SITE DRAINAGE AND SUDS TECHNICAL MEMORANDUM

TO	Knockharley Landfill Ltd.	**Doc No.**	IE0037027.9697-TM12.V1
DATE	**09 November 2025**	**CONFIDENTIALITY**	**Restricted**
SUBJECT	**Knockharley Landfill Expansion Drainage and SUDS Planning Level Design**		

INTRODUCTION

The Knockharley Landfill, referred to herein as 'the landfill' is preparing a planning submission to seek approval to expand the landfill by nine cells, increasing the landfill area by approximately 17.68 ha and bringing the total number of cells to 39. The total site footprint will be 109 ha. A layout and drawings of the proposed development can be found in Appendix A.

To facilitate the expansion, diversion of a section of the Knockharley Stream to the northwest of the site is required.

The purpose of this Technical Memorandum is to document the surface water drainage and attenuation infrastructure that will be required due to the landfill expansion, as well as the sizing of the stream diversion.

KNOCKHARLEY EXPANSION – SITE STORMWATER MANAGEMENT

Background & Site Overview

The purpose of the SUDS elements on site, particularly the swales and attenuation ponds, is to improve the quantity and quality of the site stormwater discharge to resemble greenfield conditions. The current surface water infrastructure system is split into two catchments, draining by gravity to the north and to the south.

The northern system receives surface water flows from the northern half of the landfill; to illustrate this, a simplistic process flow diagram is shown in Figure 1. The holding pond is primarily designed to contain and isolate any contaminated surface water if spillage occurs from the facility, and the attenuation pond is primarily designed to attenuate and manage the discharge rate and volume of stormwater (EIAR, 2018). The wetland polishes any remaining sediment over and above statutory requirements.

The southern system receives surface water flows from the landfill, access road, leachate and biological facilities. The process flow diagram of the southern drainage system is shown in Figure 2.

Figure 1: Northern Stormwater Process Flow Diagram

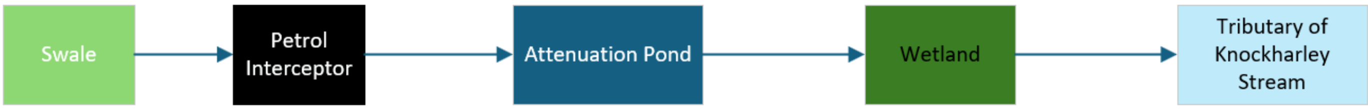


Figure 2: Southern Stormwater Process Flow Diagram

Design Criteria

The surface water system has been designed to meet the requirements set out in the Greater Dublin Regional Code of Practice for Drainage Works V6.0, in line with the Greater Dublin Strategic Drainage Study (GDSDS) and the CIRIA SuDS Manual C753, summarised in Table 1.

The main criteria items listed in GDSDS is:

- Criterion 1: River water quality protection;
- Criterion 2: River regime protection;
- Criterion 3: Level of service/flooding of the site; and,
- Criterion 4: River flood protection.

Criterion 3 has not been considered in this memorandum, as flood risk assessment has been addressed separately by others.

Within Criterion 4, sub-criteria 4.1 and 4.2 (relating to disposal of runoff via infiltration) are not applicable since the ponds are lined with an HDPE geomembrane and will not permit any infiltration.

Sub-criterion 4.3 (i.e., restriction of discharge) has been considered; however, application of this criterion will result in an unfeasibly large attenuation pond for the Southern system[1]. As an alternative to sub-criterion 4.3, guidance taken from the SuDS Manual (CIRIA C753) has been adopted.

Table 1 - Stormwater Design Criteria, Section 16 Greater Dublin Regional Code of Practice for Drainage Works

Criteria	Sub-criterion	AEP[2]	Design Objective
Criterion 1: River water quality protection	1.1	<1	Interception storage of at least 5 mm, and preferably 10 mm, of rainfall where runoff to the receiving water can be prevented.
	1.2	<1	Where initial runoff from at least 5 mm of rainfall cannot be intercepted, treatment of runoff (treatment volume) is required. Retention pond (if used) to have a minimum pool volume equivalent to 15 mm rainfall.
Criterion 2: River regime protection	2.1	1	Discharge rate equal to 1-year greenfield site peak runoff rate or 2 l/s/ha, whichever is greater. Site critical duration storm to be used to assess attenuation storage volume.

[1] If sub-criterion 4.3 was to be implemented for the Southern Attenuation Pond, the pond would occupy an area of 28,500 m^2, more than twice the area of the proposed pond.
[2] Annual Exceedance Probability

Criteria	Sub-criterion	AEP[2]	Design Objective
	2.2	100	Discharge rate equal to 1-in-100-year greenfield site peak runoff rate. Site critical duration storm to be used to assess attenuation storage volume.
CIRIA C753, section 3.3.1 (b) *(Alternative to sub-criterion 4.3)*	Bullet point 1	100	The additional runoff volume (long-term storage)[3] should be discharged from the site at a rate of 2 l/s/ha or less, while still allowing greenfield runoff peak flow rates to be applied for the greenfield runoff volume

The facility stormwater infrastructure has been sized for the closure phase, which is defined as the phase after the facility has been capped but prior to the decommissioning of the hardstanding areas such as roads, etc. which will be rehabilitated prior to post-closure.

Hydrological and Hydraulic Analysis

CLIMATE CHANGE

A 10% increase in the precipitation depth has been incorporated to allow for climate change, based on section 3.12 of the Greater Dublin Regional Code of Practice. Reference has also been paid to the Flood Risk Management, Climate Change Sectoral Adaptation Plan (CCSAP), prepared by the OPW under the National Adaptation Framework, which recommends an increase of 20% in extreme rainfall depth for the Mid-Range Future Scenario (MRFS). However, given the short time-frame over which active closure will occur, and the relatively short facility life-time, application of this more extreme climate change allowances in design was considered excessively conservative.

A high-level analysis of storage requirements during post-closure has been undertaken to confirm that the High-Range Future Scenario 30% increase in extreme rainfall can be accommodated in the proposed attenuation ponds.

Climate change has only been applied to the post-development scenarios and not to calculate the greenfield flow rate or volume.

CATCHMENT DELINEATION

Catchments reporting to the northern and southern ponds were delineated based on the proposed expanded site layout during closure. During the closure phase, the landfill will be capped, and all run-off generated from the landfill will be directed to the site's stormwater system whereas during the operational phase, stormwater generated on the landfill will need to be treated prior to release to the environment and will be managed separately. Therefore, closure was selected as the design scenario as it represents the largest contributing catchment area and, by extension, will generate the highest flow rates and volumes. It should be noted that during the operational phase, significantly less run-off will report to the stormwater

[3] i.e., the difference between the predicted development runoff volume and the estimated greenfield volume for the 100 AEP event, termed the “long-term storage”

system. The catchment delineation of the site is shown in Figure 3 and a summary of the contributing areas has been tabulated in Table 2.

Table 2: Catchment areas reporting to ponds - Closure scenario

Catchment	Capped/Landfill Area (ha)	Hardstanding Area (ha)	Greenfield Area (ha)	Area within planning boundary that does not report to ponds (ha)	Total Area (ha)
Northern	23.60	4.44	16.77	23.67	68.48
Southern	29.13	8.35	27.13	4.04	68.65
				Total Site Area (ha)	137.13

Figure 3: Catchment delineation (not to scale)

The effective catchment areas (i.e., the sub-catchment areas multiplied by their run-off generation potential) have been calculated to be 20.57 ha and 30.18 ha for the northern and southern catchments, respectively. Prior to the landfill expansion, the effective catchment areas have been reported to be 19.66 ha and 17.45 ha for the northern and southern catchments, respectively (EIAR, 2018).

Calculation of the effective catchment areas is summarised in Table 3. The Standard Percentage Runoff (SPR) for the greenfield site has been estimated from the WRAP[4] soil classification. It is understood that a significant portion of the site is covered with deep well well-drained soil. For this reason, a WRAP class 2 soil type has been adopted for this site, with a corresponding SPR of 30%. This is also a conservative assumption in the sense that a low SPR will produce a low greenfield flow rate and volume. It has been assumed that capping of the landfill will also have a WRAP classification of 2 since the landfill capping will be greater than 800 mm and will consist of permeable material[5].

Table 3: Catchment Factors

Catchment Type	Run-off Factor	Notes
Landfill & Ash Area	30%	WRAP Soil Type 2
Hardstanding Area	100%	100% impermeable
Greenfield Area	30%	WRAP Soil Type 2

GREENFIELD RUN-OFF RATE:

The greenfield peak run-off rate for the site has been calculated considering the total contributing catchment area and using the IH124 method. Refer to Appendix B for detailed calculations of the greenfield rates.

The greenfield peak run-off rate for the various criteria, provided in the previous section, has been summarised below.

<u>Criterion 2.1:</u> *(Discharge rate for the 1:1 AEP event ≤ 1:1 AEP greenfield site peak runoff rate or 2 l/s/ha, whichever is greater)*

- The 1:1 Annual Exceedance Probability (AEP) greenfield peak runoff rate was calculated to be 207 l/s; or,
- Using the 2 l/s/ha allowance equates to a maximum allowable discharge rate of 218 l/s.

<u>Criterion 2.2:</u> *(Discharge rate for the 1:100 AEP event ≤ 1:100 AEP greenfield site peak runoff rate)*

- The 1:100 Annual Exceedance Probability (AEP) greenfield peak runoff rate was calculated to be 636 l/s.

<u>CIRIA C753, section 3.3.1 (b):</u> *(1:100 AEP long-term storage should be discharged at a rate of 2 l/s/ha or less; short term storage shall be discharged at greenfield equivalent or less)*

- Long-term storage: Using the 2 l/s/ha allowance equates to a maximum allowable discharge rate of 218 l/s; and,

4 Winter Rainfall Acceptance Potential - Sourced from HR Wallingford and IH126
5 Based on table 2.1, WRAP Classification Scheme, IH126

- Short-term storage: The 1:100 Annual Exceedance Probability (AEP) greenfield peak runoff rate was calculated to be 636 l/s.

GREENFIELD RUN-OFF VOLUME:

The 1:100 AEP, 6-hour, greenfield event has been calculated to generate a total stormwater runoff volume of 18,798 m^3, using the SPR for the site and the corresponding precipitation depth.

ATTENUATION REQUIREMENTS

Both ponds are lined with an impermeable HDPE membrane and as such can't infiltrate long-term storage and therefore the attenuated volume will be released via outlet pipes.

To meet all the criteria, the combined discharge rate from the two discharge points, the northern and southern attenuation ponds, for the post-development site will be limited to:

- A maximum discharge rate of 218 l/s for all attenuated stormwater from a 1:1 AEP event; and,
- A maximum discharge rate of 218 l/s for long-term storage; and,
- A maximum discharge rate of 636 l/s for short-term storage for up to 18 798 m^3.

Attenuation Ponds

NORTHERN ATTENUATION POND[6]

The Northern Attenuation Pond has been designed to only provide long-term storage (no short-term storage) which will discharge at a maximum rate of 69 l/s and 109 l/s during a 1:1 and 1:100 AEP event, respectively. It has been assumed that a Hydro-brake, or similar, will be installed on the outlet pipe to regulate the discharge rate.

The total required storage volume for various storm events have been plotted in Figure 5. As can be seen, the total storage volume required to attenuate the 1:100 AEP event is 7,901 m^3.

A schematic of the proposed northern stormwater attenuation system is shown in Figure 4. The current attenuation capacity provided by the Northern Attenuation Pond and the Holding Pond will be sufficient to accommodate the design storage volume, however modification will be required to the discharge pipe arrangements. The two ponds are hydraulicly connected, with a large diameter pipe, and will function as a single attenuation system.

An emergency spillway has been provided above the 1:100 AEP level to safely pass any exceedance flows.

Refer to Appendix C for all calculations.

[6] For simplicity both the Northern Attenuation Pond as well as the Holding Pond which will act as a single system will be referred to as only the "Northern Attenuation Pond" in the report.

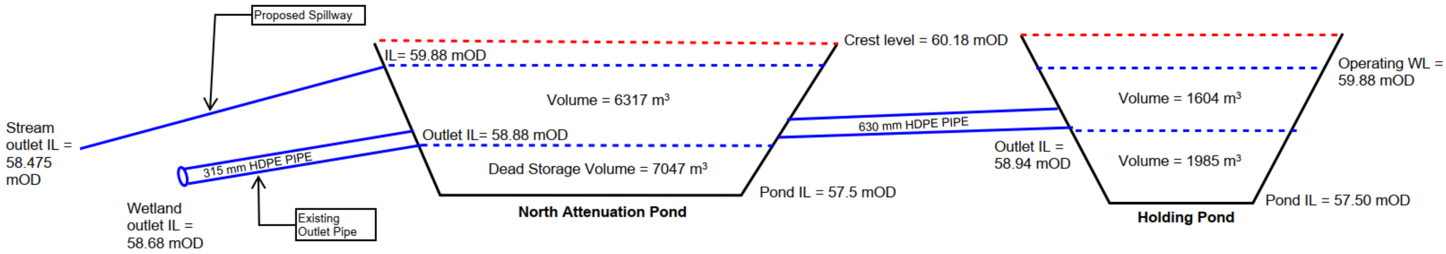


Figure 4: Northern Attenuation and Holding Pond

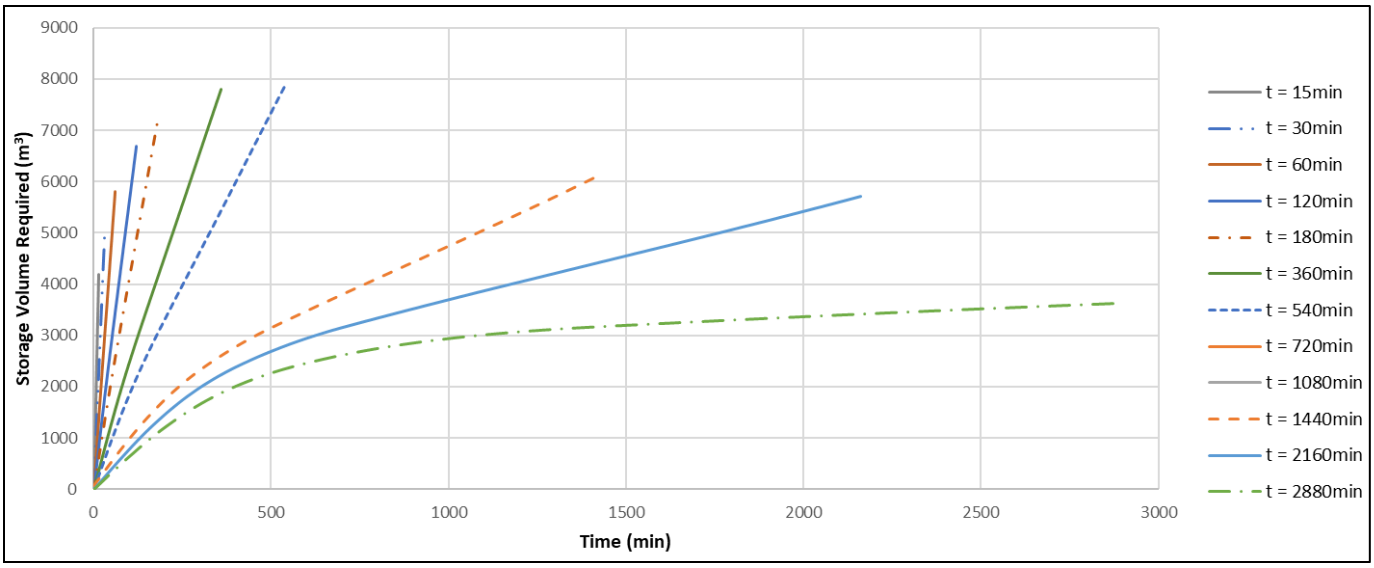


Figure 5: Northern System 1:100 AEP Attenuation Volume Required vs Storm Duration.

SOUTHERN ATTENUATION POND

The Southern Attenuation Pond has been designed to provide long-term and short-term storage. The long-term storage will discharge at a maximum rate of 102 l/s and 107 l/s during a 1:1 and 1:100 AEP event, respectively. The maximum short-term storage volume will be 11,491 m^3 and will be discharged at a maximum rate of 505 l/s. It has been assumed that an orifice, or similar, will be installed on the outlet pipe to regulate the discharge rate.

The total required storage volume for various storm events has been plotted in Figure 6 and Figure 7, for the 1:1 and 1:100 AEP events, respectively. As can be seen, the total storage volume required to attenuate the 1:100 AEP event is 9,464m^3. Similarly, the minimum long-term storage volume required in the pond is 3,249 m^3 to attenuate the 1:1 AEP event and to meet sub-criterion 2.1.

A schematic of the proposed southern stormwater attenuation system is shown in Figure 8. The current attenuation capacity provided by the Southern Attenuation Pond will need to be increased from approximately 6,900 m^3 to the proposed total capacity of approximately 12,700 m^3. Additionally, modifications to the configuration of the outlet pipes will be required.

An emergency spillway has been provided above the 1:100 AEP level to safely pass any exceedance flows.

Refer to Appendix D for the calculations.

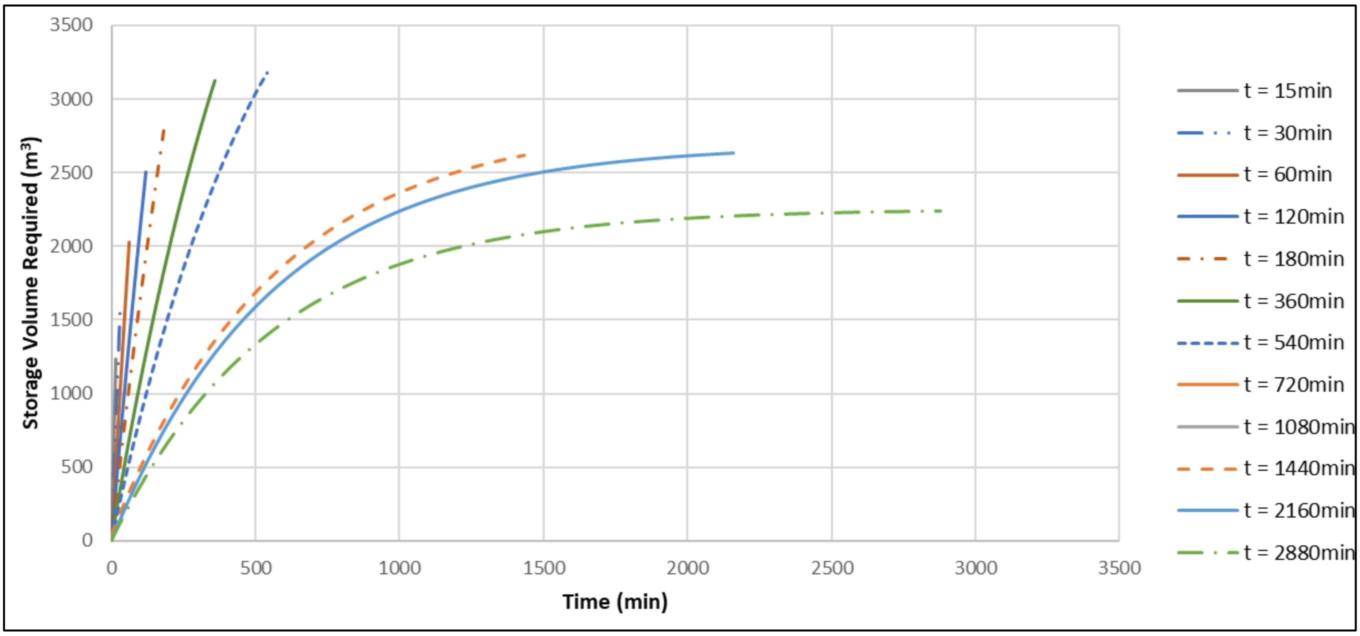


Figure 6: Southern System 1:1 AEP Attenuation Volume Required vs Storm Durations.

