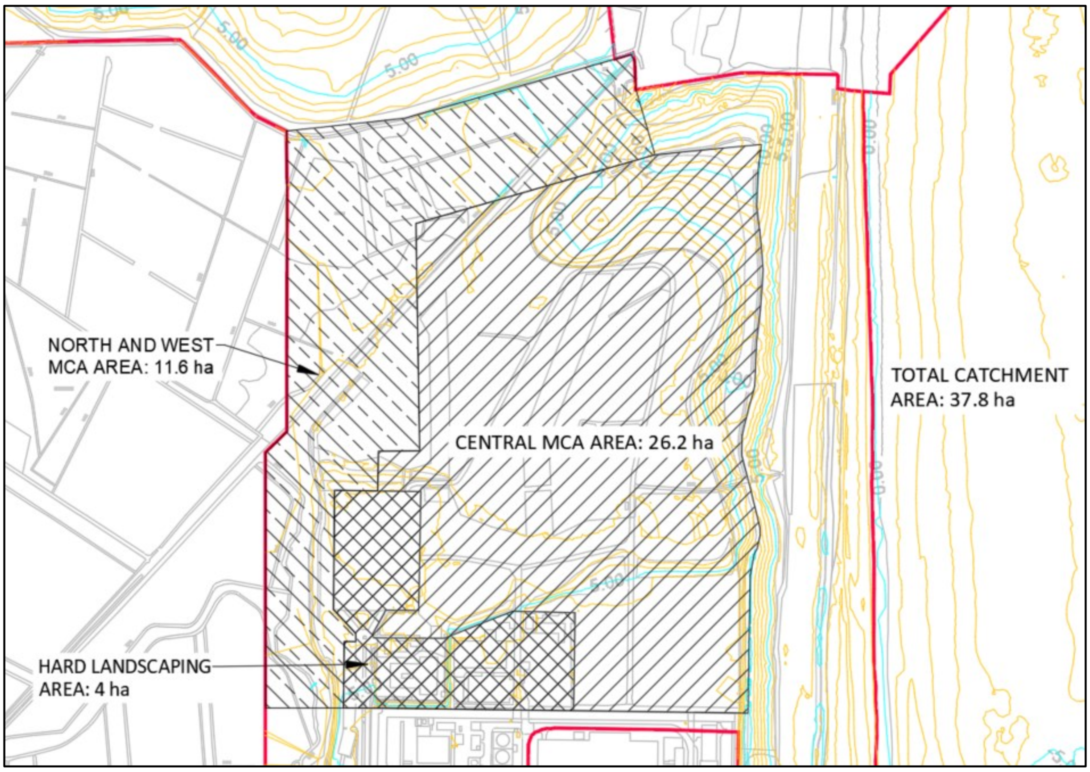
2 SURFACE WATER MANAGEMENT

2.1 Overview

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

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

Figure 2-1 - Approximate surface water catchment area



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

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

2.2 Surface Water Discharge

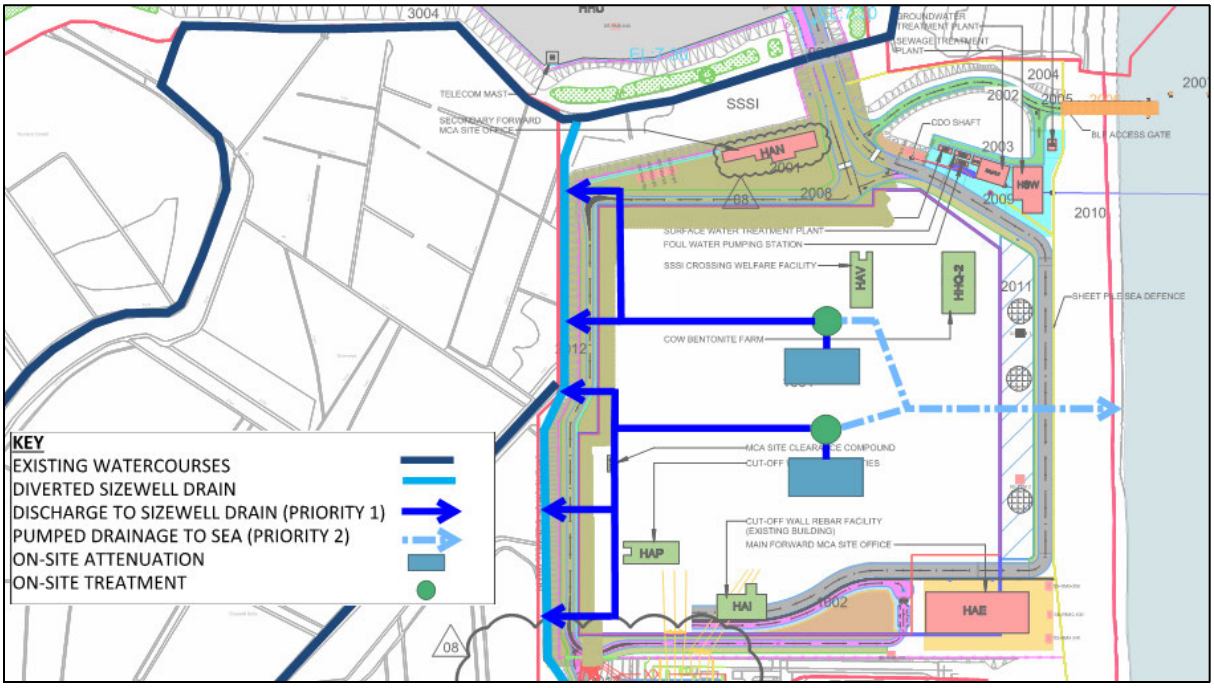
2.2.1 It is important to mimic existing conditions on the site to ensure the SSSI water levels, including ground water, are not affected by the construction works. This means surface water will discharge primarily via infiltration, using temporary infiltration ponds and other Sustainable Drainage (SuDS)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

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

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

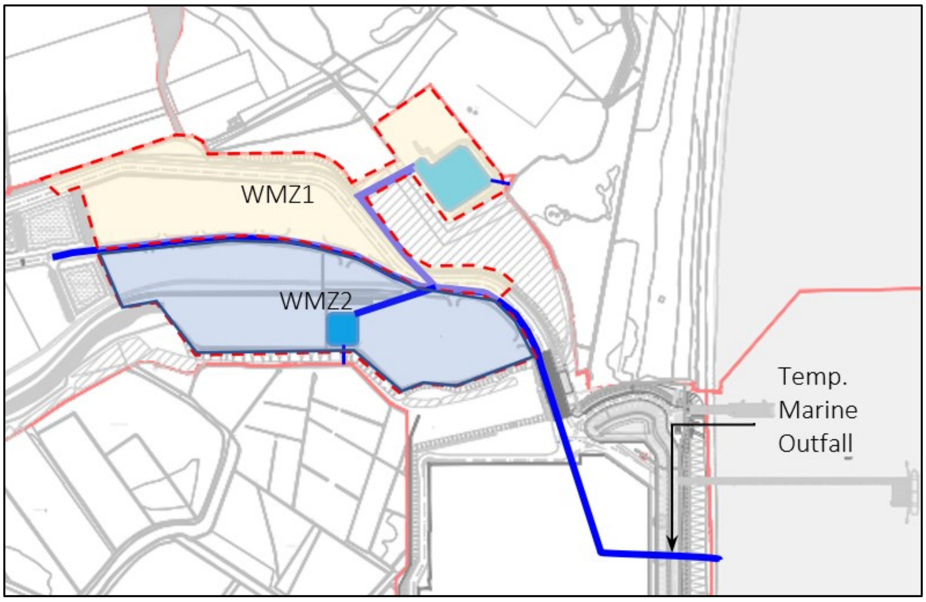
2.2.4 The Sizewell Marshes (including the Sizewell Drain) are known to flood occasionally due to either extreme rainfall events or other external factors, such as the Leiston Drain downstream being blocked or the Minsmere sluice inhibiting surface water flow to sea. In these scenarios, the Sizewell drain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

