Norwich Western Link

Drainage Strategy Report

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Contents

Tables

Figures

Appendices

Drainage Strategy Report Appendix 9: Ringland Lane Flood Modelling Report (4.04.09)

Drainage Strategy Report Appendix 10: Basins Options Technical Note (4.04.10)

Drainage Strategy Report Appendix 11: Topsoil for Infiltration Basins Technical Note (4.04.11)

Drainage Strategy Report Appendix 12: Landscape Amenity proposals (4.04.12)

Drainage Strategy Report Appendix 13: A47 Stub Drainage (4.04.13)

Drainage Strategy Report Appendix 14: Ground Investigation Reports (extracts) (4.04.14)

Drainage Strategy Report Appendix 15: Construction Surface Water Management Strategy (4.04.15)

Drainage Strategy Report Appendix 16: Drainage Strategy Drawings & HCD Standard Details (4.04.16)

Executive Summary

The Applicant has prepared the design of the highway and drainage for the Proposed Scheme in support of a planning application.

The surface water drainage strategy proposes a Sustainable Drainage System to manage highway surface runoff following the four pillars of SuDS. The system includes filter strips, swales, filter drains, infiltration and attenuation basins, sedimentation forebays, scrapes, planting of reeds and long grasses within the basins. There are also facilities to provide amenity for the public. Infiltration basin designs are based on the results of infiltration tests (undertaken during ground investigations in 2021 and 2022) and follow the SuDS manual and LLFA guidelines. Discharge rates from the proposed attenuation basins will be controlled to greenfield runoff rates or 2 l/s/ha, whichever is the lower, before either discharging to the local stream or neighbouring catchment by agreement with third-parties.

Surface water runoff (from upland catchments flowing along natural flow paths) will be intercepted and collected by a network of drainage ditches (pre-earthworks drainage). Runoff from the natural catchment areas will be diverted around and through the highway footprint in ditches and culverts, directed towards the nearest watercourse or local infiltration features. Upland catchment runoff will be separated from highway runoff as the latter will have pre-treatment before discharge.

Side roads intersected by the Proposed Scheme will drain to local ditches

1 Abbreviations

AOD	Above Ordinance Datum
BGL	Below ground level
CDE	Common Data Environment
DC	Document Control
DCM	Document Control Manager
DMRB	Design Manual for Roads & Bridges
EA	Environment Agency
EDMS	Electronic Data Management System
ES	Environmental Statement
FC NWL	Ferrovial Construction Norwich Western Link
FEH	Flood Estimation Handbook
FER	Ferrovial Construction (The Contractor)
FoS	Factor of Safety
FSR	Flood Studies Report
FWMA	Flood Water Management Act 2010
GI	Ground Investigation
GSWC	Grass Surface Water Channels
GW	Groundwater
IDB	Internal Drainage Board
LLFA	Lead Local Flood Authority
LFRMS	Local Flood Risk Management Strategy
MADD	a factor that the design software uses to define the amount of peripheral storage through small branch lines, gully connections and other sewerage that is not explicitly defined in the network to be analysed
MAOD	meters above ordinance datum
M/Hr	Meters per hour
M/S	Meters per second
NCC	Norfolk County Council (Client)
NDR	Northern Distributor Road
NGR	National rid Reference
NH	National Highways
NMU	Non-motorised user
NPPF	National Planning Policy Framework
NWL	Norwich Western Link
PCV	Pollution Control Valve
PED	Pre-Earthwork Ditches
RAF	Royal Air Force
RUK	Ramboll UK Ltd.
QA	Quality Assurance
QM	Quality Manager
RMA	Risk Management Authorities
WSP	WSP Ltd Consultants

2 Introduction

2.1 Appointment and brief

The Applicant has prepared the design of the highway and drainage for the Proposed Scheme.

2.2 Scope of the report

This report is developed for the Proposed Scheme from the previous strategy prepared by WSP for Norfolk County Council (NCC) in June 2020 (ref. 1 and 2). Since this report, the highway alignment has been refined to account for ecological constraints, including the mitigation of disturbance to ancient woodland along the route. This drainage strategy includes design considerations and constraints that have been applied for key consultees, such as the Environment Agency (EA) and NCC, acting as Lead Local Flood Authority and the IDB to comment/approve in principle prior to planning submission.

This report describes the surface water drainage strategy for the Proposed Scheme. Other water-related assessments include:

- Flood risk assessment (WSP) (Reference 3.12.02)
- Water quality assessment (WSP) (Reference 3.12.01)

The report excludes drainage aspects relating to temporary works, described in the Construction Surface Water Management Plan (Appendix 15 (Reference 4.04.15))

2.3 Limitations

Parts of this report are based on the interpretation and assessment of data provided by third parties. The Applicant cannot be held responsible for the accuracy of third-party data. The conclusions and findings of this report may change if the data is amended or updated after the date of consultation.

2.4 Construction (Design and Management) Regulations

The Construction (Design and Management) Regulations 2015 (CDM Regulations) identifies duties on all parties involved in a construction project, including those promoting the Proposed Scheme. One of the designer's responsibilities under clause 9 (1) is to ensure that the client organisation, in this instance Norfolk County Council, is made aware of their duties under the CDM Regulations.

3 Existing Site

3.1 Proposed Scheme location

The Site Boundary extends in a northeast to southwest trend with the far northeast of the Proposed Scheme situated off the A1067 Fakenham Road roundabout (national grid reference 614853, 315625) through to the far southwest of the Proposed Scheme located north of the A47 (NGR 609696, 312490).

Figure 1- Proposed Scheme Location

3.2 Site description

The area within the Site Boundary and surrounding areas comprise woodland situated south of the Wensum Valley floodplain and agricultural land situated north of Ringland Lane and in the Easton Estate in the far southwest of the site. The surface within the Site Boundary is variable and predominantly comprises stubble fields, arable land used for pig farming and woodland areas.

The surrounding area comprises similar land uses with the villages of Ringland, Taverham, Attlebridge, Weston Longfield, Weston Green, Honningham and RAF Attlebridge airfield located in the surrounding areas.

3.3 Existing topography

The general topography within the Site Boundary falls from north-west to southeast and follows three natural valleys running in that direction. Two valleys have one or more watercourses running through them. The third valley runs alongside and to the north of Ringland Lane.

3.4 Existing waterbodies

The river Wensum (a main river) and two parallel watercourses run within the northern section of the Site Boundary. The maintenance responsibility of these waterbodies is as follows:

River Wensum: the Environment Agency,

Ordinary Watercourse ref. OWC5 (MN 20 - Ringland/Morton Hall (DRN112G0101): Norfolk Rivers Internal Drainage Board,

Watercourse ref. OWC7: Riparian watercourse, privately maintained.

A tributary of the River Tud also known as the Foxburrow stream, lies within the southern section of the Site Boundary. The Ringland Lane valley does not contain a watercourse other than the drainage ditches cut beside the road.

Each of the above watercourses, with the exception of the riparian watercourse, pass beneath the Proposed Scheme.

3.5 Existing Drainage

There are no existing foul sewers within the Site Boundary.

Existing highway drainage is located at the A1067 and consists of kerb and gullies with a carrier drain that conveys highway runoff to existing ponds located at the roundabout junction with the A1270. These are referenced in the Proposed Scheme as NDR Basin 1 and NDR Basin 1A.

Along the Proposed Scheme, four side roads (Ringland Lane, Weston Road, Breck Lane and Broadway) are intersected which do not have any positive drainage infrastructure. Drainage of these side roads is by informal over-the-edge to ditches. Existing ditches affected by the works will be connected to the permanent works where required to maintain continuity. These are at the following locations:

- Existing ditches around the A1067 roundabout.
- Existing natural pond by the A1067
- Ringland Lane flow path
- Existing ditch and ephemeral flow paths at the Morton Green Bridge (GB4).
- The Broadway existing ditches
- Foxburrow Stream
- Ephemeral flow path and natural pond at Ch. 5125

4 Surface Water Management Policy Context

4.1 National Planning Policy Framework (NPPF) – December 2023

The latest NPPF was published on 19 December 2023, superseding previous editions of the framework document. One of the overarching objectives of the NPPF is the encouragement of growth and acknowledgement that decision makers should adopt a presumption in favour of sustainable development. Paragraph 11 of the document states:

"Plans and decisions should apply a presumption in favour of sustainable development. For decision-making this means:

- approving development proposals that accord with an up-to-date development plan without delay or
- where there are no relevant development plan policies, or the policies which are most important for determining the application are out of date, granting permission unless:
  - the application of policies in this Framework that protect areas or assets of particular importance provides a clear reason for using the development proposed, or
  - any adverse impacts of doing so would significantly and demonstrably outweigh the benefits, when assessed against the policies in this Framework taken as a whole."

Paragraph 173 of the NPPF states that:

When determining any planning applications, local planning authorities should ensure that flood risk is not increased elsewhere. Where appropriate, applications should be supported by a site-specific flood risk assessment. Development should only be allowed in areas at risk of flooding where in the light of this assessment (and the sequential and exception tests, as applicable) it can be demonstrated that:

- within the site, the most vulnerable development is located in areas of lowest flood risk, unless there are overriding reasons to prefer a different location;

- the development is appropriately flood resistant and resilient such that, in the event of a flood, it could be quickly brought back into use without significant refurbishment;
- it incorporates sustainable drainage systems, unless there is clear evidence that this would be inappropriate;
- any residual risk can be safely managed and
- safe access and escape routes are included where appropriate, as part of an agreed emergency plan.

4.2 Sustainable Drainage Systems Written Statement HCWS 161 (December 2014)

The Secretary of State for Communities Local Government laid a Written Ministerial Statement in the House of Commons on 18th December 2014 setting out changes to planning that will apply for major development from 6 April 2015. This confirms that in considering planning applications, local planning authorities should consult the relevant Lead Local Flood Authority (LLFA) on the management of surface water; satisfy themselves that the proposed minimum standards of operation are appropriate and ensure through the use of planning conditions or planning obligations that there are clear arrangements in place for ongoing maintenance over the lifetime of the development.

Therefore, from 6 April 2015 local planning policy and decisions on planning applications to major developments are required to ensure that Sustainable Drainage Systems (SuDS) are used for the management of surface water. Major development is development involving any one of the following:

- The winning and working of minerals for the use of land for mineral working deposits,
- Waste development,
- Provision of 10 dwellings or more,
- The provision of a building or building where the floor space to be created by the development is 1,000 square metres or more or,
- Development carried out on the site having an area of 1 hectare or more.

4.3 Defra Sustainable Drainage Systems Non-Statutory Technical Standards for Sustainable Drainage Systems (March 2015)

This document sets out non-statutory technical standards for sustainable drainage systems. It is used in junction with the National Planning Policy Framework and Planning Practice Guidance.

For greenfield developments, the peak runoff rate from the development to any highway drain, sewer or surface water body in the 1 in 1 year rainfall event and the 1 in 100 year rainfall event should not exceed the peak greenfield runoff rate for the same event.

Where reasonably practicable, for greenfield development, the runoff volume from the development to any highway drain, sewer or surface water body the 1 in 100 year, 6 hour rainfall event should not exceed the greenfield runoff volume for the same event.

Where it is not reasonably practical to constrain the volume of runoff to any drain, sewer or surface water body in accordance with the above, the runoff volume must be discharged at a rate that does not adversely affect flood risk.

The drainage system must be designed so that, unless an area is designated to hold and/or convey water as part of the design, flooding does not occur on any part of the site for a 1 in 30 year rainfall event.

The drainage system must be designed so that, unless an area is designated to hold and or convey water as part of the design, flooding does not occur during a 1 in 100 year rainfall event in any part of a building including a basement; or in any utility plant susceptible to water e.g., a pumping station or an electricity substation, within the development.

The design of the site must ensure that, so far as is reasonably practicable, flows resulting from rainfall in excess of a 1 in 100 year rainfall event are managed in exceedance routes that minimise the risks to people and property.

4.4 Flood Water Management Act (FWMA) (2010) & Norfolk Local Flood Risk Management Strategy (2021)

The FWMA (2010) was first proposed as the legislative vehicle to implement the European Floods Directive, however due to delays in the bill, it was not implemented within the time frame set out by the Floods Directive, hence the implementation of the Floods Directive and the FWMA was delayed until 2010.

Schedule 3 of the FWMA sets out requirements for new drainage systems for developments with an emphasis on Sustainable Drainage Systems (SuDS). Formal requirements of how Schedule 3 is to be implemented in England has not been published but NCC policies and guidance reflect the requirements for SuDS.

The FWMA provided the legislative basis for several recommendations in the Pitt review. In October 2010, Section 9 of the FWMA came into force requiring all LLFAs in England to develop, review, update as well as apply and monitor the application of a strategy for local flood risk in the area. This is known as a Local Flood Risk Management Strategy (LFRMS).

The Norfolk LFRMS (2015, reviewed 2021) provides a more detailed overview of local flood risk across Norfolk from local sources such as surface water, groundwater and ordinary watercourses. It also outlines the strategy to manage flood risk. As a first step in this process NCC prepared a Preliminary Flood Risk Assessment (PFRA).

Within the Norfolk LFRMS:

- Policy UC10: Planning, outlines that the LLFA will expect planning authorities “to take account of flood risk identified by Surface Water Management Plan modelling, Strategic Flood Risk Assessments and other sources of flood risk modelling (such as the flood risk mapping provided by the Environment Agency) and either avoid locating new development within areas that are at risk of flooding, or if that is not possible, ensure that designs fully mitigate for the expected flood risk.”
- Policy UC11: Securing Sustainable Drainage indicates that the LLFA “shall, using all available legislative and regulatory measures, seek to secure the implementation of high quality, multi-functional SuDS, which follow the most up to date guidance, in new development.”

- Policy UC13: Adapting to Climate Change, outlines that, when developing policy, risk management authorities "must take into account the predicted impacts of climate change including the changes in sea level and more frequent extreme weather events. In doing so, Risk Management Authorities will use the most up to date advice available, including UKCIP Climate Change Projections".

4.5 North Norfolk Strategic Flood Risk Assessment

To continue fulfilling the recommendations of the Pitt review locally, Norfolk County Council as Lead Local Flood Authority is required to develop a Strategic Flood Risk Assessment (SFRA). The SFRA sets out the flood risk constraints to help inform the Local Plan, Neighbourhood Planning, and the determination of planning applications in North Norfolk.

4.6 Highways Guidance for Development, Drainage – Norfolk County Council (LLFA)

The online NCC Drainage guidance is a live document and states that developments must provide adequate drainage for surface water, with disposal of surface water from new highways to be through a sustainable drainage system, which incorporates adequate water quality treatment measures where possible.

NCC have stated that they:

"seek to reduce the rate of surface water run-off through the use of Sustainable Urban Drainage Systems SuDS, which may incorporate filter strips and swales (which can be under-drained to allow for crossing points and maintain connectivity), filter drains, permeable surfaces, bio-retention areas, infiltration devices and basins or ponds.

These systems are more sustainable than conventional drainage methods as they:

- Manage runoff flow rate, reducing the impact of urbanisation on flooding
- Protect or enhance water quality
- Are sympathetic to the environmental setting and the needs of the local community
- Provide a habitat for wildlife in urban watercourses
- Encourage natural groundwater recharge (where appropriate)

They do this by dealing with runoff close to where the rain falls, managing potential pollution at the source, and protecting water resources from point pollution (such as accidental spills)."

All schemes should attenuate runoff to the pre-development greenfield runoff rate and volumes for all rainfall events up to and including the 1% annual probability (1 in 100 year) plus 40% climate change. Note that the climate change up-lift changed from 40% to 45% during the development of this scheme.

4.7 Lead Local Flood Authority Statutory Consultee for Planning Guidance Document, October 2022

"This guidance document is intended to support the development of Norfolk County Council (NCC), as Lead Local Flood Authority's (LLFA) role as a statutory consultee to planning, and to inform stakeholders in this process such as Local Planning Authorities (LPAs) and developers."

- "Part C aims to provide guidance for developers on the information required by the LLFA from applicants to enable it to provide responses to major planning applications."

Reference is made to the guidance in this document on important decision making during the design of SuDS on the NWL as it is considered a 'major' development:

- "All developments with an area greater than or equal to 2 hectares".
- "Any major development applications that have a local flood risk and are on an obvious flow route or include extensive surface water or fluvial flooding on the site. Significant ponding of surface water over a large proportion of the site boundary also falls within this category. Further information on screening applications against local flood risk ….." and
- "Sites adjacent to, or within, areas with records of local flooding (as evidenced and provided by the LLFA)…."

4.8 Climate Change Allowances – Updated 2022

The Climate Change Adaptation Sub-Committee Progress Report 2014 stated that increased flood risk is the greatest threat to the UK from climate change. Models of the climate system suggest floods of the type experienced in England and Wales in autumn

2000, and between December 2013 and February 2014, have become more likely because of increased concentrations of greenhouse gases in the atmosphere.

More frequent short-duration, high intensity rainfall and more frequent periods of long-duration rainfall could be expected. Sea levels are also expected to continue to rise.

Latest guidance for application of climate allowances to developments is available from the government Flood Risk Assessments website for peak rainfall intensity, peak river flow and sea level rise.

The Norwich Western Link is considered essential transport infrastructure (considered “Essential Infrastructure” in flood risk terms) and should be reviewed against the following climate change allowances:

Flood criteria: Peak runoff

The climate change factors for the area were amended in 2022 and 2023 and are as follows:

- 1 in 1 year return period rainfall events: 20%
- 1 in 10 year return period rainfall event: 40% (note 1)
- 1 in 30 year return period rainfall events: 40%
- 1 in 100 return period rainfall events: 45% (note 2)

Notes. The principal changes are:

1) Identification of climate change allowance ref. LLFA letter FW2023_0343 dated 27/4/2023.

2) the raising of the 1 in 100 return period from 40 to 45%. Raised at a meeting with LLFA on 21 September 2022.

5 Surface Water Drainage Strategy

5.1 Existing Surface Water Drainage

The existing area within the Site Boundary consists primarily of agricultural land and woodland (with roads crossing). Observations of these roads and surrounding fields indicated that surface water that falls upon the existing Site Boundary is either drained via natural infiltration into the ground or runs off into the existing drainage system. This system consists of interconnecting drainage ditches across the farmland. Existing ditches and waterbodies are as follows:

- Existing ditches around the A1067 roundabout.
- Existing natural pond by the A1067
- Ringland Lane flow path
- Existing ditch and ephemeral flow paths at the Morton Green Bridge (GB4).
- The Broadway existing ditches
- Foxburrow Stream
- Ephemeral flow path and natural pond at Ch. 5125

These existing surface water drainage ditches eventually convey the surface water runoff either eastwards or westwards towards the waterbodies such as ordinary watercourses parallel to the River Wensum and the Tud tributary (Foxburrow stream) and an existing pond south-west of the Tud tributary.

A flood risk assessment of the Proposed Scheme is described in a flood risk assessment by WSP (Reference 13.12.02).

5.2 Proposed Highway Drainage

5.2.1 Design standards and guidance

The drainage strategy for the Proposed Scheme has been developed in accordance with, but without limitation to, the following documents contained within the Design Manual for Roads & Bridges (DMRB).

i) CG 502 Certification of drainage design;

ii) CG 501 Design of highway drainage systems;

iii) CD 521 Hydraulic Design of Road-Edge Surface Water Channels;

iv) CD 524 of Pavement Details;

v) CD 533 Determination of Pipe Bedding Combinations for Drainage Works;

vi) CD 526 Spacing of Road Gullies;

vii) CD 532 Vegetated Drainage Systems for Highway Runoff;

viii) CD 522 Drainage of Runoff from Natural Catchments;

ix) CD 529 Design of Outfall and Culvert Details;

x) LA 113 Road Drainage and the Water Environment.

The below is a list of design guidance that has been referred to throughout the development of the drainage strategy including without limitation, the following:

i) BRE Digest 365 Soakaway Design ref DG 365 dated 2016

ii) CIRIA report C753 The SuDS Manual 2015

iii) NCC LLFA Guidance Document Version 6.1 October 2022

iv) Relevant sections and paragraphs of the NPPF 2023

v) CIRIA C683 The Rock Manual 2007

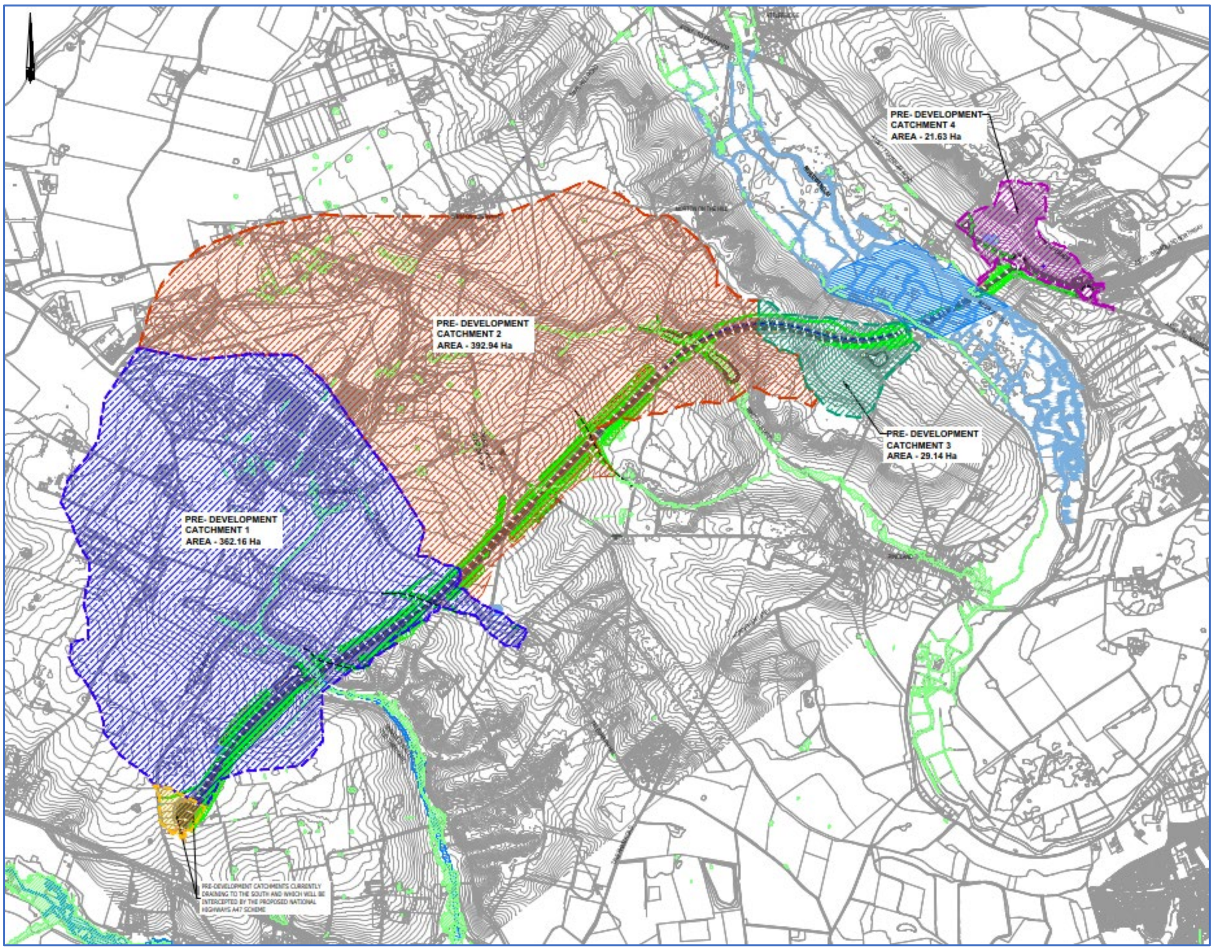
vi) BS 8582:2013 – Code of Practice for Surface Water Management for Development Sites.

vii) CIRIA C742 Manual on scour at bridges and other hydraulic structures 2002

5.2.2 Existing catchments

The route of the Proposed Scheme passes through a rural area and intersects several hydrological catchments along its length. These hydrological catchments are defined principally by local topography and existing drainage features such as watercourses or land drains. The existing catchments intersected by the Proposed Scheme are shown on Figure 2.

Figure 2- Existing natural catchment plan



5.2.3 Ground Conditions

The Proposed Scheme crosses several geologies that mainly comprise glaciofluvial deposits, comprising the Sheringham Cliffs Formation overlying the Lowestoft Formation. Both these formations have units of permeable granular material and relatively impermeable units of sandy silty clay and silt. Chalk forms the bedrock.

Chalk is an erodible material and may be subject to dissolution features which are underground pockets where voids in the chalk could be infilled by the overlying material. This could cause a potential issue for infiltration basins if the infill material is clay rather than a free draining material, which could affect the drainage base or permeability characteristics. However, the Ground Investigations undertaken have only identified potential dissolution features within the Wensum Flood Plain and none in the vicinity of any of the proposed basins.