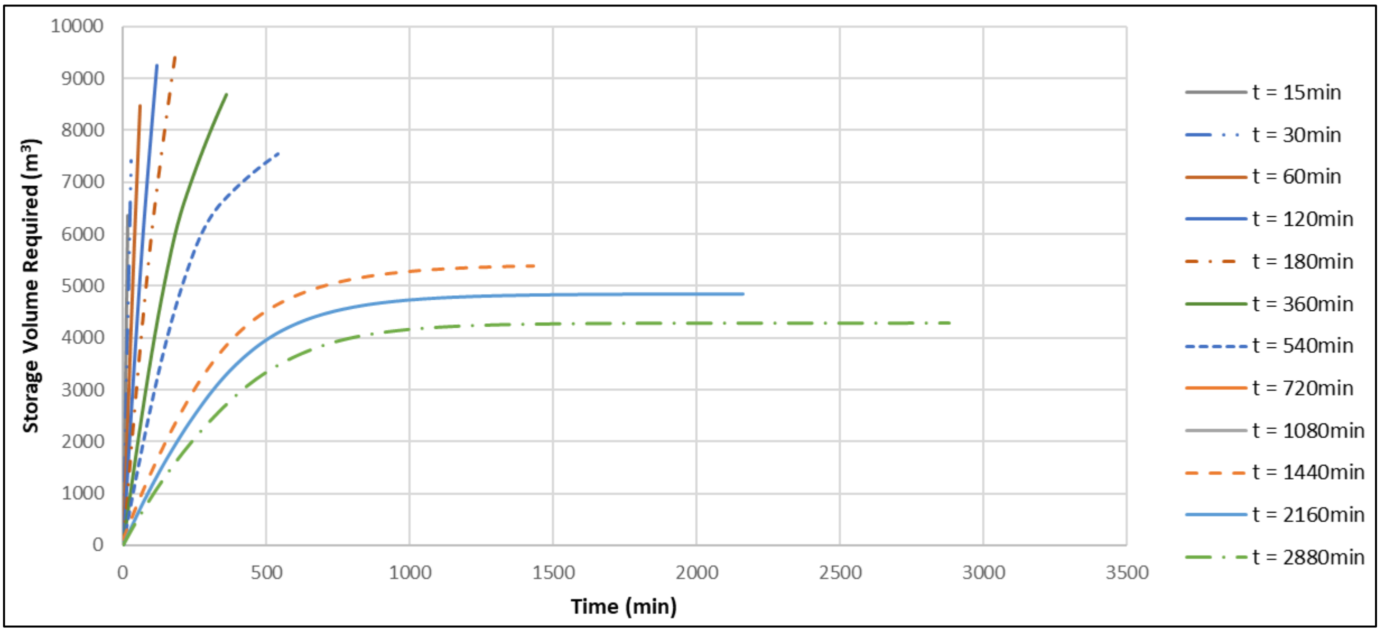


Figure 7: Southern System 1:100 AEP Attenuation Volume Required vs Storm Durations.

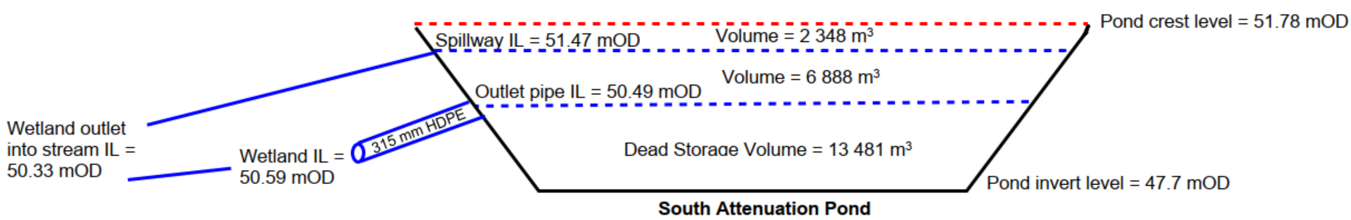


Figure 8: South Attenuation Pond sketch

ATTENUATION PONDS SUMMARY

A summary of the attenuation ponds design has been provided in Table 4.

Table 4 Summary of Attenuation Pond Design Output

Criteria	Required	Northern Pond	Southern Pond	Total	Corresponding Criteria
Maximum 1:1 AEP Discharge Rate	< 218 l/s	69 l/s	102 l/s	171 l/s	Sub-criterion 2.1
Maximum Long-term Discharge Rate	< 218 l/s	109 l/s	107 l/s	216 l/s	Sub-criterion 2.2 & CIRIA C753, section 3.3.1 (b)
Maximum Short-term 1:100 AEP Discharge Rate	< 636 l/s	109 l/s	505 l/s	614 l/s	CIRIA C753, section 3.3.1 (b)
Maximum Short-term 1:100 AEP Discharge Volume	< 18 798 m^3	2 807 m^3	11 491 m^3	14 298 m^3	CIRIA C753, section 3.3.1 (b)

Swales

Grassed swales have been proposed to provide treatment and convey surface water from the landfill to the attenuation ponds. A hydrological analysis was undertaken to assess the peak runoff rates generated from the facility reporting to the swales during the design period (closure).

The maximum catchment areas reporting to the perimeter swales have been delineated as shown in Figure 9, along with the swale drainage routes for the proposed additional swales associated with Project West. Further details of the existing and proposed drainage arrangements are presented in Drawings 17, 17A and 17B which can be found in Appendix A to this document.

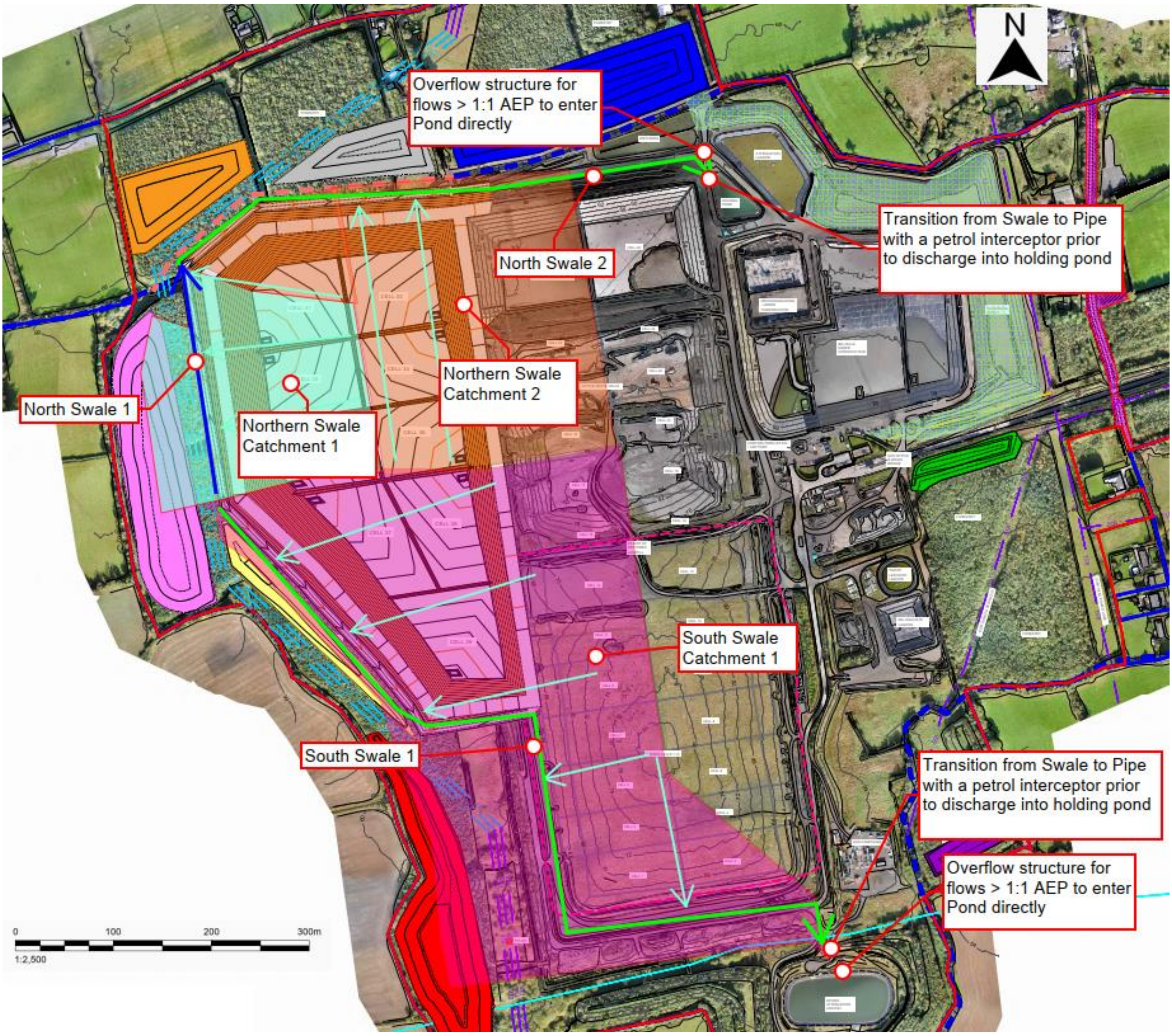


Figure 9: Swale routes and catchment delineation

A summary of the inputs for the Rational Method calculations are shown in Table 5.

Table 5: Rational Method Inputs.

Catchment Name	Average Slope (m/m)	Time of Concentration (minutes)	1:100 AEP Rainfall Intensity including climate change factor (mm/hr)	Runoff Coefficient [1]
North Swale 1	0.009	21	88.9	0.48
North Swale 2	0.009	41	47.7	0.48
South Swale 1	0.017	35	53.9	0.48

1 Based on Developed Grassed Areas – Fair condition (grass cover 50% to 75%) slopes 2-7% (Chow et.al, 1998)

For simplicity, only one swale dimension is proposed, which can be refined during the detailed design phase. The swale will be 600 mm deep, with 1V : 3H side slopes and a bottom width of 1 m, as shown in Figure 10.

Hydraulic Toolbox™ software was used to size the swales, refer to Appendix E for the calculations. A Manning's n value of 0.030 was applied, representing a grass-lined channel. The inputs and results are shown in Table 6, and demonstrate that all swales can convey the design 1:100 AEP flow rate with a minimum freeboard of 150 mm.

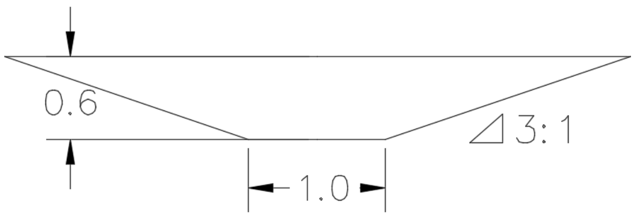


Figure 10: Proposed Swale Dimensions.

Table 6: Swale performance summary.

Name	Catchment Area (ha)	Minimum Slope (%)	1:100 AEP Peak Flow Rate (m^3/s)	Flow Depth (m)	Freeboard (m)
North Swale 1	6.59	0.9%	0.70	0.321	0.279
North Swale 2	20.03	0.9%	1.28	0.429	0.171
South Swale 1	26.39	1.7%	1.90	0.445	0.155

During the operational phase, the catchment reporting to the swales will be significantly smaller and only a nominally sized swale will be constructed on site until the closure stage has been reached.

Sediment Removal

Sediment is the primary pollutant, due to the exposed construction area and the likelihood of erosion during storm events, that needs to be managed within the drainage system. The Sustainable Drainage Systems (SuDS) at the landfill comprise vegetated swales, attenuation ponds, and wetlands, all of which facilitate the removal of sediment from surface water flows, and will satisfy both sub-criteria 1.1 and 1.2.

Ponds

All ponds have sufficient storage for sediment build-up, and more than 1 m depth of dead storage volumes have been provided in each attenuation pond. In addition to storage, the ponds provide extended retention times to facilitate the settling of sediment particles.

Swales

Residence time within the swales should be at least 18 minutes, and maximum flow velocities should be 0.3 m/s in swales to facilitate good pollutant removal (CIRIA, 2015). The Manning's Formula was used to assess the swale performances under low flow conditions (1:1 AEP flows). A Manning's n of 0.35 was used, representing the high roughness caused by grass during low flows.

The velocity and residence time are achieved for all swales and shown in Table 9. As can be seen the low flow velocity and retention time provided by the swales will encourage sediment removal.

Table 7: Low flow swale performance.

Name	1:1 AEP Peak Flow (m^3/s)	Average Velocity (m/s)	Swale Length (m)	Retention Time (minutes)
North Swale 1	0.11	0.112	290	43
North Swale 2	0.21	0.132	937	118
South Swale 1	0.31	0.185	1 151	104

Petrol Interceptors

If additional petrol interceptors (i.e. in addition to those currently installed) are deemed necessary as part of the proposed drainage system for Project West, they shall be installed at a point upstream of the discharge into the attenuation ponds.

Since the risk of significant hydrocarbon spillage within the stormwater catchment is expected to be low, a by-pass separator will be used, which is able to treat low flows. It is proposed that a NSBE030 Kingspan by-pass petrol interceptor, or similar, be installed which is capable of treating 30 l/s and by-pass a peak flow rate of 300 l/s. This system is capable of treating the first flush, which is at least 10% of the peak design flow rate of 210 l/s and 270 l/s for the northern and southern stormwater systems, respectively.

Any exceedance flows beyond the interceptor bypass capacity will be directed directly from the swales into the attenuation pond via a spillway.

Assumptions and Limitations

The attenuation ponds and other surface water drainage infrastructure have been sized based on the contributing catchments and site characteristics during the closure phase. This phase is defined as the period when the entire landfill is capped and contributing to the stormwater system, while hardstanding areas remain in place. During the operational phase, runoff generated within the landfill area is classified as dirty water and will be collected and treated separately via an internal drainage system which will not be connected to the stormwater system. As a result, the stormwater system will be oversized for the operational phase, with the critical design scenario being the closure phase.

While the updated drainage and attenuation system for the site has been designed for the worst-case scenario in terms of effective contributing catchment during the active closure of Project West, it is acknowledged that the system will need to be upgraded and expanded prior to this point in the site development. It is assumed that the proposed expansion of the southern attenuation pond capacity, and reconfiguration of the outlet arrangements at both ponds, will be constructed prior to closure of the existing landfill footprint and thus before the existing system capacity is exceeded. The proposed construction timing will be finalised during detailed closure planning.

Furthermore, it was assumed that any surface water external to the landfill planning boundary will not enter drainage channels within the site. There are existing farm drains to the south of the site that facilitate this and the Knockharley Stream to the north.

STREAM DIVERSION

A section of the Knockharley Stream along the northwestern corner of the landfill footprint shall be diverted to facilitate the expansion of the landfill. As part of this diversion, a new culvert will be installed along the realigned stream channel. Refer to Appendix A for a drawing of the stream diversion.

While the diversion and culvert constitute planning-exempt drainage works, they are described in this memorandum as part of the wider site drainage system for completeness. Refer to Appendix F for the Section 50 application and stream diversion calculations which are to be submitted to the OPW subject to planning approval by An Bord Pleanála.

The Flood Studies Update 3-Variable (FSU-3) method was used to calculate the 1:100 AEP design flow rate. This resulted in a peak flow of 3.80 m^3/s when accounting for a 20% increase due to climate change. This climate change allowance of 20% increase in peak flood flow is also in line with the MRFS set out in the CCSAP.

The diversion was sized using Manning’s equation and considering a 1:100 AEP storm event. The inputs and results are summarised in Table 8 and Table 9 respectively.

Table 8: Stream Diversion Inputs.

Shape	Base width (m)	Side slopes (H:V)	Depth (m)	Length (m)	Invert Level u/s (mOD)	Invert Level d/s (mOD)	Channel Slope (1:x)	Manning n
Trapezoidal	2.5	1.5	>1.7	414	62.92	59.46	1:125	0.03

Table 9: Manning's Formula Results for Stream Diversion.

Flow depth (m)	Average Velocity (m/s)	Critical Depth (m)	Maximum Shear Stress (N/m)	Freeboard (m)[1]
0.63	2.08	0.55	49	1.07

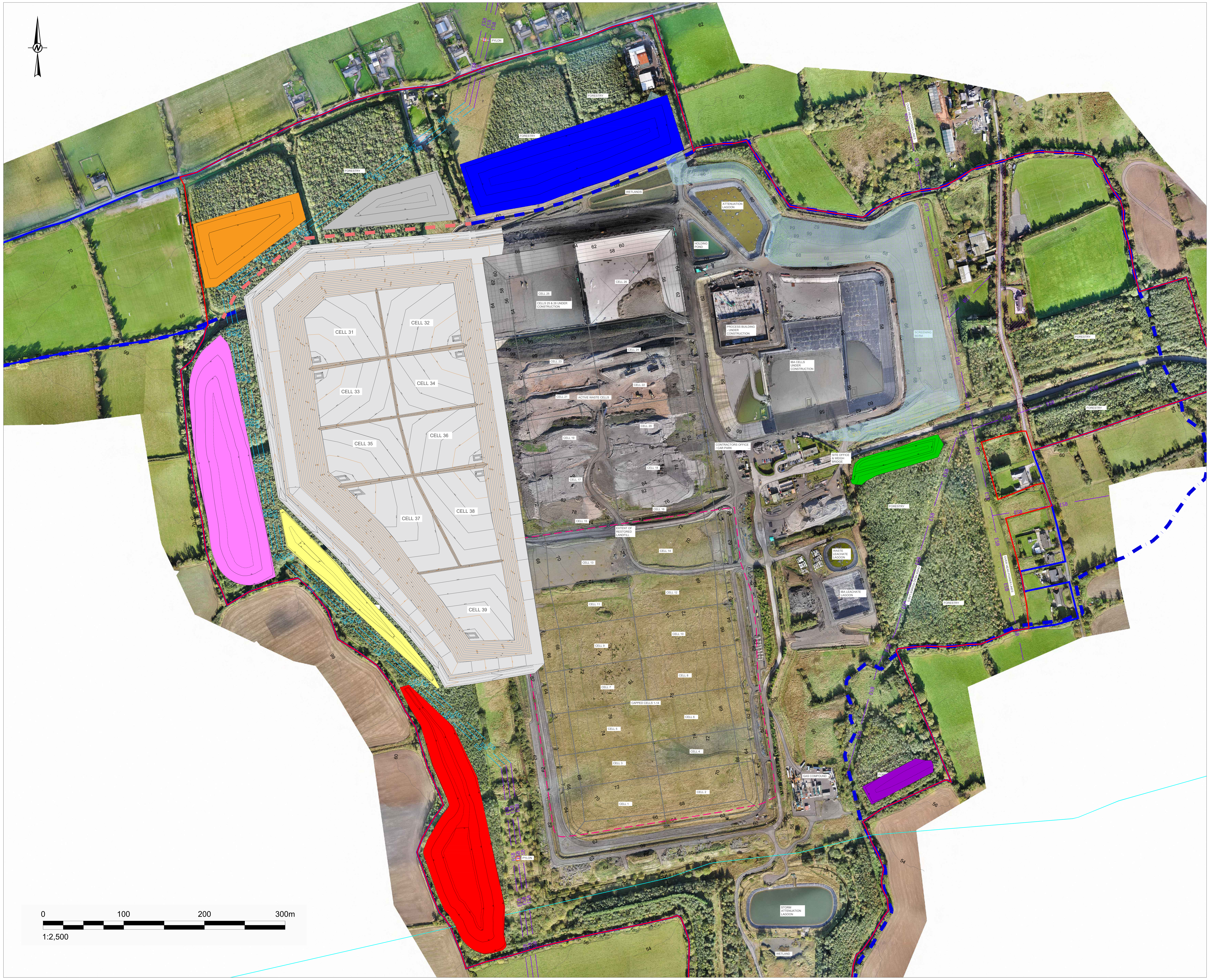
[1] Based on minimum depth in swale

The proposed diversion channel will have adequate capacity to convey the design flow rate, with significant freeboard allowance.