Figure 2-2 - Schematic showing proposed discharges



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

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



2.3 Temporary Marine Outfall

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

2.3.2 Permitting, Operation, and Usage

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

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

2.3.5 **Proposed Arrangement**

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

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

Figure 2-4 - Proposed outfall location

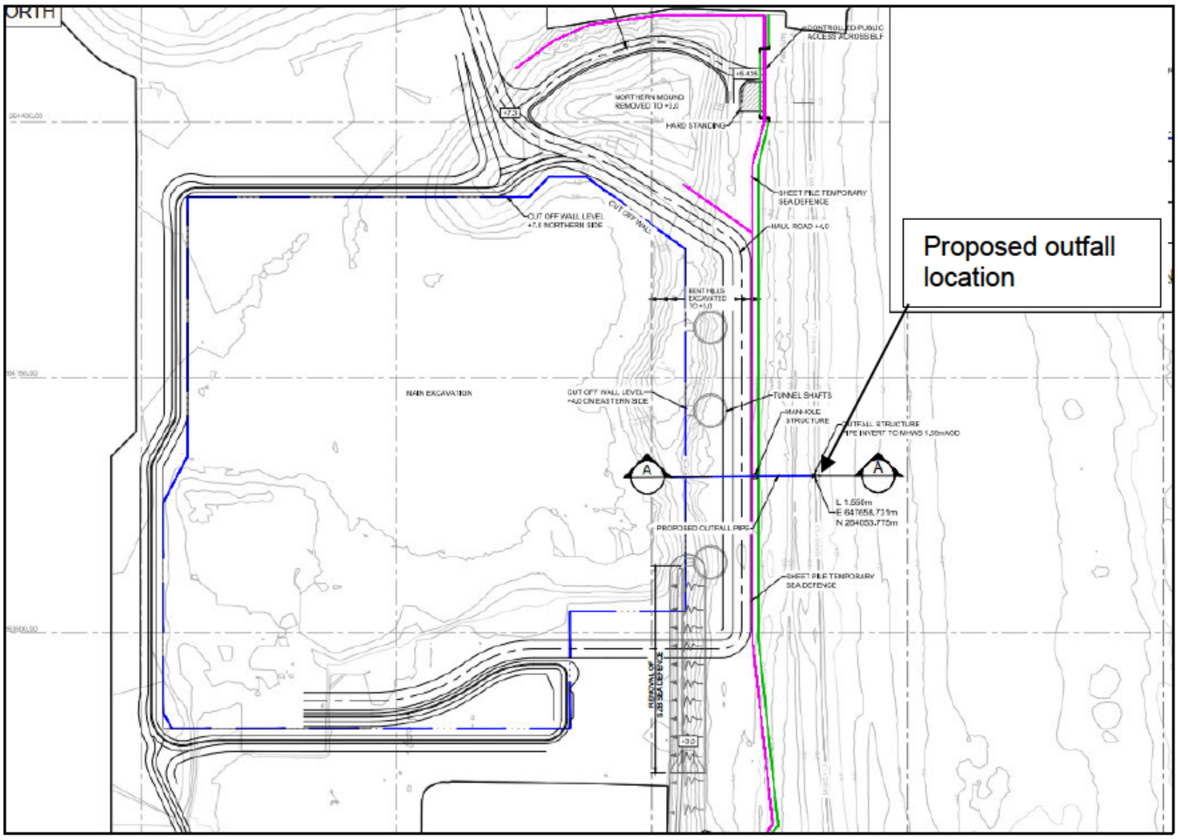
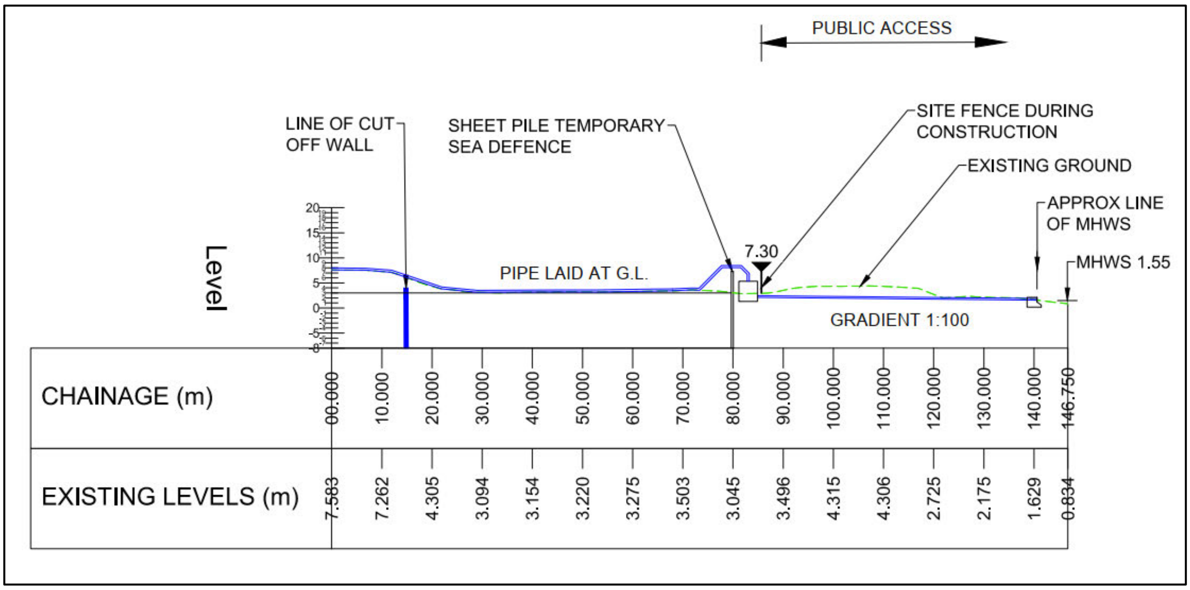


Figure 2-5 - Indicative section discharge pipe from MCA to outfall



3 SUMMARY

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

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

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

ANNEX 2A.15: WMZ1 SURFACE WATER TREATMENT ASSESSMENT

CONTRACTOR DOCUMENT FRONT SHEET

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DOCUMENT DETAILS

PROJECT		ORIGINATOR CODE		VOLUME		LOCATION		TYPE		ROLE		SEQUENTIAL NUMBER
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DOCUMENT STATUS	S3	DOCUMENT PURPOSE	S3 - Fit for Internal Review and Comment	TOTAL PAGES (Including this page)	22

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ATKINS NUMBER	N/A	CONTRACTOR REVISION	01

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REVISION STATUS/SUMMARY OF CHANGES

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Technical Note

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Subject:	**WMZ1 Surface Water Treatment Assessment**
Author:	DH