Site investigations were carried out by WSP and Harrison Geotechnical Ltd. Reference should be made to the following ground investigation reports. Summarised copies are included in Appendix 14 (Reference 4.04.14):

Factual Ground Investigation Reports:

- 70061370-WSP-RP-GEO-0002 dated November 2020
- NCCT41793-HAG-VGT-FSC-RP-GI-0001 dated February 2022,
- NCCT41793-HAG-VGT-FSC-RP-GI-0002 dated October 2022
- NCCT41793-HAG-VGT-FSC-RP-GI-0003 'Woodland Campaign' dated November 2022

Drainage basin A1067/1, Basin 2 and Basin 5 will be situated in the relatively permeable silty or clayey gravelly sand of the Sheringham Cliffs Formation.

Drainage Basin 1 is situated partly in relatively impermeable silty or clayey gravelly sand of the Sheringham Cliffs Formation and structureless chalk.

Basins 3 and 4 are situated in an area still within the Sheringham Cliffs Formation, with bands of lower permeability silty clay and sandy silt, together with zones of more permeable silty gravel.

Towards the southern end of the Proposed Scheme the alignment will cut through the Lowestoft Formation, which comprises mainly relatively impermeable cohesive deposits in the cutting towards the A47, but with a layer of slightly silty sand towards or at the base of basin 6.

5.2.4 Surface Water Discharge

Planning Practice Guidance for flood risk and coastal change advises that runoff from new developments is discharged in line with the following hierarchy: infiltration to ground, discharge to surface watercourse, discharge to sewer.

1. Infiltration: Subsoil within the Site Boundary has variable infiltration capabilities and site investigations have proved the feasibility of using infiltration as a source control SuDS technique.
2. Existing Watercourse: Outfalls to various watercourses, where available, are planned to serve a number of sub catchments within the Proposed Scheme area.

At the outfall locations, infiltration methods are not feasible due to high groundwater conditions.

3. Existing sewer: this discharge method is low in the hierarchy and is not used. No existing sewers are available.

5.2.5 Infiltration testing

A series of ground investigations has taken place to determine the feasibility of infiltration at discharge points in the Proposed Scheme highway drainage network.

Infiltration testing results are described in summaries of ground investigation reports included in Appendix 14 (Reference 4.04.14). An interpretation of the percolation values for each basin is described below.

The results of the early site investigation work in 2020 indicated that disposal of highway runoff was feasible by the infiltration method at basin locations A1067/1, 2, 3, 4 and 6. Preliminary basin design identified desired locations and elevations for infiltration basins. Subsequent testing focused on developed basin locations and at planned basin base elevations.

Infiltration testing results for A1067-1, 2, 3, 4 and 6 are described below summarised in Appendix 1 (Reference 4.04.01).

A precautionary approach is taken to the selection of a design infiltration rate for each basin taken from the range of test results. The approach is to use the lowest infiltration rate from the test results. This is described in the following paragraphs.

NWL Basin 1 at the A1067/A1270 roundabout discharges to an existing infiltration basin, NDR Basin 1A, which is not proposed to be modified by the Proposed Scheme. The capacity of the NDR Basin 1A has been assessed by others to confirm that it can accept the peak discharge from NWL Basin 1. An infiltration rate of 0.342 m/hr and a factor of safety of 5 was applied in the assessment which is based on the original design parameters. Refer to 7.9 (and appendix 8 (Reference 4.04.08)) for further details.

Basin A1067/1

Drainage basin A1067/1 will be situated in the relatively permeable silty or clayey gravelly sand of the Sheringham Cliffs Formation.

Figure 3 - Basin A1067/1 Trial Pit Locations

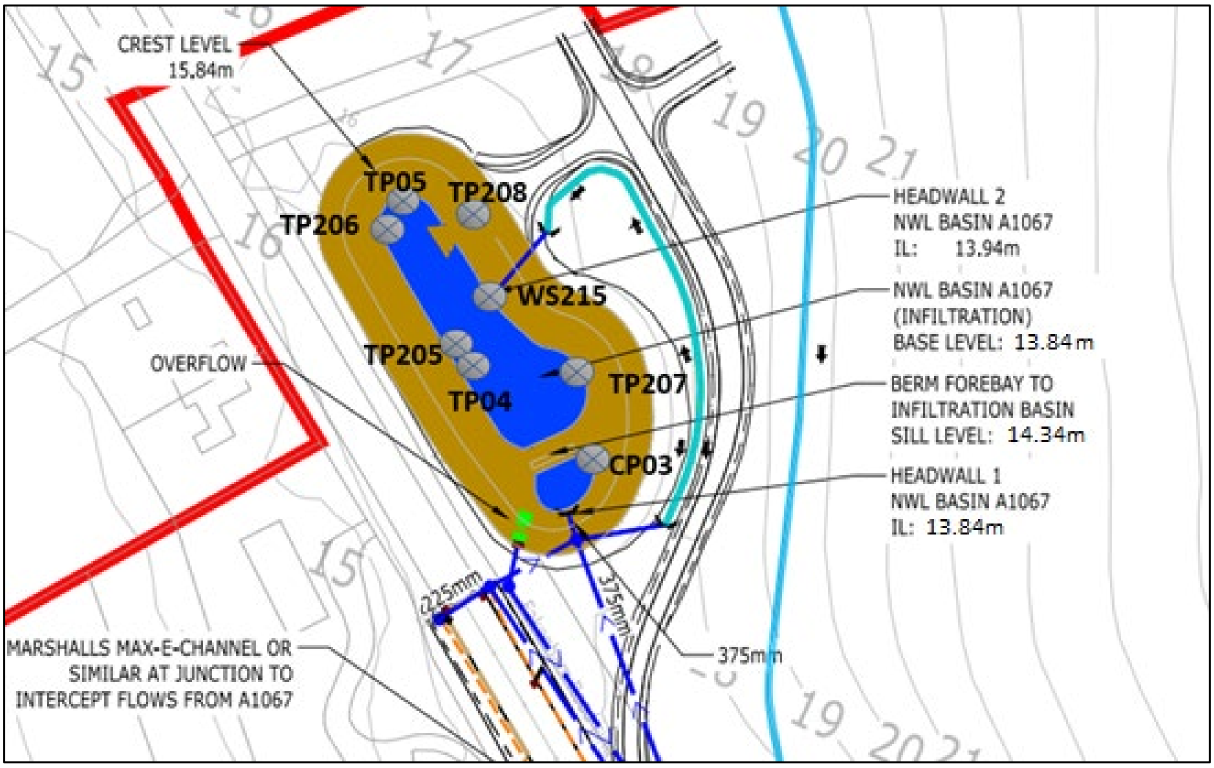


Table 1 Basin A1067/1 Infiltration Test Results

Trial pit	Depth of trial pit (m)	Ground Level (mAOD)	Trial Pit Base Level (mAOD)	Infiltration Value (m/hr)	Infiltration Value (m/s)	Notes
TP208	1.6	16.29	14.69	0.01498	4.16E-06	Gravelly, very clayey f to c (fine to coarse) SAND
TP206	1.8	15.94	14.14	0.01436	3.99E-06	Gravelly, slightly clayey f to c SAND
TP205	1.8	15.99	14.19	0.01361	3.78E-06	Gravelly clayey f to c SAND
TP207	**3.1**	**16.75**	**13.65**	**0.00958**	**2.66E-06**	**Slightly gravelly, very clayey f to c SAND**
TP04	4	16.01	12.01	0.00299	8.31E-07	Structureless chalk – Discount due to depth
TP05	4	16.01	12.01	0.00299	8.31E-07	Slightly gravelly slightly silty f to c SAND – Discount due to depth

Basin invert level = 13.84m

Infiltration rate used for design shown in **bold.**

Figure 4 - Basin A1067/1 Infiltration test results

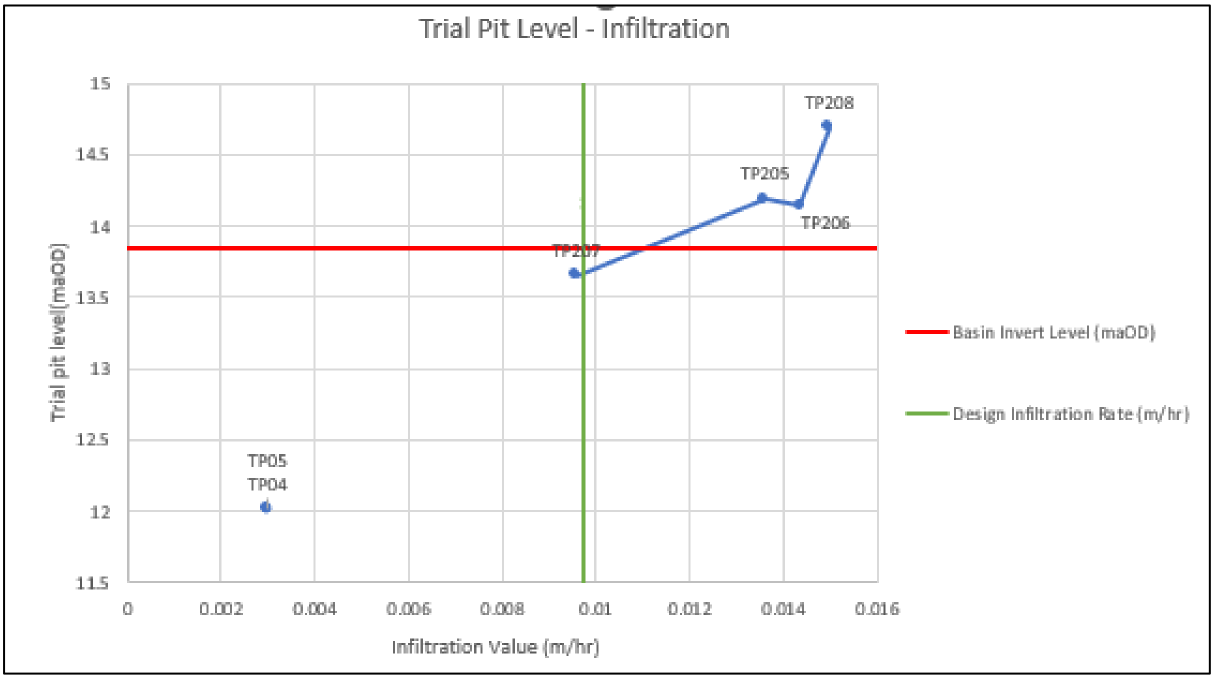


Table 1 lists the infiltration testing and results from trial pits dug across the site of the proposed basin. Two of the trial pits (TPs 04 and 05) have been discounted as the base material is in chalk. The red line on the graph indicates the level of the proposed basin. The lowest infiltration rate of 2.66 x10^{-6} m/s (0.00985 m/hr) in TP207 is taken forward for design in accordance with the Norfolk CC LLFA guidelines.

Basin 2

Drainage basin 2 will be situated in the relatively permeable silty or clayey gravelly sand of the Sheringham Cliffs Formation. Two options for a location for this basin and trial pit positions are shown below.

Figure 5 – Basin 2 Trial Pit Locations

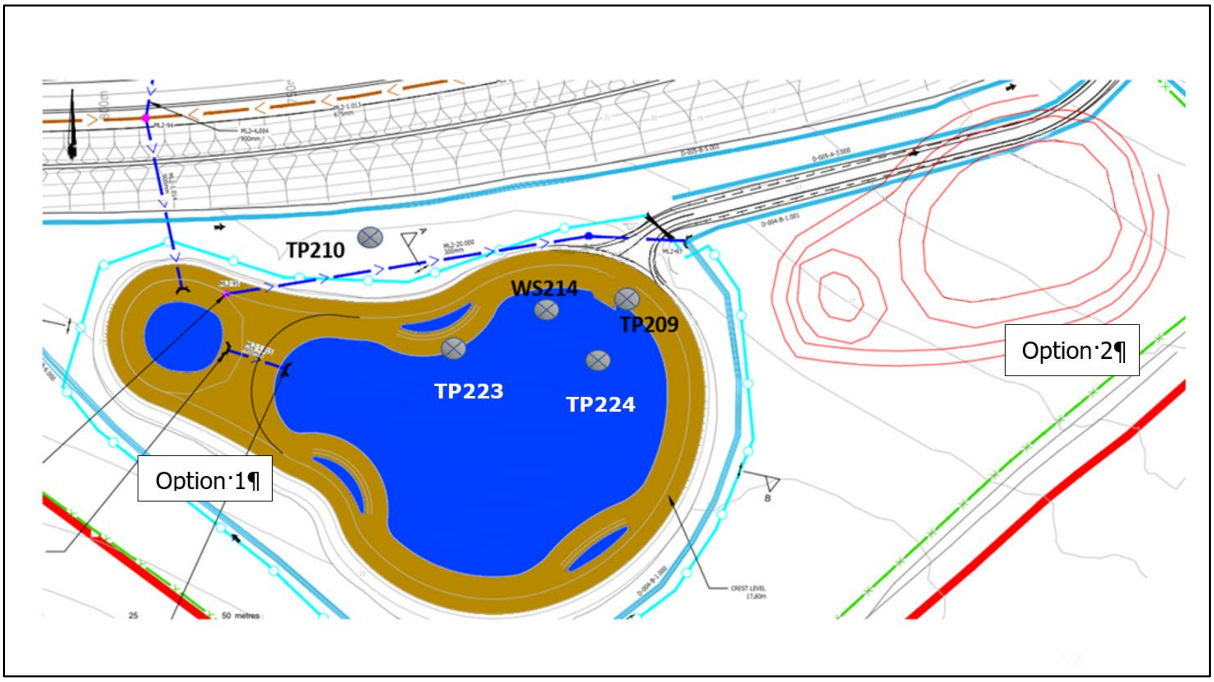


Table 2 Basin 2 Option 1 Infiltration Test Results

Trial pit	Depth of trial pit (m)	Ground Level (mAOD)	Trial Pit Base Level (mAOD)	Infiltration value (m/hr)	Infiltration value (m/s)	Notes
TP209	2.8	18.36	15.56	0.04	1.11E-05	Fine to medium SAND
TP210	1.4	17.39	15.99	0.0296	8.22E-06	Gravelly, clayey f to c SAND
TP224B	2.7	18.69	15.99	0.0228	6.33E-06	Slightly clayey f to c SAND
TP223*	**1.4**	**18.56**	**17.16**	**0.0216**	**6.00E-06**	**Slightly gravelly, slightly clayey f to c SAND**

Option 1 Basin invert level = 16.1m

Infiltration rate used for design shown in **bold**

*TP223 selected as this is the lowest infiltration rate. Design infiltration rate 0.0216 m/hr (6.00E-06 m/s)

Figure 6 – Basin 2 (Option 1) Infiltration test results

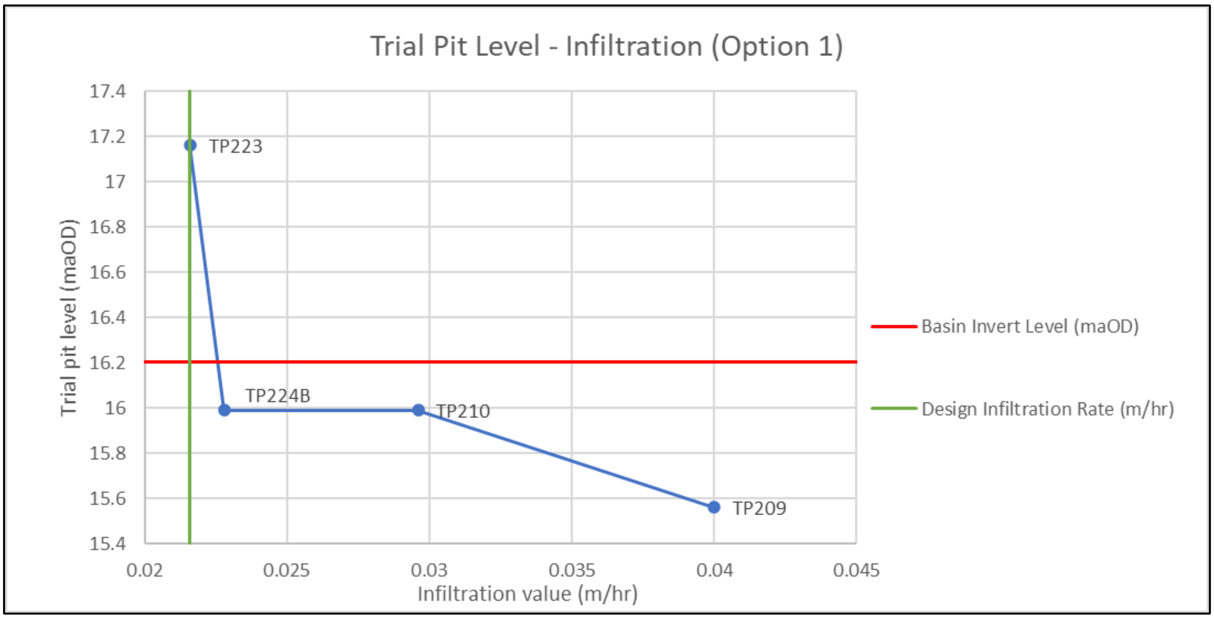


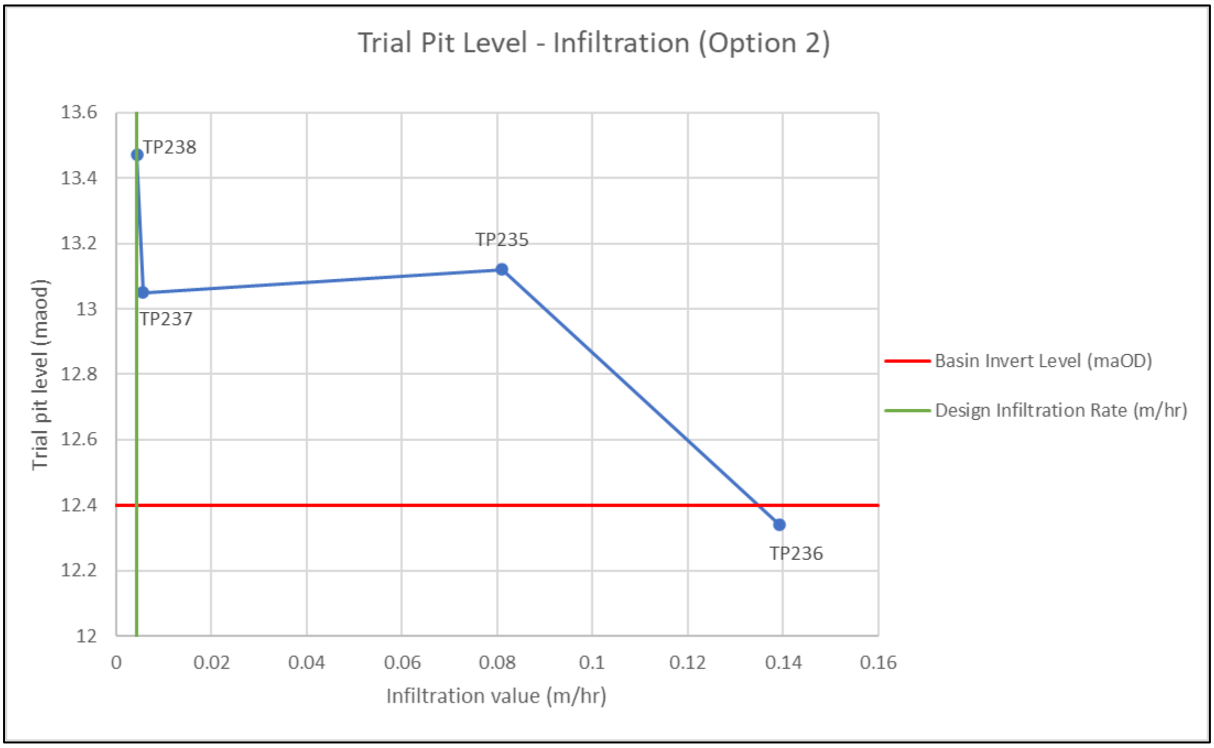
Table 3 Basin 2 Option 2 Infiltration Test Results

Trial pit	Depth of trial pit (m)	Ground Level (mAOD)	Trial Pit Base Level (mAOD)	Infiltration value (m/hr)	Infiltration value (m/s)	Notes
TP236	2.2	14.54	12.34	0.1393	3.87E-05	Slightly gravelly silty f to m SAND
TP235	1.65	14.77	13.12	0.081	2.25E-05	Gravelly, slightly clayey f to c SAND
TP237	2.9	15.95	13.05	0.0057	1.58E-06	Gravelly, slightly clayey f to c SAND
TP238*	2.4	15.87	13.47	0.0044	1.22E-06	Gravelly, slightly clayey f to c SAND

Option 2 Basin invert level = 12.4m

TP238 selected as this is the lowest infiltration rate. Design infiltration rate – 0.0044 m/hr (1.22E-06 m/s). Basin option 2 was not taken forward.

Figure 7 – Basin 2 (Option 2) Infiltration test results



Two options for a basin position were chosen to be studied as shown above. A comparison of the infiltration test results by trial pit depth and proposed basin level at the time of the study is shown. Basin Option 1 was chosen based on giving the higher infiltration rate. The infiltration rate chosen for design of basin 2 is the lowest of the four results from TPs 223, 224B, 209 and 210, as recommended by the LLFA Technical Guidance, being 1.22×10^{-6} m/s (0.0044 m/hr). Confirmatory testing will be carried out at the preconstruction stage with additional trial pits to ensure the infiltration rate is representative along the footprint of the basin.

Basin 3

Basins 3 will be situated in an area still within the Sheringham Cliffs Formation, with bands of lower permeability silty clay and sandy silt, together with zones of more permeable silty gravel.