CONCLUSION

The stormwater infrastructure for the Knockharley expansion has been designed in accordance with the Greater Dublin Regional Code of Practice for Drainage Works and the CIRIA SuDS Manual C753. The system includes additional swales, petrol interceptors, and changes and expansion of the existing attenuation ponds to manage surface water effectively. In addition, a stream diversion with a new culvert is proposed to facilitate the landfill expansion. The diversion has been appropriately sized, and a Section 50 application will be submitted to the OPW in support of the culvert installation, subject to planning approval by An Bord Pleanála.

APPENDIX A DRAWINGS OF PROPOSED DEVELOPMENT



NOTE(S)

1. DO NOT SCALE FROM THIS DRAWING
2. ALL LEVELS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
3. ALL DIMENSIONS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
4. ALL LEVELS AND COORDINATES SHOWN ARE TO IRISH NATIONAL GRID

LEGEND

- PLANNING BOUNDARY
- OWNERSHIP BOUNDARY
- AREA WITHIN PLANNING BOUNDARY, NOT WITHIN SITE
- EXISTING OVERHEAD POWER LINES
- GAS MAIN
- KNOCKHARLEY STREAM
- PROPOSED NEW POWERLINE
- RESTORED LANDFILL
- PROJECT WEST FOOTPRINT
- PROPOSED KNOCKHARLEY STREAM DIVERSION
- EXISTING EASTERN BERM
- PROPOSED NORTH EAST BERM BLUE
- PROPOSED NORTH BERM GREY
- PROPOSED NORTH WEST BERM ORANGE
- PERMITTED WEST BERM PINK
- PROPOSED SOUTH WEST BERM YELLOW
- PERMITTED SOUTH BERM RED
- PROPOSED SOUTH EAST BERM PURPLE
- PERMITTED EAST BERM GREEN

VOLUMES

TOTAL CLAY PERMITTED BERM VOLUMES:
VOLUME OF FILL FOR WEST BERM PINK = 154,492 m^3
VOLUME OF FILL FOR SOUTH WEST BERM RED = 133,774 m^3
VOLUME OF FILL FOR EAST BERM GREEN = 7,792 m^3
TOTAL VOLUME OF FILL PERMITTED BERMS = **296,058 m^3**.

TOTAL CLAY PROPOSED BERM VOLUMES:
VOLUME OF FILL FOR NORTH EAST BERM BLUE = 191,790 m^3
VOLUME OF FILL FOR NORTH BERM GREY = 38,651 m^3
VOLUME OF FILL FOR NORTH WEST BERM ORANGE = 52,938 m^3
VOLUME OF FILL FOR SOUTH WEST BERM YELLOW = 21,962 m^3
VOLUME OF FILL FOR SOUTH EAST BERM PURPLE = 7,367 m^3
TOTAL VOLUME OF FILL PROPOSED BERMS = **312,708 m^3**.
POTENTIAL VOLUME OF FILL FOR ALL BERMS = 608,766 m^3.

TOTAL REQUIRED CUT = 1,722,074 m^3.
TOTAL REQUIRED FILL = 693,047 m^3.
TOTAL EXCESS CUT = 1,029,027 m^3.

REMAINING VOLUME AFTER BERMS OFF SITE = 420,261 m^3.

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2025-05-13	PLANNING - DRAFT CONCEPT 3 WITH BERMS FOR DISCUSSION ONLY	DC	POB	AL	AL

CLIENT
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PROJECT
PROJECT WEST LANDFILL DEVELOPMENT

TITLE
CONCEPT 3 - RELOCATED POWERLINES WITH BERMS - MIN

PROJECT NO.	SCALE	REV.	of	DRAWING
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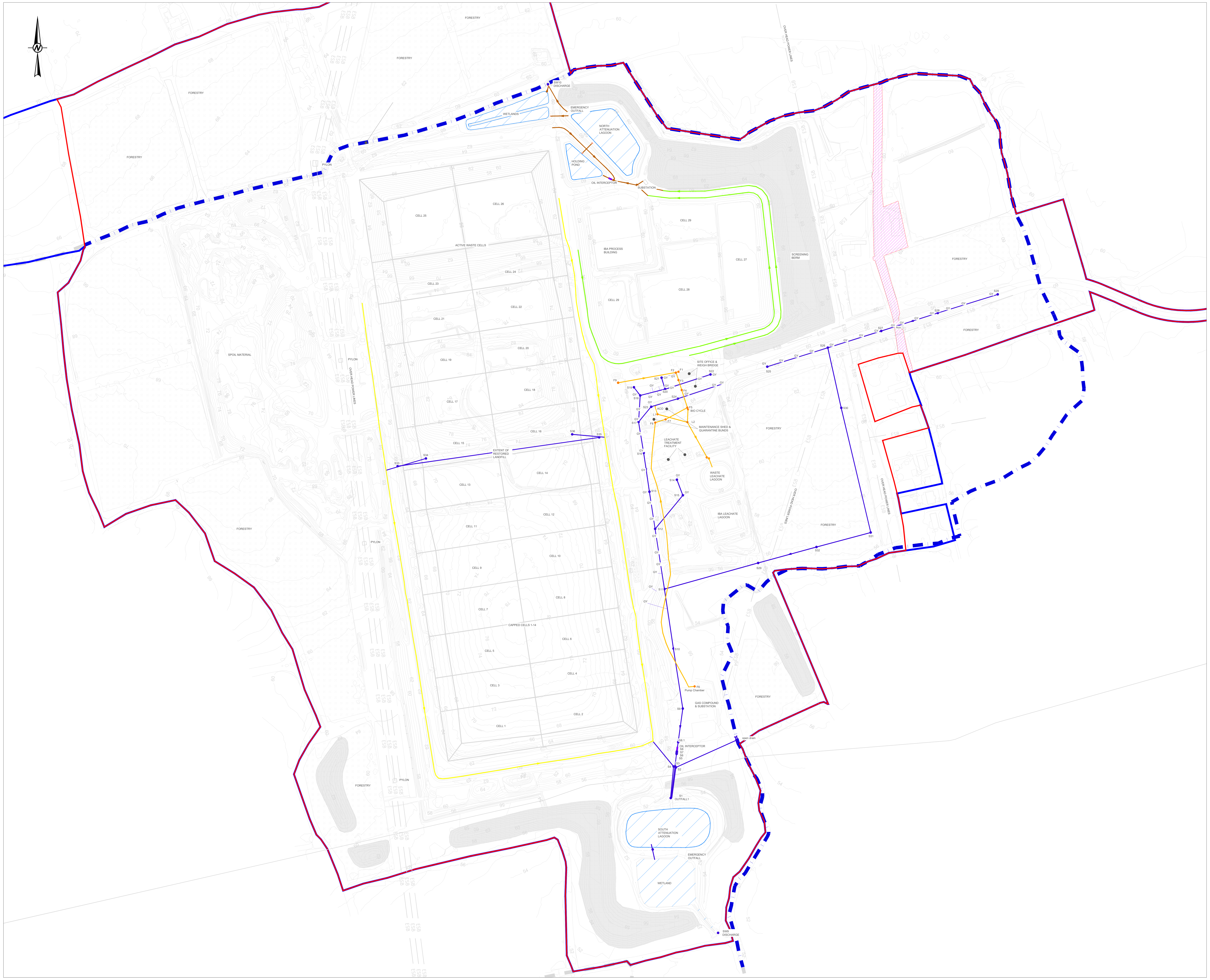

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LEGEND

- PLANNING BOUNDARY
- OWNERSHIP BOUNDARY
- AREA WITHIN PLANNING BOUNDARY, NOT WITHIN SITE
- EXISTING WATERCOURSE
- WATER BODIES
- EXISTING SOUTH SURFACE WATER PIPELINE
- EXISTING SOUTH SURFACE WATER SWALE
- EXISTING NORTH SURFACE PIPELINE
- EXISTING NORTH SURFACE WATER SWALE
- EXISTING FOUL WATER PIPELINE
- EXISTING OIL INTERCEPTORS

0 120 240 360m

1:3,000

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
B	2025-06-27	FOR PLANNING	DC	DC	NV	AL
A	2025-06-25	FOR PLANNING	DC	POB	AL	AL

CLIENT

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KLL

CONSULTANT

wsp

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www.wsp.com

PROJECT

PROPOSED FURTHER LANDFILL DEVELOPMENT

TITLE

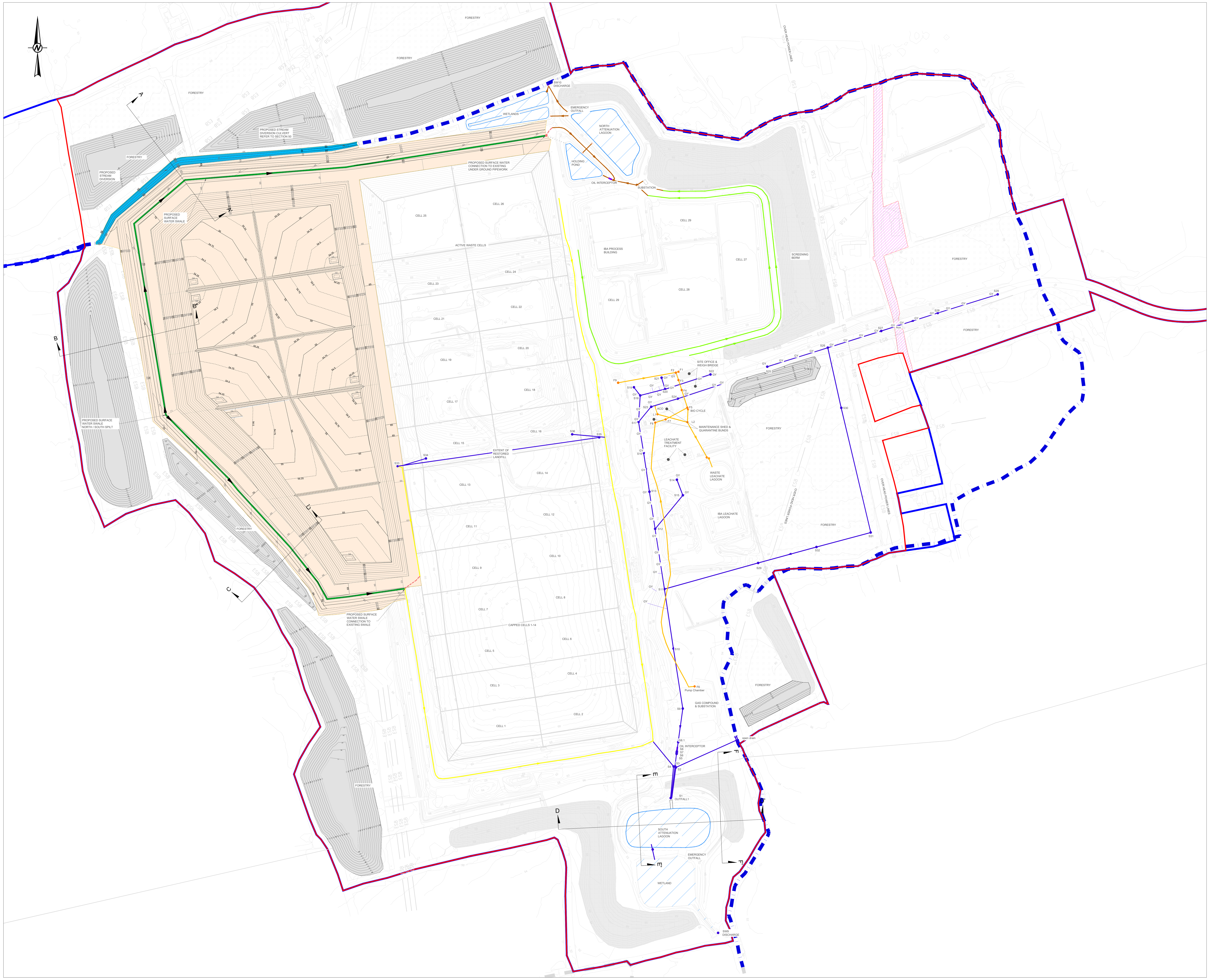
EXISTING FOUL AND SURFACE WATER LAYOUT

PROJECT NO.	SCALE	REV.	of	DRAWING
IE0037029.9627	1/3000 @ A1	B		017

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

25 mm

0



NOTE(S)

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3. ALL DIMENSIONS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
4. ALL LEVELS AND COORDINATES SHOWN ARE TO IRISH NATIONAL GRID

LEGEND

- PLANNING BOUNDARY
- OWNERSHIP BOUNDARY
- AREA WITHIN PLANNING BOUNDARY, NOT WITHIN SITE
- EXISTING WATERCOURSE
- WATER BODIES
- EXISTING SOUTH SURFACE WATER PIPELINE
- EXISTING SOUTH SURFACE WATER SWALE
- EXISTING NORTH SURFACE PIPELINE
- EXISTING NORTH SURFACE WATER SWALE
- EXISTING FOUL WATER PIPELINE
- EXISTING OIL INTERCEPTOR
- PROPOSED WEST LANDFILL EXTENSION FOOTPRINT
- PROPOSED STREAM DIVERSION
- PROPOSED NEW SURFACE WATER SWALE
- PROPOSED NEW PIPELINE
- B B' SECTION LINES

0 120 240 360m

1:3,000

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
B	2025-06-27	FOR PLANNING	DC	DC	NV	AL
A	2025-06-25	FOR PLANNING	DC	POB	AL	AL

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PROJECT

PROPOSED FURTHER LANDFILL DEVELOPMENT

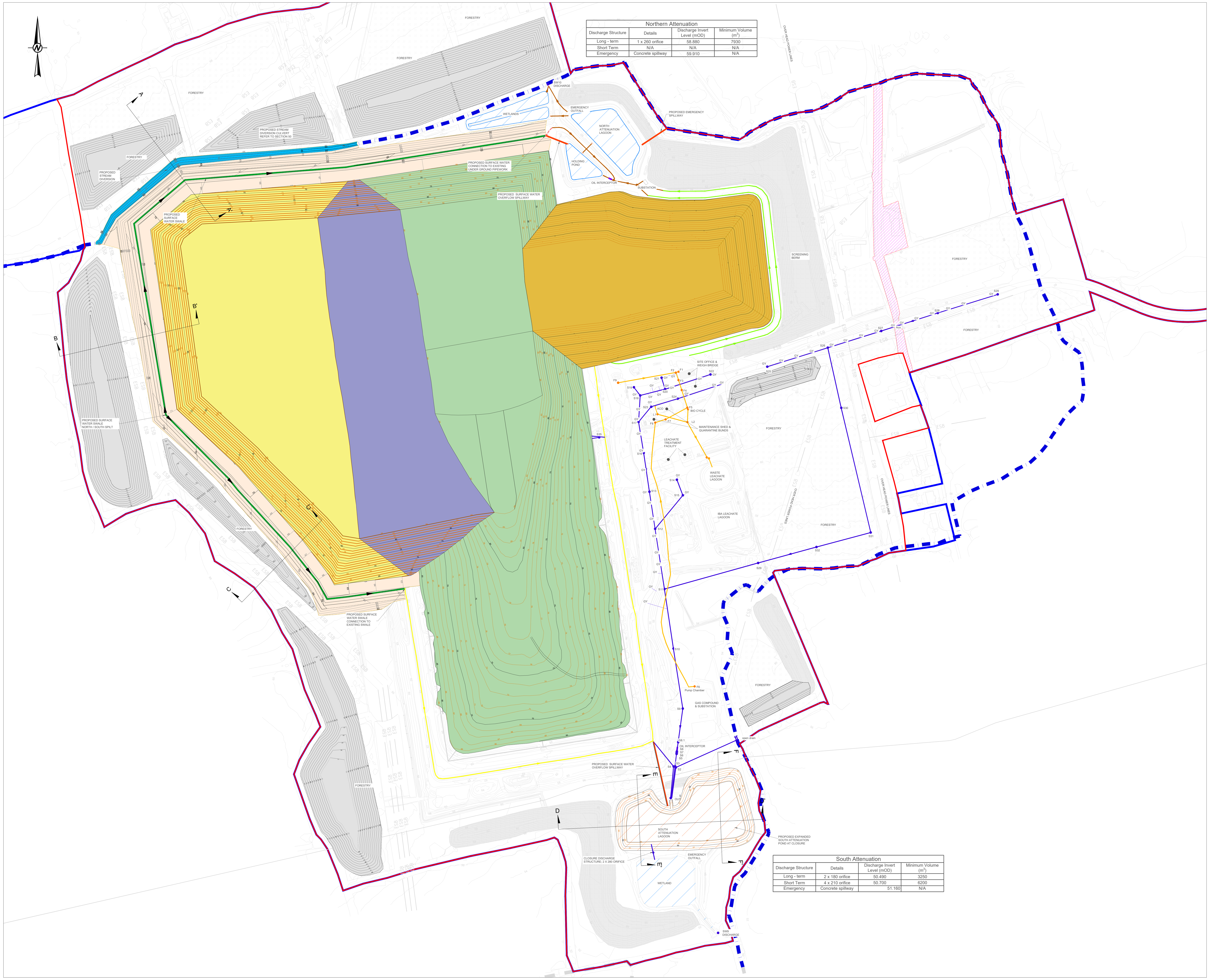
TITLE

PROPOSED SURFACE WATER LAYOUT - ACTIVE LANDFILL

PROJECT NO.	SCALE	REV.	of	DRAWING
IE0037029.9627	1/3000 @ A1	B		017A

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ISO A1

25 mm



Northern Attenuation			
Discharge Structure	Details	Discharge Invert Level (mOD)	Minimum Volume (m^3)
Long - term	1 x 260 orifice	58.880	7930
Short Term	N/A	N/A	N/A
Emergency	Concrete spillway	59.910	N/A

South Attenuation			
Discharge Structure	Details	Discharge Invert Level (mOD)	Minimum Volume (m^3)
Long - term	2 x 180 orifice	50.490	3250
Short Term	4 x 210 orifice	50.700	6200
Emergency	Concrete spillway	51.160	N/A

NOTE(S)

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LEGEND

- PLANNING BOUNDARY
- OWNERSHIP BOUNDARY
- AREA WITHIN PLANNING BOUNDARY, NOT WITHIN SITE
- EXISTING WATERCOURSE
- WATER BODIES
- EXISTING SOUTH SURFACE WATER PIPELINE
- EXISTING SOUTH SURFACE WATER SWALE
- EXISTING NORTH SURFACE PIPELINE
- EXISTING NORTH SURFACE WATER SWALE
- EXISTING FOUL WATER PIPELINE
- EXISTING OIL INTERCEPTOR
- PROPOSED WEST LANDFILL EXTENSION FOOTPRINT
- PROPOSED STREAM DIVERSION
- PROPOSED NEW SURFACE WATER SWALE
- PROPOSED NEW PIPELINE
- B B' SECTION LINES
- PROPOSED SOUTH ATTENUATION POND EXPANSION
- PROPOSED EMERGENCY SPILLWAY AT CLOSURE
- PROJECT WEST LANDFILL EXTENSION CAP
- PROJECT WEST LANDFILL EXTENSION PIGGYBACK CAP
- EXISTING LANDFILL CAP
- IBA FACILITY CAP

0 120 240 360m
1:3,000

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
B	2025-06-27	FOR PLANNING	DC	DC	NV	AL
A	2025-06-25	FOR PLANNING	DC	POB	AL	AL