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PW Revision	Status	Purpose description	Originated	Checked	Reviewed	Authorised	Date
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Client signoff

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

Glossary

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

1. Introduction

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

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

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

2. WMZ 1 Catchment Overview

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

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

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

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

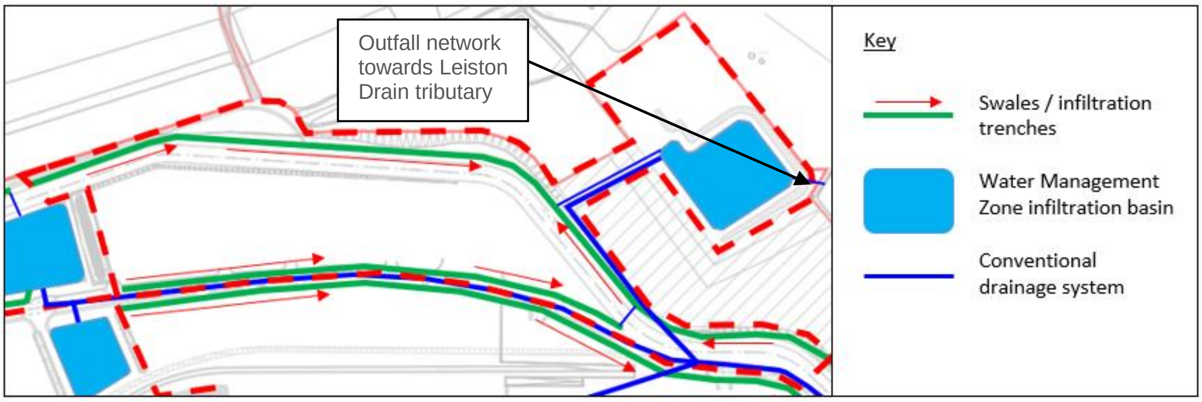


Figure 2-1 – WMZ1 surface water drainage overview

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

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

3. Surface Water Treatment Design

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

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

3.1. Sediment Laden Runoff

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

The following SuDS features are proposed within WMZ 1:

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

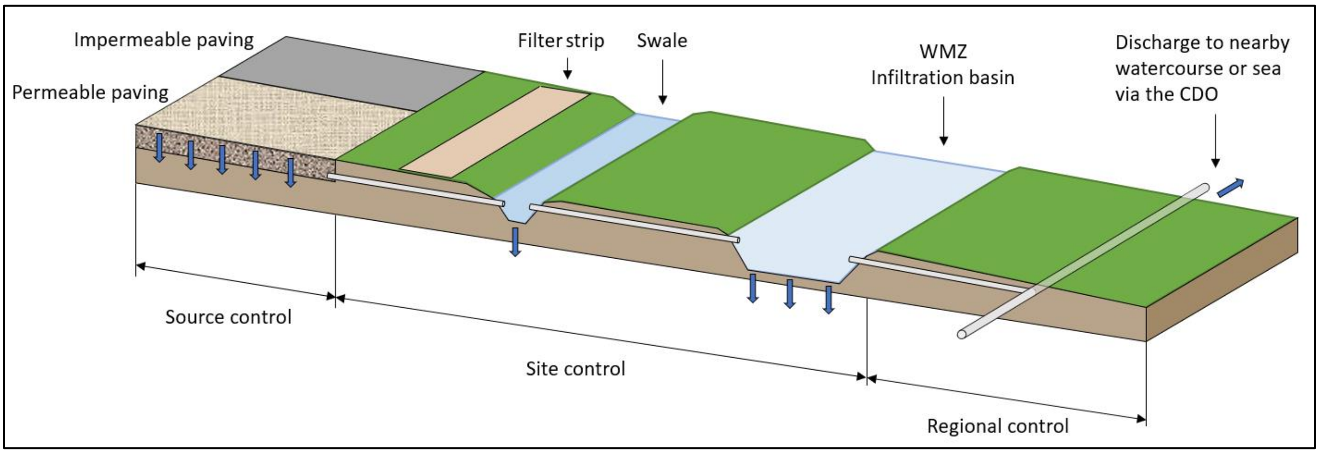


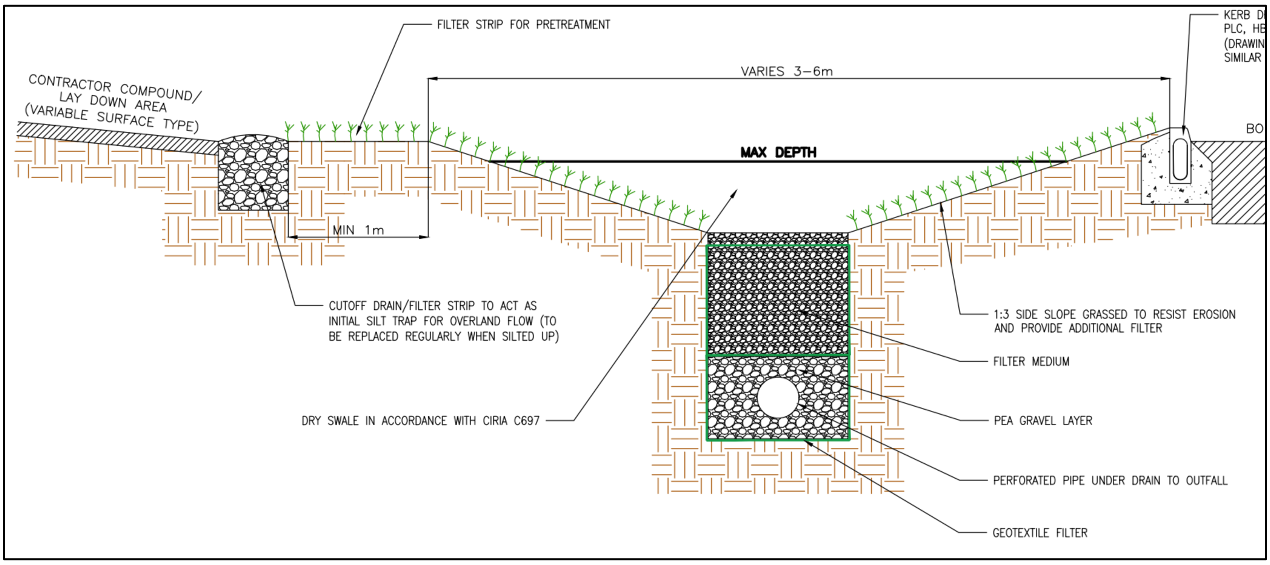
Figure 3-1 - Surface water capture, treatment, and discharge plan