Figure 8 – Basin 3 Trial Pit Locations

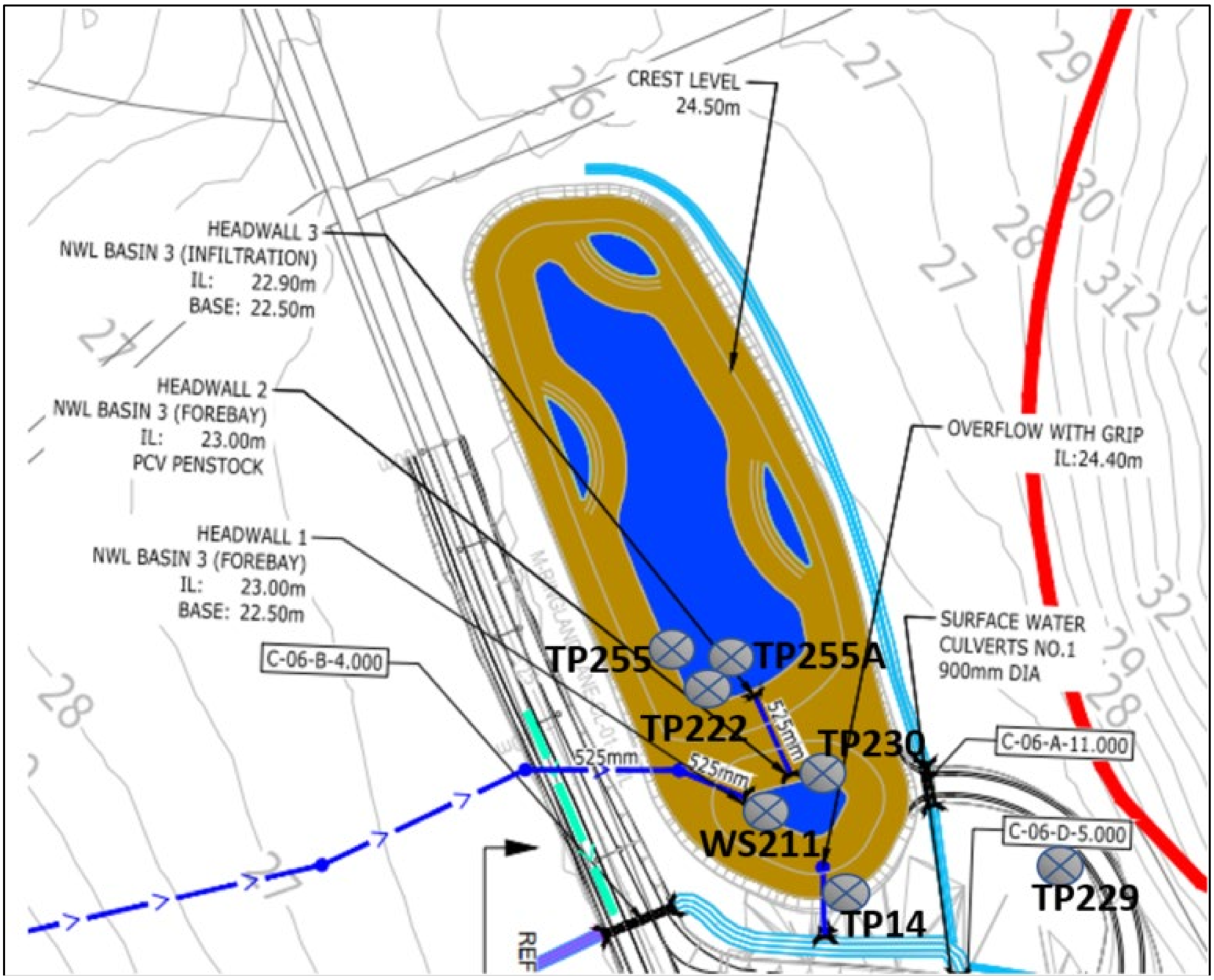
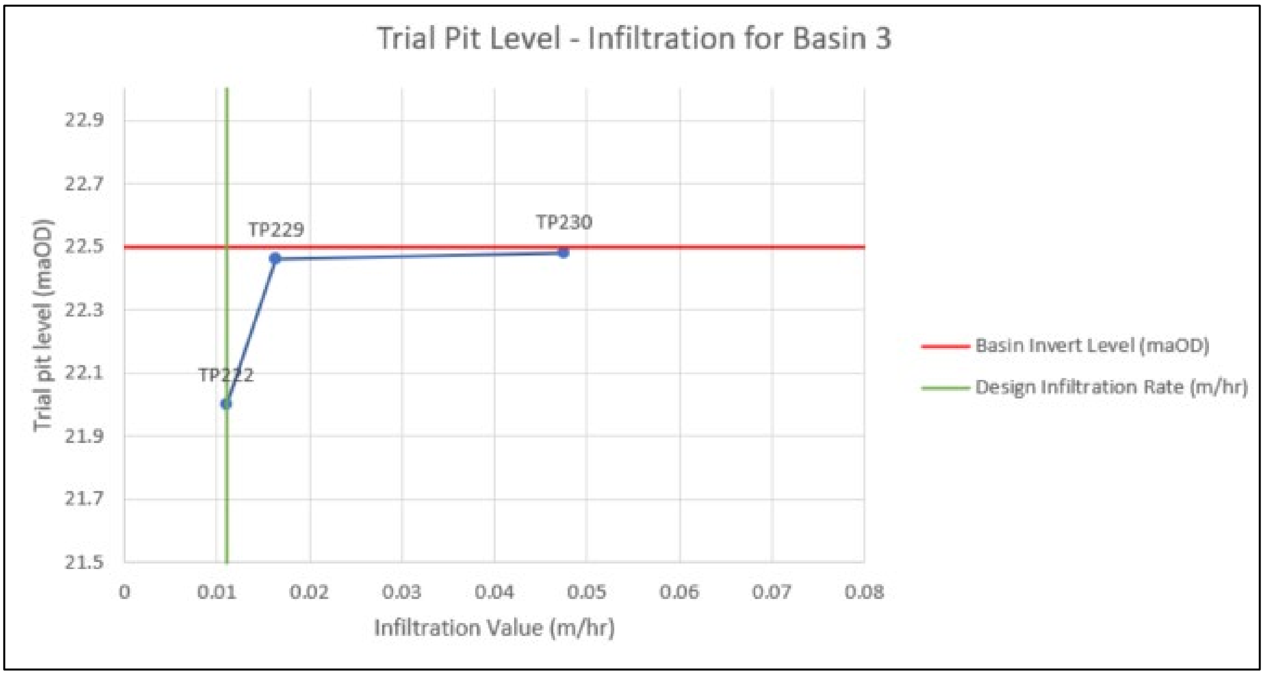


Table 4 Basin 3 Infiltration Test Results

Trial pit	Depth of trial pit (m)	Ground Level (mAOD)	Trial Pit Base Level (mAOD)	Infiltration value (m/s)	Infiltration value (m/hr)	Notes
TP13	4	23.72	19.72	8.91E-07	0.0032	Slightly gravelly very sandy CLAY
TP14	4	23.70	19.70	5.22E-05	0.1879	Made ground to 0.50, Slightly clayey silty fine to coarse SAND 0.50 – 0.80, Firm occasionally stiff, sandy gravelly CLAY 0.80 – 2.20, over structureless CHALK to 4.0m.
TP222	**2.15**	**24.15**	**22.00**	**3.05E-06**	**0.0110**	**Slightly gravelly sandy CLAY**
TP229	1.7	24.16	22.46	4.55E-06	0.0164	Sandy gravelly SILT
TP230	1.7	24.18	22.48	1.32E-05	0.0475	Coarse GRAVEL

*Infiltration rate used for design shown in **bold**

Figure 9 – Basin 3 infiltration results



As the planned basin depth is at 2m depth, the first set of results from trial pits at 4m depth have been discounted. The planned basin base level for infiltration is +22.50m. Results show variable soil materials with a range of infiltration rates reflecting the soil variability.

The infiltration rate chosen for design of the basin is the lowest of the three results from TPs 222, 229 and 230, as recommended by the LLFA Technical Guidance, being 3.05×10^{-6} m/s (0.011 m/hr). Confirmatory testing will be carried out at the preconstruction stage with additional trial pits to ensure the infiltration rate is representative along the footprint of the basin.

Basin 4
Basin 4 will be situated in an area within the Sheringham Cliffs Formation, with bands of lower permeability silty clay and sandy silt, together with zones of more permeable silty gravel.

Figure 10 – Basin 4 Infiltration Test Locations

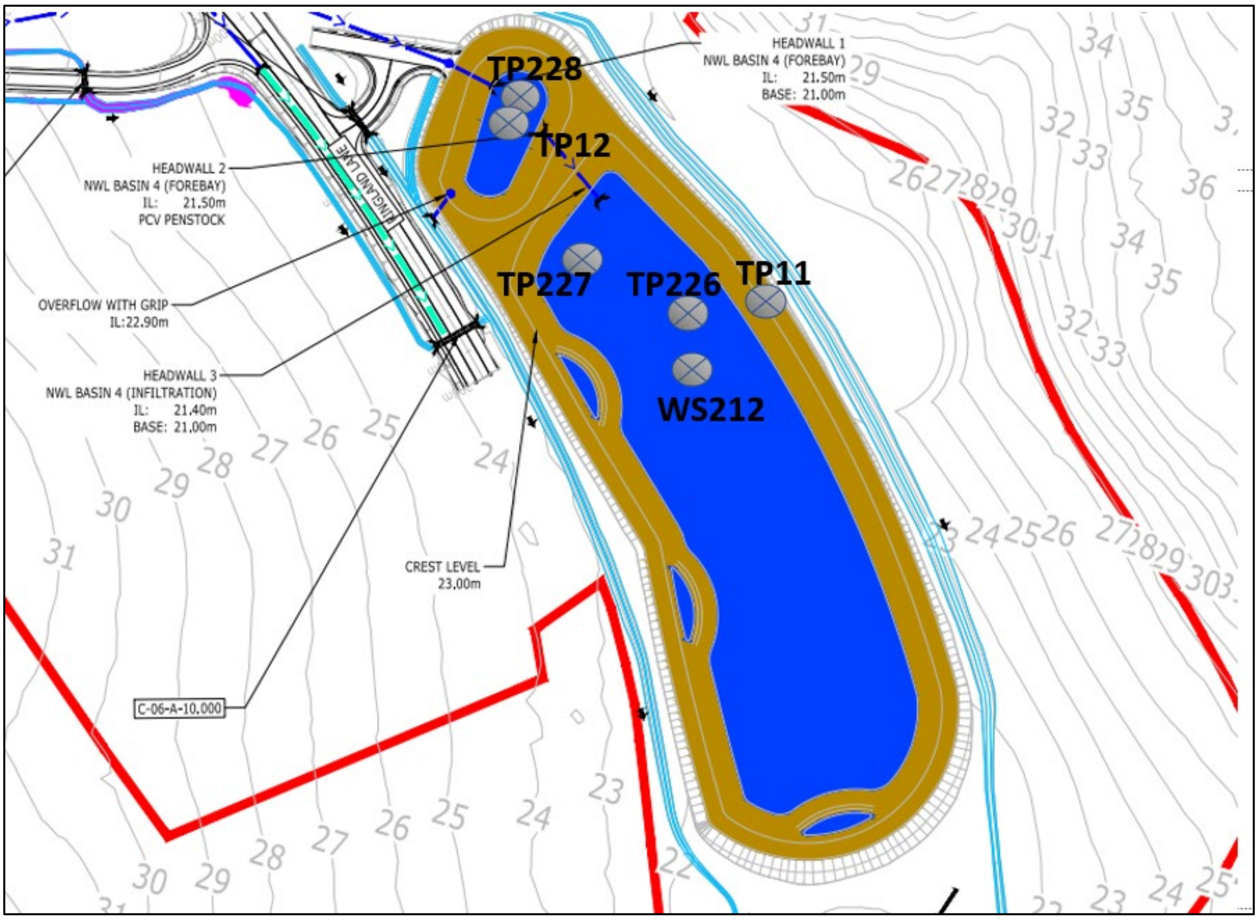


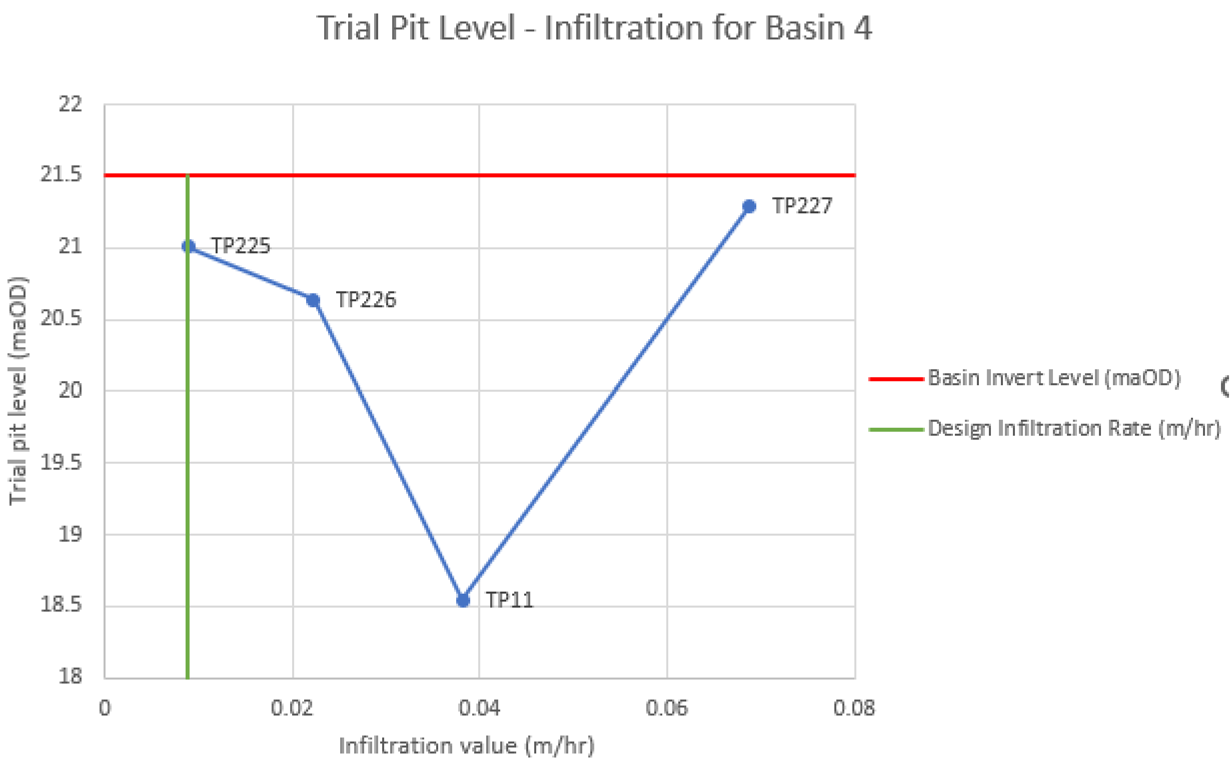
Table 5 Basin 4 Infiltration Test Results

Trial pit	Depth of trial pit (m)	Ground Level (mAOD)	Trial Pit Base Level (mAOD)	Infiltration value (m/hr)	Infiltration value (m/s)	Notes
TP227	0.95	22.24	21.29	0.0688	1.91E-05	Gravelly Clay
TP11	4	22.54	18.54	0.0382	1.06E-05	Structureless chalk (Discounted – 3m below basin)
TP226	1.6	22.24	20.64	0.0222	6.17E-06	Gravelly Silt
TP225	2.4	23.41	21.01	0.0089	2.47E-06	Sand

Infiltration rate used for design shown in **bold** (lowest value of the different geologies)

Basin invert level = 21.5m

Figure 11 – Basin 4 Infiltration Test Results



The basin invert level range considered during the feasibility study was between +21 and +21.5 mAOD. Four trial pits show four different geologies in the region of the basin i.e. clay, chalk, silt and sand. TP11 has been discounted as this in the structureless chalk and 3m below the base of the basin. TP228 has also been discounted as this is in the basin forebay area and has a much higher infiltration rate.

The infiltration rate chosen for design of the basin is the lowest of the three results from TPs 225, 226 and 227, as recommended by the LLFA Technical Guidance. It is 2.47×10^{-6} m/s (0.0089 m/hr). Confirmatory testing will be carried out at the preconstruction stage with additional trial pits to ensure the infiltration rate is representative along the footprint of the basin.

Basin 6

Figure 12 - Basin 6 Infiltration Test Locations

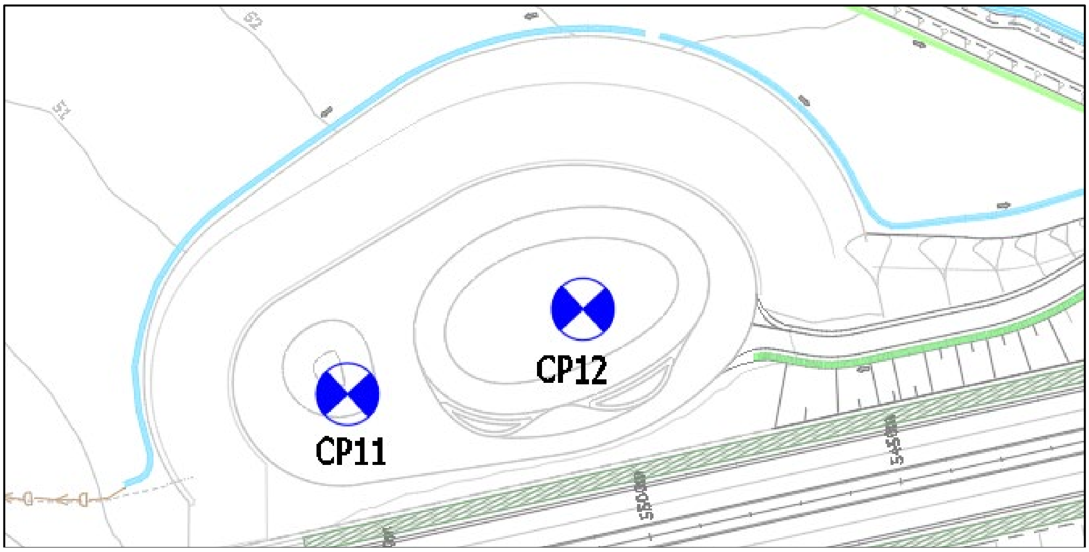


Table 6 Basin 6 Infiltration Test Results

Borehole	Depth (m)	Infiltration values (m/s)	Infiltration values (m/hr)	Soil description
CP12	9	3.90E-07 3.60E-07 2.40E-07	0.0014 0.0013 0.0009	Topsoil to 0.35, sandy gravelly CLAY 0.35 – 0.70, slightly gravelly silty CLAY 0.70 – 3.80, slightly gravelly slightly silty CLAY to 6.7, slightly silty fine to medium SAND to 8.9, slightly silty fine to coarse SAND and fine to coarse GRAVEL with occasional thin bands of sandy CLAY
CP11	12	3.10E-06 8.10E-07 1.10E-06	0.0112 0.00292 0.0040	Topsoil to 0.35, sandy gravelly CLAY 0.35 – 2.20

Infiltration rate used for design shown in **bold**

The results in Table 6 indicate that the infiltration rate is less than 1.0 x10^{-6} m/s and as such infiltration is not a viable option (SuDS Manual ref. 25.2.1) at the location of Basin 6.

Summary of infiltration test results

Table 7 Summary of infiltration test results by basin

Basin no.	Infiltration rate m/s (m/hr)	Infiltration rate m/s (m/hr)
A1067/1	0.00985	2.66×10^{-6}
2	0.0216	6.00×10^{-6}
3	0.01100	3.05×10^{-6}
4	0.0089	2.47×10^{-6}
6	0.00292	8.1×10^{-7}

5.2.6 Existing Groundwater

Groundwater monitoring has been undertaken as part of the ground investigation and will be continuing throughout the pre-construction period. This will determine any propensity for groundwater table levels to rise within one metre of the base of pond structures. This is to provide an alert for any encroachment of groundwater into the unsaturated zone of infiltration basins and potential for disturbance of pond linings in attenuation basins due to hydraulic uplift.

The most recent groundwater monitoring results (Appendix 14 (Reference 4.04.14), report dated October 2022), indicate that in general, apart from localised perched water regimes and in areas close to watercourses, the groundwater is several metres below the base of the road formation and drainage basins with two exceptions being basins 5 & 6. Groundwater reports at basins 5 & 6 are described below. There is evidence of perched groundwater at basin 6. At the sites of basins 3 and 4 groundwater was at relatively shallow depth but still with a sufficient unsaturated zone below the proposed base of the basins.

Basin 5

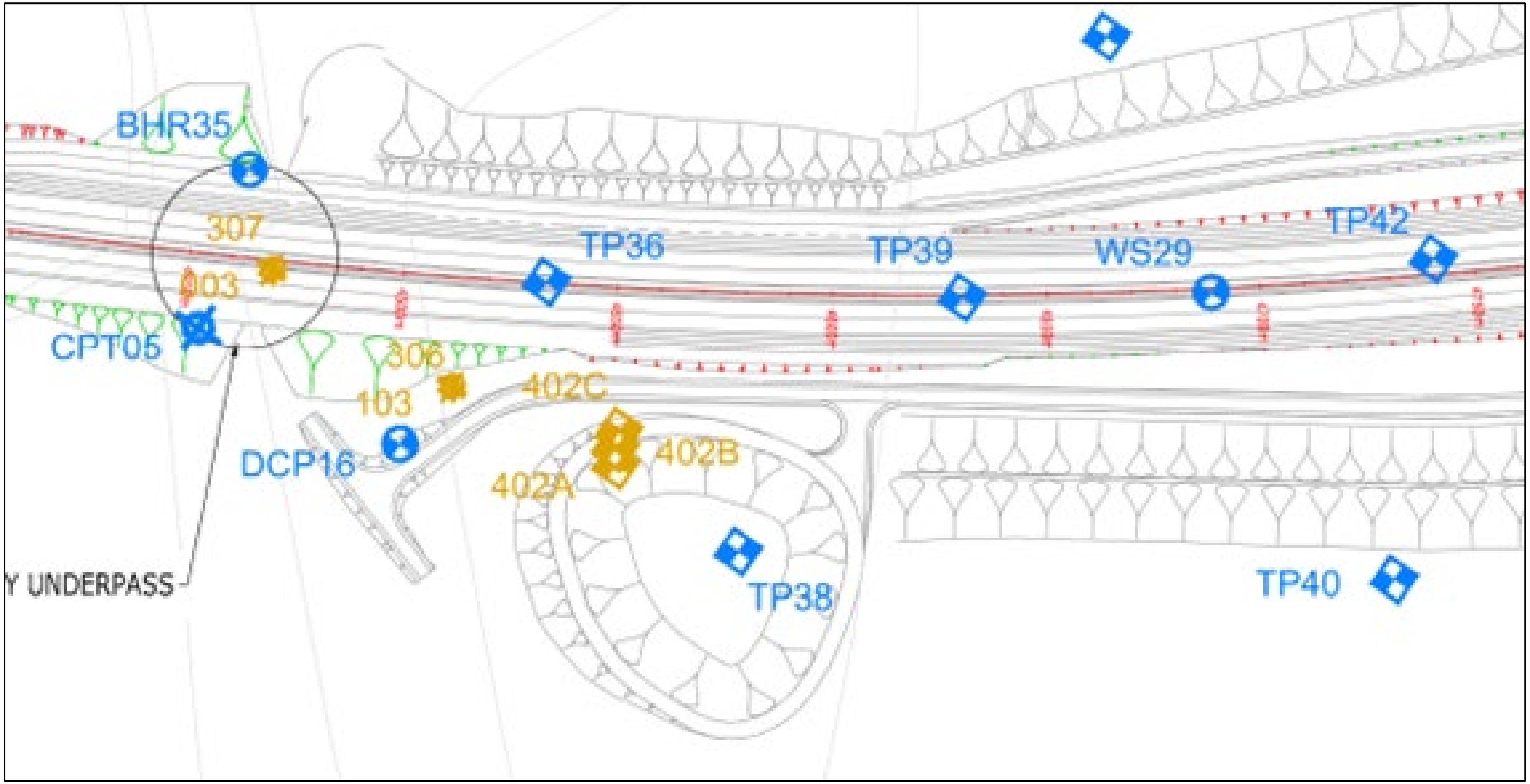
Groundwater monitoring has identified that the highest groundwater level could be some 2m above the base of NWL Basin 5 (+39.30m AOD). Refer to the groundwater long section in **Figure 14** and drawing PK1002-RAM-HDG-MLE-DE-DZ-0546 (Reference 2.08.01). There is a risk that the groundwater level could be higher than this.

Owing to the fact that monitoring has been undertaken during a particularly dry period, a search of historical borehole information was undertaken to confirm groundwater observations at Basin 5. However, no additional sources of information were found.

Table 8 Basin 5 Groundwater monitoring results

Location ID	Highest GW Depth recorded (m bgl)	Highest GW Elevation recorded (m AOD)	Lowest GW Depth recorded (m bgl)	Lowest GW Elevation recorded (m AOD)
BHR35	-0.75	36.93	10.5	25.68
WS29	2.96	40.39	38.90	40.39

Figure 13 – Basin 5 Borehole Plan



Borehole BR35 (GL +36.20) monitoring results between Feb and July show a maximum positive pressure of 0.75m (+36.95) at the stream side.