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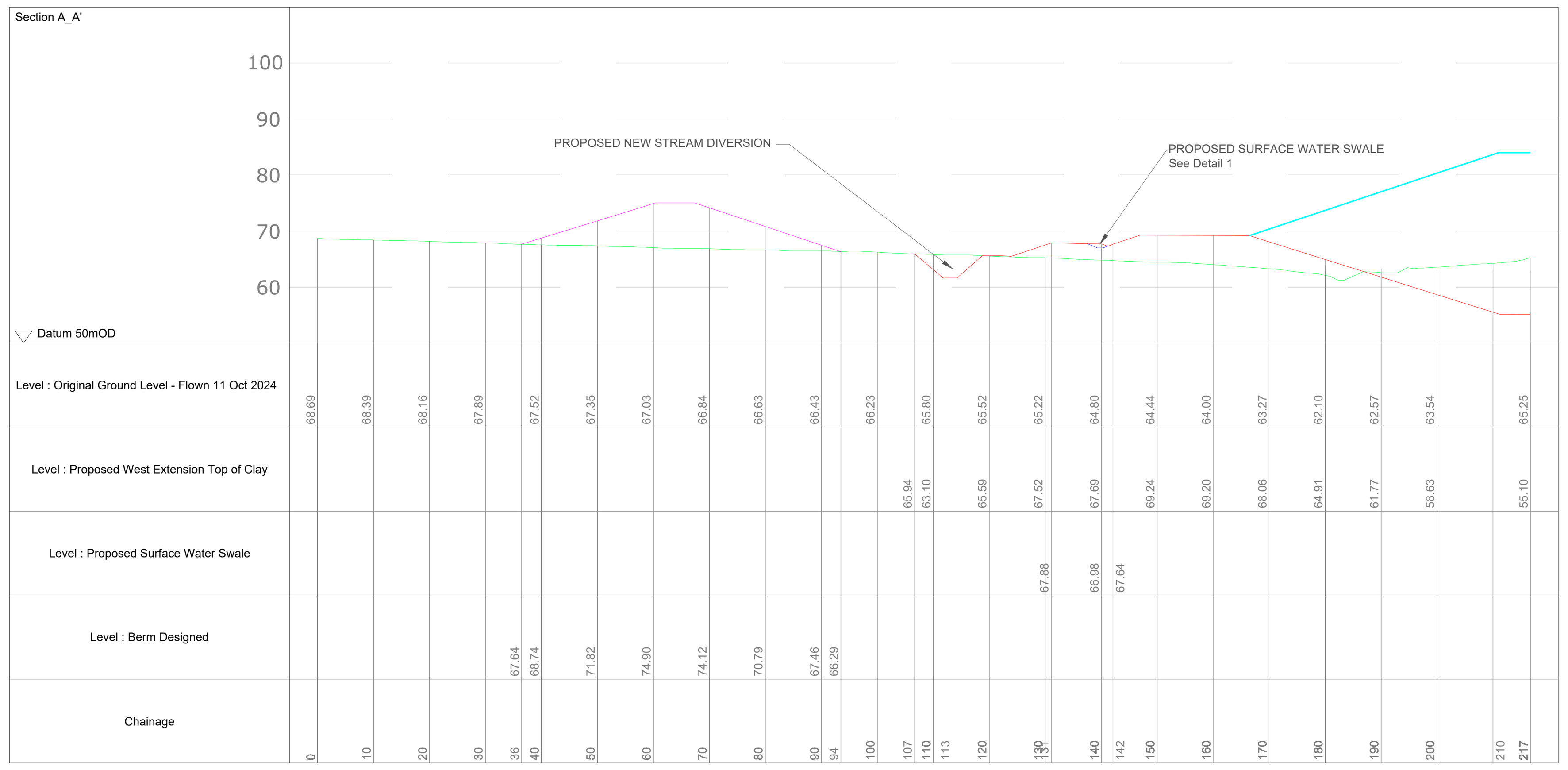
PROJECT
PROPOSED FURTHER LANDFILL DEVELOPMENT

TITLE
PROPOSED SURFACE WATER LAYOUT - CLOSURE LANDFILL

PROJECT NO.	SCALE	REV.	of	DRAWING
IE0037029.9627	1/3000 @ A1	B		017B



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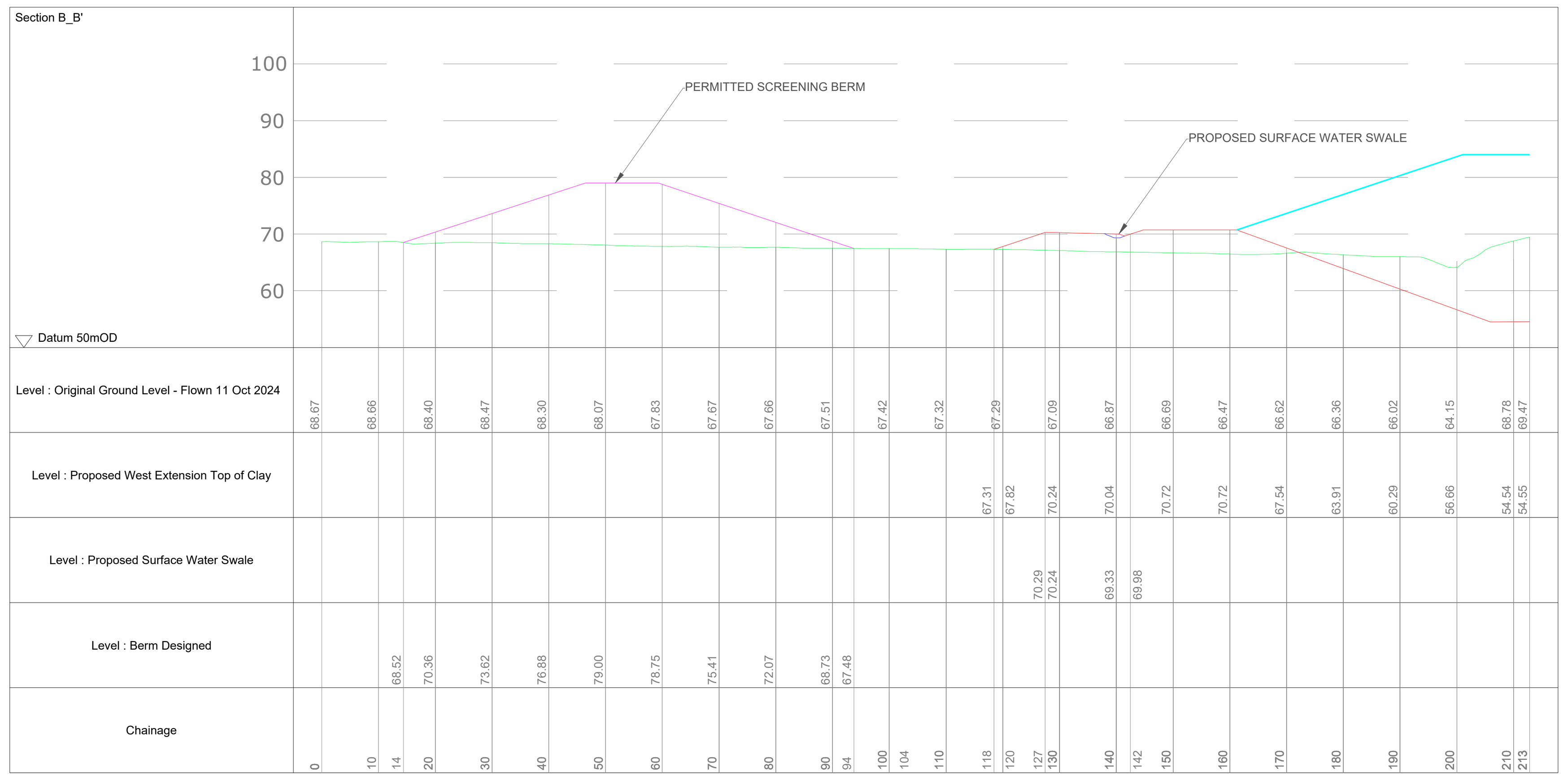

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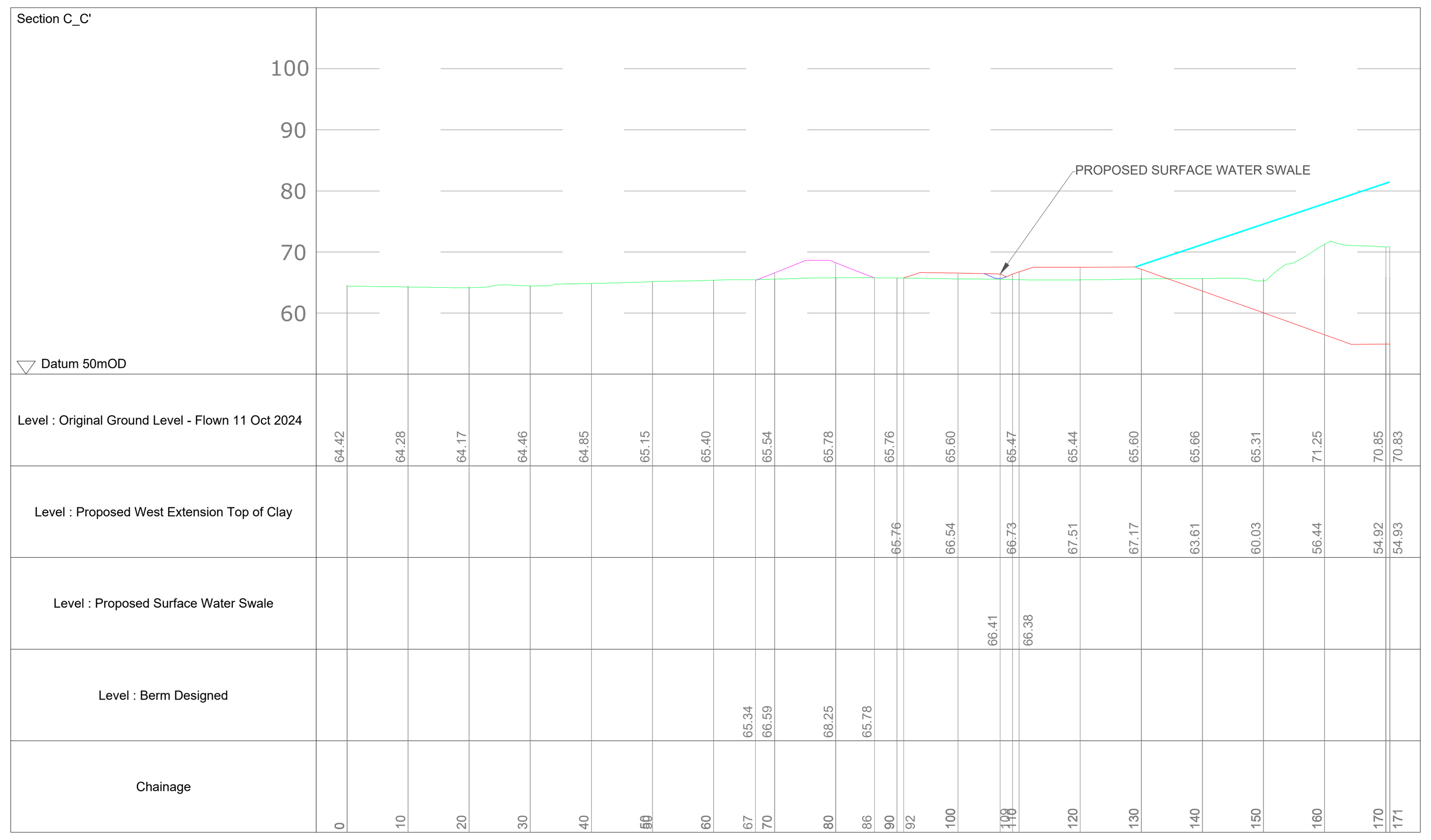

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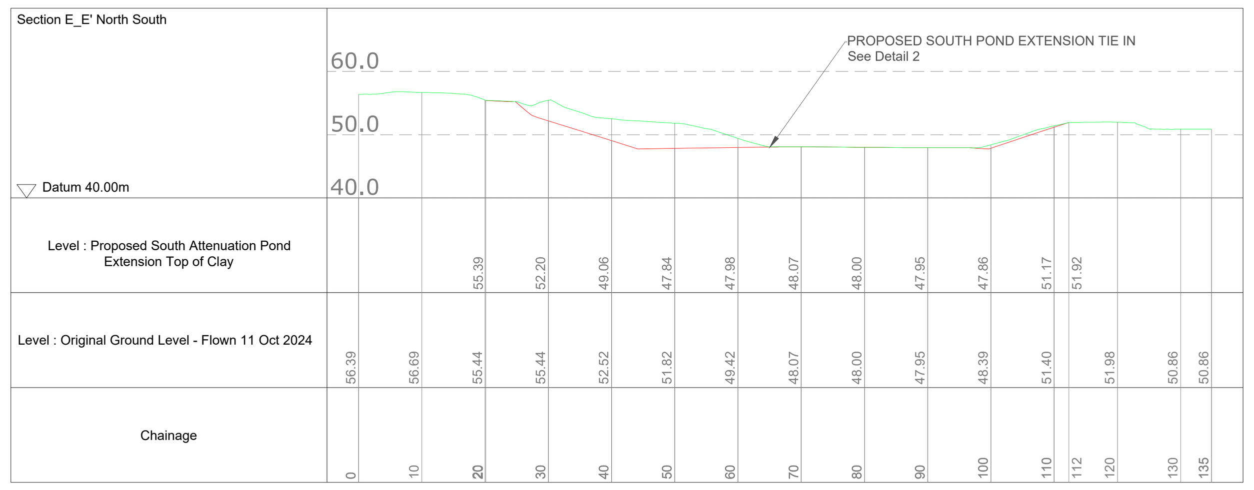

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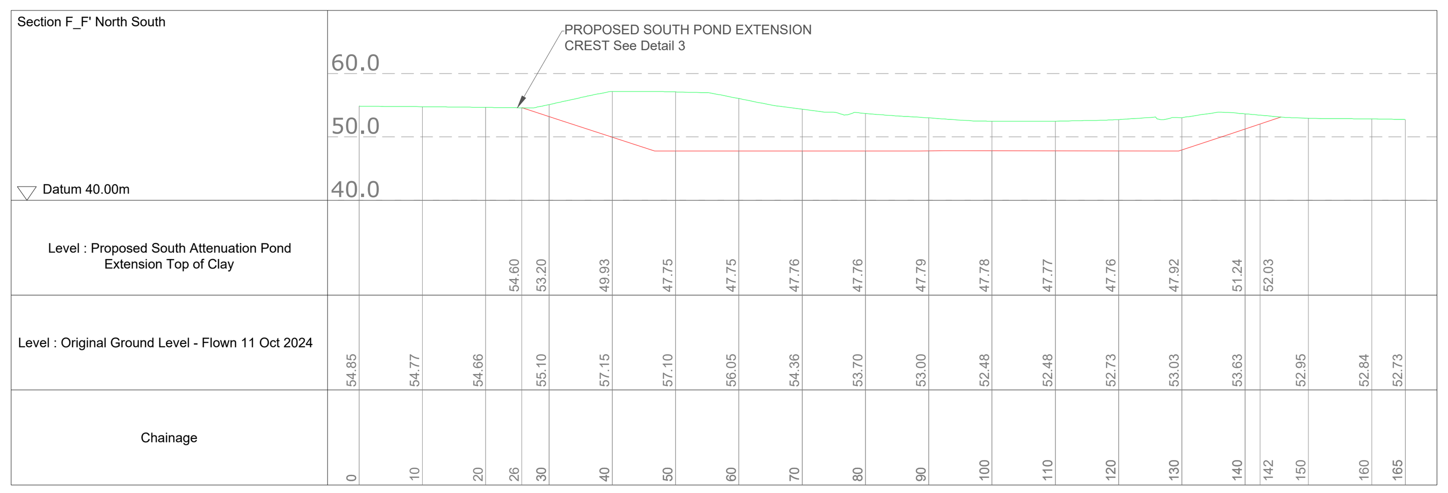

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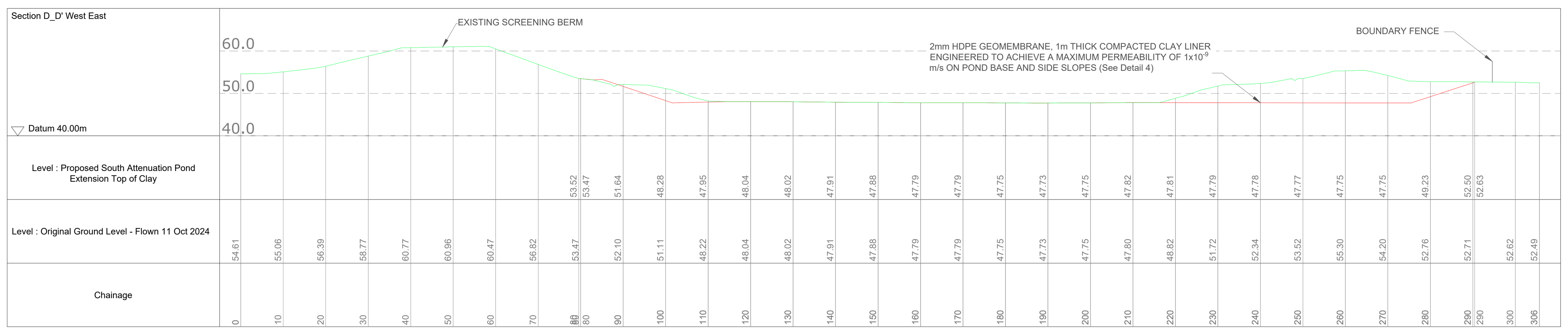

CROSS SECTION DD: (Scale: 1/1000 A1)

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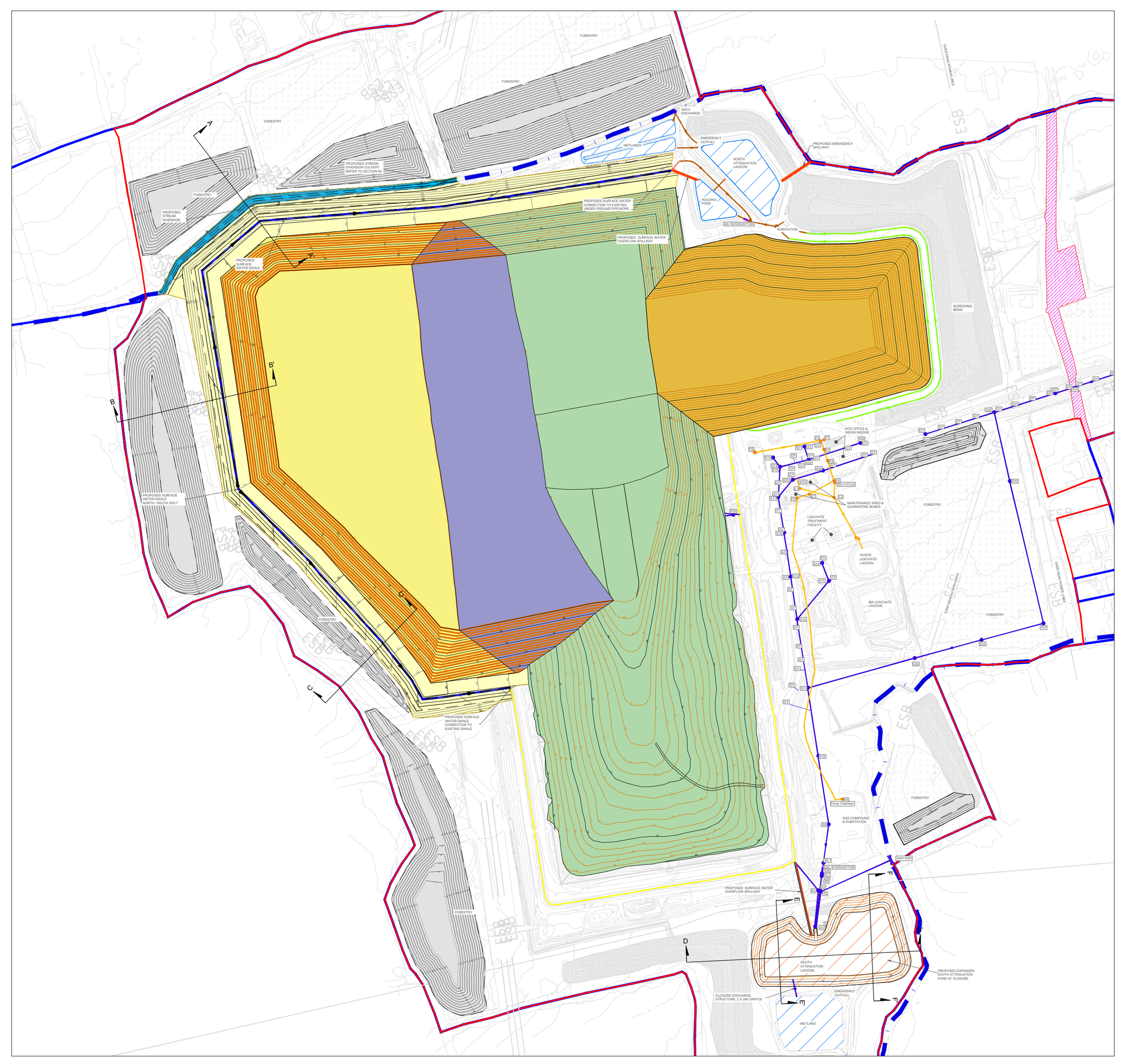
4. ALL LEVELS AND COORDINATES SHOWN ARE TO IRISH NATIONAL GRID