3.1.1. Access Roads / Contractor Compounds

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

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

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



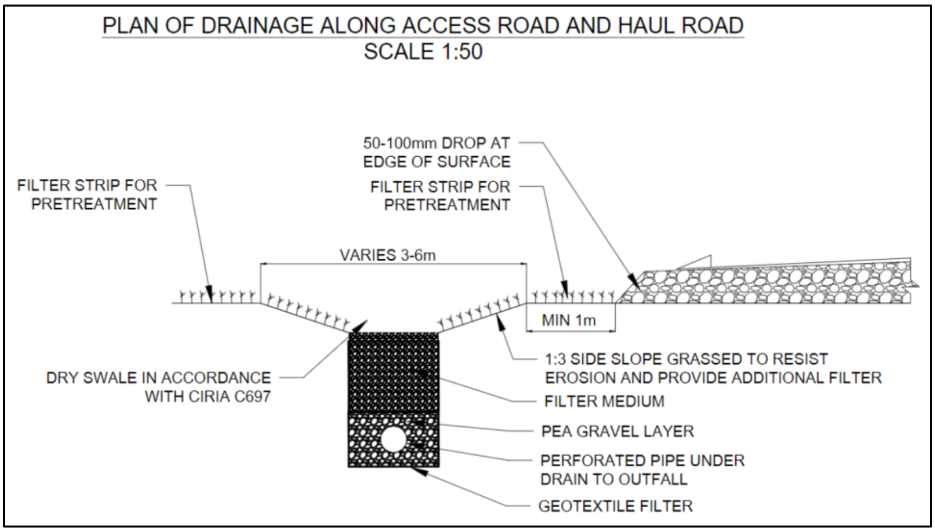


Figure 3-2 – Typical filter strip, swale and infiltration trench arrangement adjacent to Contractor Compounds and Access Roads

3.2. Oil and Chemical Spills

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

3.2.1. Fuel Farm Runoff

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

3.3. Other Treatment

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

3.3.1. TBM Slurry Treatment Plant / Bentonite Farm

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

3.3.2. Road Sweeper Compound

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

4. Treatment Assessment

4.1. Simple Index Approach (SIA)

As set out in Section 26.7.1 of the CIRIA C753 The SuDS Manual, the Simple Index Approach (SIA) is used to assess water quality risk management and ultimately determine what SuDS measures are required to treat different types of developments. This section presents the assessment of WMZ 1 using the SIA. The steps are set out as:

Step 1 – Allocate suitable pollution hazard indices for the proposed land use
Step 2 – Select SuDS with a total pollution mitigation index that equals or exceeds the pollution hazard index
Step 3 – Where the discharge is to protected surface waters or groundwater, consider the need for a more precautionary approach

Given the intended use of the WMZ 1 site we can allocate pollution indices (Table 26.2 CIRIA C753 The SuDS Manual) as one of the two categories, shown in Table 4-1.

Table 4-1 – Pollution hazard indices for different land use classifications

Land Use	Pollution hazard levels	Total suspended solids (TSS)	Metals	Hydrocarbons
Commercial yard and delivery areas, non-residential car parking with frequent change (e.g. hospitals, retail), all roads except low traffic roads and trunk roads/motorways.	Medium	0.7	0.6	0.7
Sites with heavy pollution (e.g. haulage yards, lorry parks, highly frequented lorry approaches to industrial estates, waste sites), sites where chemicals and fuels (other than domestic fuel oil) are to be delivered, handled, stored, used or manufactured; industrial sites; trunk roads and motorways.	High	0.8	0.8	0.9

Table 4-2 summarises the intended use of each site within WMZ1 and the assigned pollution hazard level.

Table 4-2 – WMZ1 pollution hazard indices assigned

WMZ 1 Area	Description of proposed use	Assigned pollution hazard levels
Haul Road	Used for transporting material (for earthworks) between the TCA and MCA	High
Main Access Road	Used as main access for vehicles from TCA to the MCA	High
Workshop compound	Compound containing workshops for metal, formwork and joinery	Medium
Plant Workshop & Storage	Storage and maintenance area for plant	Medium
Fuel Farm	Facility used to supply different fuel types to support the construction plant	High*
Road Sweeper Compound	Dedicated area to dispose of sediment removed by road sweepers from site roads	High
Fire & Rescue Centre	Operations space with welfare and a covered area for emergency vehicles and equipment	Medium
Emergency Response Facility	Operations space with welfare and a covered area for emergency vehicles and equipment	Medium

* *Runoff treatment to be supplemented by proprietary components, e.g. oil interceptor prior to conveyance through drainage network*

Indicative SuDS pollution mitigation indices for discharge to surface waters are stated in Table 26.3 of the CIRIA C753 The SuDS Manual, copied into Table 4-3.

Table 4-3 – Indicative SuDS mitigation indices for discharges to surface waters

	Mitigation indices		
Type of SuDS component	TSS	Metals	Hydrocarbons
Filter strip	0.4	0.4	0.5
Filter drain	0.4	0.4	0.4
Swales	0.5	0.6	0.6
Bioretention system	0.8	0.8	0.8
Permeable pavement	0.7	0.6	0.7
Detention basin	0.5	0.5	0.6
Pond	0.7	0.7	0.5
Wetland	0.8	0.8	0.8

Indicative SuDS pollution mitigation indices for discharge to groundwater are stated in Table 26.4 of the CIRIA C753 The SuDS Manual, copied into Table 4-4.

Table 4-4 - Indicative SuDS mitigation indices for discharges to groundwater

	Mitigation indices		
Type of SuDS component	TSS	Metals	Hydrocarbons
A layer of dense vegetation underlain by a soil with good contaminant attenuation potential of at least 300 mm in depth	0.6	0.5	0.6
A soil with good contaminant potential of at least 300 mm in depth	0.4	0.3	0.3
Infiltration trench (where a suitable depth of filtration material is included that provides treatment, i.e. graded gravel with sufficient smaller particles but single size coarse aggregate such as 20 mm gravel) underlain by a soil with good contaminant attenuation potential of at least 300 mm in depth	0.4	0.4	0.4
Constructed permeable pavement (where a suitable filtration layer is included that provides treatment, and including a geotextile at the base separating the foundation from the subgrade) underlain by a soil with good contaminant attenuation potential of at least 300 mm in depth	0.7	0.6	0.7
Bioretention underlain by a soil with good contaminant attenuation potential of at least 300 mm in depth	0.8	0.8	0.8

Assessing the proposed SuDS features for each of the WMZ 1 areas we can determine a total pollution mitigation index (Table 26.3 & 26.4 CIRIA C753 The SuDS Manual) for each. To fully assess the pathways for which surface water runoff may undertake from point of capture to discharge, three pathways have been identified:

1. **Pathway 1** – surface water runoff passes through a filter strip before entering a swale, where water infiltrates into the underlying trench and discharged to ground.
2. **Pathway 2** – surface water runoff passes through a filter strip before entering a swale and conveyed to WMZ 1 basin, where water is discharged to the ground.
3. **Pathway 3** – surface water runoff passes through a filter strip before entering a swale and conveyed to WMZ 1 basin, where water is discharged to a surface water (outfall O1 or CDO).

This assessment is based on the current proposals and will be updated during the next design phase to reflect additional SuDS elements. Where additional SuDS features are not considered appropriate at this design stage, proprietary features are proposed in these areas and is discussed in Section 4.2.1. Appendix B presents the summary table from the SIA tool for the worst-case scenario (i.e. high pollution hazard level, poorest groundwater discharge index) for each discharge pathway.

As per the SuDS Manual, where more than one SuDS component is proposed in series, the total mitigation index is given by the sum of the mitigation index for the primary component plus half of the mitigation index for additional components. A factor of 0.5 is used to account for the reduced performance of secondary or tertiary components associated with already reduced inflow conditions. Where the total aggregated mitigation index is greater than 1, then the outcome is fixed at ">0.95", to suggest the proposed components are likely to have a very high potential for reducing pollutant levels in the runoff and should be sufficient for any proposed land use.