Window sample WS29 struck water at +38.9 mAOD

Basin 5 design base level = +39.3 mAOD

Figure 14 - Basin 5 Geological Long section

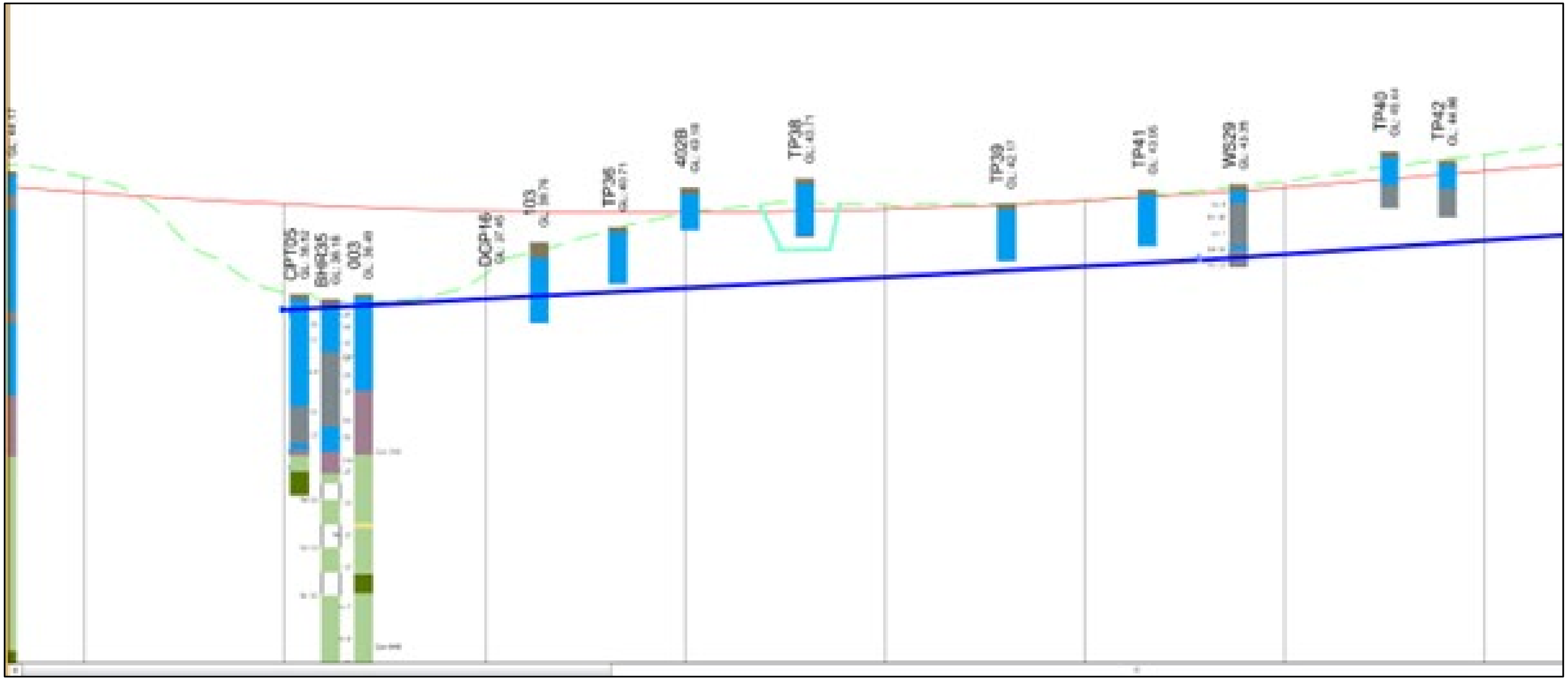
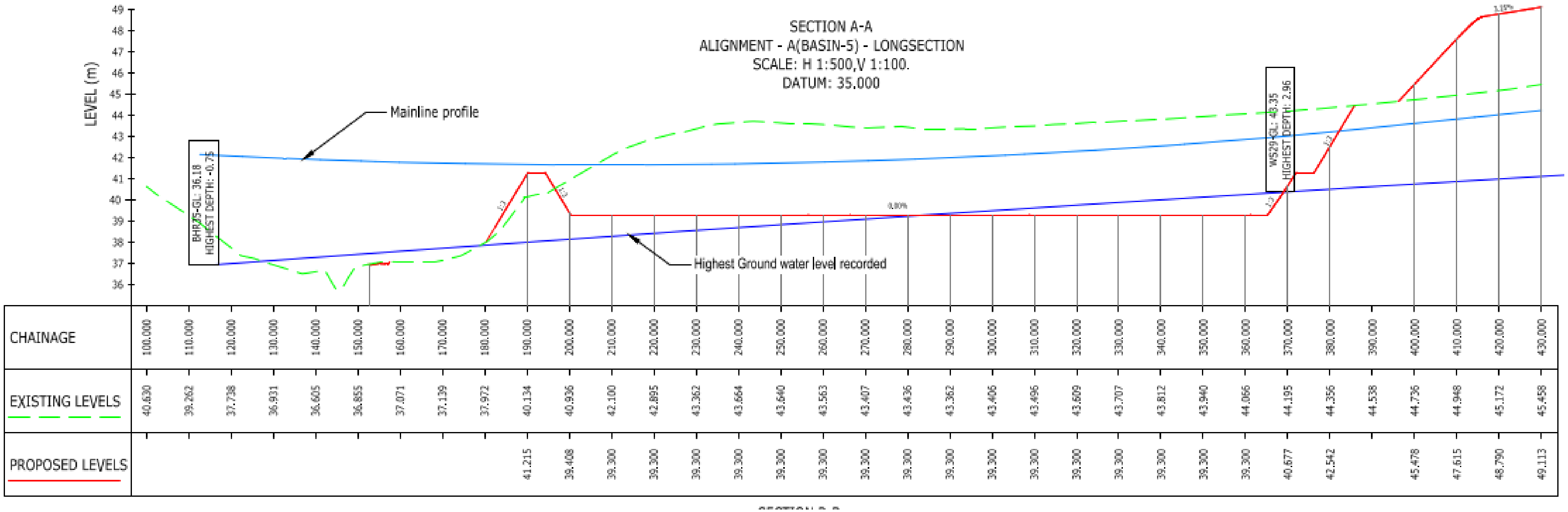


Figure 15 - Basin 5 Long section



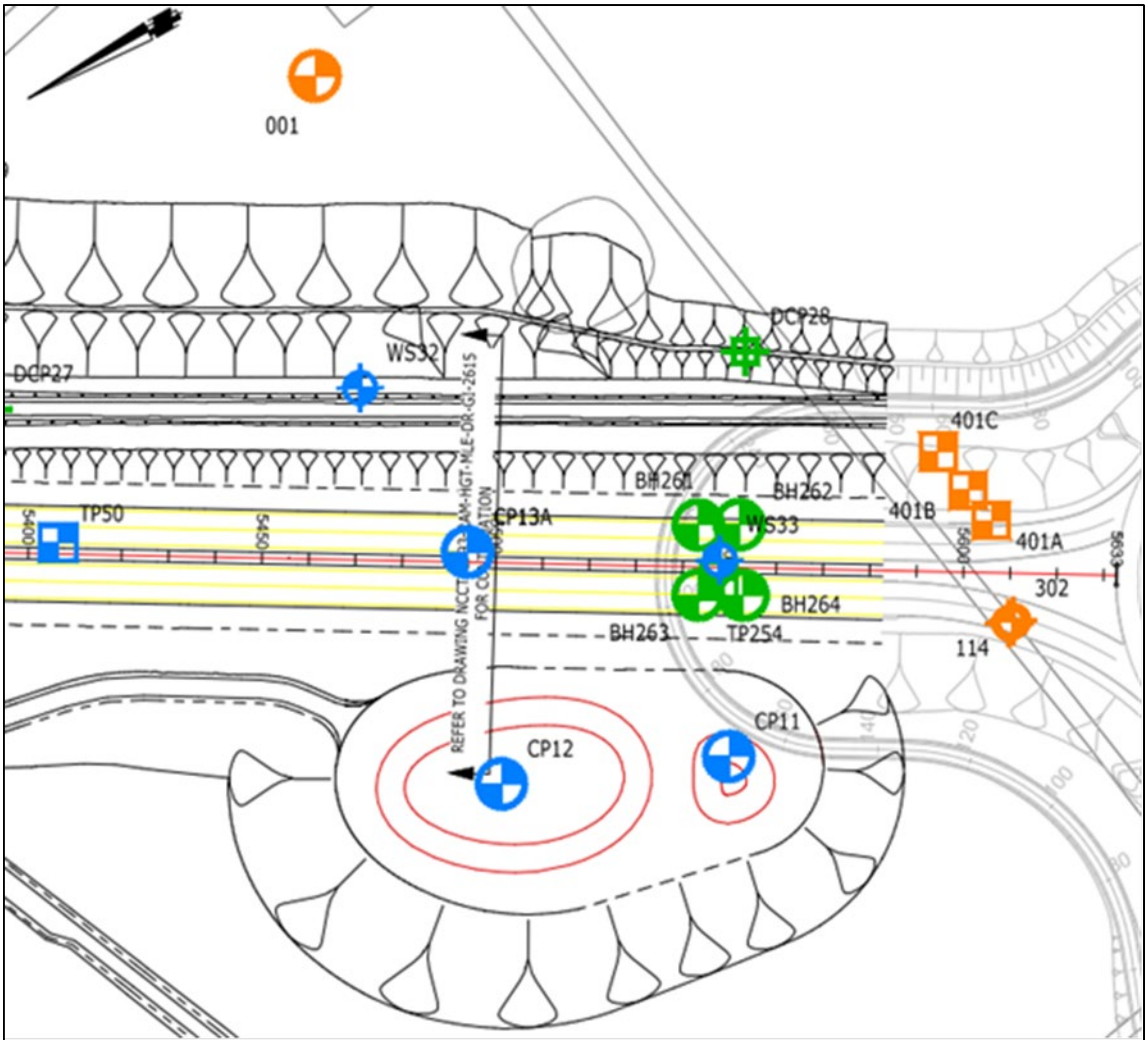
An interpretation of a groundwater profile using the data from the monitoring site is shown above. This indicates that the highest groundwater level is predicted to be 35 approx. +40.3m i.e., 1m above the base of basin 5 of +39.3m.

Conclusions are:

- the highest, anticipated groundwater level is +40.3, i.e., 1.0m above the base of basin 5;
- the basin lining would be compromised without a dewatering system in place.

Basin 6

Figure 16 – Borehole 6 Borehole plan



CP12 and CP11 were both installed within the Lowestoft Formation (Granular).

The highest monitored GW level at CP12 is 8.96 mbgl recorded in February 2023 and is above the Basin 6 base level.

A high groundwater level is noted in BH261 and BH262 within a cohesive layer with high granular content. These boreholes are circa. 40-50m away from CP11. This suggests water from the surface drains slowly through the Lowestoft Formation (Till) into the Lowestoft Formation (Granular).

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Table 9 Basin 6 Groundwater monitoring results

Location ID	Highest GW Depth recorded (m bgl)	Highest GW Elevation recorded (m AOD)	Lowest GW Depth recorded (m bgl)	Lowest GW recorded Elevation (m AOD)
BH261	0.47	49.5	4.4	45.57
BH262	0.39	49.42	1.08	48.73
BH263	1.08	49.19	5	45.27
BH264	8.13	42.07	8.13	42.07
CP12	8.96	42.72	9.41	42.27
WS33	0.57	49.51	3.66	46.42
114, 302, 401A, 401B, 401C, CP11, CP13, CP13A, DCP28, WS32	No groundwater strike encountered/ groundwater monitoring recorded as dry	No groundwater strike encountered/ groundwater monitoring recorded as dry	No groundwater strike encountered/ groundwater monitoring recorded as dry	No groundwater strike encountered/ groundwater monitoring recorded as dry

Figure 17 - Basin 6 cross-section

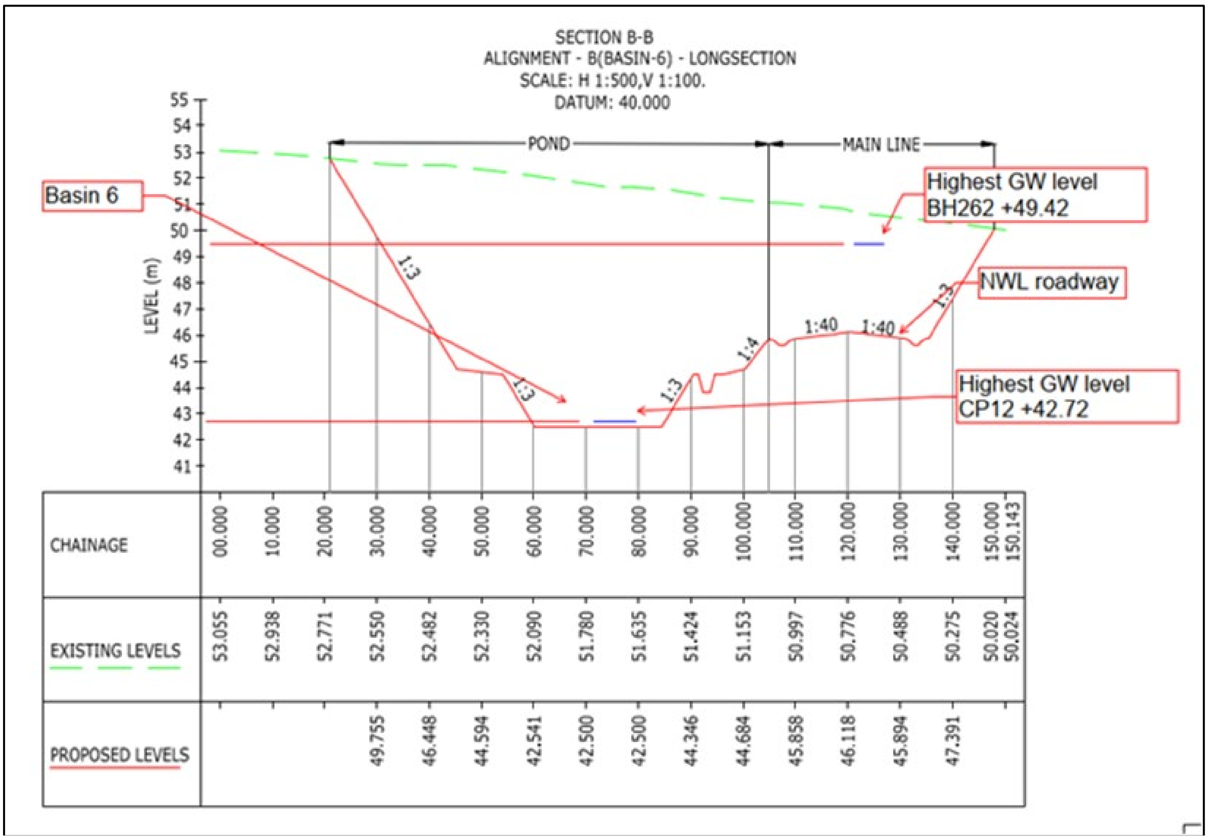


Figure 17 and Table 9 Basin 6 Groundwater monitoring results show that the highest, monitored groundwater level is +42.72 from CP12. The base level of basin 6 is +42.50.

Conclusions are:

- There is perched groundwater within the area of basin 6 and adjacent proposed Proposed Scheme;
- the highest, anticipated groundwater level in the basin area is +42.72;
- this equates to 0.22m above the base of basin 6;
- the basin lining would be compromised without a dewatering system in place.

Further data can be found in Ground Investigation Report NCCT41793-RAM-HGT-FSC-RP-GI-0002 in Appendix 14 (Reference 4.04.14).

5.2.7 Groundwater mitigation measures

Further to the conclusions on groundwater monitoring above, measures are proposed to reduce the groundwater levels around basins 5 and 6.

These include:

- laying a drainage blanket beneath the basin lining. The drainage blanket will enable groundwater redirection around the basin side slopes and below the impermeable liner. The basin lining prevents seepage of highway runoff into the groundwater;
- a filter drain within or beneath the blanket to directly discharge groundwater downstream to either the Tud tributary at basin 5 or to the culvert system beneath the planned A47 junction;
- a non-return valve to allow any excess water pressure beneath the lining to dissipate into the basin and discharge through the basin outlet (detail agreed with NCC maintenance team on 22/06/23– refer to Appendix 2 (Reference 4.04.02));
- narrow filter drains around the periphery of the access road. These would feed into and irrigate the planting in the scrapes;
- a permanent groundwater monitoring station built into a dedicated groundwater monitoring manhole in the drainage blanket. The manhole would be accessible at the access road level. This is to monitor any residual risk of high groundwater lifting and distorting the basin lining should the drainage blanket become blocked. A piezometer would be installed within the manhole with a datalogger and transmitter to give real-time readings to the operator by SMS transmission or similar. The station will be powered by a solar panel located in the access road adjacent to the groundwater monitoring manhole.

The groundwater risk assessment and associated consultations with the EA has been undertaken by WSP and is contained in the water quality assessments and the flood risk assessment (Reference 3.12.01 and 13.12.02 respectively).

5.2.8 SuDS Selection

Edge of road drainage will be designed in accordance with DMRB standards as described above, and embrace, where possible, the four pillars of SuDS:

- Water quantity (Flooding risk)
- Water quality (Pollution control)
- Amenity
- Biodiversity

It is good practice for SuDS to include an element of water quality treatment prior to discharging highway runoff to receiving waterbodies. The SuDS strategy for the proposed development has been developed from the principles outlined within the CIRIA C753 SuDS Design Manual along with BS 8582:2013 – Code of Practice for Surface Water management for Development Sites. A SuDS hierarchy has been followed in applying the use of sustainable drainage techniques. This has been set out in Table 10 with justifications provided where techniques are deemed feasible.

Highway surface water runoff is known to have contain pollutants including sediment and soluble metals. The SuDS features used will treat the runoff through settlement, filtering and uptake by vegetation. Water quality assessments have been undertaken in accordance with LA 113 to determine if additional mitigation measures to those described in Table 10 are required to meet water quality standards set out in the Environment Statement. The water quality assessment forms Appendix 12.1 of the Environmental Statement (Reference 3.12.01) and a summary of the mitigation measures is provided in section 10.

Landscaping and planting around drainage basins will be designed to not only provide pollution control but also to provide road users with enhanced amenity.

Table 10 SuDS Selection

SuDS Type	Can they be incorporated in the site?	Justification
Green roofs	No	No buildings within the Proposed Scheme
Basins and ponds	Yes	It is proposed to use a mix of attenuation and infiltration basins within the Proposed Scheme. Basins will also provide a level of treatment, notably with the inclusion of a sediment forebay, as well as amenity and biodiversity benefits.

SuDS Type	Can they be incorporated in the site?	Justification
Filter strips / Swales / Ditches	Yes	Filter strips will be used alongside swales to collect and convey surface water runoff from the highway to the basins. Drainage ditches will be utilised to convey surface water flows discharging from basins into the existing watercourses. Minor access road and NMU drainage will be conveyed along ditches with attenuation behind check dams prior to discharge to a watercourse. Both techniques will provide treatment of storm water.
Infiltration	Yes	Infiltration is proposed at several basin and open field sites alongside the NWL. Whilst the infiltration rates are low, the sites do not have a suitable watercourse nearby to discharge to.
Permeable surfaces	No	Permeable surfacing is not suitable for heavily trafficked areas as covered by the Proposed Scheme.
Filter drains	Yes	Filter drains are proposed to be used in the highway verge to collect and convey surface water to the basins. Filter drains supplement the capacity of swales.
Tanked systems	No	Not favoured by NCC due to poor accessibility for maintenance and limited water quality benefits.

5.2.9 Wensum Viaduct Deck Drainage

Along the viaduct section, surface water runoff from each carriageway shall be collected at the road edge using combined kerb drains (CKD). Drop pipes shall connect the CKD through the deck to a carrier drain underslung beneath each carriageway. The carrier drain shall include suitable support cradles. No drain sumps will be provided and access points shall be provided to the carrier drain at regular intervals which will be accessible if required from the underside during principal bridge inspections regime (every 6 years). The bridge deck outlets are hinged to allow for access at road level, and hatchboxes on the underside carrier pipe will be provided to enable more detailed inspection and maintenance during the principal inspections when suspended scaffolding and mobile working platforms are used. The bridge drainage design is shown in drawings PK1002-RAM-HDG-MLE-DR-DZ-0504 to 0505 (Reference 2.08.00) and PK1002-RAM-SBR-BR1-DR-CB-1796 (Reference 2.06.01).

The viaduct drainage network has been modelled for the design storm events described in CG 501 to determine the size of carrier drains to ensure no flooding of the carriageway for the 1 in 5 year event + 20% climate change. The hydraulic modelling identified surface level flooding for the 100 year + 45% climate change at locations shown in drawing PK1002-RAM-HDG-MLE-DR-DZ-0560 (Reference 2.08.02). The flood exceedance volumes calculated by hydraulic modelling have been translated to flood extents, maximum flood depths and exceedance duration as shown. All flood exceedances occur for the 100 year and are short-lived during the peak storms durations of 30 minutes due to downstream capacity in the CKD system. Results of the hydraulic analysis are provided in appendix 5 (Reference 4.04.05) (Catchment 2). In accordance with CG 501, there is no risk of flooding outside the Proposed Scheme highway boundary.

5.2.10 Ringland Lane Flood Modelling

As part of the Flood Risk Assessment (Reference 3.12.02), the flood alleviation measures for the overland flow path at Ringland Lane have been subject to further detailed modelling and assessment. This has included the provision of a flood bund and flow control structure to regulate flows upstream to mitigate flood risk to Keeper and Dell private property downstream of the project road.

The flood modelling report is provided in Appendix 9 (Reference 4.04.09).

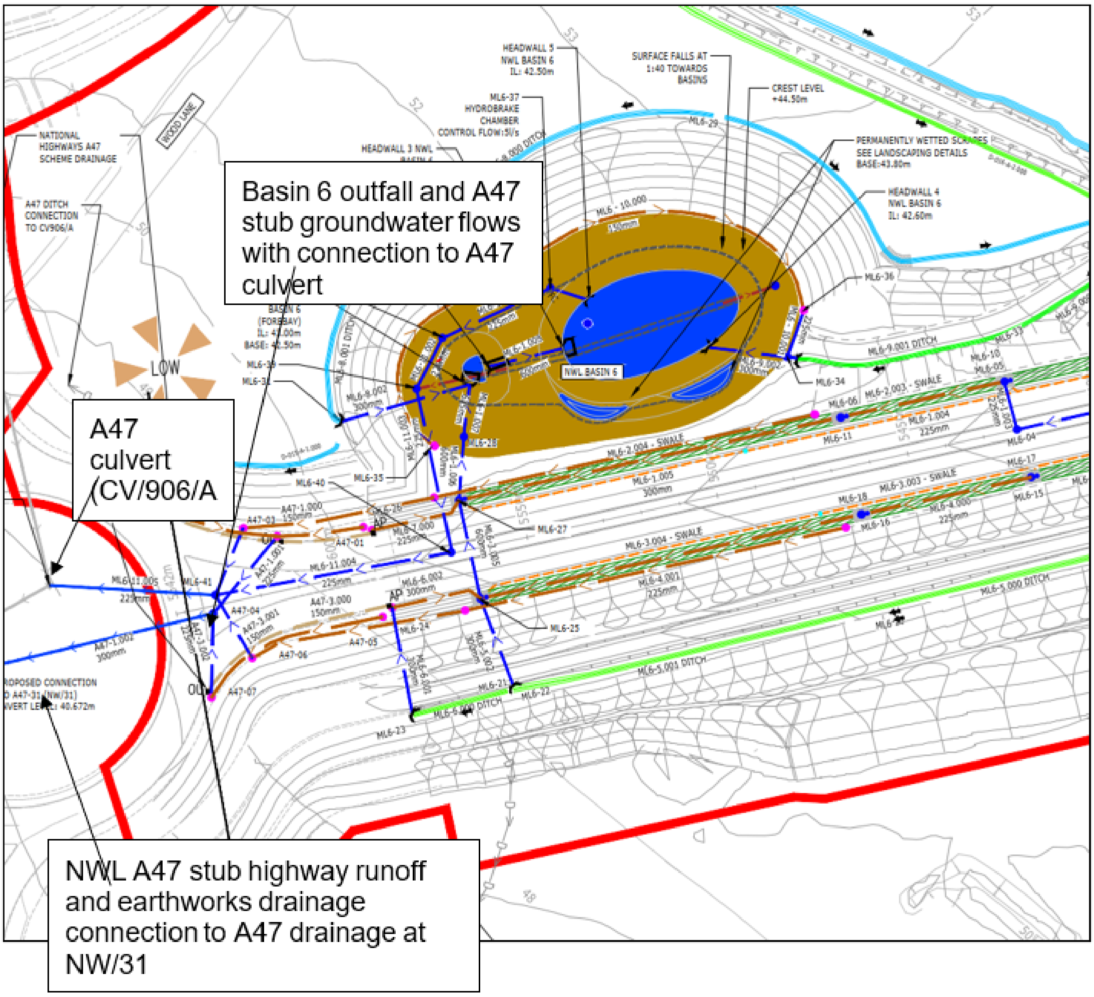
5.2.11 A47 Stub drainage

At the southern end of the Proposed Scheme, basin 6 discharges to the National Highways (NH) drainage network of the proposed A47 North Tuddenham to Eastern dualling scheme roundabout. The drainage of the Proposed Scheme connection to the A47 roundabout which is approximately 50m long and has an earthwork cutting also requires an outfall.

Options for the highway drainage for this A47 stub connection were explored which included relocating or deepening basin 6, but these were discounted due to increasing impact on land take and risk of intensifying groundwater problems. The option proposed involves the following and the layout is shown in Figure 18.

- continue to discharge from basin 6 into the proposed NH A47 scheme drainage system at culvert CV/906/A with an allowance for groundwater flows up to a total of 5 l/s discharge rate; and
- outfall of the NWL A47 stub highway runoff including earthworks runoff to the NH A47 drainage system at manhole NW/31 which leads to the NH A47 Basin.

Figure 18 - A47 Stub drainage and connection to NH drainage



A report covering the highway and drainage design of this section is included in Appendix 13 (Reference 4.04.13) (ref NCCT41793-RAM-HGN-ZZZ-TN-CH-0003).

When combining the Proposed Scheme A47 stub model with the NH A47 drainage model there were differences in design parameters used in modelling. The parameters affected were:

- MADD coefficient – the Proposed Scheme used 0 and NH used 2. This parameter is a global setting and cannot be applied to different pipe runs,
- Climate Change allowance – the Proposed Scheme used 45% and NH used 40%. This parameter can be adjusted indirectly by adjusting paved area to suit the climate change uplift.
- Percentage impermeable (PIMP) for cuttings – the Proposed Scheme used 14% and NH used 26% in accordance with CD 521 Table 5.6.2.

Several scenarios with different combinations of modelling parameters were explored (See Appendix 13 (Reference 4.04.13)). It was agreed to use “model 4” with different climate change allowances (40% for NH A47 drainage and 45% for the Proposed Scheme A47 stub drainage), MADD coefficient of 0 and different PIMP values (26% for NH A47 drainage and 14% for the Proposed Scheme A47 stub drainage) for the combined model.