LEGEND

- ORIGINAL GROUND LEVEL OCTOBER 2024
- PROPOSED DESIGN TOP OF CLAY
- PROPOSED SWALE
- PROPOSED BERMS
- PROPOSED CAP

0 40 80 120m
1:1,000

KEY PLAN



REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
B	2025-06-27	FOR PLANNING	DC	DC	NV	AL
A	2025-06-25	FOR PLANNING	DC	POB	AL	AL

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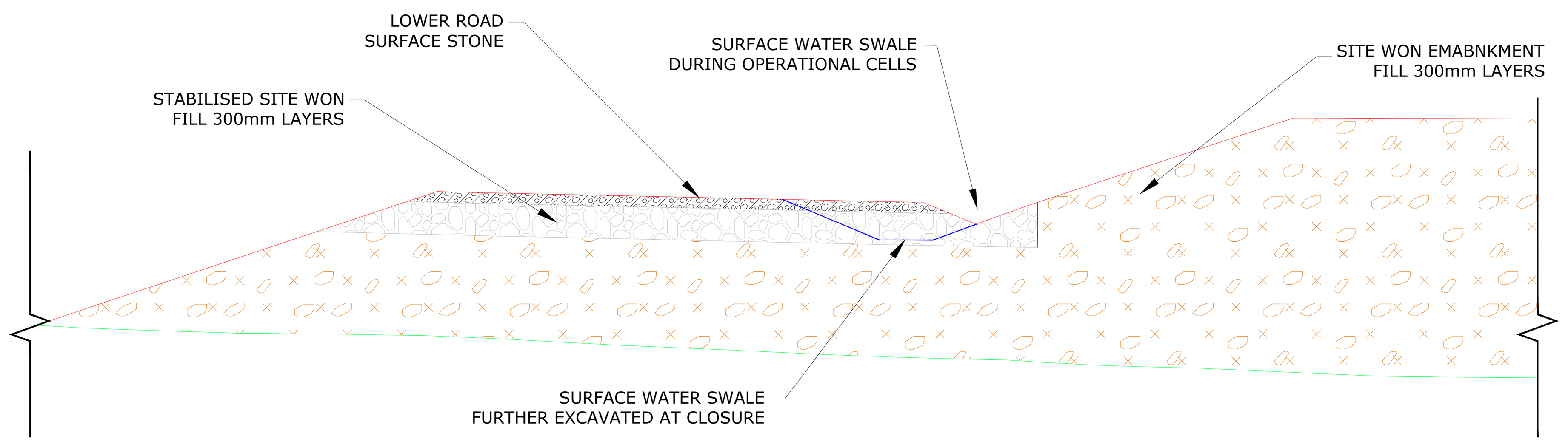
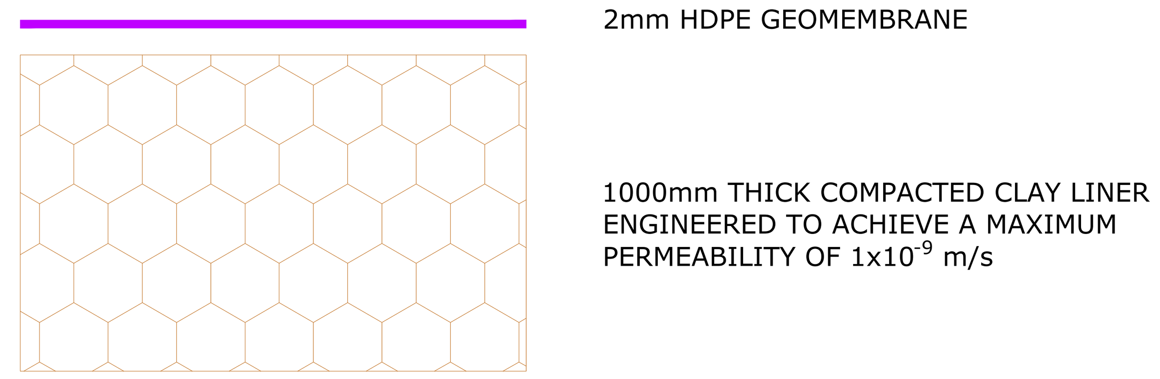
PROJECT
PROPOSED FURTHER LANDFILL DEVELOPMENT

TITLE
PROPOSED SURFACE WATER LAYOUT SECTIONS

PROJECT NO.	SCALE	REV.	of	DRAWING
IE0037027.9697	1/1000 @ A1	B		18



DETAIL 1; TYPICAL SURFACE WATER SWALE ACTIVE AND CLOSURE. (Scale: 1/100 A1)



DETAIL 4; TYPICAL SOUTH ATTENUATION POND LINING SYSTEM (Scale: 1/20 A1)

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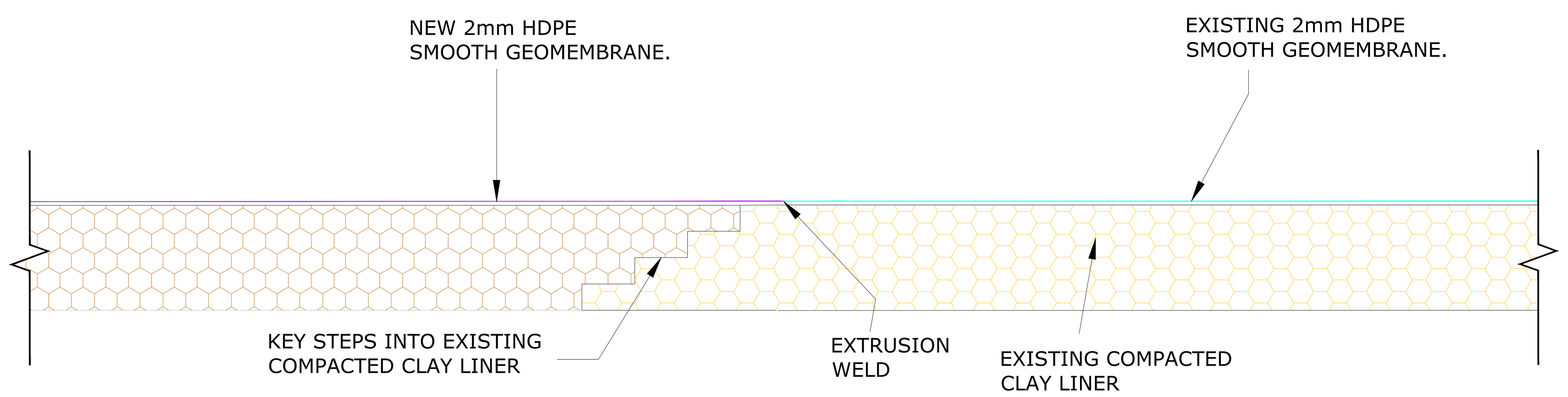
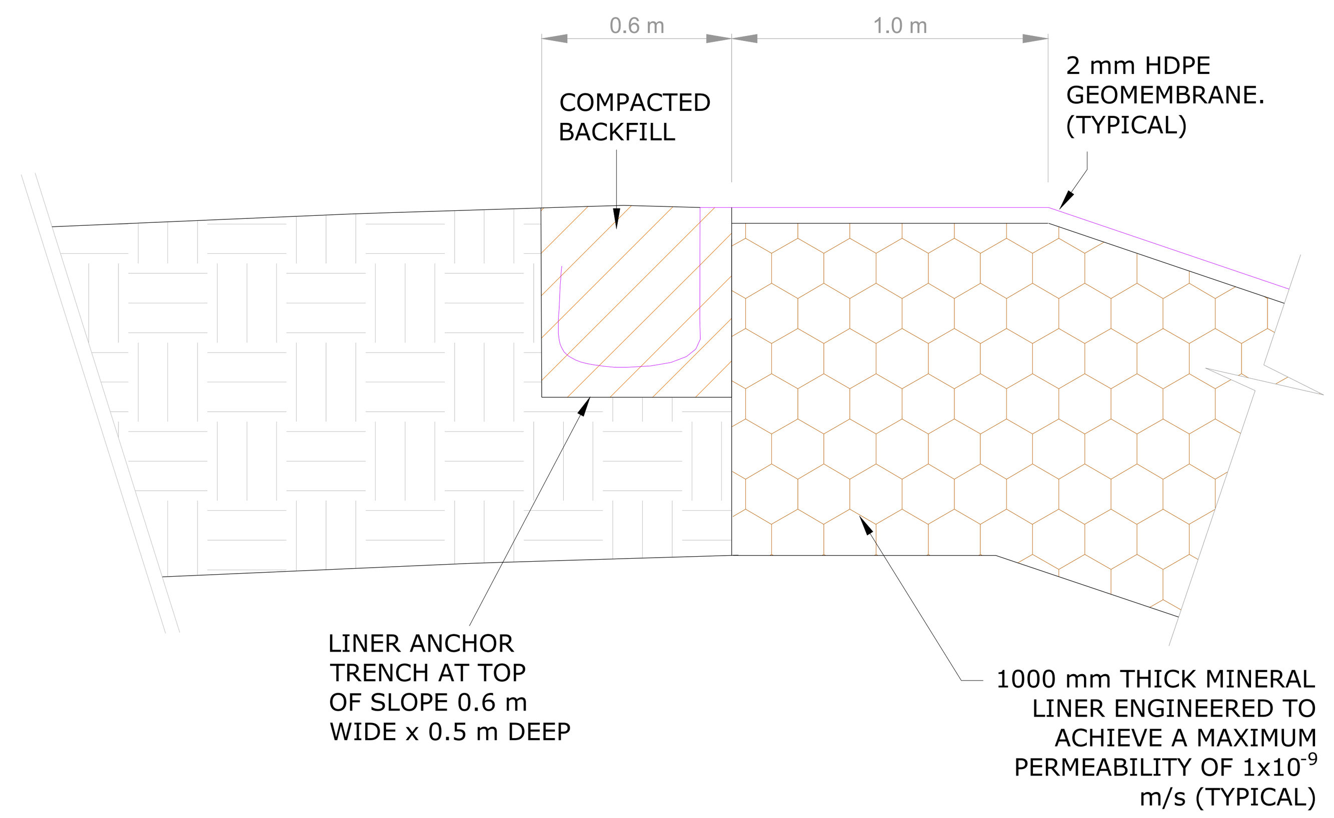
4. ALL LEVELS AND COORDINATES SHOWN ARE TO IRISH NATIONAL GRID

LEGEND

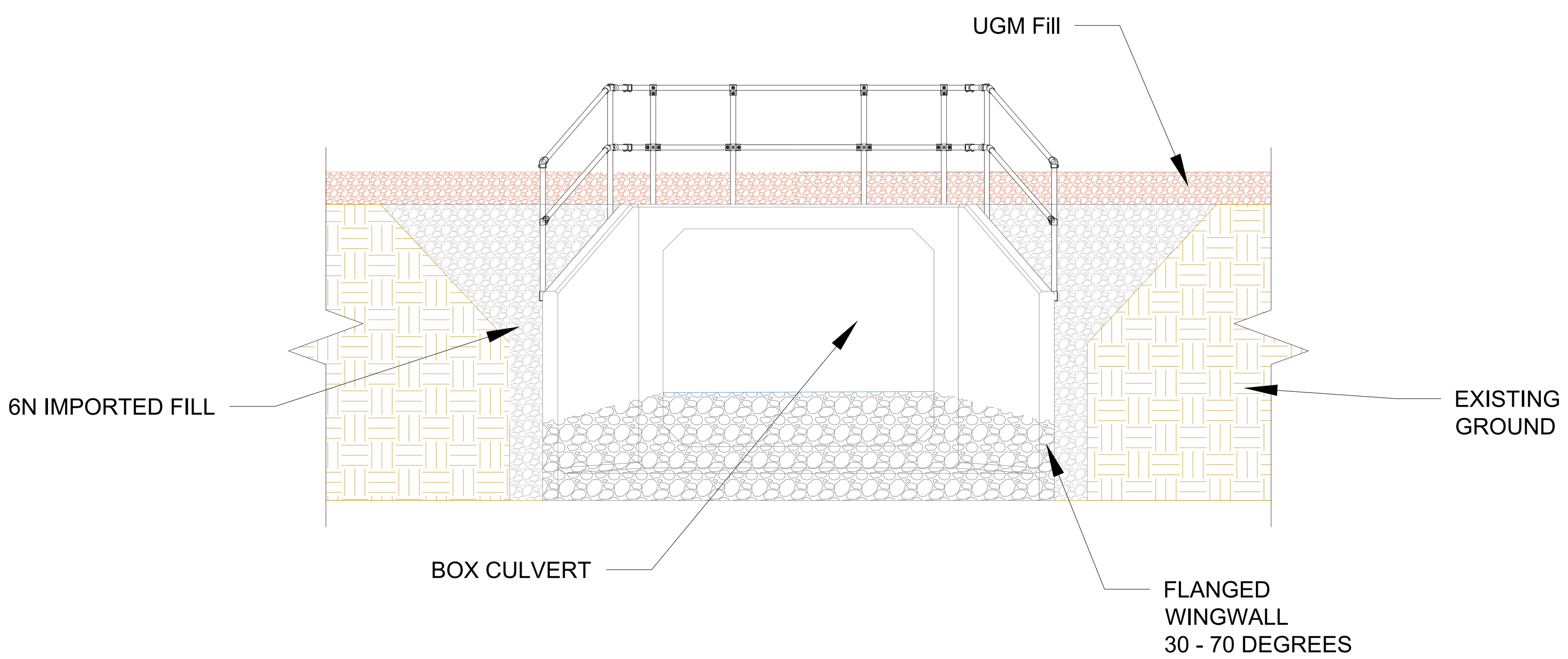
- ORIGINAL GROUND LEVEL OCTOBER 2024
- PROPOSED DESIGN TOP OF CLAY
- PROPOSED SWALE



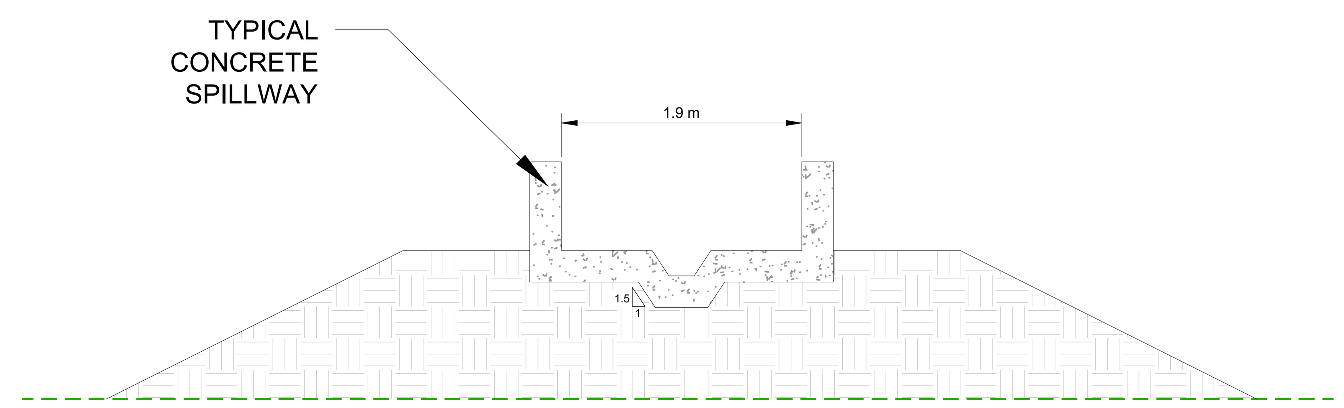
DETAIL 2; TYPICAL SOUTH ATTENUATION POND EXTENSION TIE IN. (Scale: 1/50 A1)



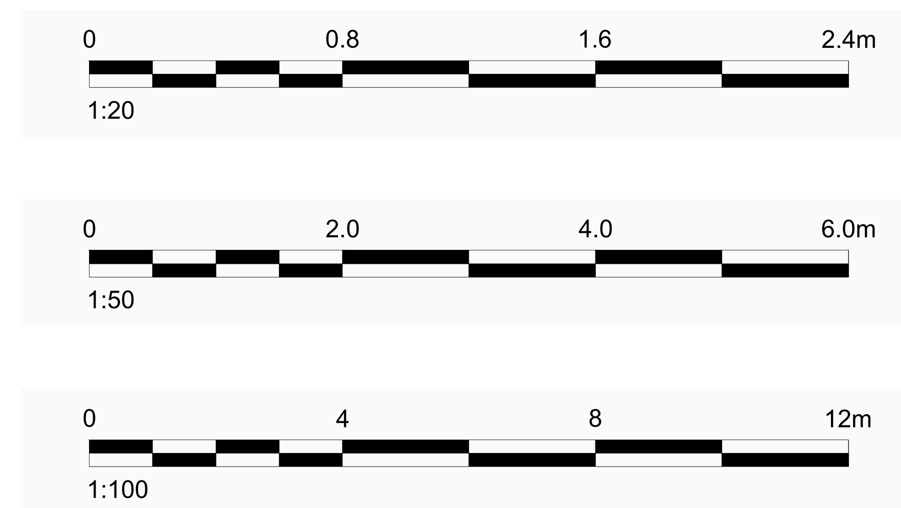
DETAIL 3; TYPICAL SOUTH ATTENUATION POND ANCHOR TRENCH AT CREST. (Scale: 1/20 A1)



DETAIL 5; TYPICAL CONCRETE CULVERT FOR SPILLWAY AT CLOSURE (Scale: 1/20 A1)



DETAIL 6; TYPICAL CONCRETE FOR SPILLWAY AT CLOSURE (Scale: 1/50 A1)



REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
B	2025-06-27	FOR PLANNING	DC	DC	NV	AL
A	2025-06-25	FOR PLANNING	DC	POB	AL	AL

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PROJECT
PROPOSED FURTHER LANDFILL DEVELOPMENT

TITLE
PROPOSED SURFACE WATER LAYOUT DETAILS

PROJECT NO.	SCALE	REV.	of	DRAWING
IE0037027.9697	Various @ A1	B		18A

APPENDIX B GREENFIELDS RUN-OFF CALCULATIONS

Greenfield runoff rate estimation tool

hrwallingford

www.uksuds.com | Greenfield runoff rate estimation tool (https://www.uksuds.com/)

This is an estimation of the greenfield runoff rates that are used to meet normal best practice criteria in line with Environment Agency guidance “Rainfall runoff management for developments”, SC030219 (2013), the SuDS Manual C753 (CIRIA, 2015) and the non-statutory standards for SuDS (Defra, 2015). This information on greenfield runoff rates may be the basis for setting consents for the drainage of surface water runoff from sites.