Table 4-5 – Pathway 1: SuDS mitigation indices for discharges to groundwater at source (via swales)

WMZ 1 area	Assigned pollution hazard levels	SuDS features proposed	Total SuDS mitigation index		
			TSS	Metals	Hydrocarbons
Haul Road	High	- Filter strip - Swale	0.85 (>0.8)	0.9 (>0.8)	>0.95 (>0.9)

Main Access Road	High	- Filter strip - Swale	0.85 (>0.8)	0.9 (>0.8)	>0.95 (>0.9)
Workshop compound	Medium	- Filter strip - Swale	0.85 (>0.7)	0.9 (>0.6)	>0.95 (>0.7)
Fuel Farm	High	- None	N/A (<0.8)*	N/A (<0.8)*	N/A (<0.9)*
Plant Workshop & Storage	Medium	- Filter strip - Swale	0.85 (>0.7)	0.9 (>0.6)	>0.95 (>0.7)
Road Sweeper Compound	High	- Filter strip - Swale	0.85 (>0.8)	0.9 (>0.8)	>0.95 (>0.9)
Fire & Rescue Centre	Medium	- Filter strip - Swale	0.85 (>0.7)	0.9 (>0.6)	>0.95 (>0.7)
Emergency Response Facility	Medium	- Filter strip - Swale	0.85 (>0.7)	0.9 (>0.6)	>0.95 (>0.7)

** Primary treatment to be provided by proprietary components (e.g. oil interceptor). Performance to be discussed and agreed with LLFA. Captured runoff likely to be conveyed to WMZ1 basin, and not expected to discharge at source hence not applicable.*

Table 4-6 – Pathway 2: SuDS mitigation indices for discharges to groundwater via basin

WMZ 1 area	Assigned pollution hazard levels	SuDS features proposed	Total SuDS mitigation Index		
			TSS	Metals	Hydrocarbons
Haul Road	High	- Filter strip - Swale - Basin	>0.95 (>0.8)	>0.95 (>0.8)	>0.95 (>0.9)
Main Access Road	High	- Filter strip - Swale - Basin	>0.95 (>0.8)	>0.95 (>0.8)	>0.95 (>0.9)
Workshop compound	Medium	- Filter strip - Swale - Basin	>0.95 (>0.7)	>0.95 (>0.6)	>0.95 (>0.7)
Fuel Farm	High	- Basin	0.7 (<0.8)*	0.65 (<0.8)*	0.75 (<0.9)*
Plant Workshop & Storage	Medium	- Filter strip - Swale - Basin	>0.95 (>0.7)	>0.95 (>0.6)	>0.95 (>0.7)
Road Sweeper Compound	High	- Filter strip - Swale - Basin	>0.95 (>0.8)	>0.95 (>0.8)	>0.95 (>0.9)
Fire & Rescue Centre	Medium	- Filter strip - Swale - Basin	>0.95 (>0.7)	>0.95 (>0.6)	>0.95 (>0.7)
Emergency Response Facility	Medium	- Filter strip - Swale - Basin	>0.95 (>0.7)	>0.95 (>0.6)	>0.95 (>0.7)

** Primary treatment to be provided by proprietary components (e.g. oil interceptor). Performance to be discussed and agreed with LLFA.*

Table 4-7 – Pathway 3: SuDS mitigation indices for discharges to surface waters

WMZ 1 area	Assigned pollution hazard levels	SuDS features proposed	Total SuDS mitigation Index		
			TSS	Metals	Hydrocarbons
Haul Road	High	- Filter strip - Swale - Basin	0.9 (>0.8)	0.95 (>0.8)	>0.95 (>0.9)
Main Access Road	High	- Filter strip - Swale - Basin	0.9 (>0.8)	0.95 (>0.8)	>0.95 (>0.9)
Workshop compound	Medium	- Filter strip - Swale - Basin	0.9 (>0.7)	0.95 (>0.6)	>0.95 (>0.7)
Fuel Farm	High	- Basin	0.5 (<0.8)*	0.5 (<0.8)*	0.6 (<0.9)*
Plant Workshop & Storage	Medium	- Filter strip - Swale - Basin	0.9 (>0.7)	0.95 (>0.6)	>0.95 (>0.7)
Road Sweeper Compound	High	- Filter strip - Swale - Basin	0.9 (>0.8)	0.95 (>0.8)	>0.95 (>0.9)
Fire & Rescue Centre	Medium	- Filter strip - Swale - Basin	0.9 (>0.7)	0.95 (>0.6)	>0.95 (>0.7)
Emergency Response Facility	Medium	- Filter strip - Swale - Basin	0.9 (>0.7)	0.95 (>0.6)	>0.95 (>0.7)

** Primary treatment to be provided by proprietary components (e.g. oil interceptor). Performance to be discussed and agreed with LLFA.*

4.2. Summary of Assessment

The SIA approach uses the worst-case pollutant and mitigation indices, and so is a conservative assessment on the treatment potential of a drainage design. The assessment demonstrates that treatment from SuDS alone can provide effective capture and treatment of pollutants from surface water runoff, with the exception of the Fuel Farm area. As stated above, proprietary components are proposed to further reduce the risk of untreated water for this area.

Filter strips and grassed verges act as the initial level of treatment to trap sediment for all pathways within WMZ 1, protecting the longevity and effectiveness of the swales.

Pathway 1, where treatment is provided through discharge to groundwater via the swales, produces the worst-case mitigation indices, yet still demonstrates sufficient treatment potential.

The inclusion of Pathway 2, discharge to groundwater from the WMZ 1 basin, is the best-case treatment train, however, further design development will be required on raising the invert level of the WMZ 1 basin to enable infiltration. Pathway 3 also meets all treatment requirements, although direct discharge to surface waters may be subject to more stringent water quality requirements (Section 6).

4.2.1. Proprietary drainage methods water quality risk management

Where traditional drainage systems are proposed, features such as silt traps and catch pits in all gullies and manholes, and where required separators will be implemented to manage water quality risks from TSS, metals and hydrocarbons. The concreted Fuel Farm area is proposed to be bunded and drain through an oil separator to mitigate the risk of hydrocarbon spillage during refuelling and long-term parking of Heavy Goods Vehicles. The design will comply with the initiatives and best practice guidance for pollution prevention for impermeable areas. At this stage, this is deemed to be the most practicable solution. The benefit of capturing potential oil spills and treating them in a proprietary device of specific performance requirements is that risk of pollution to downstream sensitive areas is greatly reduced. Proprietary drainage features also provide a level of certainty from past performance that is not immediately offered by vegetative solutions in the early months (sometimes 12 to 18 months) as they establish.

In places, such systems may be used a fail-safe method of treatment to supplement primary treatment observed using SuDS techniques and will be explored in future design stages.