The Proposed Scheme A47 stub drainage was designed to comply with all the design criteria described in 7.11 but for the assessment of the NH A47 drainage, only the 100 year + climate change allowance was used. The model results in Appendix 13 (Reference 4.04.13) show that pipes within the NH A47 drainage system that would need to be upgraded in order meet design criteria described in section 7.11 below. Although the NH A47 basin peak water level had increased, the minimum 300mm freeboard is still achieved so no updates are required to the NH A47 basin size.

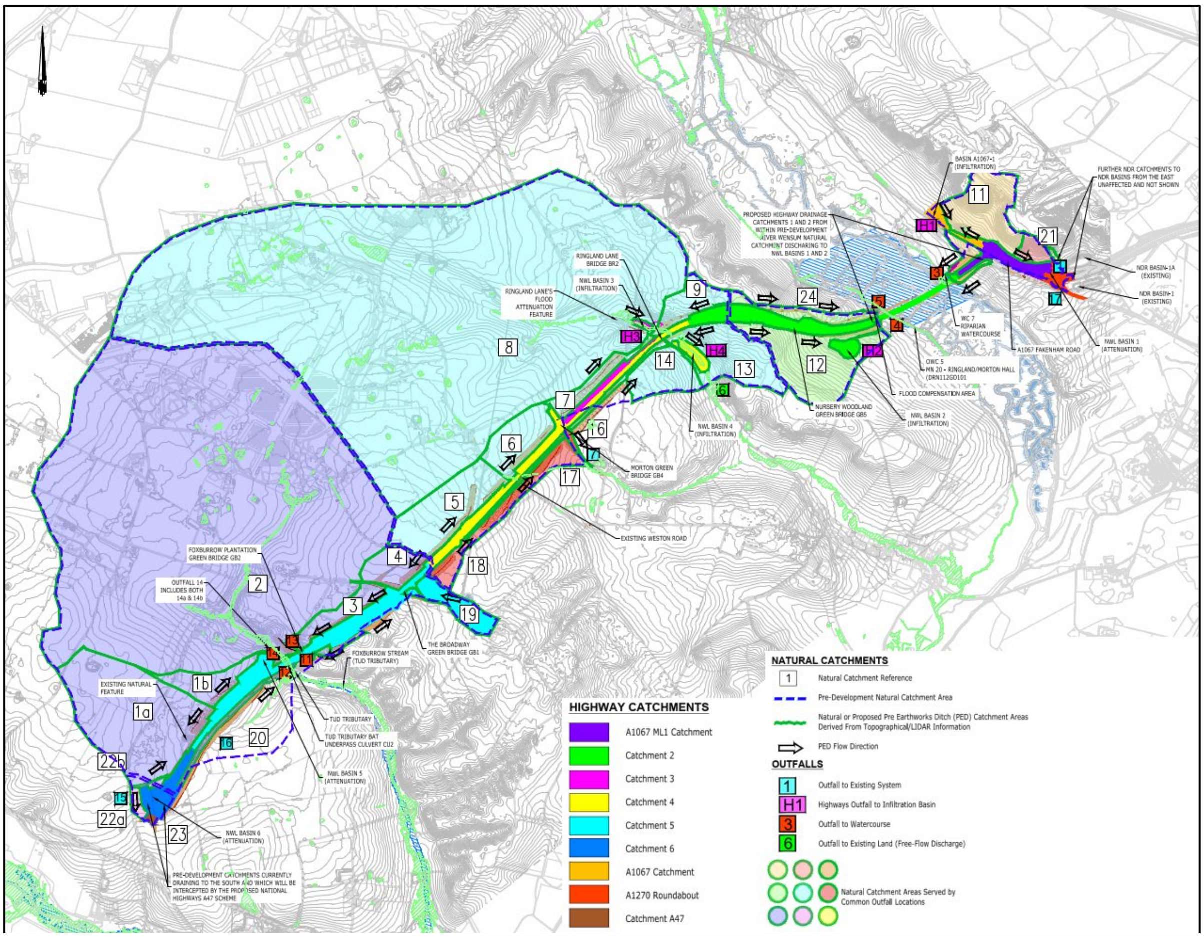
Further details of presentations, correspondence and the combined Highways/Drainage A47 Technical Note are provided in Appendix 13 (Reference 4.04.13).

6 Proposed Catchments

The upland catchments areas are shown on Figure 19 (from Drawing PK1002-RAM-HDG-MLE-DR-DZ-0502 Reference 2.08.00). Pre-development catchments are shown with dashed boundary lines and post-development catchments are coloured.

The illustration demonstrates that there is an increase in catchment area to outfall 6 as a result of the Proposed Scheme. Pre-development, catchments 5 and 6 flowed into an existing ditch that is now intersected by the Proposed Scheme. The resultant increased upland catchment flows at outfall 6 are managed within the spreader ditch area described later in this report. Refer also to the Flood Risk Assessment Report (Reference 3.12.02).

Figure 19 - Natural catchment area comparison to developed highway catchment areas



The Proposed Scheme crosses the natural drainage paths of the upland areas. These upland areas are sub-divided into natural catchments as shown above in Figure 19. Each natural catchment is intercepted by a Pre-Earthworks Ditch (PED) and runoff is conveyed in the direction of the arrows to a discharge point shown in the figure. Highway drainage through the drainage network into a watercourse via an attenuation basin or to an infiltration basin. Four different destinations for outfalls are illustrated:

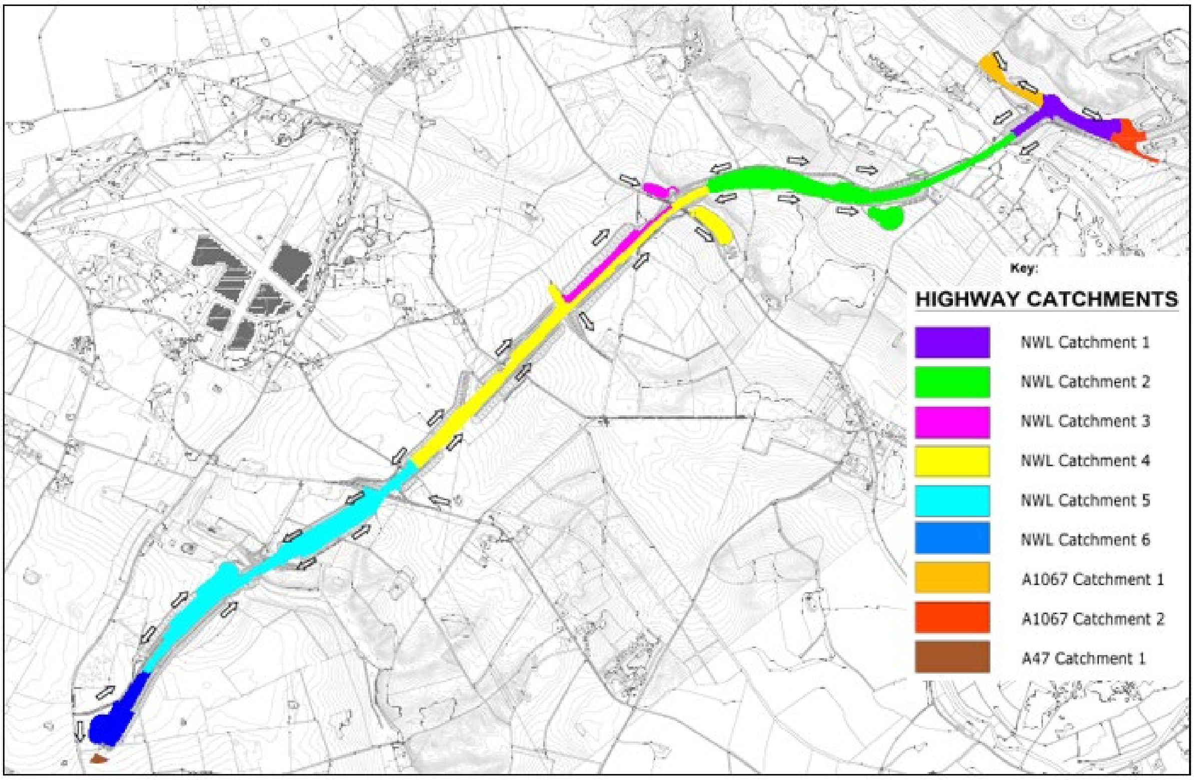
- outfall to an existing system: this would be an existing ditch or drainage system;
- highway drainage outfall to infiltration basin;
- outfall to a watercourse, via attenuation basins;
- outfall to existing land (free-flow discharge)

The discharge point of each upland sub-catchment is engineered to be the same as for the pre-development with one principal exception. This is at Ringland Lane (outfall 6). The total sub catchment areas before and after development vary and these are shown on Drawing PK1002-RAM-HDG-MLE-DR-DZ-0502 (Reference 2.08.00) and abstracted below in Table 11. For outfall 6, the catchment area has increased from 354.57 to 368.98 ha. The impact of this and the proposed mitigation is described in the Flood Risk Assessment.

Table 11 A comparison of pre and post development upland catchment areas.

Outfall	Final discharge	Pre-development catchment (ha)	Post-development greenfield catchment (ha)	Post-development highway catchment* (ha)
1	A1067 System	3.70	2.43	3.43
17	A1067 System	1.87	0.00	1.50
3	Riparian Watercourse - Wensum Floodplain	16.40	14.80	0.00
4 and 5	OWC5 - Wensum Floodplain	29.14	22.37	0.00
6	Ringland Lane Valley	354.57	368.98	0.00
11, 12, 13 and 14	Existing ditch to Wensum River	355.81	329.15	17.24
15	A47 drainage system	4.60	0.96	4.30
16	Existing ditch to Foxburrow Stream	43.14	35.29	0.00

Figure 20 - Highway sub-catchments



With the exception of the catchment for oufall 6, pre-development catchment areas have reduced, with the difference being made up by post-development highway catchments areas that consist of impermable areas. As stated in sections 4.3 and 4.6, additional runoff generated by the post-development areas will be attenuated to greenfield runoff rates.

The surface water drainage for the Proposed Scheme highway has been divided into nine individual sub-catchments (Figure 20). The sub-catchments serve the Proposed Scheme highway from the A1270 roundabout through a proposed new roundabout and along the Proposed Scheme dual carriageway to the junction with the A47. Reference should be made to drawing PK1002-RAM-HDG-MLE-DR-DZ-0502 (Reference 2.08.00) which identifies these catchments.

The Proposed Scheme highway sub-catchments are further sub-divided into zones of different permeability. These are designated 100% impermeable surfaces for roads and swales and 14% for verges. These zones are illustrated in Drawings nos. PK1002-RAM-HDG-MLE-DR-DZ-0520 to 0525 (Reference 2.08.05). The zones and their values are used as input to the hydraulic modelling described section 7.

7 Drainage Systems

The following section outlines the various methods of conveyance of the Proposed Scheme highway runoff prior to discharge to either attenuation or infiltration basins.

7.1 Swales

The design includes for conveyance of edge-of-road drainage using conveyance swales or grassed surface water channels (GSWC). The requirements for GSWC are described in DMRB CD 521, CD 532 and the Ciria SuDS Manual. Guidance in the SuDS Manual states that where gradients are less than 1.5%, underdrains should be added to allow runoff that infiltrates through the swale to be collected and conveyed to the downstream outfall. An impermeable liner is included in the design of the underdrain to contain the highway runoff.

Where swale gradients are less than 1.5% the SuDS Manual guidance has been adhered to except at the hogging sections of highway where the risk of flooding is low, i.e., at chainages +1600m, +3500m and +5200m (see **Figure 21** and **Figure 22**)

Inspection chambers are built on the underdrains at intervals not greater than 200m.

7.2 Filter drains

Filter drains are be laid in the base of swales (under-drains as discussed above) and placed at the base of embankments, cuttings and environmental bunds to provide a positive drainage to slopes greater than 1 in 3. They are designed in accordance with the criteria described in 7.12.

7.3 Carrier drains

A network of carrier drains will be installed to convey the highway runoff to the infiltration/attenuation basins. Carrier drains are designed in accordance with the criteria described in 7.12.

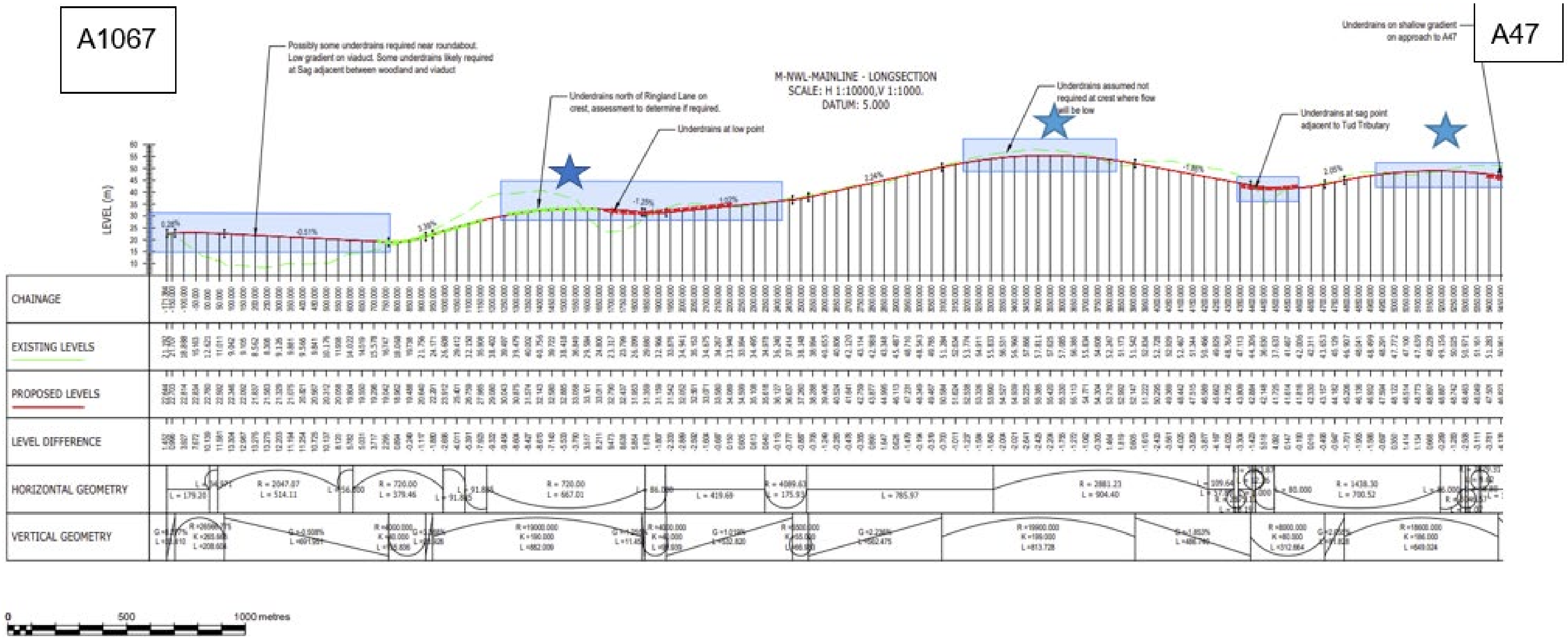
Pre-cast concrete headwalls or synthetic bagwork, filled with topsoil or topsoil and sand mix, will be provided at the pipe inlets and outlets to basins to prevent the areas from scouring (see 7.6).

7.4 Narrow filter drains/fin drains

A network of sub-surface narrow filter drains, or fin drains is proposed along the route of the Proposed Scheme highway to provide free drainage to the pavement subgrade, which includes sub-base and capping layers.

Figure 21 - Swale Underdrains. Plan of NWL showing where swale underdrains are omitted.

Figure 22 - Swale Underdrains Long section of NWL showing where swale underdrains are omitted.



7.5 Ditches

Where the earthworks of the Proposed Scheme intersect a natural catchment, pre-earthwork ditches (PED) and culverts are proposed to intercept natural runoff and convey it away from the external Proposed Scheme earthworks and towards infiltration features and adjacent watercourses where a suitable watercourse is available.

The general arrangement of the PEDs, the associated catchment areas, design flow paths and planned outfalls from the PEDs are described on drawing PK1002-RAM-HDG-MLE-DR-DZ-0502 (Reference 2.08.00). Table 12 also shows the outfall location/type for each intersected natural catchment area and identifies the approving authority. There are three proposed outfalls into managed watercourses and ten that require approval by the LLFA and one outfall to an existing basin that will require the approval by NCC. There are no planned outfalls directly into a main river.

Table 12 Schedule of outfalls, upland sub-catchments served and approval authorities

Outfall1	Outfall Type1	Discharging to2	Upland Catchment nos served	Highway Catchment no. served	Approval Authority3
1	Concrete headwall	NDR basin 1	21	1	NCC
3	Synthetic bags filled with topsoil or topsoil and sand mix connection	Riparian watercourse : WC7	11	n/a	IDB, privately maintained
4	Synthetic bags filled with topsoil or topsoil and sand mix connection	OWC: WC5 DRN112G0 101	12	n/a	IDB
5	Synthetic bags filled with topsoil or topsoil and sand mix connection	OWC: WC5 DRN112G0 101	24	n/a	IDB

Outfall1	Outfall Type1	Discharging to2	Upland Catchment nos served	Highway Catchment no. served	Approval Authority3
6	Spreader ditch	Field	5,6,7, 8,9,13, 14,15,17,18	n/a	LLFA
7	Ditch	existing ditch	16	n/a	LLFA
11	Lined ditch	Tud tributary	near 3	n/a	LLFA
12	Lined ditch	Tud tributary	1a	n/a	LLFA
13	Lined ditch	Tud tributary	3	n/a	LLFA
14 a and b	Lined ditch	Tud tributary	1b	n/a	LLFA

1 For upland catchments refer to Drg no PK1002-RAM-HDG-MLE-DR-DZ-0502 (Reference 2.08.00)

2 OWC Ordinary Watercourse

3 NCC, LLFA Lead Local Flood Authority, IDB Norfolk Rivers Internal Drainage Board

The hydraulic design of PEDs incorporates peak upland catchment flow rates for a return period of 1 in 100 years as determined from the greenfield runoff for the area plus 45% for climate change in accordance with EA guidelines for the wider catchment. Greenfield runoff rates have been obtained from the HR Wallingford website. The flow rates vary for different sub catchments, their size and equate to the following:

- 4.34 l/s/ha for small catchments up to 59 ha
- 3.43 l/s/ha for the large catchment 356 ha leading to outfall 6

The hydraulic assessment of PEDs was carried out using Manning's equation to estimate flow capacity based on the surface water runoff from the associated sub-catchment area accumulated with the area upstream. Design gradients were assessed from longitudinal profiles generated along the existing terrain using Autodesk Civils 3D.

Where the run of a PED is directed uphill, the ditch will either be made deeper to ensure a constant fall or, where greater than 1.5m deep, passed into a short piped ditch.

At the southern end of the Proposed Scheme, the topography of catchments 20, 22a and 23 naturally fall to the south and to the area planned by National Highways for a junction with the A47 at Wood Lane. Drainage of these areas will involve a connection to the NH drainage at the proposed junction (see section 5.2.11).

7.5.1 Scour protection

Where high velocities are reached due to steep gradients, a ditch lining system will be needed to act as scour protection for the PED. Guidance on deciding when scour protection is required is taken from:

1. "CIRIA C742 Manual on scour at bridges and other hydraulic structures".
2. "Technical note- Calculation of scour depth at bridge piers" by RUK, reference NCCT41793-RAM-SBR-BR1-TN-NZ-0003 date 7/11/2022. This is in relation to scour protection to piers beneath the NWL viaduct.
3. "CIRIA C683 2007, The Rock Manual. The use of rock in hydraulic engineering (2nd edition)".

Table 13 reproduces Table 1 from the technical note 'Calculation of scour depth at bridge piers'. This identifies the critical local point velocity above which scour of the soil material in the ditch would take place. The soil description within the subsoil of the ditches lies between fine sand and gravel and thus the critical velocity is judged to be 0.5m/s.

Table 13 Extract from Technical note Calculation of scour depth at bridge piers

Table 1 – Selected critical velocities for very fine sand to gravel particle sizes (CIRIA C683)

Material	Sieve size D (mm)	Critical velocity (m/s)
Gravel	5-2	0.60
Coarse sand	2-0.5	0.40
Fine sand	0.5-0.1	0.25
Very fine sand	0.1-0.02	0.20

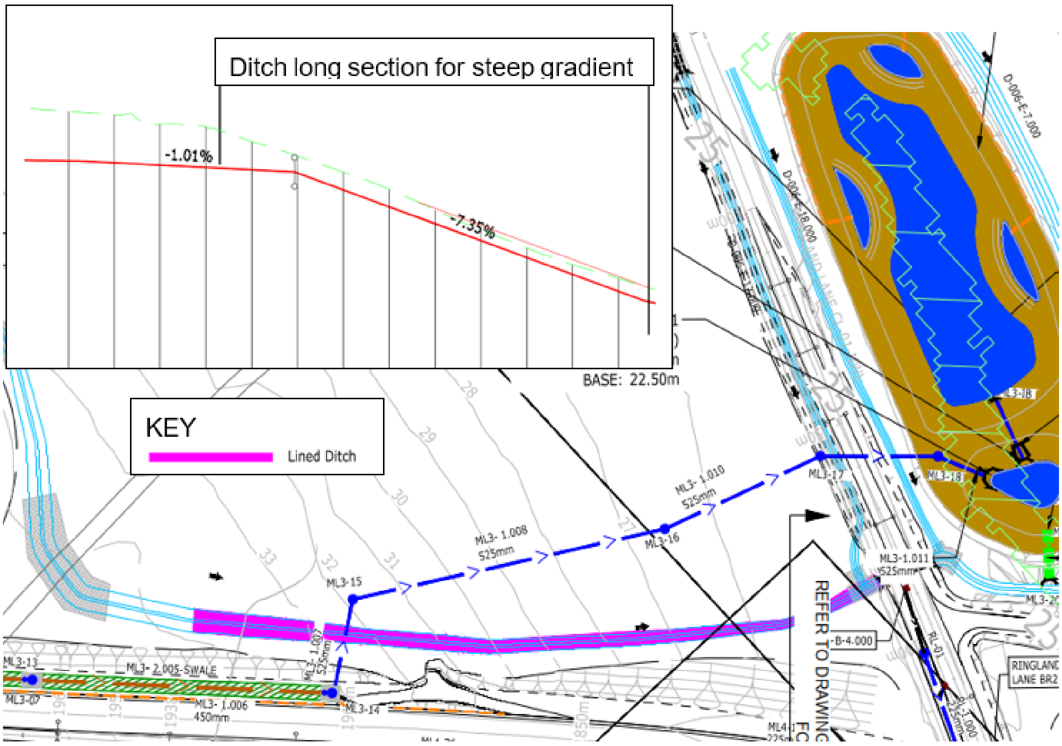
As the ditches will become established with vegetation, from the principles of Sediment Transport Theory, the threshold velocity for mobilisation of sediment particles has been increased to 1 m/s. This value is an average of the range of maximum allowable velocities for vegetated soils in Table 31.1 of the SuDS Manual which has a range of 0.6 to 1.5 m/s.

The actual depth of the design flows in ditches, described in section 7.5 has been calculated and used to determine the actual velocity. By rearranging Manning's open channel flow equation, where the gradient of the ditch is steep enough for the velocity to exceed 1 m/s, erosion protection measures are provided in the design.

The lengths of ditch where lining is required to prevent scour are highlighted on the drawings in pink with an example shown in Figure 23. The calculations for threshold gradient are included in Appendix 6 (Reference 4.04.06) for ditch calculations.

In the example shown, for ditch D-006-C-1.005, the depth of flow in the ditch for the 100 year event (26.51 l/s) has been calculated as 0.51m which will generate a flow of 1 m/s where the gradient is steeper than 3.5%. the section of the ditch shown in the pink in figure 22 has a gradient of 7. 35%.

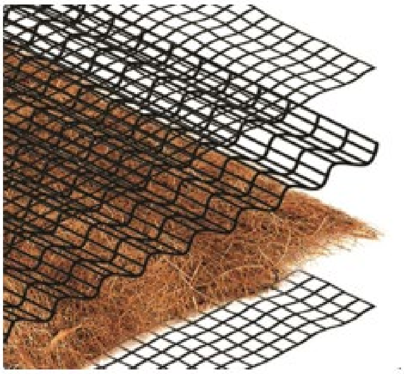
Figure 23 – Typical ditch lining locations



A study was made of suitable ditch lining materials including concrete canvas, concrete bagging, gabions, soil-filled bags and a mesh system. A suitability study was undertaken and the following was concluded:

- Concrete canvas and concrete bagging were considered not suitably aesthetic in the rural environment and were ruled out by NCC/LLFA in favour of alternative products.
- For straight lengths of steep gradient ditches with high velocities, a product called 'Vmax C350' composite turn reinforcement mat was determined to have the best qualities for scour protection and aesthetics in a rural environment, as well as to reduce the carbon footprint. This system is illustrated below in Figure 24. The mats would provide long-terms erosion protection and facilitate vegetation establishment. The mats will have resilience to the water flow velocities required in the design.
- Ditches will be lined at acute changes in direction such as at right-angle and tee-junctions. To prevent scour of the ditch sides and base, these areas will be lined with synthetic bags filled topsoil or topsoil and sand mix using 'Flex MSE' as illustrated in Figure 25. This is on the direction of NCC, reference 5. Each bag is linked to its neighbour with fixings. The bags are seeded to promote growth of grass to enable the ditches to blend in with the rural environment. A presentation on the use of this material for lining ditches was given to NCC on 24 April 2023, and a copy is included in Appendix 4 (Reference 4.04.04).