Project details

Date	11/06/2025
Calculated by	
Reference	
Model version	2.0.1

Location

Site name	
Site location	

Site easting	100850
Site northing	426066

Site details

Total site area (ha)	109.42	ha

Greenfield runoff

Method

Method	IH124

IH124

	My value		Map value
SAAR (mm)	894 mm		894
How should SPR be derived?	WRAP soil type		
WRAP soil type	2		2
SPR	0.3		
QBar (IH124) (l/s)	243.6 l/s		

Growth curve factors

	My value		Map value
Hydrological region	12		12
1 year growth factor	0.85		
2 year growth factor	0.95		
10 year growth factor	1.72		
30 year growth factor	2.13		
100 year growth factor	2.61		
200 year growth factor	2.86		

Results

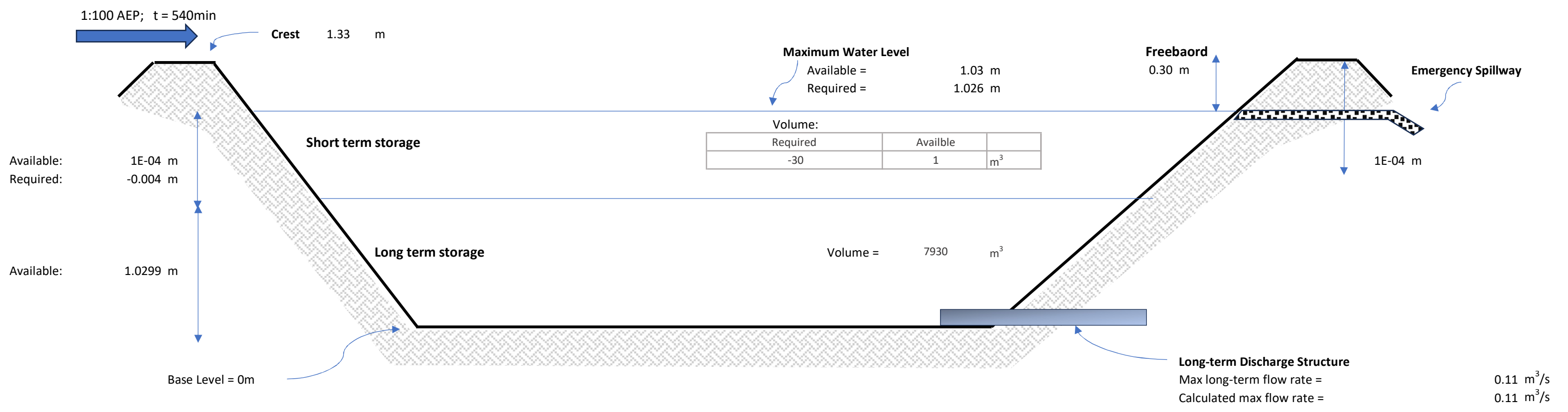
Method	IH124
Flow rate 1 year (l/s)	207 l/s
Flow rate 2 year (l/s)	231.3 l/s
Flow rate 10 years (l/s)	419 l/s
Flow rate 30 years (l/s)	518.8 l/s
Flow rate 100 years (l/s)	635.7 l/s
Flow rate 200 years (l/s)	696.6 l/s

Disclaimer

APPENDIX C NORTHERN ATTENUATION POND CALCULATIONS

	Project		Knockharley Landfill	Job No.	IE0037027.9697 - Project West		
			Project West	Made By	BoC	Date	18/06/2025
	Calc. Title		Sizing of SuDs Attenuation Pond	Chkd by	NV	Date	
				Sheet No.	3/3	Rev	00

Reference | Calculations

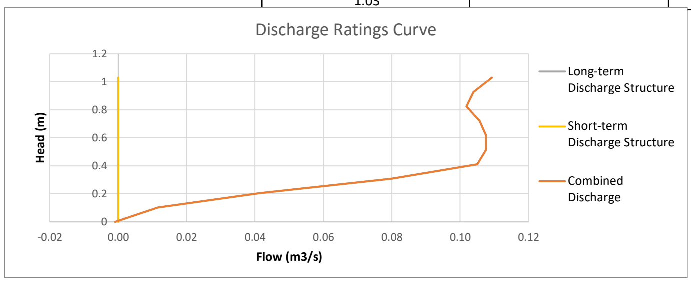


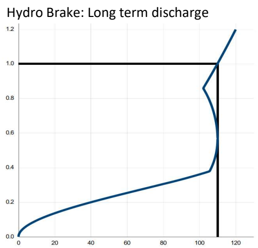
Emg. Spilway IVL	1.03	m
Short-term Discharge Structure IVL	1.03	m
Long-term Discharge Structure IVL	0	m
Freeboard Depth	0.3	m

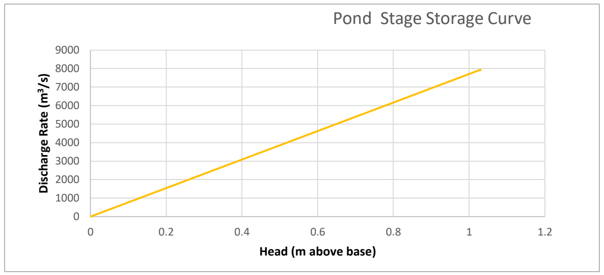
	Long-term Discharge Structure	Short-term Discharge Structure	
Invert level	0	1.0299	m
Pipe Size	0.26	0.00	m
Nr of orifice in parallel	1	0	
Head (m)	Orifice Discharge (m3/s)	Oriface Discharge	Combined Discharge
0	0.00	0.00	0.00
0.103	0.01	0.00	0.01
0.206	0.04	0.00	0.04
0.309	0.08	0.00	0.08
0.412	0.10	0.00	0.10
0.515	0.11	0.00	0.11
0.618	0.11	0.00	0.11
0.721	0.11	0.00	0.11
0.824	0.10	0.00	0.10
0.927	0.10	0.00	0.10
1.03	0.11	0.00	0.11

Pond Stage Storage Table

	Depth	volume
	m	m³
Base & Low flow discharge level	0	0
Long-term Discharge Structure IVL	1.0299	7930.23
Emg. Spilway IVL	1.03	7931







Project	Knockharley Landfill Project West	Job No.	IE0037027.9697 - Project West
		Made By	BoC
		Date	18/06/2025
Calc. Title	Sizing of SuDs Attenuation Pond	Chkd by	NV
		Date	
		Sheet No.	1/3
		Rev	00

Reference | Calculations

This spreadsheet has been developed to size an attenuation pond for the control of discharge rate and volume in accordance with the CIRIA SuDS Manual (C753) and the GDRCP.
The pond and outlet have been sized based on the following assumptions:

- The short and long termstorage volumes are designed to attenuate the 1:100 Annual Exceedance Probability (AEP) event while limiting discharge to below the 1:100 AEP greenfield runoff rate (Ref: SuDS Manual Section 24.10).
- The long term storage volume is assumed to provide long-term storage capacity for runoff exceeding the greenfield volume from a 1:100 AEP, 6-hour duration event (i.e. Long-term storage = Developed runoff volume – Greenfield runoff volume) (Ref: SuDS Manual Section 24.10).
- Long-term storage is discharged at a rate of 2 L/s/ha (Ref: SuDS Manual Section 24.10).
- Only the greenfield runoff volume is discharged at the greenfield runoff rate (Ref: SuDS Manual Section 24.10).
- Volumetric control is sized considerting only the 6-hour, 1:100 AEP event. (Ref: SuDS Manual Section 24.10.4).
- The inflow hydrograph is assumed to be uniform and calculated as total precipitation divided by storm duration.

Table: Catchment Properties

Catchment	Area (ha)	Run-off Factor	Factored Area (ha)	Notes
Landfill & Ash Area	23.60	30%	7.08	*WRAP SOIL Type 2*
Hardstanding Area	4.44	100%	4.44	*100% impermeable*
Greenfield Area Contributing	16.77	30%	5.03	*WRAP SOIL Type 2*
Total Contributing Area	44.81	*N/A*	16.55	

89.6187435

	Pond Stage Storage Table	
	Depth	**volume**
	m	m^3
Base & Low Flow Discharge IVL	0	0
Outlet Spillway/Orifice IVL	1.03	7930
Emg. Spillway level IVL	1.03	7931

3000

Volume available :

Short term	1	m^3
Long term	7930	m^3
Total	7931	m^3

Outlet Rating Table		
Head	**Flow Rate**	Combined rating curve for all outlet structures
m	m^3/s	
0	0.00	Base & Low flow discharge level
0.103	0.01	
0.206	0.04	
0.309	0.08	
0.412	0.10	
0.515	0.11	
0.618	0.11	
0.721	0.11	
0.824	0.10	
0.927	0.10	
1.03	0.11	

<u>Greenfield</u>

NERC, 1975	WRAP soil type	=	2		*Moderately permeable soils, often loamy. Some infiltration, moderate runoff.*
Based on WRAP	SPR value for the Greenfield site	=	30%	-	
SUDS Manual 24.10.2	1:100 AEP, 6-hours Rainfall Depth	=	52.9	mm	
	1:100 AEP, 6-hours Greenfield Flow Volume	=	7111	m^3	
IH124 Input	1:100 AEP Greenfield Flow Rate	=	**0.283**	m^3/s	
2l/s/ha	Long term Discharge Rate	=	**0.110**	m^3/s	*(110l/s of the total 218 l/s allowed for the site)*

<u>Post Development</u>

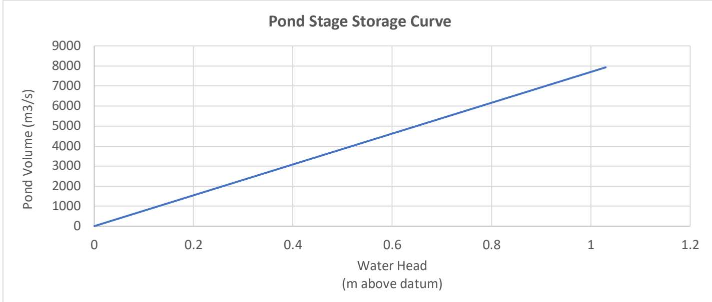
	Effective Catchment Area	=	165532	m^2	
	Climate Change Factor	=	1.1		
	Critical Storm Duration		**1:100 AEP; t = 540min**		
Critical Duration	Maximum **Depth** in Pond	=	1.03	m	
	Maximum depth of Short term storage portion	=	0.00	m	
	Max Short Term Discharge **Rate**	=	0.109	m^3/s	Good
	Max Short Term Attenuatoin **Storage** Volume Required	=	-30	m^3	Good, Required Volume is less than availble storage
	Short Term **Discharge** Volume	=	2807	m^3	Good, Short-term discharge volume is less than Greenfield allowable
	Total Attenuatoin Pond Volume Required	=	7901	m^3	Good
	Additional volume storage required	=	-30	m^3	
	Max Long Term Discharge Rate	=	0.109	m^3/s	Good
	Max Long Term Attenuatoin Storage Volume	=	7930	m^3/s	

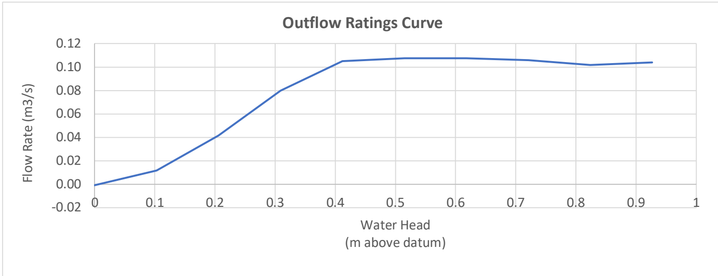
Table: Rainfall Depth(mm) without Climate Change

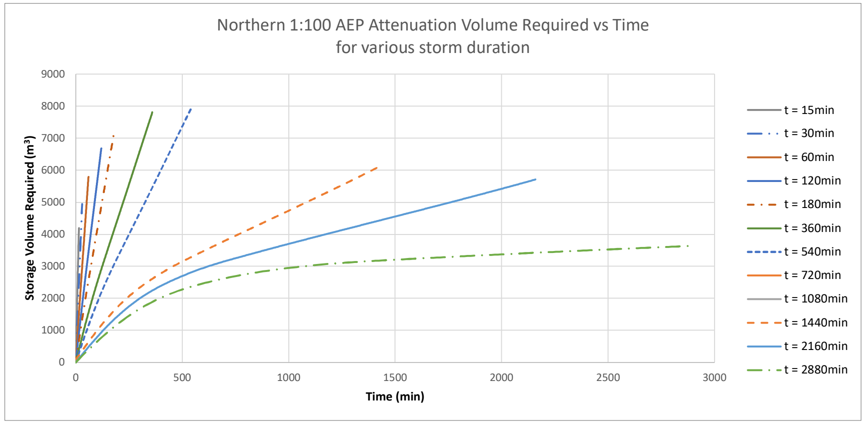
Duration		Return Period (years)					
(min)	(hours)	1	2	5	10	30	100
15	0.25	4.5	6.2	7.2	10.2	12.6	23.3
30	0.5	5.9	8	9.1	12.8	15.6	27.8
60	1	7.7	10.3	11.6	16	19.2	33.3
120	2	10	13.2	14.8	20	23.8	39.8
180	3	11.7	15.2	17.1	22.7	26.9	44.2
360	6	15.2	19.6	21.7	28.4	33.2	52.9
540	9	17.8	22.6	25	32.4	37.6	58.7
720	12	19.9	25.1	27.7	35.5	41.1	63.3
1080	18	23.3	29	31.9	40.5	46.5	70.3
1440	24	26	32.2	35.3	44.4	50.8	75.8
2160	36	36.9	44.7	48.5	59.8	67.5	96.7
2880	48	41.3	49.8	53.9	66	74.3	105.3

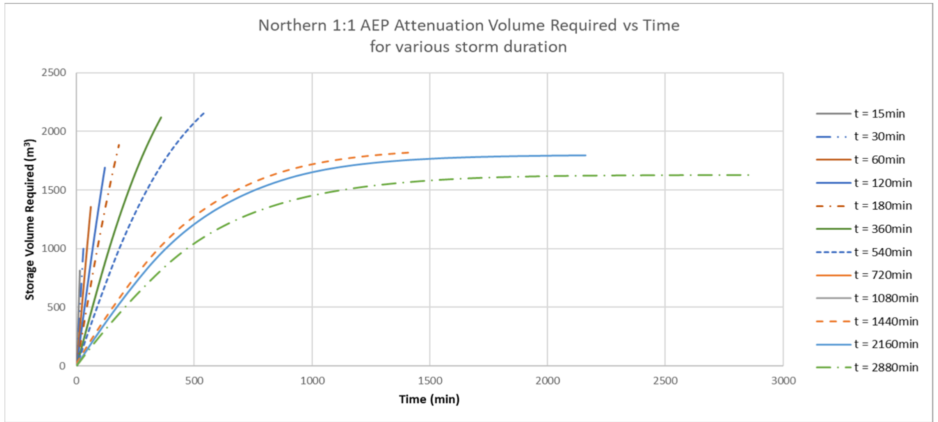
Table: Rainfall Depth(mm) with Climate Change

Duration		Return Period (years)					
(min)	(hours)	1	2	5	10	30	100
15	0.25	4.95	6.82	7.92	11.22	13.86	25.63
30	0.5	6.49	8.8	10.01	14.08	17.16	30.58
60	1	8.47	11.33	12.76	17.6	21.12	36.63
120	2	11	14.52	16.28	22	26.18	43.78
180	3	12.87	16.72	18.81	24.97	29.59	48.62
360	6	16.72	21.56	23.87	31.24	36.52	58.19
540	9	19.58	24.86	27.5	35.64	41.36	64.57
720	12	21.89	27.61	30.47	39.05	45.21	69.63
1080	18	25.63	31.9	35.09	44.55	51.15	77.33
1440	24	28.6	35.42	38.83	48.84	55.88	83.38
2160	36	40.59	49.17	53.35	65.78	74.25	106.37
2880	48	45.43	54.78	59.29	72.6	81.73	115.83





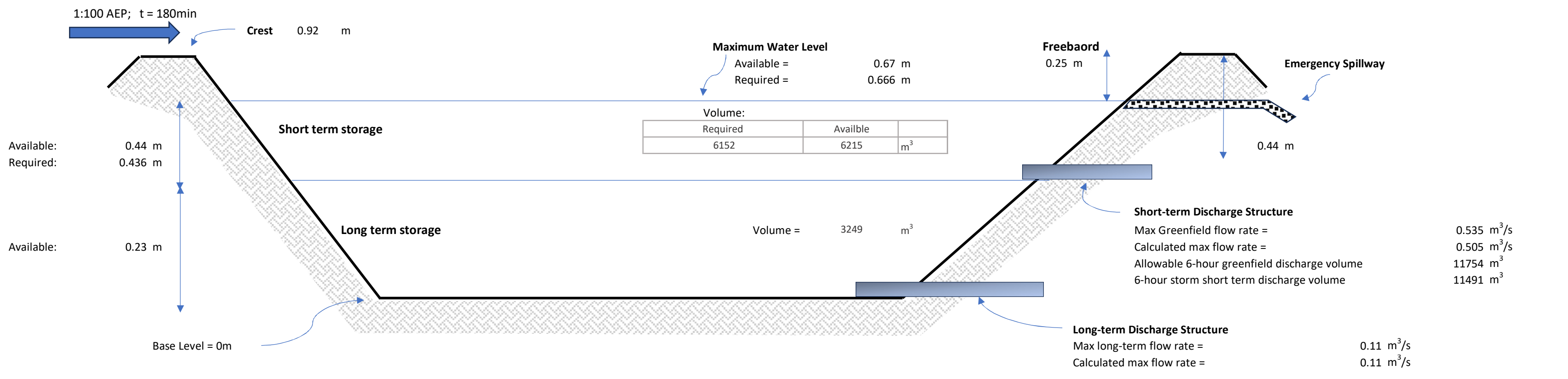




APPENDIX D SOUTHERN ATTENUATION POND CALCULATIONS

	Project		Knockharley Landfill Project West	Job No.	IE0037027.9697 - Project West		
				Made By	BoC	Date	18/06/2025
	Calc. Title		Sizing of SuDs Attenuation Pond	Chkd by	NV	Date	
				Sheet No.	3/3	Rev	00

Reference | Calculations

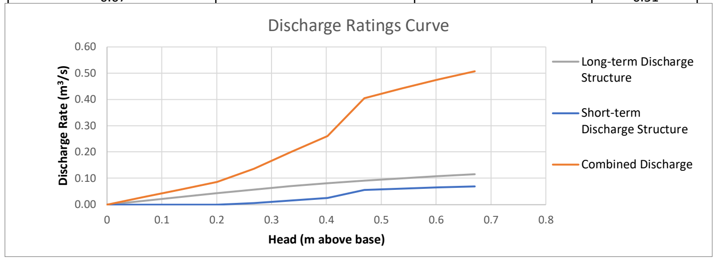


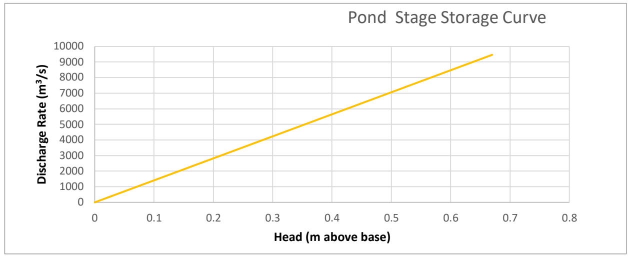
Emg. Spilway IVL	0.67	m
Short-term Discharge Structure IVL	0.23	m
Long-term Discharge Structure IVL	0	m
Freeboard Depth	0.25	m

	Long-term Discharge Structure	Short-term Discharge Structure	
Invert level	0	0.23	m
Pipe Size	0.28	0.21	m
Nr of orifice in parallel	2	4	
Head (m)	Orifice Discharge (m3/s)	Oriface Discharge (m3/s)	Combined Discharge
0	0.00	0.00	0.00
0.067	0.01	0.00	0.03
0.134	0.03	0.00	0.06
0.201	0.04	0.00	0.09
0.268	0.06	0.01	0.14
0.335	0.07	0.01	0.20
0.402	0.08	0.02	0.26
0.469	0.09	0.06	0.40
0.536	0.10	0.06	0.44
0.603	0.11	0.06	0.48
0.67	0.12	0.07	0.51

Pond Stage Storage Table

	Depth	volume
	m	m^3
Base & Low flow discharge level	0	0
Long-term Discharge Structure IVL	0.23	3248.835821
Emg. Spilway IVL	0.67	9464





Reference	Calculations

This spreadsheet has been developed to size an attenuation pond for the control of discharge rate and volume in accordance with the CIRIA SuDS Manual (C753) and the GDRCP.
The pond and outlet have been sized based on the following assumptions:

- The short and long termstorage volumes are designed to attenuate the 1:100 Annual Exceedance Probability (AEP) event while limiting discharge to below the 1:100 AEP greenfield runoff rate (Ref: SuDS Manual Section 24.10).
- The long term storage volume is assumed to provide long-term storage capacity for runoff exceeding the greenfield volume from a 1:100 AEP, 6-hour duration event (i.e. Long-term storage = Developed runoff volume – Greenfield runoff volume) (Ref: SuDS Manual Section 24.10).
- Long-term storage is discharged at a rate of 2 L/s/ha (Ref: SuDS Manual Section 24.10).
- Only the greenfield runoff volume is discharged at the greenfield runoff rate (Ref: SuDS Manual Section 24.10).
- Volumetric control is sized considerting only the 6-hour, 1:100 AEP event. (Ref: SuDS Manual Section 24.10.4).
- The inflow hydrograph is assumed to be uniform and calculated as total precipitation divided by storm duration.