5. Maintenance

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

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

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

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

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

6. Surface Water Quality

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

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

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

Criteria	Treatment Level Required at Monitoring Point	Sample Type	Notes
Visible oil or grease	No significant trace present so far as is reasonably practicable	Visual inspection	No significant trace

Suspended Solids (measured after drying at 105°C)	60mg/l (to local watercourse) 250mg/l (to sea)	Spot sample	Maximum Allowable Concentration (MAC)
pH	6 to 9	Spot sample	Minimum and maximum

7. Monitoring and Sampling

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

Table 7-1 – Proposed monitoring manhole location

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

8. Conclusion

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

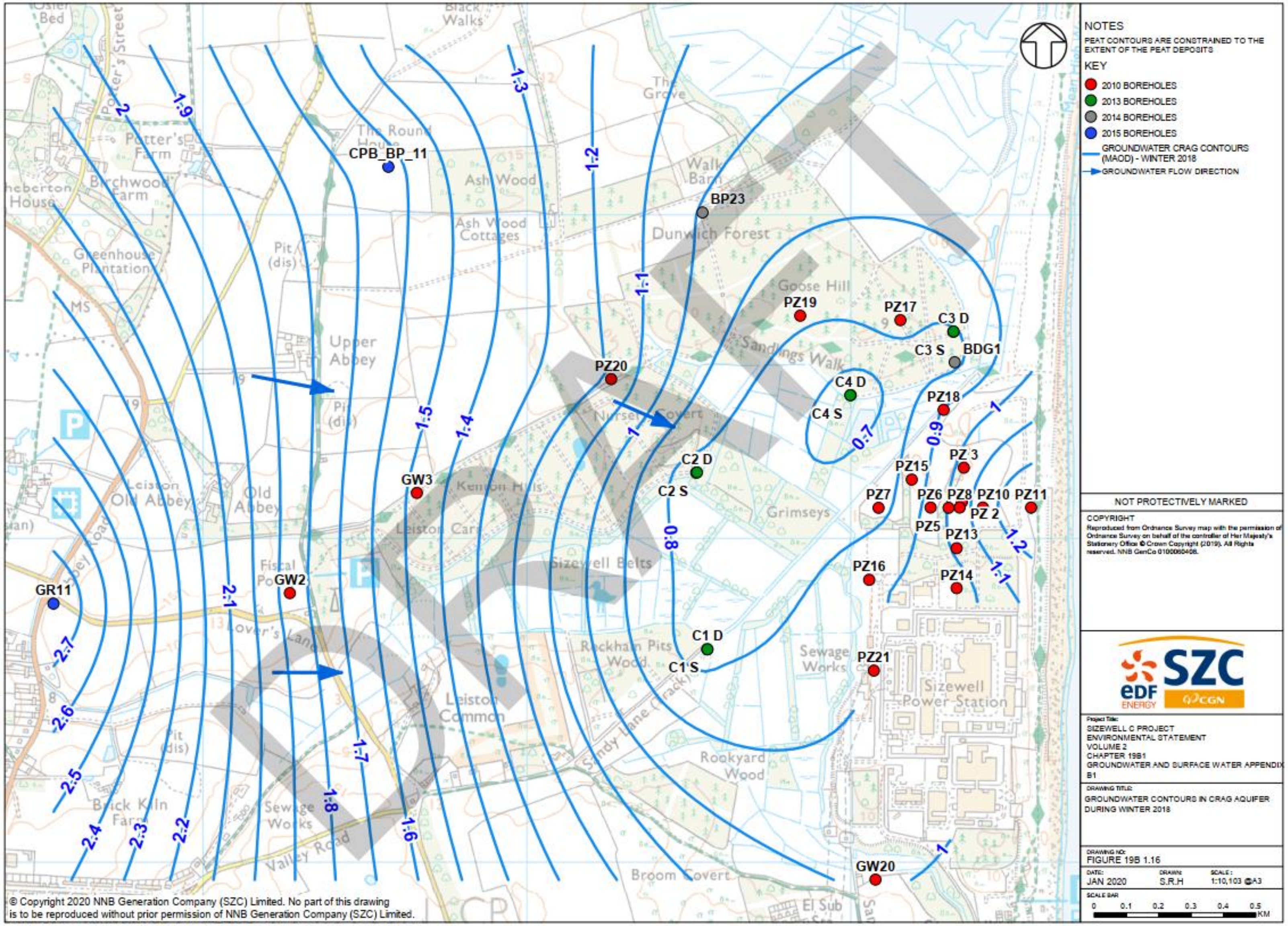
There are a number of forward actions associated with further assessment of treatment of WMZ 1 that will be undertaken at the next design stage:

- Update to the design of WMZ 1 basin to allow for infiltration when further ground investigation campaign infiltration data is received.
- Assessment of water treatment to be completed for remaining WMZs across the MDS.
- Development of proprietary drainage methods across contractor compounds and access/haul roads, building on lessons from HPC.
- Confirm water quality requirements with CEFAS and the EA.

9. Appendices

Appendix A. Groundwater Levels

A.1. Groundwater Contour Drawing


NOTES
PEAT CONTOURS ARE CONSTRAINED TO THE EXTENT OF THE PEAT DEPOSITS
KEY
2010 BOREHOLES
2013 BOREHOLES
2014 BOREHOLES
2015 BOREHOLES
GROUNDWATER CRAG CONTOURS (MAOD) - WINTER 2018
GROUNDWATER FLOW DIRECTION
CPB_BP_11
BP23
PZ19
PZ17
C3 D
C3 S
BDG1
PZ20
C4 D
C4 S
PZ18
C2 D
C2 S
GW3
PZ15
PZ 3
PZ7
PZ6
PZ8
PZ10
PZ11
PZ 2
PZ5
PZ13
PZ16
PZ14
GR11
GW2
C1 D
C1 S
PZ21
GW20
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Project Title:
SIZEWELL C PROJECT
ENVIRONMENTAL STATEMENT
VOLUME 2
CHAPTER 19B1
GROUNDWATER AND SURFACE WATER APPENDIX B1
DRAWING TITLE:
GROUNDWATER CONTOURS IN CRAG AQUIFER DURING WINTER 2018
DRAWING NO:
FIGURE 19B 1.16
DATE: JAN 2020
DRAWN: S.R.H
SCALE: 1:10,103 @A3
SCALE BAR
0 0.1 0.2 0.3 0.4 0.5 KM
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Appendix B. SIA Assessment

Items B.1, B.2 and B.3 are the summary tables from the SIA tool from CIRIA C753 The SuDS Manual for each discharge pathway considered. For each pathway, the worst-case scenario is shown e.g. ‘High’ pollution hazard level and least favourable groundwater protection type, where infiltration is proposed.

B.1. SIA Summary – Pathway 1

Pathway 1 - Surface water runoff passes through a filter strip before entering a swale, where water is discharged to the ground via infiltration through an infiltration trench.

SUMMARY TABLE	
Land Use Type	Other industrial site area
Pollution Hazard Level	High
Pollution Hazard Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.9
SuDS components proposed	
Component 1	Filter strip
Component 2	Swale
Component 3	None
SuDS Pollution Mitigation Indices	
TSS	0.65
Metals	0.7
Hydrocarbons	0.8
Groundwater protection type	Infiltration trench with suitable depth of filtration material underlain by 300 mm minimum depth of soils with good contamination attenuation potential
Groundwater protection Pollution Mitigation Indices	
TSS	0.4
Metals	0.4
Hydrocarbons	0.4
Combined Pollution Mitigation Indices	
TSS	0.85
Metals	0.9
Hydrocarbons	>0.95
Acceptability of Pollution Mitigation	
TSS	Sufficient
Metals	Sufficient
Hydrocarbons	Sufficient

B.2. SIA Summary – Pathway 2

Pathway 2 - Surface water runoff passes through a filter strip before entering a swale and conveyed to WMZ 1 basin, where water is discharged to the ground via infiltration at the basin.