Figure 24 - Ditch lining materials for straight lengths



Index Property	Test Method	Typical
Thickness	ASTM D6525	18.54 mm
Resiliency	ASTM 6524	90%
Density	ASTM D792	0.917 g/cm^3
Mass/Unit Area	ASTM 6566	624 g/m^2
UV Stability	ASTM D4355 /1000 hr	86%
Porosity	ECTC Guidelines	99%
Stiffness	ASTM D1388	275990 mg-cm
Light Penetration	ASTM D6567	7.2%
Tensile Strength - MD	ASTM D6818	8.70 kN/m
Elongation - MD	ASTM D6818	45.3%
Tensile Strength - TD	ASTM D6818	10.20 kN/m
Elongation - TD	ASTM D6818	19.5%

Standard Roll Sizes	
Width	2.0 m
Length	20.0 m
Weight ± 10%	23 kg
Thread	40 m^2

- Matrix made of 100% coconut fibre
- Top and bottom netting made of UV-stabilized polypropylene and middle, corrugated UV-stabilised polypropylene

Figure 25 - Vegetated ditch lining system at acute changes in direction:

7.5.2 Attenuation in Ditches

Separate to the PEDs, highway ditches will convey surface water from the NMU and maintenance access tracks into the PED system. A series of check-dams are located along these ditches in order to attenuate flows before discharging to watercourses for up to the 100 year + 45% climate change event. Analysis of peak run-off from the NMU and maintenance access tracks has shown that check-dams at 24m intervals provides sufficient attenuation within the ditches to minimise discharge rates to watercourses to greenfield rates.

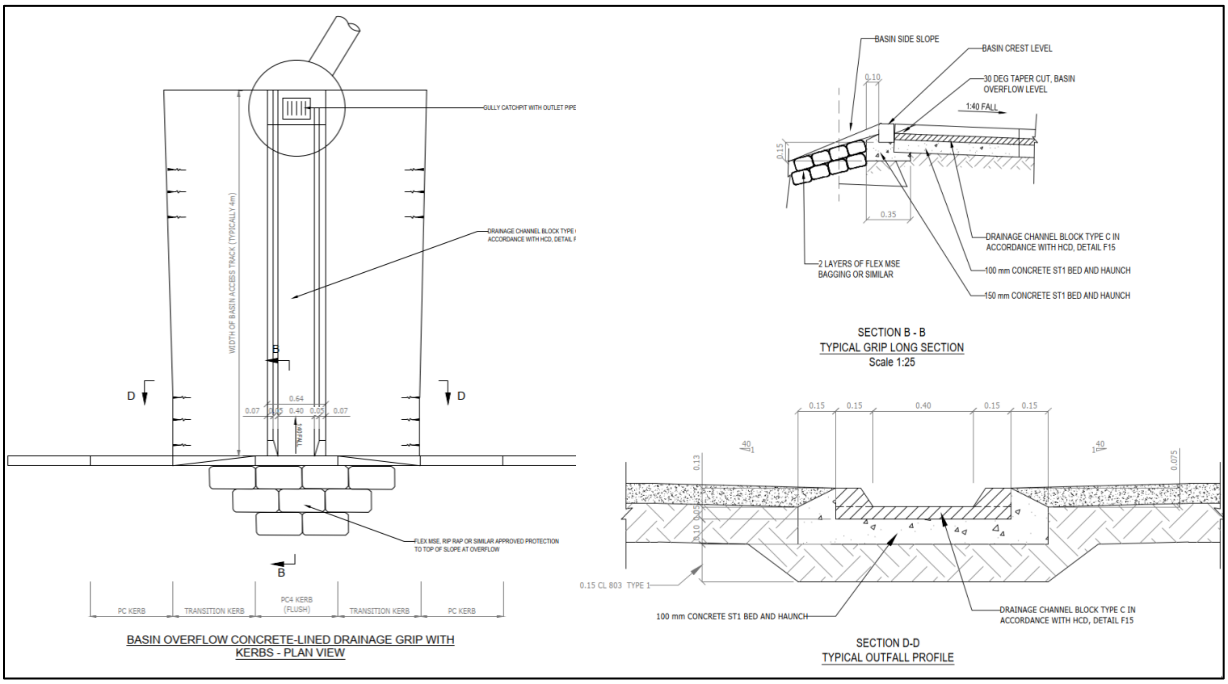
7.6 Discharge

It is proposed that the highway surface water runoff is discharged either by infiltration to the ground or will be attenuated prior to discharge to an existing watercourse or existing basin.

Basin overflows will be provided to each attenuation and infiltration basin with outfall overflow from the forebay. This is to prevent the overtopping of any basin in an extreme event such as a blockage between forebay and main basin. The risk of not having an overflow is damage the crest of the basin and potential flooding and injury to receptors downstream of the basin.

The overflow consists of a weir which is set into the side of the basin leading to a watercourse through a field ditch. Details are shown in Figure 26. The overflow discharge rates are based on the full-bore capacity of the trapezoidal channel that flows to the field ditch. Flowrates are given in Table 15 and in each basin drawing.

Figure 26 - Overflow Weir Detail at Basins



7.7 Infiltration basins

NWL Catchments 2, 3, 4, and A1067 Catchment 1 discharge to shallow infiltration basins with a depth of 2m or less below ground level. The establishment of infiltration rates from site testing is described in section 5.2.

A1067/1 catchment discharges, via a sediment forebay feature (A1067/1 Forebay 1), to a shallow infiltration basin (A1067/1 Basin 1). Reference Drawing No. PK1002-RAM-HDG-MLE-DE-DZ-0541 (Reference 2.08.01)

NWL Catchment 2 discharges, via a sediment forebay feature (NWL Forebay 2), to a shallow infiltration basin (NWL Basin 2). Reference Drawing No. PK1002-RAM-HDG-MLE-DE-DZ-0543 (Reference 2.08.001)

NWL Catchment 3 discharges, via a sediment forebay feature (NWL Forebay 3), to a shallow infiltration basin (NWL Basin 3). Reference Drawing No. PK1002-RAM-HDG-MLE-DE-DZ-0544 (Reference 2.08.01)

NWL Catchment 4 discharges, via a sediment forebay feature (NWL Forebay 4), to a shallow infiltration basin (NWL Basin 4). Reference Drawing No. PK1002-RAM-HDG-MLE-DE-DZ-0545 (Reference 2.08.01).

The history of the development of basins 3 and 4 over 2020 and 2022 is described in a technical note NCCT41793-RAM-HDG-MLE-TN-DZ-0002 included in Appendix 10 (Reference 4.04.10).

Design Infiltration values

Refer to Table 7 for the design infiltration rate used for each infiltration basin and Appendix 1 (Reference 4.04.01) for a summary of the different infiltration test results. A factor of safety (FoS) is applied to the results of percolation testing at each site in line with The SuDS Manual Table 25.2 (reproduced in Figure 27). Although in most cases it can be assessed that there is no damage or inconvenience as a consequence of failure, the basins are to be adopted by NCC highways and it is required to use the middle column of Table 25.2 for the FoS as stated in the LLFA Guidance Document.

Figure 27 - Table 25.2 from the SuDS Manual

Table 25.2: Suggested factors of safety, F, for use in hydraulic design of infiltration systems (designed using Bettess (1996). Note: not relevant for BRE method)

Consequences of failure

Size of Area to be drained	No damage or inconvenience	Minor damage to external areas or inconvenience (eg surface water on car parking)	Damage to buildings or structures, or major inconvenience (eg flooding of roads)
<100 m^3	1.5	2	10
100 – 1000 m^3	1.5	3	10
>1000 m^3	1.5	5	10

Infiltration Basin Design

The infiltration basins have been sized based on the incoming rate of flow from the upstream carrier drains and the design infiltration rate. Design aspects for each infiltration basin are provided in Table 7. Infiltration through the base and sides is assumed using the Bettress method (1996) with the values establish from test results at each site with an applied factor of safety (FoS) as described in the table above extracted from The SuDS Manual.

All basins have been designed with a minimum 300mm freeboard.

The half drain time has been determined and where a time of 24 hours is exceeded, a check has been undertaken to ensure that additional runoff volume equivalent to a 1 in 10 year storm can be stored within the available volume and freeboard. A climate change allowance of 40% is added to 1 in 10 year storm as directed by LLFA.

7.8 Attenuation basins

NWL catchments 1, 5 and 6 and A1067 catchment 2 discharge to attenuation basins. The justification for this is described below. Refer also to detailed basin drawings PK1002-RAM-HDG-MLE-DE-DZ-0541 to 548 (Reference 2.08.01).

NWL Catchment 1 (drawing PK1002-RAM-HDG-MLE-DE-DZ-0542 (Reference 2.08.01)) discharges, via NWL Basin 1, at a controlled rate to the existing infiltration basin located north-east of the A1067/NDR junction (NDR Basin 1A). NDR Basin 1A is designed to intercept overland flow from an approximately 117 hectare catchment to the north of the NDR. After assessment, it is considered that there is sufficient capacity within this basin to accommodate additional inflow, with no modifications to NDR Basin 1A required.

- Based on previous work undertaken by others it is considered that a controlled discharge rate of 43 l/s would provide a reasonable balance between the required storage volume in NWL Basin 1 and the maximum depth of water in NDR Basin 1A. Justification and details for the proposed drainage strategy for NWL Catchment 1 can be found in Appendix 8: NDR Basin 1A Drainage Analysis – Technical Note, WSP, August 2023 (Reference 4.04.08).

NWL Catchment 5 (drawing PK1002-RAM-HDG-MLE-DE-DZ-0546 (Reference 2.08.01)) discharges to an attenuation basin with integral sediment forebay (NWL Basin 5) which outfalls to the Tud tributary (Foxburrow stream). Infiltration as a method of disposal is not feasible due to the high groundwater table at the site. Discharge rates from this basin have been restricted to the lesser of greenfield runoff rates (Table 14) or 2 l/s/ha of catchment area by the 'Simple' method (LLFA Guidance Document ref. paragraph 14.13). The catchment area is 6.77 ha. The greenfield runoff rates, as determined by the calculator available from HR Wallingford, are described in Appendix 7 (Reference 4.04.07). It is proposed to have a single flow control to limit the discharge to the watercourse.

Table 14 Design flow control rates for NWL catchment 5

Rainfall return period	Greenfield runoff rate (l/s)	Flow control rate (l/s)
1 in 1 year	13.82	10.2
1 in 30 year	31.71	15.7
1 in 100 year	40.32	18.5

The A1067 consists of catchments discharging to basins A1067-1 to the north-west and NWL Basin 1 to the south-east. The existing A1067/A1270 roundabout currently drains to NDR basin 1 and NDR basin 1A. There will be a minor increase in impermeable area of the existing roundabout (widening to the north to accommodate future traffic flows) – however there will be a reduction in contributing area from the upland catchment area and A1067 carriageway which, once dualled, will discharge to the new NWL Basin 1.

NWL Catchment 6 (drawing PK1002-RAM-HDG-MLE-DE-DZ-0547 and 0548 (Reference 2.08.01)) discharges, via a sediment forebay feature (NWL Forebay 6) to an attenuation basin (NWL Basin 6). The discharge from basin 6 is controlled using a hydrobrake to a rate of 4 l/s. The discharge from basin 6 will be connected to NH drainage system for the proposed National Highways A47 scheme.

Basin 6 was initially considered for discharge through infiltration. However, during site testing in 2022, the soil at the basin elevation was found to have a poor drainage capability i.e., having an infiltration rate of less than $1x10^{-6}$ m/s which is the minimum practical value recommended for infiltration (ref SuDS Manual ref. 25.2.1). This rate would result in very long drain-down times in an infiltration basin following the design extreme event. Refer also to Appendix 1 (Reference 4.04.01) for infiltration test results. In addition to soakaway testing, groundwater monitoring undertaken at basin 6 and the surrounding area show that groundwater reached levels close to the invert of the proposed basin so infiltration will not be possible at this location. Section 5.2.6 provides detailed information of the groundwater monitoring information. Basin 6 will be an attenuation basin for the above reasons.

Attenuation Basin Design

The attenuation basins have been sized based on the incoming flow from the upstream highway catchment. The discharge rate at each location is controlled to the lower of either:

- the greenfield runoff rates for each respective return rainfall event from the equivalent catchment (complex method); or
- 2 l/s/ha of catchment (simple method).
- For basin 6, the discharge rate has been agreed as 4 l/s as described in 5.2.11.

The additional volume required for long-term storage can be accommodated within the forebay and main basin volumes and as such, no additional storage is needed. The design aspects for each attenuation basin are provided in Table 15 Basin design summary.

All basins have been designed with a minimum 300mm freeboard and an overflow through the hydrobrake chamber. The overflow details and levels are shown in the basin detail drawings.

Information and safety warning signs will be erected around each basin as follows:

- direction signs pointing to pollution control devices (PCD) for alerting fire services or maintenance crews (normally pole-mounted);
- client notice board pole-mounted adjacent to the entrance gate; and
- deep water warning signs to members of the public; affixed to the peripheral fence at suitable locations and centres.

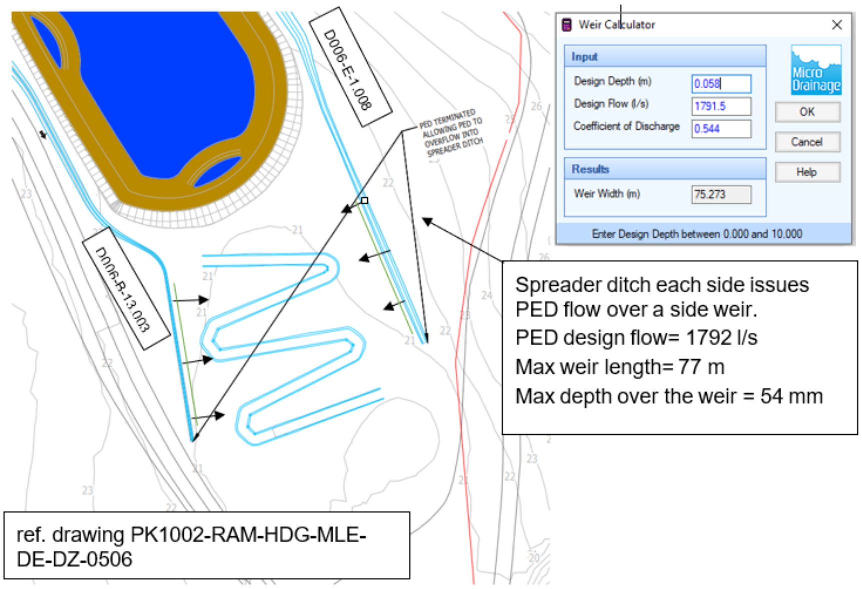
7.9 Spreader Ditch

A spreader ditch is proposed in the natural valley that receives and distributes flows from the PEDs at either side of basin 4. This arrangement is required to discharge PED flows at outfall 6 to the field without a watercourse and reflects the existing natural overland flow regime in this area.

The spreader ditch will be located between the two PEDs serving the upland catchment above Ringland Lane and is a form of SuDS source control. Each of the PEDs run almost flat along the north and south sides of the valley and end at the 21m contour mark. PED flows for the extreme (100 year) event will overtop like an overflow weir and continue towards the spreader ditch located at the lower valley level where it is contained and conveyed through the meandering channel. The long, wide, meandering channel arrangement promotes attenuation, as well as natural infiltration to assist with managing flood risk of the upland catchment flows.

The arrangement is illustrated in Figure 28, as well as an extract of the hydraulic modelling to simulate the effect of an overflow weir for the larger PED flow (D006-E-1.008). The Ringland Lane flood modelling study described in 5.2.10 and detailed in the technical note in Appendix 9 (Reference 4.04.09) provides a more detailed 2D analysis of overland flows through this area and gives further details of how this arrangement performs for up to the 100 year event.

Figure 28 - Spreader Ditch Flow Theory

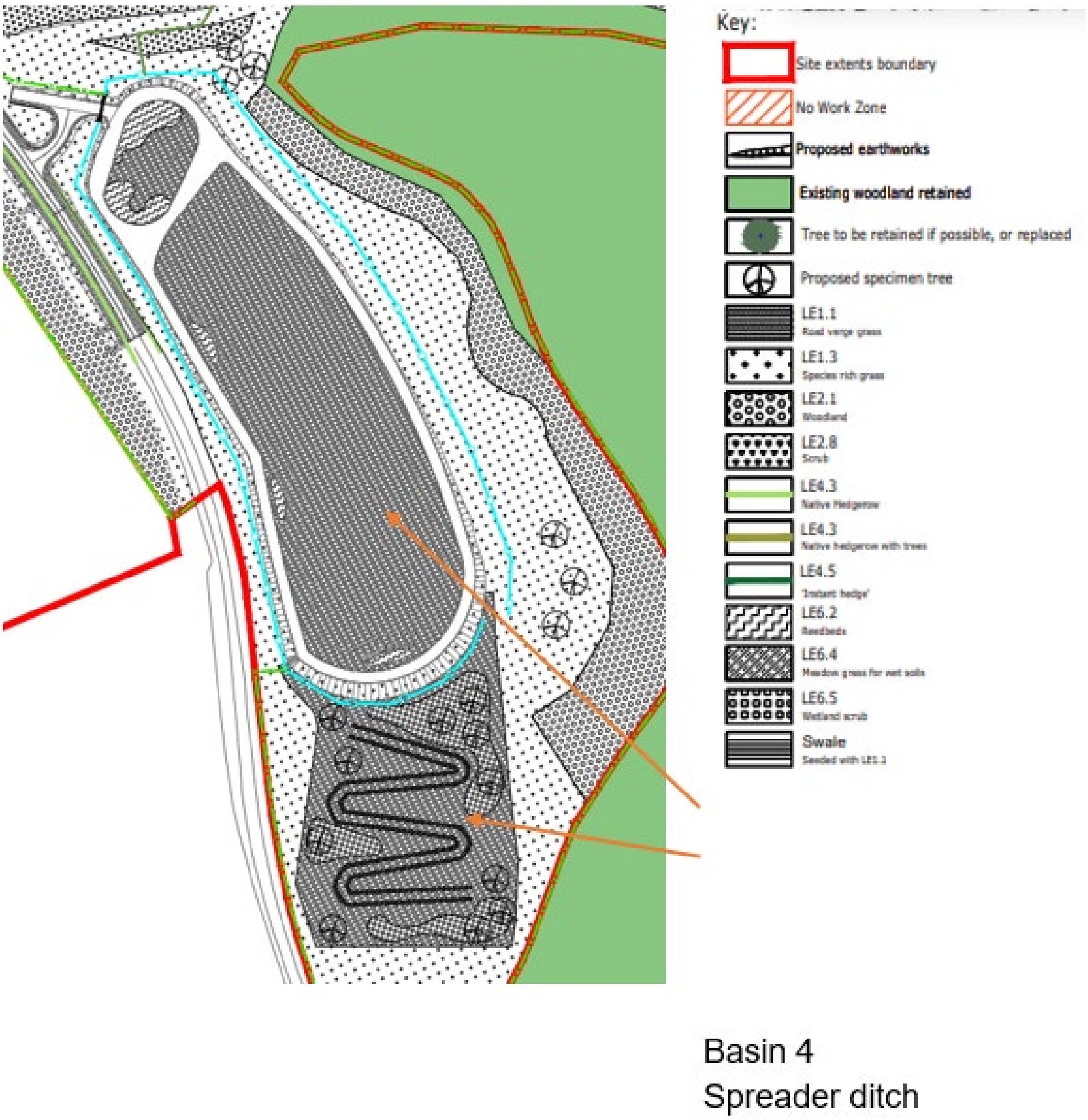


The area will be vegetated, making use of the water and increasing biodiversity, as shown in Figure 29.

In summary objectives of the spreader ditch are:

- to maintain the pre-development upland flow path without increasing flood risk and with minimum disruption to the field surface;
- to provide biodiversity and amenity.

Figure 29 - Spreader ditch biodiversity opportunities created by planting and adding a meandering ditch.



7.10 Amenity and Biodiversity

The inclusion of basins provides opportunities for amenity and biodiversity net gain through enhanced landscaping. Full landscaping proposals are described on drawings nos. PK1007-RAM-ELS-MLE-DR-NZ-0001 to 0011 inclusive (Reference 2.07.00), and extracts are illustrated here.

Amenities for the public will be provided by erecting information boards and benches along the non-motorised user (NMU) routes, offering vistas across basins and ponds. Refer to the proposals in Appendix 12 (Reference 4.04.12).

Figure 30 - Information boards and benches

- Make use of the proposed/existing NMU/PRoW network
- Provide post mounted QR code points where the public could access online information boards which could provide detail on:
  - Local landscape
  - Natural history
  - Cultural heritage
  - Information about the NWL project

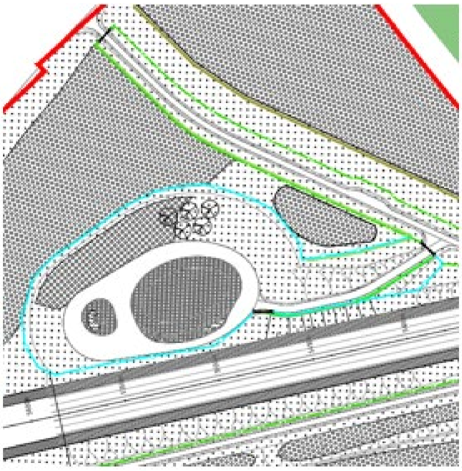
Incorporate benches along the NMU/PRoW network with views across ponds and the local landscape – potential to use site won timber

Amenity Value (3rd pillar of SuDS)

Biodiversity gains will come with planting opportunities in permanently wet and dry areas of basins. See landscaping plans below and the WSP Biodiversity report (Reference 3.10.33).

Figure 31 - Basin Plans Showing Typical Landscaping Proposals

Basin 5



Basin 6

LE6.4_Meadow grass for wet soils (Emorsgate EG8) @ 5g/m2

Botanical Name	Common Name	Specification	Mix %
Agrostis capillaris	Common bent	Seed; British native-origin	12.5
Alopecurus pratensis	Meadow foxtail	Seed; British native-origin	3.75
Anthoxanthum odoratum	Sweet vernal grass	Seed; British native-origin	3.75
Briza media	Quaking grass	Seed; British native-origin	3.75
Cynosurus cristatus	Crested dogstail	Seed; British native-origin	30
Deschampsia cespitosa	Tufted hair grass	Seed; British native-origin	2.5
Festuca rubra	Slender creeping red fescue	Seed; British native-origin	40
Hordeum secalinum	Meadow barley	Seed; British native-origin	3.75

Meadow grass areas require 200mm topsoil, 250mm subsoil.

Overall area (m2)= 42890
Total LE6.4 seed required (g): 214450

LE6.2_Reed Planting @ 5 plants / m2

Botanical Name	**Common Name**	**Specification**	**Mix %**
Butomus umbellatus	Flowering rush	Clump; 3 buds; minimum 1 year old; British native-original	10
Carex acuta	Slender-tufted sedge	Clump; 3 buds; minimum 1 year old; British native-original	10
Carex paniculata	Greater tussock sedge	Clump; 3 buds; minimum 1 year old; British native-original	15
Carex riparia	Greater pond sedge	Clump; 3 buds; minimum 1 year old; British native-original	15
Carex rostrata	Bottle sedge	Clump; 3 buds; minimum 1 year old; British native-original	10
Carex vulpina	Fox sedge	Clump; 3 buds; minimum 1 year old; British native-original	10
Eleocharis palustris	Common spike-rush	Clump; 3 buds; minimum 1 year old; British native-original	15
Juncus acutiflorus	Sharp-flowered rush	Clump; 3 buds; minimum 1 year old; British native-original	15

Reed planting areas require 300mm soil depth.

Overall area (m2)=	1656
Total LE6.2 seed required (g):	8280

Basins are non-uniform in shape to reflect natural ponds and lakes and will be sympathetic to the surrounding location. Scrapes (shallow areas) are created in the basin edges with a depth of 0.3m an additional 0.3m of topsoil above the basin liner. Each basin will have a sediment forebay with an overall depth of between 0.3m and 0.5m. These will assist to create permanent wet areas without affecting the operation of the basin. Scrapes and forebays will be planted with reeds and sedges to aid removal of contaminants in runoff.