Table: Catchment Properties

Catchment	Area (ha)	Run-off Factor	Factored Area (ha)	Notes
Landfill & Ash Area	29.13	30%	8.74	*WRAP SOIL Type 2*
Hardstanding Area	8.35	100%	8.35	*100% impermeable*
Greenfield Area Contributing	27.13	30%	8.14	*WRAP SOIL Type 2*
Total Contributing Area	64.61	*N/A*	25.23	

	Pond Stage Storage Table	
	Depth	volume
	m	m^3
Base & Low Flow Discharge IVL	0	0
Outlet Spillway/Orifice IVL	0.23	3249
Emg. Spillway level IVL	0.67	9464

4500

Volume available :

Short term	6215	m^3
Long term	3249	m^3
Total	9464	m^3

Outlet Rating Table — Combined rating curve for all outlet structures

Head	Flow Rate	
m	m^3/s	
0	0.00	Base & Low flow discharge level
0.067	0.03	
0.134	0.06	
0.201	0.09	
0.268	0.14	
0.335	0.20	
0.402	0.26	
0.469	0.40	
0.536	0.44	
0.603	0.48	
0.67	0.51	

Greenfield

NERC, 1975	WRAP soil type	=	2		*Moderately permeable soils, often loamy. Some infiltration, moderate runoff.*
Based on WRAP	SPR value for the Greenfield site	=	30%	-	
SUDS Manual 24.10.2	1:100 AEP, 6-hours Rainfall Depth	=	52.9	mm	
	1:100 AEP, 6-hours Greenfield Flow Volume	=	11754	m^3	
IH124 Input	1:100 AEP Greenfield Flow Rate	=	0.54	m^3/s	
2l/s/ha	Long term Discharge Rate	=	0.108	m^3/s	*(108l/s of the total 218 l/s allowed for the site)*

Post Development

	Effective Catchment Area	=	252317	m^2	
	Climate Change Factor	=	1.1		
	Critical Storm Duration		**1:100 AEP; t = 180min**		
Critical Duration	Maximum **Depth** in Pond	=	0.67	m	
	Maximum depth of Short term storage portion	=	0.44	m	
	Max Short Term Discharge **Rate**	=	0.505	m^3/s	Good
	Max Short Term Attenuatoin **Storage** Volume Required	=	6152	m^3	Good, Required Volume is less than availble storage
	Short Term **Discharge** Volume	=	11491	m^3	Good, Short-term discharge volume is less than Greenfield allowable
	Total Attenuatoin Pond Volume Required	=	9401	m^3	Good
	Additional volume storage required	=	-63	m^3	
	Max Long Term Discharge Rate	=	0.107	m^3/s	Good
	Max Long Term Attenuatoin Storage Volume	=	3249	m^3/s	

Table: Rainfall Depth(mm) without Climate Change

Duration		Return Period (years)					
(min)	(hours)	1	2	5	10	30	100
15	0.25	4.5	6.2	7.2	10.2	12.6	23.3
30	0.5	5.9	8	9.1	12.8	15.6	27.8
60	1	7.7	10.3	11.6	16	19.2	33.3
120	2	10	13.2	14.8	20	23.8	39.8
180	3	11.7	15.2	17.1	22.7	26.9	44.2
360	6	15.2	19.6	21.7	28.4	33.2	52.9
540	9	17.8	22.6	25	32.4	37.6	58.7
720	12	19.9	25.1	27.7	35.5	41.1	63.3
1080	18	23.3	29	31.9	40.5	46.5	70.3
1440	24	26	32.2	35.3	44.4	50.8	75.8
2160	36	36.9	44.7	48.5	59.8	67.5	96.7
2880	48	41.3	49.8	53.9	66	74.3	105.3

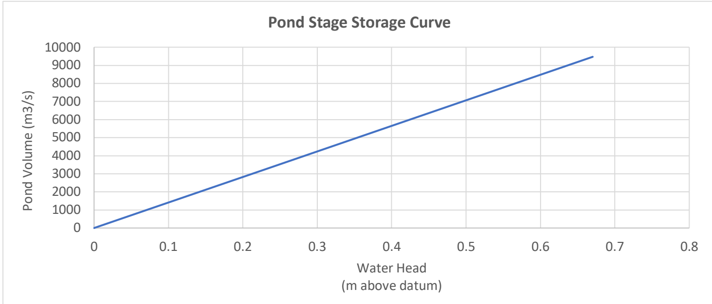
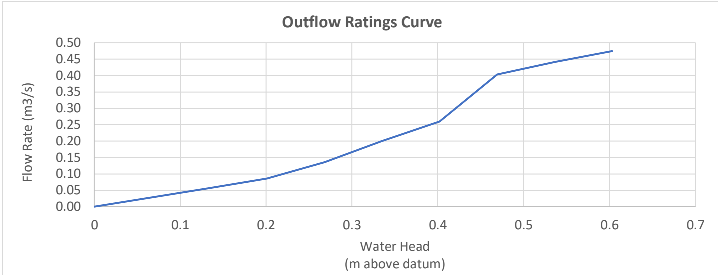
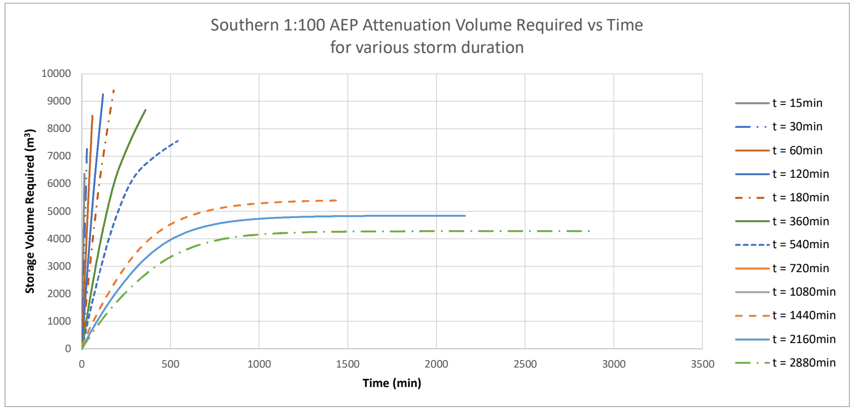
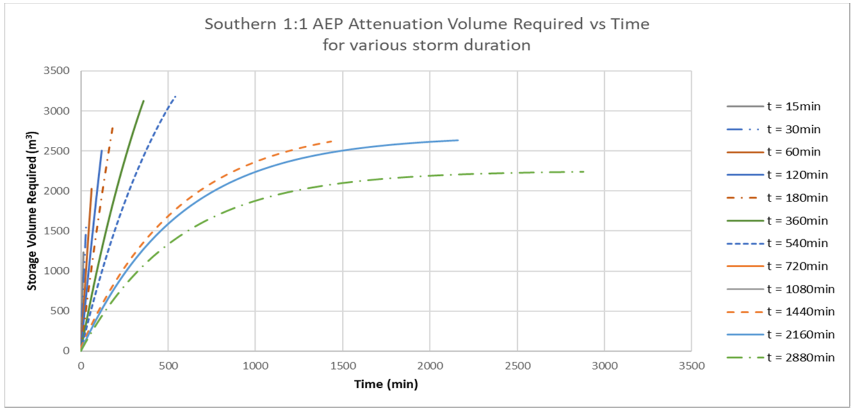


Table: Rainfall Depth(mm) with Climate Change

Duration		Return Period (years)					
(min)	(hours)	1	2	5	10	30	100
15	0.25	4.95	6.82	7.92	11.22	13.86	25.63
30	0.5	6.49	8.8	10.01	14.08	17.16	30.58
60	1	8.47	11.33	12.76	17.6	21.12	36.63
120	2	11	14.52	16.28	22	26.18	43.78
180	3	12.87	16.72	18.81	24.97	29.59	48.62
360	6	16.72	21.56	23.87	31.24	36.52	58.19
540	9	19.58	24.86	27.5	35.64	41.36	64.57
720	12	21.89	27.61	30.47	39.05	45.21	69.63
1080	18	25.63	31.9	35.09	44.55	51.15	77.33
1440	24	28.6	35.42	38.83	48.84	55.88	83.38
2160	36	40.59	49.17	53.35	65.78	74.25	106.37
2880	48	45.43	54.78	59.29	72.6	81.73	115.83





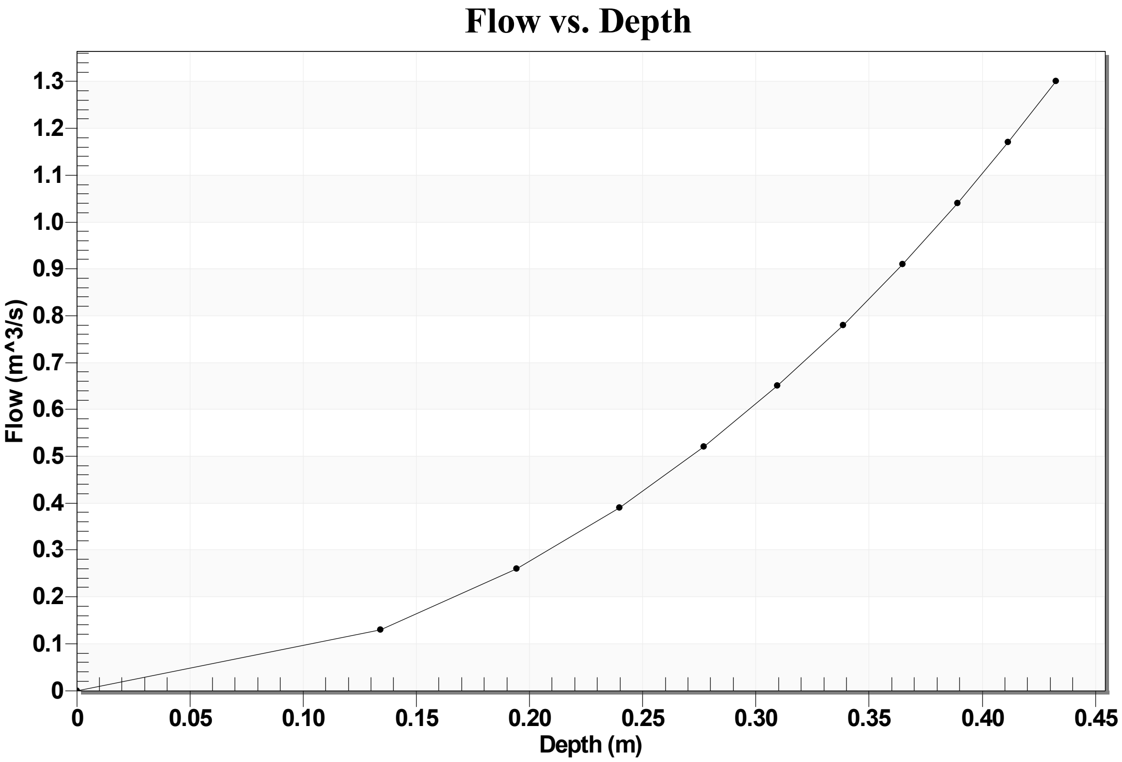


APPENDIX E SWALE HYDRAULIC CAPACITY CALCULATIONS

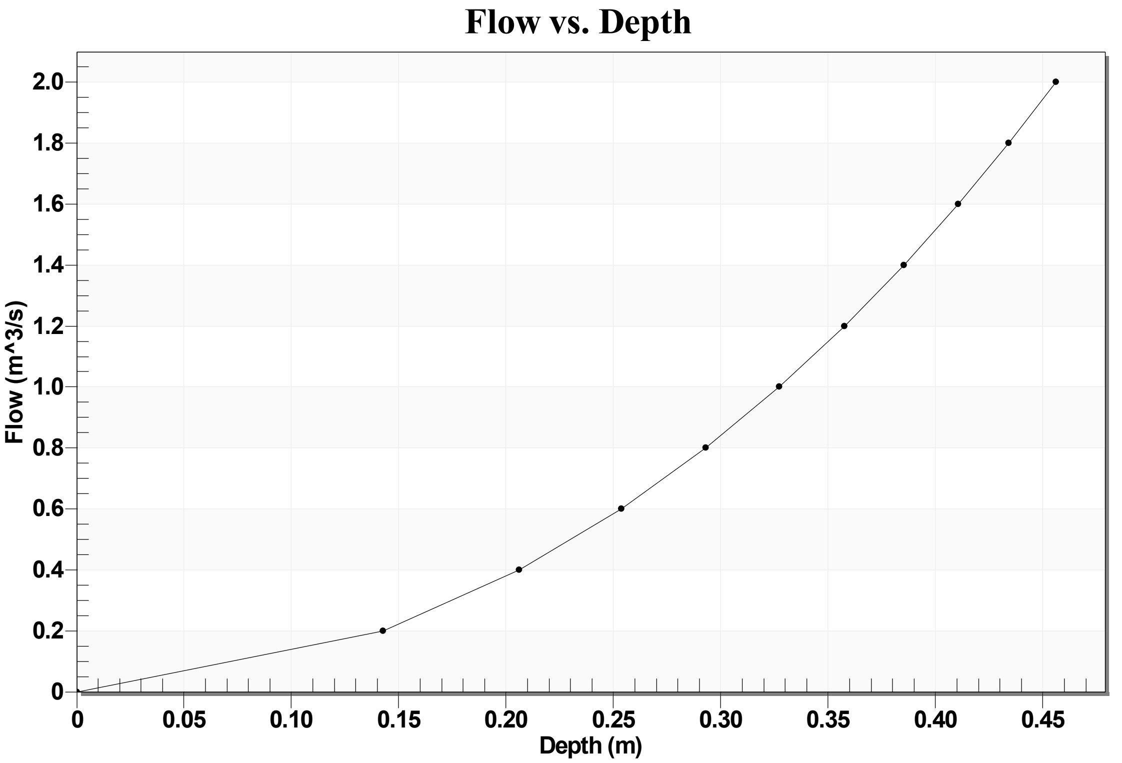
Hydraulic Toolbox Results: Swale Hydraulic Capacity
Peak 1:100 Flow Rate

Parameter	0.9% Swale	1.7% Swale
Channel Type	Trapezoidal	Trapezoidal
Side Slope 1 (Z1)	3.0	3.0
Side Slope 2 (Z2)	3.0	3.0
Channel Width (m)	1.0	1.0
Longitudinal Slope (m/m)	0.009	0.017
Manning's n	0.030	0.030
Flow (cms)	1.280	19.000
Depth (m)	0.4294	1.2314
Area of Flow (m^2)	0.9824	5.7801
Wetted Perimeter (m)	3.7156	8.7878
Hydraulic Radius (m)	0.2644	0.6577
Average Velocity (m/s)	1.3029	3.2872
Top Width (m)	3.5762	8.3881
Froude Number	0.7935	1.2640
Critical Depth (m)	0.3818	1.3665
Critical Velocity (m/s)	1.5629	2.7266
Critical Slope (m/m)	0.0147	0.0103
Critical Top Width (m)	3.29	9.20
Calculated Max Shear Stress (N/m^2)	37.8789	205.1901
Calculated Avg Shear Stress (N/m^2)	23.3264	109.6041

0.9% Swale:



1.2% Swale



APPENDIX F STREAM DIVERSION & SECTION 50 APPLICATION

Knockharley Landfill Limited

Knockharley S50

Culvert Hydrologic and Hydraulic Assessment

May 2025

Confidential

Knockharley Landfill Limited

Knockharley S50

Culvert Hydrologic and Hydraulic Assessment

Type of document (version) Confidential

Project no. IE0037027.9697

Date: May 2025

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Quality control

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1 Overview

Knockharley Landfill Limited (KLL) operates a residual waste management facility, known as Knockharley Landfill (“the Site”) at Knockharley, Kentstown, Navan, Co. Meath (includes townlands of Tuiterath and Flemingstown). The Site is located adjacent to the N2 National Primary Route approximately 7 km south of Slane, Co. Meath.

Currently there is only a single access point to the site via an existing culvert over the Knockharley stream. A new culvert is required for this stream to facilitate emergency access to the Site. The culvert will be located in the Knockharley Stream which is a tributary stream of the River Nanny and flows from west to east through the northern portion of the site.

This Section 50 assessment therefore relates to construction of a Knockharley stream crossing and new flood culvert.

WSP Ireland Consulting Limited (‘WSP’) has been commissioned by KLL to undertake a hydrological and hydraulic assessment to support the submission of a Section 50 (S50) application for the proposed culvert. This report has been prepared to summarise the methodologies employed and outcomes from the completed hydrologic and hydraulic assessments. The completed Section 50 form is found in Appendix A.

1.1 Proposed Culvert

As part of the expansion of the Knockharley Landfill, a new culvert is required on the Knockharley Stream to facilitate a new access road crossing over the stream.

Drawings of the proposed works are included in Appendix B and photos of the existing channel are included in Appendix C of this application.

The proposed culvert will be a concrete box culvert which will be installed over an 8 m section of the Knockharley Stream as detailed in Table 1-1 and as indicated on the drawings provided in Appendix B.

Following correspondence and site meetings with Inland Fisheries, design requirements for the culvert construction were agreed on and incorporated. Specifically, that the culvert will be buried to a minimum depth of 0.5 m below the stream bed and that the gradient in the culvert should not exceed 3%. The material used to embed the culvert is assumed to be similar to that of the existing stream bed.

Table 1-1 - Proposed culvert details

Form	Size (m) (H:W)	Length (m)	US Invert Level (m)	DS Invert Level (m)	Slope
Box Culvert	2 m x 2.5 m (0.5 m embedded below ground level)	8	59.52	59.46	0.008

Construction of the proposed culvert will be subject to the approval of the Office of Public Works (OPW), under Section 50 of the Arterial Drainage Act, 1945. The purpose of this report is to verify that the proposed culvert meets the assessment criteria and design standards of the OPW.

2 Hydrologic Assessment

2.1 Design Criteria

The design of watercourse crossings will be subject to the approval of the OPW, under Section 50 of the Arterial Drainage Act, 1945. The following standards are required by the OPW:

- A bridge or culvert must be capable of passing a fluvial flood flow with a 1% annual exceedance probability (AEP) or 1 in 100 year flow without significantly changing the hydraulic characteristics of the watercourse.
- If the land potentially affected does not include dwellings and infrastructure, a culvert must be capable of operating under the above design conditions while causing a hydraulic loss of no more than 300 mm (excluding the culvert gradient).
- If the land potentially affected includes dwellings and infrastructure, it must be demonstrated that those dwellings and/or infrastructure are not adversely affected by constructing the bridge or culvert.
- Minimum culvert diameter, height and width is 900 mm to facilitate maintenance access and reduce the likelihood of debris blockage.
- Increase flood flow by 20% as an allowance for Climate Change
- Increase flows by a factor of 1.6, where there are existing drainage schemes upstream

2.2 Catchment Location and Characteristics

The stream entering the site from the western boundary at Knockharley is a 1st order tributary of the River Nanny (Fehily Timoney & Company, 2018). It flows from the west in an easterly direction.