SUMMARY TABLE	
Land Use Type	Other industrial site area
Pollution Hazard Level	High
Pollution Hazard Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.9
SuDS components proposed	
Component 1	Filter strip
Component 2	Swale
Component 3	Detention basin
SuDS Pollution Mitigation Indices	
TSS	0.9
Metals	0.95
Hydrocarbons	>0.95
Groundwater protection type	300 mm minimum depth of soils with good contamination attenuation potential
Groundwater protection Pollution Mitigation Indices	
TSS	0.4
Metals	0.3
Hydrocarbons	0.3
Combined Pollution Mitigation Indices	
TSS	>0.95
Metals	>0.95
Hydrocarbons	>0.95
Acceptability of Pollution Mitigation	
TSS	Sufficient
Metals	Sufficient
Hydrocarbons	Sufficient

B.3. SIA Summary – Pathway 3

Pathway 3 - surface water runoff passes through a filter strip before entering a swale and conveyed to WMZ 1 basin, where water is discharged to a surface water (outfall O1 or CDO).

SUMMARY TABLE	
Land Use Type	Other industrial site area
Pollution Hazard Level	High
Pollution Hazard Indices	
TSS	0.8
Metals	0.8
Hydrocarbons	0.9
SuDS components proposed	
Component 1	Filter strip
Component 2	Swale
Component 3	Detention basin
SuDS Pollution Mitigation Indices	
TSS	0.9
Metals	0.95
Hydrocarbons	>0.95
Groundwater protection type	None
Groundwater protection Pollution Mitigation Indices	
TSS	0
Metals	0
Hydrocarbons	0
Combined Pollution Mitigation Indices	
TSS	0.9
Metals	0.95
Hydrocarbons	>0.95
Acceptability of Pollution Mitigation	
TSS	Sufficient
Metals	Sufficient
Hydrocarbons	Sufficient

ANNEX 2A.16: REVIEW OF EXISTING INFILTRATION AND PERMEABILITY TEST DATA

CONTRACTOR DOCUMENT FRONT SHEET

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DOCUMENT DETAILS

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NNB NUMBER		TEAMCENTER NUMBER	

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01	26/02/21	RM	Engineering Geologist	ES	Civil Engineer	AL	Area Lead

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Technical Note

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Author:	RM		
Date:		Project No.:	5199744
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01	S3	For Issue	RM	ES	AP	AL	26/02/21

Client signoff

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

1. Introduction

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

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

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

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

2. Summary of Existing Data

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

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

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

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

3. Review of Existing Data

3.1. Review Procedure

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

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

Table 3-1 – Explanation of Confidence Categories

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

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

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

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

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

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

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

Table 3-2 – Review of infiltration test data

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
2020, (2021, Fugro)	WMZ3_2020-3-TP-A	Trial Pit	BRE 365	3	3.40 x 0.70 x 2.30; 0.30	Slightly gravelly slightly silty SAND	Gravel infill to 0.30m bgl to support unstable pit. Well screen not used.	1.34E-06, 1.32E-05, 1.16E-05	1	Test durations do not allow infiltration rate to level off or pit to fully empty. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rates for Tests 2 and 3 were not calculated correctly following BRE 365.
2020, (2020, Fugro)	ACA_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 2.90; 0.45/0.50/0.50	SAND and GRAVEL	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commenced 0.50m bgl. Standing water was noted at 2.90m bgl (to 3.50m) before test started; this standing water level is taken as the test base depth.	3.34E-6 3.31E-6 5.43E-6	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth appear approximately correct. Infiltration rates were not calculated correctly following BRE 365 due to durations of tests. Note gravel at slightly lower depth than depth to start of test in Test 1, so gravel fraction % may require review.
2020, (2020, Fugro)	ACA_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.50	SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	5.61E-5 2.18E-5 1.48E-5	3	Testing undertaken in borehole. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	CAMPUS_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.40/0.50/0.50	Gravelly SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	7.99E-6 9.99E-6 6.14E-6	3 (Note: Category of 1 for Test 2, but of little significance as would use lowest value [Test 3] for design)	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth appear approximately correct. Infiltration rate for Test 2 was not calculated correctly following BRE 365 due to duration of test. Note gravel at slightly lower depth than depth to start of test in Test 1, so gravel fraction % may require review.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
2020, (2020, Fugro)	CAMPUS_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 2.74; 0.48/0.44/0.46	Slightly gravelly sandy CLAY	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commenced 0.50m bgl. Standing water was noted at 2.74m bgl (to 3.50m) before test started; this standing water level is taken as the test base depth.	1.17E-5 1.85E-5 1.36E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rates for all tests were not calculated correctly following BRE 365. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review.
2020, (2020, Fugro)	WMZ1_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.25; 0.25	SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	2.01E-4 1.69E-4 1.24E-4	2	Testing undertaken in borehole. Note gravel at lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	WMZ1_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.40; 0.45	SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	1.95E-4 5.73E-5 5.39E-5	3	Testing undertaken in borehole. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	WMZ1_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 1.20; 0.45	SAND with pockets of clay	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl. Standing water was noted at 1.20m bgl (to 3.50m) before test started; this standing water level is	1.28E-4 1.20E-4 1.14E-4	3	Testing undertaken in borehole. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, testing and calculations in general accordance with BRE 365.

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

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
							the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.			
2020, (2020, Fugro)	WMZ3_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.22/0.25/0.25	Slightly gravelly SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	7.02E-5 3.85E-5 3.34E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Note gravel at lower depth than depth to start of test, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ3_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 1.62; 0.60	SAND and GRAVEL	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.60m bgl. Standing water was noted at 1.62m bgl (to 2.75m) before test started; this standing water level is taken as the test base depth.	1.96E-5 1.48E-5 1.80E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ4_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.50	SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	1.30E-4 5.63E-5 3.83E-5	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ4_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.56/0.45/0.20	Gravelly SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled	5.64E-5 3.86E-5 3.11E-5	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
							space. Gravel filter commences at 0.50m bgl.			Note gravel at lower depth than depth to start of test in Tests 2 and 3, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ4_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.46/0.45/0.10	SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	2.27E-4 1.30E-4 8.14E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Note gravel at lower depth than depth to start of test, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ5_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.56; 0.95/0.97/0.95	SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 1.00m bgl.	4.23E-4 3.17E-4 2.28E-4	3	Testing undertaken in borehole. Effective depth calculated approximately correctly following BRE 365. Note gravel at slightly lower depth than depth to start of Tests 2 and 3, so gravel fraction % may require review. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ5_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 2.93; 0.60/0.40/0.40	Slightly gravelly SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.40m bgl.	3.29E-5 2.08E-5 1.71E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ5_2020-3	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.50/0.27/0.46	Slightly gravelly SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	7.22E-6 7.09E-6 6.00E-6	1	Testing undertaken in borehole. Test durations do not allow infiltration rate to level off or hole to fully empty. In absence of longer test duration, calculations of effective depth do not appear correct. Note gravel at lower depth than depth to start of test in Tests 2 and 3, so gravel fraction % may require review.

GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by GI Contractor – Tests 1, 2 and 3 (lowest highlighted) [m/s]	Confidence Category (not in conjunction with other results)	Comments on Confidence Category
										Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ6_2020-1	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.40; 0.70	SAND and GRAVEL	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.70m bgl.	5.41E-4 1.77E-4 7.99E-5	3	Testing undertaken in borehole. Aside from that, testing and calculations in general accordance with BRE 365.
2020, (2020, Fugro)	WMZ6_2020-2	Borehole	WSP Technical Note 1 (2019) – Based on BRE 365	3	0.30 x 0.30 (circular) x 3.50; 0.66/0.73/0.61	Slightly gravelly SAND	Test carried out inside 225mm well screen in gravel filled borehole. Volume of gravel fraction assumed to be 57.62% of the total volume of gravel filled space. Gravel filter commences at 0.50m bgl.	1.89E-5 1.59E-5 1.62E-5	2	Testing undertaken in borehole. Effective depth for all 3 tests does not appear to have been calculated correctly following BRE 365. Infiltration rate calculations use the correct equation in accordance with BRE 365.
2020, (2020, Fugro)	WMZ6_2020-2-PIT	Trial Pit	BRE 365	3	1.10 x 0.60 x 1.30; 0.54/0.57/0.54	Slightly gravelly SAND	Test carried out inside 50mm slotted pipe in gravel filled pit. Gravel infill to 0.50m bgl.	1.82E-05, 1.09E-05, 5.58E-06	1	Test durations do not allow infiltration rate to level off or pit to fully empty. In absence of longer test duration, calculations of effective depth appear correct. Infiltration rates for Tests 2 and 3 were not calculated correctly following BRE 365 due to durations of tests.
2020, (2020, Fugro)	WMZ6_2020-2-IP-A	Inspection Pit	BRE 365	3	0.40 x 0.40 x 1.30; 0.43/0.44/0.44	Slightly gravelly SAND	Test carried out inside 225mm slotted casing in gravel filled pit. Gravel infill to 0.50m bgl.	1.42E-05, 1.05E-05, 1.01E-05	3	Testing in general accordance with BRE 365, except for test pit length. Note gravel at slightly lower depth than depth to start of test, so gravel fraction % may require review. Aside from that, calculations of effective depth and infiltration rates appear correct.
2017 (2017, SSL)	TP-WMZ-21	Trial Pit	BRE 365	2	1.00 x 0.60 x 2.40; 1.00/0.00	Slightly gravelly sandy CLAY with low cobble content, over slightly silty gravelly SAND (boundary at 0.50m)		<Test 1 not provided>, 7.76E-6	1	Two tests carried out at different depths, and no third test. Test durations do not allow infiltration rate to level off or hole to fully empty. Effective depth does not appear to have been calculated correctly following BRE 365.

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

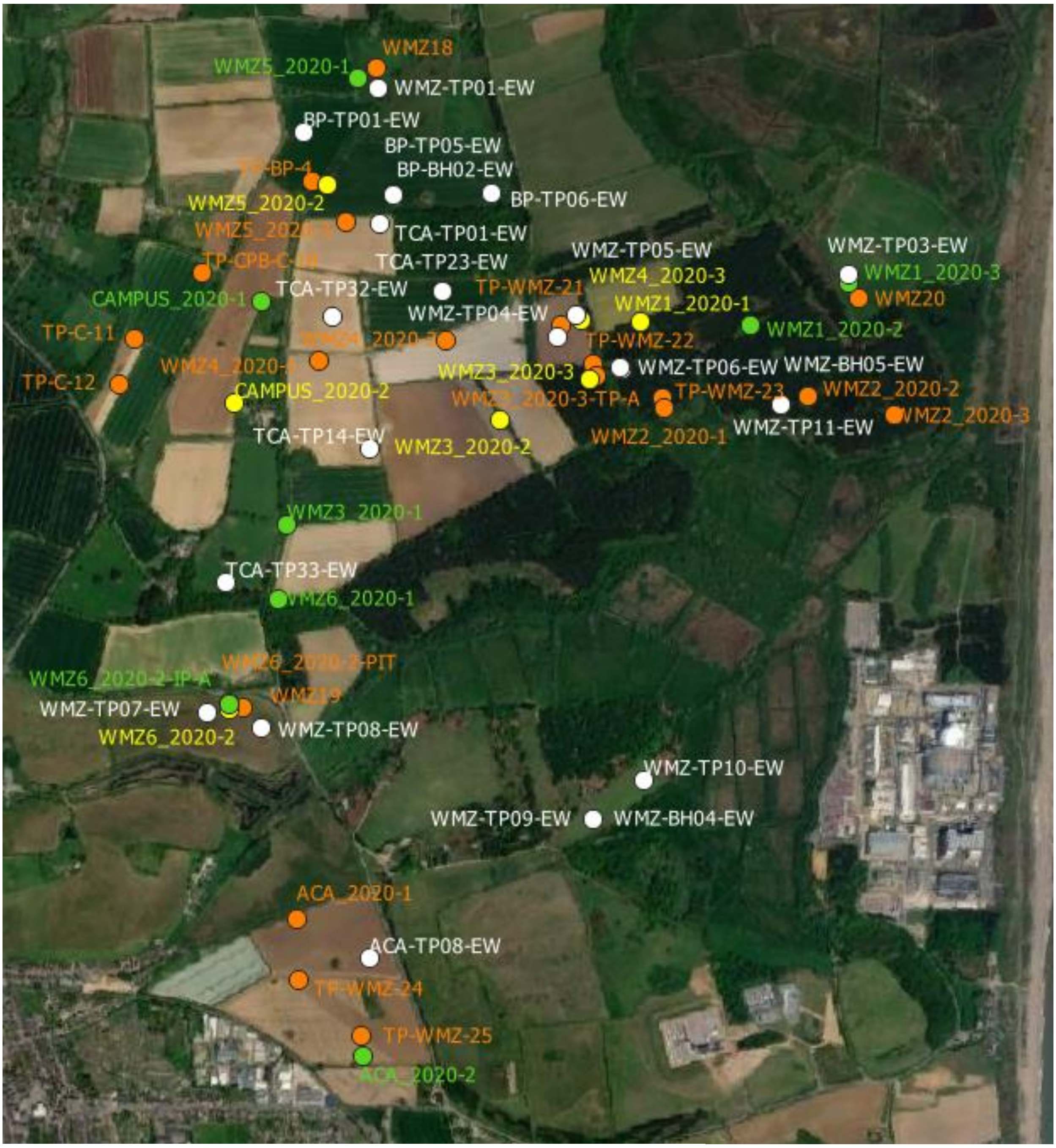
GI year (Factual Report year of issue and GI Contractor)	Exploratory Hole ID	Exploratory hole type	Standard used, according to Factual Report	Number of tests undertaken	Test dimensions – length x breadth x depth; test top depth [m x m x m; m bgl]	Test geology	Test notes	Infiltration rate calculated by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

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

Table 3-3 – Summary of permeability test data

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

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



- Proposed Infiltration & Permeability Tests in Upcoming TCA & ACA GI
- Infiltration Test_Confidence Category 4
- Infiltration Test_Confidence Category 3
- Infiltration Test_Confidence Category 2
- Infiltration Test_Confidence Category 1

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

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

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

3.2. Test Methodology

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

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

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

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

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

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

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

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

3.2.2. Potential non-conformance with BRE 365 standard

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

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

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

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

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

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

3.3. Literature Review

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

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

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

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

4. Recommendations for Future Works

4.1. Office-based Tasks

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

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

4.2. Testing Methodology for Future Investigations

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

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

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

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

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

5. Summary

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

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

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

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

6. References

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