Basins will have a 200mm topsoil layer over the 250mm subsoil or basin liner which is planted with an aquatic grass mix that can tolerate prolonged periods without water.

Basin edges are to be planted with a wet meadow grassland mix, to provide colour and interest and attract invertebrates and mammals.

All grasses will be long, only requiring a yearly maintenance cut. Long grass will promote a deterrent to birds using the basin area as a landing/ grazing area, as part of the safety management policy of the adjacent airport.

Topsoil will have a light, sandy texture to promote drainage and percolation to ground in infiltration basins. The subject of topsoil suitability within infiltration basins is described in Appendix 11 (Reference 4.04.11) which is based on the recommendations of the SuDS Manual. The technical note in Appendix 11 (Reference 4.04.11) confirms that topsoil required to meet the permeability criteria for use for infiltration basins is in the 8.1×10^{-7} to 3.05×10^{-6} m/s range which can be achieved with site-won, non-cohesive topsoil that has predicted infiltration values in the 1×10^{-7} to 1×10^{-5} m/s range.

Green bridges are planned at several road crossings along the Proposed Scheme. They will be arranged as for green roofs with planting cells separated by baffle walls. This is to retain most of the moisture to encourage growth, although drain slots are included at vertical joints.

7.11 Hydraulic Modelling

Hydraulic models for each of the carrier drain networks, including the Wensum viaduct carrier drains, have been developed using MicroDrainage (Innovyze). The calculation outputs are provided in Appendix 5 (Reference 4.04.05). The hydraulic design parameters are based on the client's requirements and in accordance with the National Highways Design Manual for Roads and Bridges (DMRB) design standards. The components of the drainage networks include edge of road drainage including carrier drains, filter drains, fin drains, gullies, swales, attenuation basins, infiltration basins, associated forebays and PEDs with attenuation.

Input parameters and design criteria for the drainage modelling include:

- Rainfall data

  a) Flood Studies Report (FSR) is not used in favour of the Flood Estimation Handbook (FEH) point descriptor method which is used to obtain the optimum forecast rainfall data;

b) 1 in 1 year rainfall event to ensure no flooding or surcharge from the formation/sub formation where filter drains where are used and no surcharge occurs in pipes, swales, and manholes. FEH1999 1 year return period rainfall profile with a climate change coefficient of 20% (1);

c) 1 in 5 year rainfall event: to ensure no flooding or surcharge from the formation/sub formation where filter drains where used and no surcharge occurs in pipes, swales, and manholes within a centre reservation. FEH2013 5 year return period rainfall profile with a climate change coefficient of 20%;

d) 1 in 10 year rainfall events for adding to the volume of infiltration basins, see (g) below;

e) 1 in 30 year rainfall event for assessing exceedance flows within the drainage network: FEH2013 30 year return period rainfall event plus 40% climate change;

f) 1 in 100 year rainfall event for assessing flood exceedance routes from carriageway to ensure no flooding outside the Proposed Scheme highway boundary, design of flood exceedance routing within the sub-catchment of drainage networks: FEH2013 100 year return period rainfall event plus 45% climate change;

g) 1 in 100 year FEH2013 100 year return period rainfall profile plus 45% climate change storm event: design of infiltration and attenuation basin capacity excluding freeboard where half drain down time is 24 hours or less;

h) 1 in 100 year FEH2013 return period rainfall profile plus 45% climate change storm event plus a 1 in 10 year FEH2013 return period rainfall profile plus 40% climate change for the design of infiltration basins where half drain down time exceeds 24 hours. The capacity also includes volume in the nominal 300mm high freeboard.

Notes:

1) FEH 2013 data does not include for 1 in 1 year return period data;

2) Refer to advice in a LLFA letter dated 27 April 2023 ref FW2023_0343_ included in Appendix 2 (Reference 4.04.02). The climate change allowance for a 3.3% AER storm, 2070s epoch Upper End Allowance is 40%.

Each sub-catchment has been sub-delineated within the software to represent the individual areas draining to each manhole, pipe or swale run. A runoff coefficient of 100% has been used for impermeable carriageway (PIMP value), access roads and basin areas. Runoff coefficient values consistent with the guidance in CD521, have been used for internal earthworks (cuttings and verges) and swales which is 14% from Tables 5.6.2 / 5.6.3 of CD 521 for Norfolk.

The highways catchment plans (drawings PK1002-RAM-HDG-MLE-DR-DZ-0520 to 0525 (Reference 2.08.05)) show the impermeable area and overall extents of each highway drainage catchment. The table in the drawings show the impermeable and permeable areas which have the permeable area runoff coefficient of 14% applied.

For flood exceedance routing the locations of flooding events and flood volumes resulting from rainfall events in (f) above are described in drawings PK1002-RAM-HDG-MLE-DR-DZ-0560 to 561 (Reference 2.08.02). The flood exceedance checks in (e) are satisfied by carrying out the checks of the more extreme event in (f), i.e. there is no surface level flooding for the event for 1 in 30 year + 40% climate change. DMRB CG 501 states that flooding for the 100 year event + climate change must not leave the highway boundary and enter third party land, so some flooding on the carriageway is permitted, which is indicated in the flood exceedance drawings.

The locations of the carriageway drainage and proposed attenuation / infiltration features are shown on the design drawings PK1002-RAM-HDG-MLE-DR-DZ-0503 to 0512 (Reference 2.08.00). Further details of the key design aspects of each sediment forebay, infiltration basin and attenuation basin are provided in Table 15 Basin design summary below.

Table 15 Basin design summary

Further details of the key design aspects of each sediment forebay, infiltration and attenuation basin are provided in Table 15 on the following pages.

Basin	Northern Section NWL Basin 1	Northern Section A1067 - 1	Middle Section NWL Basin 2	Middle Section NWL Basin 3	Middle Section NWL Basin 4	Southern section NWL Basin 5	Southern section NWL Basin 6
Chainage (m)	A1067 Dual - CH300	N/A	Southbound CH700	Northbound CH1700	Southbound CH1700	CH4550	CH5500
Gross area (ha)	3.43	1.77	9.83	2.55	9.12	13.49	3.85
Verge and embankment area (ha)	1.38	0.94	4.14	1.06	2.76	6.72	1.75
Drained equivalent impermeable area (ha)	2.05	0.83	5.69	1.49	6.36	6.77	2.10
Discharge type	Attenuation outfalling to NDR Basin	Infiltration	Infiltration	Infiltration	Infiltration	Attenuation outfalling to Tud Tributary	Attenuation
Side slopes	1:3	1:3	1:3	1:3	1:3	1:3	1:3
Lined/Unlined	Lined	Unlined	Unlined	Unlined	Unlined	Lined. Drainage blanket.	Lined. Drainage blanket.

Infiltration testing

Basin	Northern Section NWL Basin 1	Northern Section A1067 - 1	Middle Section NWL Basin 2	Middle Section NWL Basin 3	Middle Section NWL Basin 4	Southern section NWL Basin 5	Southern section NWL Basin 6
Associated trial pit(s)	TP51	TP04, TP05, TP205, TP206, TP207, TP208	TP209, TP223, TP224	TP 222, TP229, TP230	TP11, TP12, TP225, TP226. TP227, TP228	TP37, TP38	CP11, CP12
Elevation of infiltration test	16.73	varies see body of report	varies see body of report	varies see body of report	varies see body of report	40.21	42.18
Infiltration rate selection method	N/A (not applicable)	Lowest infiltration rate from TP04	Lowest infiltration rates from TP223	Lowest infiltration rate from TP222	Lowest infiltration rate used from the footprint of infiltration basin (TP225)	N/A	N/A
Factor of safety against infiltration rate	N/A	5	5	5	5	N/A	N/A
Design infiltration Rate (m/s)	N/A	8.306E-07	5.99E-06	3.05E-06	2.36E-06	N/A	N/A
Design infiltration Rate (m/hr)	N/A	0.00299	0.021564	0.0110	0.008496	N/A	N/A
Max Ground Water Level from monitoring (mAOD)	5.99 (031)	12.84 (CP03)	4.50 (BH228) (damp only)	19.18	19.18	40.39 (WS29)	42.72 (CP12)
Depth of Unsaturated Zone (m)	N/A	1.20	11.10	3.32	1.82	N/A	N/A

Basin dimensions

Basin	Northern Section NWL Basin 1	Northern Section A1067 - 1	Middle Section NWL Basin 2	Middle Section NWL Basin 3	Middle Section NWL Basin 4	Southern section NWL Basin 5	Southern section NWL Basin 6
Approx. Max Ground Level (mAOD)	22.24	17.26	18.85	24.23	22.70	43.50	51.22
Forebay	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Crest level (mAOD)	17.41	16.04	17.60	24.60 to 25.50	23.00	41.30	44.5
Basin Invert Level (mAOD)	15.41	14.04	15.60	22.50	21.00	39.30	42.5
Pipe Outfall or sill IL	15.91	14.54	16.10	23.00	21.50	39.60	43
Max Depth Below Existing Ground	6.83	3.22	3.25	1.73	1.70	4.20	8.72
Max Water Level (mAOD) (1:100 + 45%) For combined storm scenario see infiltration basin section*	16.74	15.22	16.63	23.54	22.09	40.59	44.12
Max Water Depth (m) (1:100+45%)	1.33	1.18	1.03	1.04	1.09	1.29	1.62
Crest surface area (m²)	410	255	820	467	981	7332	267
Base Surface Area (m²)	159	52	323	109	387	481	30
Standing water depth (m)	0.5	0.5	0.5	0.5	0.5	0.3	0.5
Max Water Volume (m³) (1:100 + 45%)	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Main basin	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Crest level (mAOD)	17.41	16.04	17.60	24.60 to 25.50	23.0	41.30	44.50
Basin Invert Level (mAOD)	15.41	14.04	15.60	22.50	21.00	39.30	42.50
Pipe Outfall IL	15.41	N/A	N/A	N/A	N/A	39.3	42.5
Max Depth Below Existing Ground	6.83	3.22	3.25	1.73	1.70	4.20	8.72
Max Water Level (mAOD) (1:100 + 45%) For combined storm scenario see infiltration basin section*	16.74	15.22	16.63	23.54	22.09	40.59	44.12
Max Water Depth (m) (1:100+45%)	1.33	1.33	1.18	1.04	1.09	1.29	1.62
Crest surface area (m²)	1148	1427	7554	2802	9195	7332	931
Base Surface Area (m²)	542	750	5245	1274	6437	3955	1744
Max Water Volume (m³) (1:100 + 45%)	1198.14	1106.97	5860.06	1610.51	7675.89	6728.65	2022.89

Greenfield runoff flows

Basin	Northern Section NWL Basin 1	Northern Section A1067 - 1	Middle Section NWL Basin 2	Middle Section NWL Basin 3	Middle Section NWL Basin 4	Southern section NWL Basin 5	Southern section NWL Basin 6
Critical design Storm (mins) (W-Winter; S - Summer)	240W	10080W	2160W	4320W	10080W	1440W	1440W
Greenfield runoff flows (l/s)	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Q bar	4.14	2.14	11.86	3.04	11	16.26	4.64
1 in 1 year	3.52	1.82	10.08	2.65	9.35	13.82	3.94
1 in 30 year	8.07	5.30	23.13	7.46	21.44	31.71	9.04
1 in 100 year	11.75	6.07	29.47	10.83	27.27	40.32	11.5

Attenuation

Basin	Northern Section NWL Basin 1	Northern Section A1067 - 1	Middle Section NWL Basin 2	Middle Section NWL Basin 3	Middle Section NWL Basin 4	Southern section NWL Basin 5	Southern section NWL Basin 6
Max Control Outflow 1 in 1 yr, 30yr and 100yr (l/s)	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1 in 1 yr with 20%cc	38.3	N/A	N/A	N/A	N/A	10.2	3.5
1 in 30yr with 40%cc	43.0	N/A	N/A	N/A	N/A	15.7	3.5
1 in 100yr with 45%cc	43.0	N/A	N/A	N/A	N/A	18.5	4.0
Attenuation control outlet size (mm)	270mm HydroBrake	N/A	N/A	N/A	N/A	127mm HydroBrake	86mm HydroBrake
Design storage volume (critical storm)	1198.14	N/A	N/A	N/A	N/A	6827.44	2022.77
Additional long term storage volume	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Outfall location	NDR Basin 1 A	N/A	N/A	N/A	N/A	Ordinary Watercourse	A47 drainage system

Infiltration design

Basin	**Northern Section** NWL Basin 1	**Northern Section** A1067 - 1	**Middle Section** NWL Basin 2	**Middle Section** NWL Basin 3	**Middle Section** NWL Basin 4	**Southern section** NWL Basin 5	**Southern section** NWL Basin 6
Infiltration design half drain-down Time	N/A	>24 hours	>24 hours	>24 hours	>24 hours	N/A	N/A
Additional volume for 1 in 10 yr storm + 40%cc (m³)	N/A	686.67	2952.55	927.19	4356.94	N/A	N/A
Total Water Volume (m³) (1:100 + 45% + 1:10)	N/A	1793.64	8812.61	2537.70	12032.83	N/A	N/A
*Maximum design water level (mAOD) (1:100 + 45% + 1:10)	N/A	15.80	17.1	24.0	22.7	N/A	N/A
Overflow invert level (mAOD)	17.31	15.94	17.5	24.4	22.9	39.8	44.1
Overflow Weir Capacity (l/s)	78.2	75.2	70.5	430.4	430.4	111.1	51.89
Overflow Destination	NDR Basin 1A (HydroBrake chamber)	Ordinary Watercourse	Ordinary Watercourse	PED system to spreader ditch	PED system to spreader ditch	Ordinary Watercourse (HyrdroBrake chamber)	A47 drainage system (Hydrobrake chamber)
Note	N/A	N/A	N/A	N/A	N/A	N/A	N/A

8 Side Roads

The Proposed Scheme will impact on a number of road crossings which are either accommodated in the design or stopped-up. These are:

- Ringland lane: to be incorporated in an underpass to the Proposed Scheme;
- Weston Rd to be stopped-up;
- Breck Road to be stopped-up.

8.1 Ringland Lane

Currently the drainage of this road is informal and over the edge, with several drainage grips shown on the Norfolk County Council highway drainage records (see Figure 32). An element of betterment is to be provided through the Project as per discussions with the LLFA.

The proposed betterment takes the form of

- Ditch improvements along the line of the existing roadside ditches including lined grips. These will mimic the character of the existing drainage, with a crossfall already provided on each road to direct surface runoff to the ditch, whilst providing a slightly more formalised and better maintainable system; also giving a level of treatment and attenuation recognised in CIRIA C753.
- Positive drainage in the form of gullies, kerbs and a carrier drain will be provided within the underpass. This is to protect the bridge abutment foundations from being undermined by the effects of a soakaway. The carrier drain will be connected into the adjacent basin 4.

Ditch improvements will promote disposal by infiltration within ditches given the low level of test values obtained from the adjacent basin testing.

8.2 Weston Road and Breck Road

These roads will be stopped-up either side of the Proposed Scheme. To the north of the Proposed Scheme the existing roads will be gated and utilised as part of the Non-Motorised User (NMU) network only, i.e., no vehicular access will be permitted. South of the Proposed Scheme, the existing Weston Road will be stopped allowing only local farm access only and the existing Breck Road will be fully stopped up with no access. Similar to Ringland Lane, several drainage grips are shown on the Norfolk County Council highway drainage records for Weston Road and Breck Road (see Figures 14 and 15).

The limit of betterment required is assumed to be limited to the extent of physical works to the existing roads. Additional drainage facilities are proposed at the turning heads with gullies discharging to roadside. North of the Proposed Scheme, where the fall is toward the Proposed Scheme, surface water flow along the existing Weston Road and Breck Road will be collected in the nearby ditch system. South of the Proposed Scheme, where the fall is away from the Proposed Scheme, the existing drainage will continue to operate as is does currently.

The locations of the side roads and proposed infiltration ditches for Ringland Lane are shown on the design drawings PK1002-RAM-HDG-MLE-DR-DZ-0501 to 0512 (Reference 2.08.00).

Figure 32 - Existing Drainage at Ringland Lane

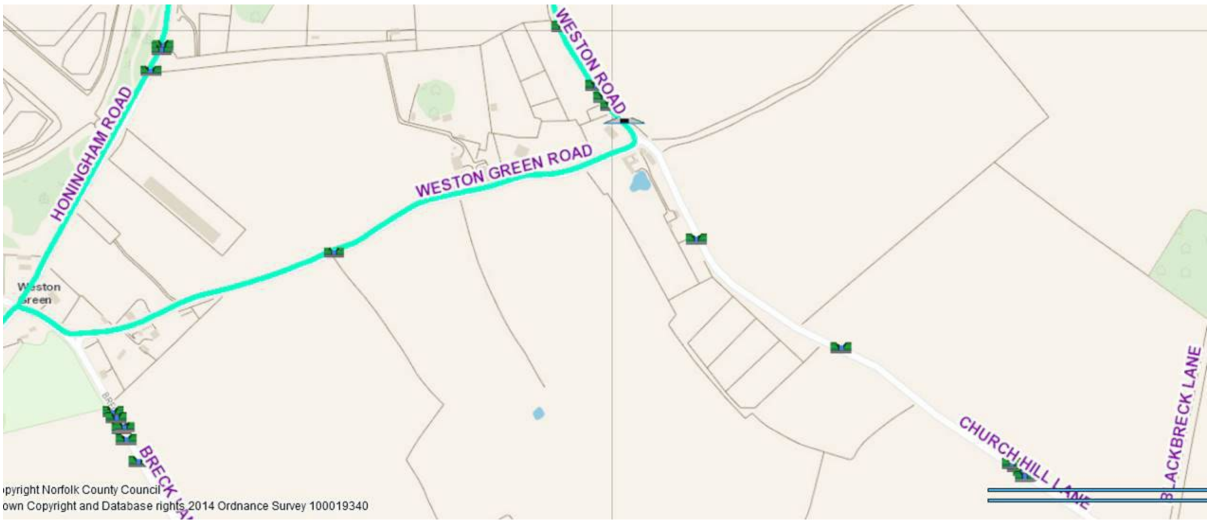


Figure 33 - Existing Drainage at Weston Road

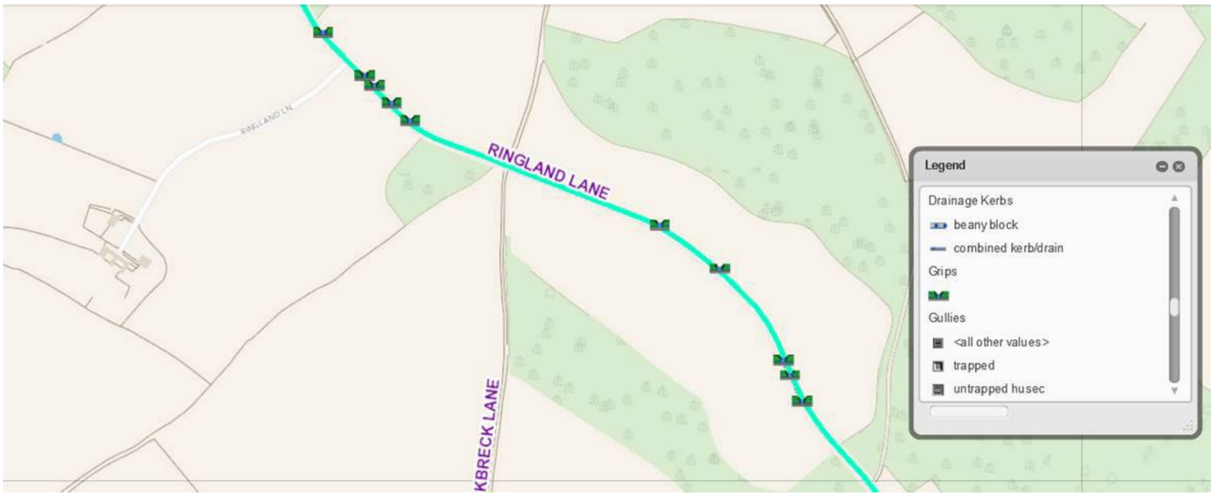


Figure 34 - Existing Drainage at Breck Road (Lane)

9 Highway crossings

The major watercourse crossing of the Proposed Scheme is the 500m long viaduct over the River Wensum from CH50 to CH550. The River Wensum is an EA Main River and a designated Special Area of Conservation (SAC). As such a clear span of the watercourse is proposed and no direct surface water discharge is proposed into the River Wensum. All surface water intercepted on the viaduct is conveyed to Catchment Basin 2.

Within the River Wensum Flood Plain there are several ordinary watercourses that fall under the jurisdiction of the IDB and private landowner. The proposed viaduct maintenance access track crosses one of these (Ordinary Watercourse WC5 ref. DRN 112G0101) where a new twin 3.3m(w) x 1.5m(h) box culvert is proposed (MA1). Pre-application discussions have taken place with IDB in relation to the form of this structure.

The second watercourse crossing along the Project is the Tud tributary at CH4470. A clear span structure (4m x 4m box culvert) over this watercourse is proposed along with a positive outfall from Basin 5. This culvert also acts as a bat underpass.

The route of the Proposed Scheme crosses a surface water flow path at CH1725 (Ringland Lane). A 900mm diameter culvert is proposed to convey surface water under the Proposed Scheme at this location.

There is also a 750mm diameter existing pond outlet at CH5125.

In addition to the two formal watercourse crossings and surface water flow path along the Proposed Scheme there are four green bridges that intersect the Proposed Scheme;

- The Broadway Green Bridge GB1 (CH3750)
- Foxburrow Plantation Green Bridge GB2 (CH4400)
- Morton Green Bridge GB4 (CH2490)
- Nursery Woodland Green Bridge GB5 (CH1000)

There is also a 600mm diameter concrete badger culvert located at CH1630. A longitudinal fall is proposed along each crossing and either an infiltration trench or connection to positive drainage system at the lower end of each structure will provide local drainage and to ensure that crossings are able to remain operational following heavy rainfall events.

The locations of the watercourse crossings, surface water flow path, wildlife crossings and green bridges are shown on drawings PK1002-RAM-HDG-MLE-DR-DZ-0501 to 0512 (Reference 2.08.00).

10 Pollution Mitigation

All infiltration and attenuation basins incorporate pollution mitigation measures, which have been agreed through discussion with the EA and LLFA, to protect the Source Protection Zone 3 (SPZ3) and Principal Aquifer over which the scheme lies.

The proposed pollution mitigation measures include the following key components: grass swales / surface water channels; sediment forebays; pollution control valve (isolation penstocks) and infiltration basins. The proposed pollutant treatment train is discussed further below.

Surface water along most of the Proposed Scheme is intercepted by lined grass surface water channels at the edge of the carriageway and conveyed by carrier drain to infiltration or attenuation basins. The only exceptions to this are: sections of concrete surface water channel at bridge structures and within the central reservation where grass channels are not feasible; at bridge crossings, including the viaduct across the River Wensum, where kerb drainage is proposed; and a small section of kerb and gully drainage at the junction with the A1067.

All surface water is collected positively from the highway and conveyed via carrier drains to a lined sediment forebay. Wet areas are provided in the forebays and scrapes to each basin to allow capture of sediment and an area for planting for treatment of water prior to disposal. The size of each sediment forebay equates to approximately 10% of the volume of the infiltration/ attenuation basin – based on minimum sizing provided in the SuDS Manual (CIRIA C753). All infiltration basins have a sediment forebay which is separate from the main infiltration basin itself. The attenuation basins (Basin 5, Basin 6 and A1270 Basin 1) have a sediment forebay which is integral with the attenuation basin. All basin surfaces are lined with up to 200mm of topsoil over 250mm subsoil. Basin 5 outfalls via a ditch to the nearby watercourse (Tud tributary), which ultimately discharges to the River Tud. Basin 6 discharges to the adjacent NH A47 project. Basin 1 discharges to the adjacent NDR Basin.

For all infiltration basins a pollution control valve (isolation penstock) is proposed at the outlet from the sediment forebay upstream of the infiltration basin. For the attenuation basins with integral sediment forebay, a pollution control valve is proposed at the outlet from the basin.

Safe access has been provided for operation and maintenance of individual chambers, outfalls, penstocks, and vortex flow controls.

The proposed SuDS treatment train is consistent with the pollution mitigation indices approach described in CIRIA C753 for surface water discharge from a trunk road. Reference should be made to the following documents with respect to the proposed pollution mitigation:

- Environmental Statement (ES) notably chapter 12, Road Drainage and Water Environment (reference 3.12.00)
- ES Chapter 12 Appendix 12.1, Drainage Network Water Quality Assessment (reference 3.12.01).
- ES Chapter 12 Appendix 12.3, Water Framework Directive Assessment;

The locations of the proposed pollution mitigation measures are shown on the Proposed Scheme drawings PK1002-RAM-HDG-MLE-DR-DZ-0503 to 0512 (Reference 2.08.00). Further details of the design aspects of the proposed sediment forebays are provided in Section 7 of this report.

Water quality assessments (HEWRAT) have been undertaken in accordance with LA 113 to determine if additional mitigation measures to those described in Table 10 are required to meet water quality standards set out in the Environmental Statement. The drainage network water quality assessment forms Appendix 12.1 of the Environmental Statement (Reference 3.12.01). In summary, the pollution mitigation measures presented in the design drawings and described above would reduce the potential impact to receiving water bodies to an acceptable level in accordance with the DMRB.

11 Operations and Maintenance

11.1 Introduction

The aim of this section is to bring together the Proposed Scheme drainage systems, setting out the various elements used in each system, their role in the drainage system and their maintenance requirements.

Maintenance of the standard drainage features such as gullies, chambers, ditches and culverts will be undertaken in line with the Transport Asset Management Plan (TAMP) in the same manner as it would for other similar routes across the county. Proprietary vegetated wall systems such as Flex MSE will require annual trimming, like ditches. The Sustainable Drainage System (SuDS) elements of the system such as swales, detention/infiltration ponds will be maintained as set out below.

Pollution control devices (PCDs) will be clearly signed. This manual will be distributed to the highway maintenance teams as well as the Fire Service. These features will form part of any emergency response standard procedures for dealing with such events. The operation manual sets out the regular maintenance of control structures and devices.

The Ciria SuDS Manual provides guidance for the maintenance activities and frequencies required for SuDs features. It is noted that the SuDs Manual guidance covers SuDS measures for all forms of infrastructure development including roads but the maintenance regime of SuDS on roads is different to that of other forms of infrastructure development, such as a residential development. The proposed drainage was discussed in meetings with NCC Operations & Maintenance teams on 27 July 2023 and 02 August 2023 to agree an appropriate maintenance regime for drainage assets. In the meetings it was noted that the SuDs drainage components are similar to those implemented on A1270 Broadland Northway and it was proposed that the SuDs measures are maintained in the same way. Refer to Appendix 2 (Reference 4.04.02) for minutes of the meetings.

NCC will be responsible for the drainage network maintenance and the agreement reached for the maintenance schedule presented hereafter.

11.2 Drainage System Components

11.2.1 Swales

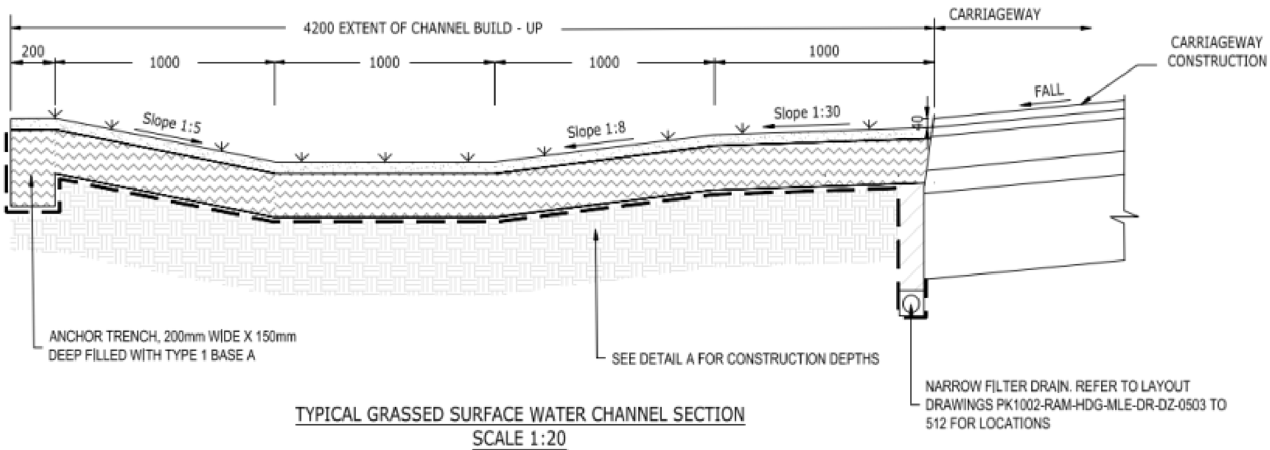
The swales are linear vegetated channels with a flat base to encourage a sheet flow of water through the vegetation. The swales collect surface water runoff laterally across a grass filter strip which acts to reduce the rate of flow and allow suspended particles to settle in the grass strip.

Key Design Features

- The swales have been designed with side slopes of 1 in 5 to allow flow across the edge and a depth of 200 mm. Refer to Figure 35 below.
- Flow rate has been restricted to 1 to 2 m/s or 1 in 50 maximum slopes to prevent erosion and ensure effective pollution control.
- The base is normally 1.0m wide to allow effective maintenance and prevent gullying of the base.
- Swales have been designed with 200mm Type 1 granular sub-base and 50mm topsoil.

At all locations the swales are lined with an impermeable (polyethylene) liner or equivalent approved impermeable barrier membrane.

Figure 35 - Lined swale cross section



Swale Maintenance

The maintenance strategy for the swale is set out below.

Swales

Maintenance Schedule	Required Action	Typical Frequency	Guidance Doc Ref
Regular Maintenance	From experience with A1270 Broadland Northway, swales require minimal maintenance. During the initial stages of handover there will be frequent inspections and remedials undertaken where necessary. It is expected that NWL swales will be similar to those on A1270 BN and require minimal Maintenance	Monthly Inspection	A1270 Broadland Northway
Regular Maintenance	Cut grass to retain grass height within specified design range 50-70 mm	Likely to be once a year. But from experience, grass cutting in swales is rarely required.	A1270 Broadland Northway
Occasional Maintenance	Inspect inlets, outlets, slopes.	Annually	A1270 Broadland Northway
Occasional Maintenance	Remove detritus, leaves, debris.	Following extreme rainfall events	A1270 Broadland Northway
Remedial Actions	Repair erosion or other damage by re-turfing or re seeding Relevel uneven surfaces and reinstate design levels Scarify and spike topsoil soil layer to improve infiltration performance, break up silt deposits and prevent compaction of the soil surface Remove build up of sediment Remove and dispose of oils or petrol residues using safe standard practices	As required, subject to outcome of monthly inspections	SuDS table 17.1

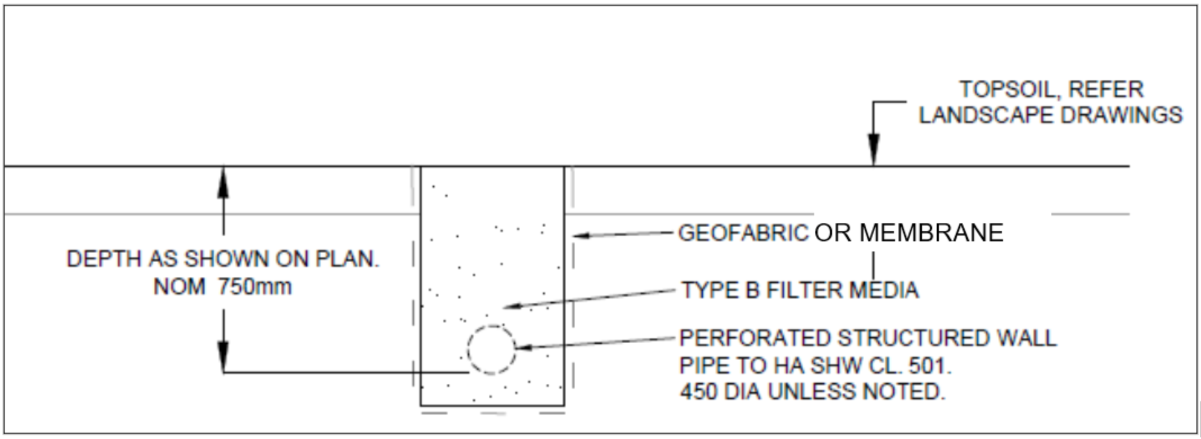
11.2.2 Filter Drains

The filter drains are linear excavations filled with stone that collect surface water runoff laterally as sheet flow. They filter surface water runoff as it passes through the filter media allowing water to infiltrate into soil or flow to the next part of the management train. The typical detail of the filter drain is shown in Figure 36 below.

The filter drains have generally been designed to deal with surface runoff from earthworks, beneath swales with a low longitudinal gradient and overland flow. A perforated pipe has been incorporated to convey water onward from the drain which includes access for rodding or jetting with open outfalls.

For filter drains laid beneath swales the geofabric is substituted by a polythene lining to prevent infiltration of highway runoff into the ground. A perforated pipe has been incorporated to convey water onward from the drain which includes access for rodding or jetting with open outfalls.

Figure 36 - Typical Filter Drain Detail



Filter drain maintenance

The maintenance strategy for the filter drains is set out below:

Filter Drains

Maintenance Schedule	Required Action	Typical Frequency	Guidance Doc Ref
Regular Maintenance	From experience with A1270 Broadland Northway, Filter drains require minimal maintenance during the initial stages of handover. There will be frequent inspections at remedials undertaken where necessary. It is expected that NWL filter drains will be similar to those on the A1270 BN and require minimal maintenance.	Monthly Inspection	A1270 Broadland Northway
Occasional Maintenance	From experience with A1270 Broadland Northway, there has been no need to carry out any walkovers, pipe inspections and maintenance.	Not expected to be required	A1270 Broadland Northway
Remedial Actions	Remove surface stone layer and set aside, replace clogged geotextile and reinstate Remove and clean filter media on site using proprietary machinery Remove and replace filter media	As required, subject to outcome of monthly inspections	N/A (not applicable)

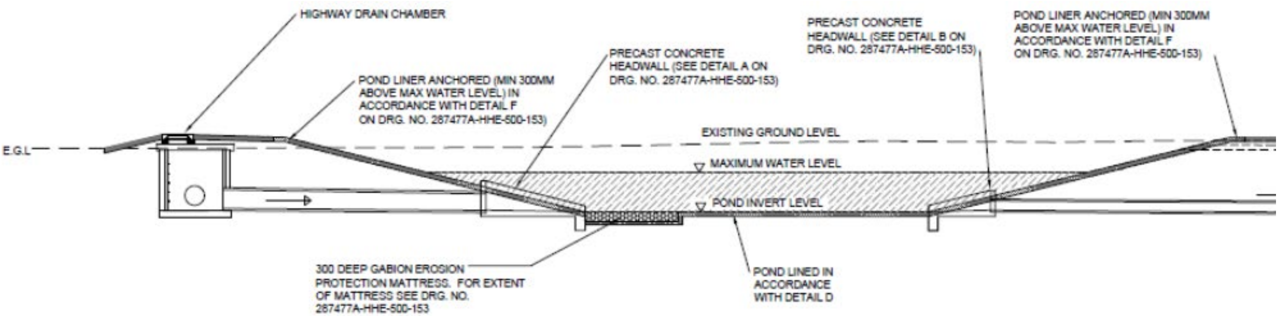
11.2.3 Detention & Infiltration Basins

The basins are vegetated ponds designed to store surface water runoff and attenuate flow. They also facilitate some settling of particulate pollutants and biological treatment of some pollutants within a continuously-wetted forebay and scrapes. The wet areas will be further enhanced by reed bed planting to accumulate suspended particles and associated heavy metals. The scrapes and forebay have another function in providing a deterrent to birds from landing on the pond. Detention ponds have been designed to be generally dry. Refer to **Figure 37** below.

Key Design Features

- Large stones are placed at the inlet to provide scour protection.
- The detention pond is lined with an impermeable geosynthetic liner.
- At infiltration basins only the forebay is lined.
- The detention pond has been designed to contain a spillage event and can be isolated from the downstream network using the pollution control valve, allowing time for the contaminated water to be removed and disposed of safely off site.
- Side slopes to the detention pond have been designed with a gradient of 1 in 3 maximum, with clear access for maintenance.
- A forebay and scrapes form an integral part of the basin.

Figure 37 - Typical cross section of detention basin



Outflow from the detention basin goes to:

- The adjacent NDR basin 1
- The surrounding watercourse (basin 5) and
- The adjacent A47 Wood Lane junction as controlled and to be developed by National Highways

The outflow is controlled by a hydrobrake by the simplistic method of 2 l/s/ha. and flow capped to at or less than the greenfield runoff for the equivalent catchment area serving the detention pond, or a minimum practical rate of 5l/s.

Basin 1: All rainfall return periods 43 l/s

Basin 5: 1 in 1 year plus 20% climate change 10.2l/s

1 in 100 year plus 45% climate change 18.5l/s

Basin 6: 1 in 1 year plus 20% climate change 3.5l/s

1 in 100 year plus 45% climate change 4.0l/s

Attenuation pond maintenance

The maintenance strategy for attention ponds is set out below.

Maintenance Schedule	Required Action	Typical Frequency	Guidance Doc Ref
Monitoring	Inspects inlets, outlets and overflows, slopes, structures, inlets and forebays for silt accumulation	Annually and following extreme events	A1270 Broadland Northway
Regular Maintenance	From experience with A1270 Broadland Northway, Basins require minimal maintenance during the initial stages of handover. There will be frequent inspections at remedials undertaken where necessary. It is expected that NWL basins will be similar to those on the A1270 BN and require minimal maintenance.	Annual inspection and following extreme rainfall events	A1270 Broadland Northway
Regular Maintenance	Cut grass in and around basin	Not required from experience with A1270 BN	A1270 Broadland Northway
Occasional Maintenance	From experience with A1270 Broadland Northway, sediment build up in ponds is low and the expectation is that NWL will be similar due to proposed swales	As required, subject to inspection	A1270 Broadland Northway

Maintenance Schedule	Required Action	Typical Frequency	Guidance Doc Ref
Remedial Actions	Inspect and repair damage to inlets, outlets, banks and overflows Repair erosion or other damage by re seeding or re turfing re-align the rip-rap Repair or rehabilitate inlets, outlets and overflows. Rehabilitate infiltration. surface using scarifying and spiking. techniques if performance deteriorates. Relevel uneven surfaces and reinstate design levels	As required subject to annual inspection and following extreme events	A1270 Broadland Northway
Remedial Actions	Spillages clean up including pumping out and disposal, remedial of contaminated areas such as topsoil replacement	As required, following spillage incident	N/A (not applicable)

11.2.4 Access for maintenance of ditches

NCC will have right of access for maintenance of ditches and SuDS features across the land encompassed by the red line boundary as shown on the drawings. The layout at site boundaries has been made to permit access for maintenance crews and vehicles.

Typical boundary corridors are created proving a 3.0m clearance between ditch and boundary fence or 3.0m between ditch and inside hedge as shown in Figure 39 and Figure 40 respectively.

Figure 38 - Typical access corridor and boundary sections

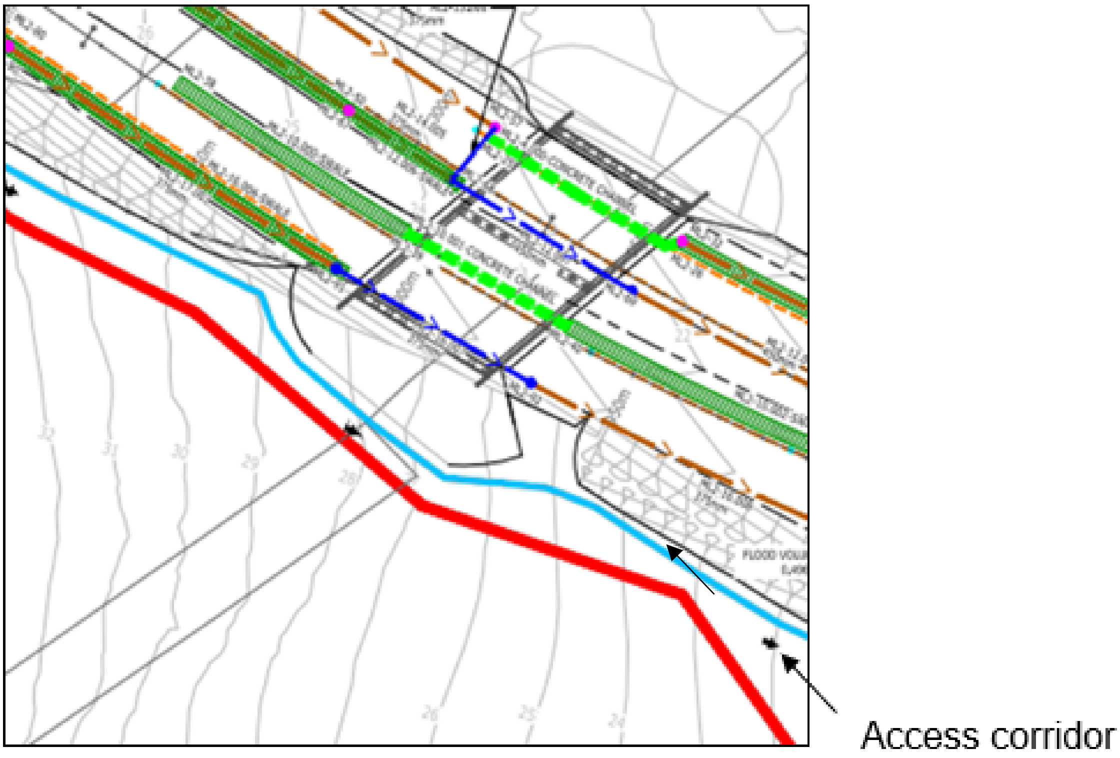


Figure 39 - Boundary Fence

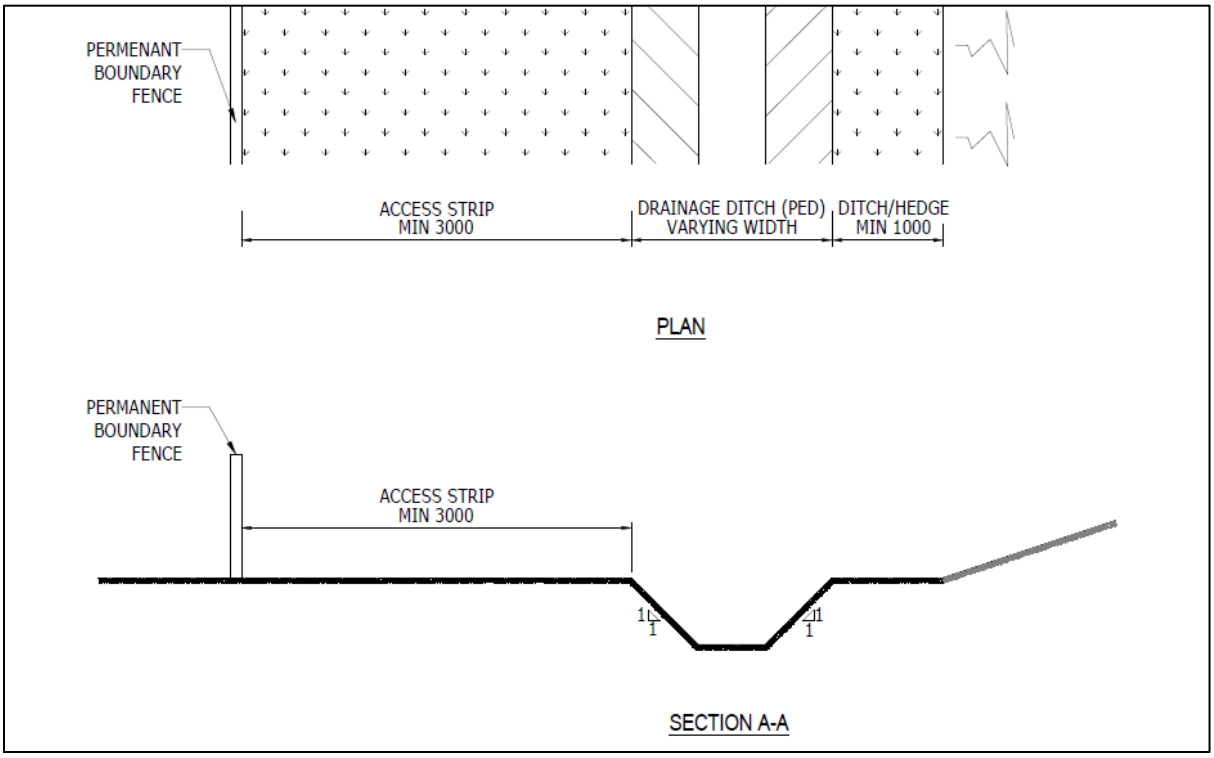
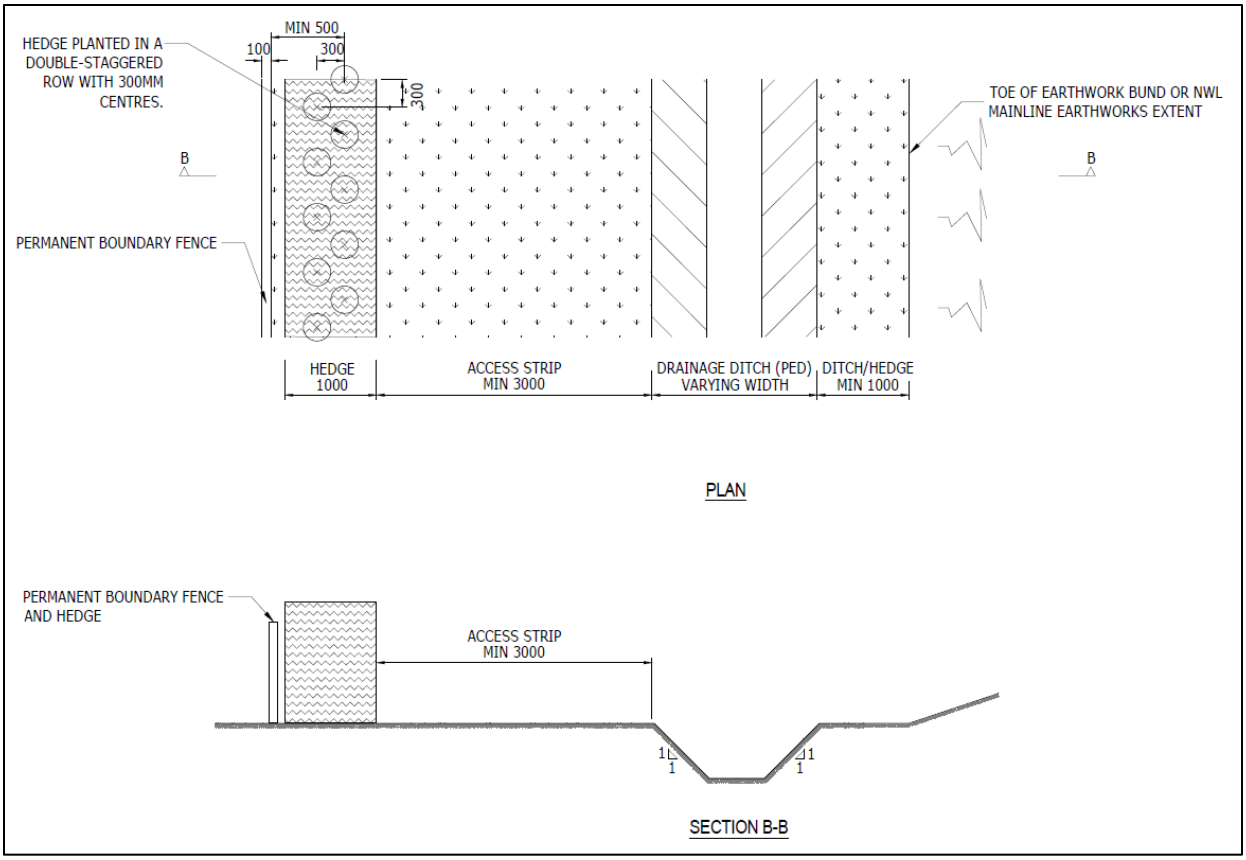


Figure 40 - Boundary Fence and Hedge



12 Third Party Liaison

LLFA

During the design development process several meetings have been held with the LLFA so that scheme proposals could be presented and discussed in advance on the planning submission with a view of agreeing design parameters/aspects.

Correspondence from LLFA is described in Appendix 2 (Reference 4.04.02).

IDB

Liaison has been undertaken with the IDB in relation to the crossing of OWC no.5 including location of viaduct piers, a maintenance access culvert and outfalls 3, 4 and 5.

Correspondence from IDB is described in Appendix 2 (Reference 4.04.02).

NCC Operations & Maintenance

Liaison undertaken to discuss and agree detail for the Non-return valve chamber at basins 5 and 6.

The maintenance regime for drainage assets was also discussed with the NCC team.

Correspondence from NCC is described in Appendix 2 (Reference 4.04.02).

National Highways (A47 tie-in)

Refer to Appendix 13 (Reference 4.04.13) for presentations and correspondence relating to the NH A47 stub.