The estimated catchment area for the proposed culvert is 1.908 km^2, which has been estimated from the OPW Flood Studies Update (FSU) data base. The catchment boundary is shown on Figure 1. The catchment is a natural undeveloped catchment comprising primarily agricultural land, with the footprint for rural roads and buildings making up a small proportion of the catchment. This catchment delineation was validated through interrogation of available topographic data.

Figure 1 - Catchment boundary for the proposed culvert (OPW FSU, 2024)

2.3 Design Flows

As previously noted, the required design flow rate is the peak run-off from a 1:100-year return period storm event, plus a 20% increase to allow for climate change.

The 1 in 100-year return period flood at the proposed culvert location was calculated using the following methods:

- FSU 3-variable method (Das and Cunnane, 2009): 3-variable equation for small catchments of area less than 25 km^2;
- the FSU 4.2A Regression (FSU WP4.2, 2012): equation based on catchment descriptors that has been developed specifically for use in smaller catchments; and
- the Rational Method and the Modified Rational Method (HR Wallingford, 1981).

The catchment descriptors and rainfall characteristics were obtained from the OPW Rainfall and Flood Estimation database that is accessed by contacting the FSU Helpdesk. Table 2-1 lists the physical catchment descriptors that were adopted for the FSU3 and FSU4.2 flood estimation methods. Table 2-2 lists the hydrological parameters adopted for the Rational Method.

Table 2-1 - Physical Catchment Descriptors For FSU-3 Variable & FSU 4.2 Regression Methods

Parameter	Value
Location Number	08_226_4
BFISOIL	0.539

SAAR - Standard Annual Average Rainfall	841.72mm
FARL - Flood Attenuation by Rivers and Lakes	1
S1085	12.444 m/km
ARTDRAIN2	0
URBEXT	0
SOIL	0.3
Centroid distance	9.152 km
Coordinates (UTM, Zone 29)	663084 E, 5947462 N

Table 2-2 - Hydrologic Parameters for the Rational Method

Parameter	Value
Catchment/Watercourse Length	3.253 km[1]
Cmax - Runoff Coefficient	0.260
Time of Concentration	1.124 hr[2]
ARF - Areal Reduction Factor @ Tc	0.953
Rainfall Depth - 2 year @Tc	11.22 mm
Rainfall Depth - 100 year @Tc	34.66 mm
Rainfall Depth ARF - 2 year @Tc	11.65 mm
Rainfall Depth ARF - 100 year @Tc	33.03 mm
i - Rainfall Intensity - 2 year	10.361 mm
i - Rainfall Intensity - 100 year	29.391 mm

1. Measured from Google Earth Pro
2. Calculated using the Bransby/Williams formula.

The mean annual flood for the catchment (Qbar) was calculated using each of the methods. The Qbar was then multiplied by a design factor (factor of standard error) of 2.06 for the FSU-3 equation and 1.69 for the FSU4.2 equation.

In line with 'The Flood Studies Report Ungauged Catchment Method Underestimates For Catchments Around Dublin' (Michael Bruen et al, 2005), to convert Qbar to the 1% AEP flow a growth factor of 2.61 was applied to the FSU calculation methods. There are no known Drainage Schemes upstream of the site and as such the estimated design flows have not been factored further, beyond the factors described above.

Results of the hydrological calculations, including a 20% increase for climate change as recommended in current OPW guidelines, are summarised Table 2-3. The hydrological calculations corresponding to these design flow estimates are shown in Appendix D of this application.

Table 2-3 - Design Flow Estimates

Method	QBAR (m^3/s)	Factor for Standard Error	Growth Factor	Q100 (m^3/s)	Q100 + 20%CC (m^3/s)
FSU 3-Variable	0.589	2.06	2.61	3.17	3.80
FSU 4.2 Regression	0.675	1.69	2.61	2.98	3.57
Rational Method	1.71	-	-	4.05	4.86

The hydraulic assessment shows that the design flow estimate for the FSU 3-variable equation is 3.17 m^3/s for the 1 in 100 year event (Q100) and 3.80 m^3/s for the Q100 including 20% increase for climate change.

The design flow estimate for the FSU 4.2 Regression method is 2.98 m^3/s for the Q100 event and 3.57 m^3/s for the Q100 including 20% increase for climate change.

The Q100 design flow calculated using the Rational method was c. 50% greater than the value given by the FSU methods. Given the catchment is predominantly rural / undeveloped, the estimates from the rational method have not been considered further in this assessment as the Rational Method is typically most appropriate for the determination of peak design discharge for urban catchments; consequently, in this scenario the resulting design flows are considered to be an overestimate.

The FEH Statistical and IoH124 methods have also been employed as additional check methods and the design flows estimated from these methods are lower than those estimated from the above FSU 3-Variable and FSU 4.2 Regression methods. Details of these calculations have also been included in Appendix D of this application.

Consequently, the design flow estimate corresponding to the FSU 3-variable method (3.80 m^3/s) has been taken forward for the purposes of the culvert design and analysis (i.e. most conservative of FSU method estimates).

3 Hydraulic Assessment

WSP has completed a hydraulic assessment to assess the hydraulic performance of the proposed culvert against S50 requirements, and the design criteria described above (Section 2.1). In addition to these design criteria, the following design considerations were reviewed as part of the hydraulic assessment:

- All losses associated with the bridge or culvert (e.g. entrance, exit, friction and pier losses).
- Any ancillary works that may affect the hydraulic performance of the bridge or culvert (e.g. erosion control works and debris screens).
- The hydraulic implications of any environmental measures incorporated into the bridge or culvert design (e.g. depression of the invert or the installation of baffles).

The stream is not tidally affected. Following, email and phone correspondence with Inland Fisheries, requirements for the culvert construction were identified. Specifically, that the culvert will be buried to a minimum depth of 0.5 m below the stream bed and that the gradient in the culvert should not exceed 3%.

3.1 Hydraulic Model & Calculation Inputs

3.1.1. Proposed Culvert Input Data (HY-8 Model)

The hydraulic analysis for the proposed culvert was undertaken using the culvert modelling software HY-8.

The tailwater data presented in Table 3-1 is found based on the properties and characteristics of the stream.

Table 3-1 - HY-8 Tailwater Input Data.

Input	Value	Source
Channel Type	Trapezoidal	Based on 2025 survey results for proposed stream and channel design
Bottom width	2.5 m	Based on 2025 survey results for proposed stream and channel design
Side Slope (H:V)	1.5 : 1	Based on 2025 survey results for proposed stream and channel design
Channel Slope (m/m)	0.008	Based 2025 average slope of proposed stream

Manning’s n (channel)	0.30	Assuming established grass
Channel invert elevation	59.460 m OD	Assumed same as culvert outlet invert elevation excluding embedment

The culvert data used in the model are shown in Table 3-2. This is based on a concrete box culvert with 500 mm embedment of material similar to that of the stream bed.

Table 3-2 - HY-8 Culvert Data Inputs.

Input	Value	Source
Flow	3.80 m^3/s	Chosen design flow
Size	2000 mm rise 2500 mm width 500 mm embedment	Proposed culvert size
Manning’s n top/sides	0.012	Concrete manning’s n
Manning’s n bottom	0.030	Assumed that the culvert base will reuse material from the existing stream where possible or will mimic the existing stream bed
Inlet loss (Ke)	0.4	Flared Wingwall (30-70º) Headwall (HY-8)
Inlet Depression	None	-

The site data for the culvert is shown in Table 3-3.

Table 3-3 - HY-8 Site Data Inputs.

Input	Value	Source
Inlet elevation	59.020 m OD	2025 survey and accounting for 500 mm embedment
Outlet elevation	58.956 m OD	2025 survey and accounting for 500 mm embedment
Culvert Slope (m/m)	0.008	2025 survey
Culvert Length	8 m	2025 survey

3.1.2. Existing Channel Input Data

The afflux, representing the head loss and upstream water level rise resulting from the culvert installation, was calculated to evaluate the culvert's impact on stream hydraulics. The Design Criteria stipulates that the afflux should be limited to a maximum of 300mm.

Determination of the normal depth in the existing channel was carried out using the Hydraulic Toolbox software, with input data provided in Table 3-4.

Table 3-4 - Hydraulic Toolbox Inputs.

Input	Value	Source
Type	Trapezoidal	2025 Survey K-K Cross Section
Side Slopes (H:V)	1:1.9	2025 Survey K-K Cross Section
Channel Width	1.4 m	2025 Survey K-K Cross Section
Longitudinal Slope (mm)	0.008	2025 Survey
Manning's Roughness	0.030	2025 Survey K-K Cross Section
Flow	3.80 m^3/s	Design Flow

4 Results

4.1 Culvert Capacity

Results from the hydraulic HY-8 model are shown in Table 4-1. The performance curve of the culvert is shown in Figure 4-1. This shows that the culvert can take up to 9 m^3/s before the headwater elevation reaches the soffit of the culvert.

The hydraulic assessment shows that the culvert is capable of passing the 1%AEP fluvial flow + 20% Climate Change without significantly changing the hydraulic characteristics of the watercourse.

Table 4-1 - HY-8 Results.

Flow (m^3/s)	Headwater Elevation (mOD)	Inlet Control Depth (m)	Outlet Control Depth (m)	Tailwater Depth (m)	Outlet velocity (m/s)	Tailwater velocity (m/s)
3.80	60.56	0.95	1.04	0.63	2.41	1.76

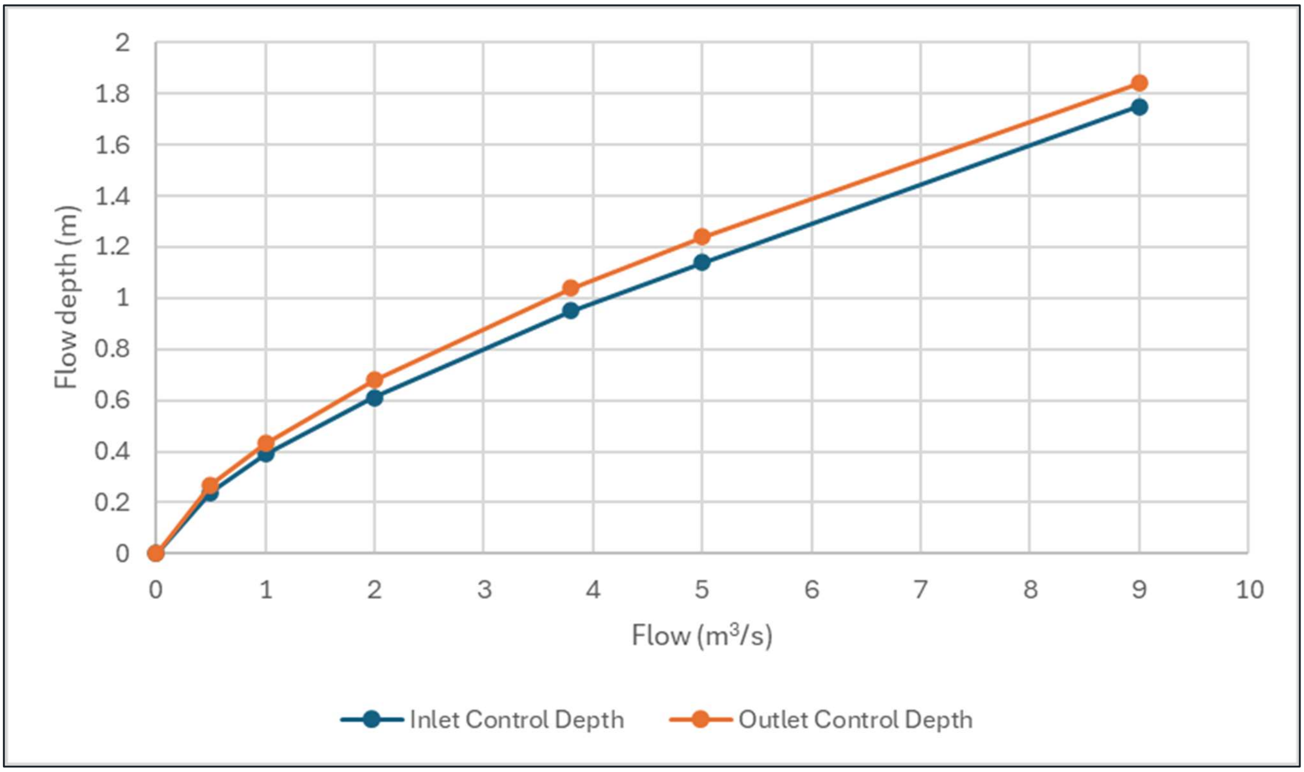

Figure 4-1 - Culvert Performance Curve.

4.2 Afflux

The normal depth of the existing channel upstream of the culvert was calculated to be 0.765 m and is shown in Figure 4-2.

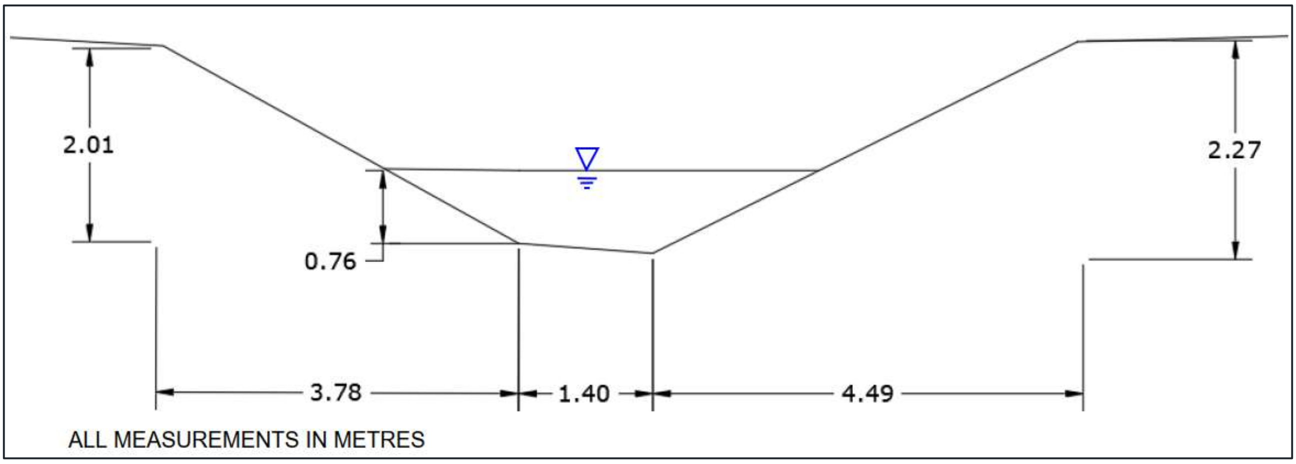

Figure 4-2 - Normal Depth in Existing Channel.

The afflux (defined as the increase in the water level upstream of a structure due to the obstruction of the flow path or otherwise described as the hydraulic loss) was calculated by comparing the normal depth in the existing channel (prior to any modifications) and the maximum water level directly upstream of the culvert. The afflux has been calculated to be 0.275 m as shown in Table 4-2,Table 4-3 which is within the requirement of no more than 300 mm.

Table 4-2 - Afflux Calculation

Culvert Head Water (m)	Pre-development Channel Normal Depth (m)	Afflux (m)
1.04	0.765	1.04 – 0.765 = 0.275

4.3 Freeboard

The available freeboard at the culvert during the 1% AEP event, inclusive of climate change allowance, has been calculated as 0.96 m, refer to Table 4-3. This means that the culvert has sufficient capacity to convey the design flow with allowance to pass moderate floating debris, thereby reducing the potential for blockage and associated backwater effects.

Several roads and residential properties exist within the upstream catchment; however, the nearest building is located approximately 160 m north of the culvert. Given that the design flow remains confined within the existing channel, no adverse impacts on nearby infrastructure are anticipated.

Table 4-3 - Freeboard Calculation.

Culvert Soffit Elevation (mOD)	Headwater Elevation (mOD)	Freeboard (m)
61.52	60.56	61.52 – 60.56 = 0.96

4.4 Required height and width

The culvert is designed as a 2500 mm x 2000 mm box culvert which is greater than the minimum requirement of 900 mm culvert diameter, height and width.

4.5 Other design considerations

To prevent scour, riprap with a D_{50} of 250 mm is recommended at the culvert outlet, as indicated on the drawings in Appendix B. Scour protection measures will be finalised during detailed design.

5 Conclusion

The hydraulic assessments have shown that the proposed culvert has adequate capacity to pass the design flood flow with sufficient freeboard and without causing more than 300 mm afflux. It is not anticipated that the flood risk in the catchment area for the 1% AEP + climate change will increase as a result of the presence of the proposed new culvert.

There are some roads and residential buildings present in the upstream catchment of the culvert. These dwellings will not be adversely affected by the proposed culvert.

References

Das, C., & Cunnane, C., (2009). Flood Studies Update Programme, WP2.2 Flood Frequency Analysis, Final Report (FSU WP2.2), NUI Galway and OPW.

Fehily Timoney & Company, 2018, Environmental Impact Assessment Report (EIAR) For Proposed Development at Knockharley Landfill

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Michael Bruen et al, 2005, 'The Flood Studies Report Ungauged Catchment Method Underestimates For Catchments Around Dublin'

Office of Public Works, 2024, Flood Studies Update (FSU) Helpdesk, Accessed 23/04/25, Available at: https://opw.hydronet.com/default.aspx?page=6&appid=169&lang=2

Appendix A

Completed Section 50 Form

qAF50 Rev1113

Construction, Replacement or Alteration of Bridges and Culverts

Application for Consent under Section 50 of the Arterial Drainage Act, 1945 & EU (Assessment and Management of Flood Risks) Regulations SI 122 of 2010

Project Name	Knockharley Landfill Culvert Crossing	Structure Ref No.	N/A

Applicant (Correspondence will issue to agent)		
Company or Organisation Name:	Knockharley Landfill Limited	
Postal Address:	Knockharley, Brownstown, Navan, Co. Meath, C15 FX09	
Contact Person:	Sean Smith	
Phone:	00353 419821650	Fax:
E-mail:	sean.smith@panda.ie	

Agent (Correspondence will issue to agent)		
Company or Organisation Name:	WSP Ireland Consulting LTD	
Postal Address:	Town Centre House, Dublin Rd, Naas, Co. Kildare, W91 TD0P	
Contact Person:	Darren Crowe	
Phone:	+ 353 87 233 3059	Fax:
E-mail:	darren.crowe@wsp.com	

Location and Parameters of crossing			
Watercourse: **Knockharley Stream**		Catchment:	Nanny-Delvin, IE_EA_08_352
Address (Townland – County):		Kentstown, Co. Meath	
Grid Reference	X:	297260 (53.64844177192011)	Y: 267268 (6.531077535033356)
Hydrometric Station(s) utilized (including reference number):		Hydrometric Area 08	
Area of Contributing Catchment:		1.908 Km^2	Road Reference:
Design Flood Flow: 3.80 m^3/s		Annual Exceedance Probability (AEP):	1 %

Statement of Authenticity	
I hereby certify that the information contained in this application form, along with all appended supporting information, has been checked by me and that all statements are true and accurate.	
Name:	Darren Crowe
Company/Organisation:	WSP Ireland Consulting Ltd
Signature:	
Date:	

Application Check List	☐
COMPLETED APPLICATION FORM	☒
SUPPORTING HYDROLOGICAL AND HYDRAULIC INFORMATION	☒
PHOTOGRAPHS COVERING SITE OF ALL PROPOSED WORKS	☒
SCALED PLAN OF BRIDGE/CULVERT/APPROACH EARTHWORKS	☒
SCALED CROSS SECTION OF BRIDGE/CULVERT/APPROACH EARTHWORKS	☒
SCALED LONG SECTION OF CHANNEL THROUGH BRIDGE/CULVERT	☒
DETAILS OF RELEVANT EXISTING STRUCTURES	☒
COMPLETED STATEMENT OF AUTHENTICITY	☒
PLAN OF CATCHMENT AREA	☒
COPY OF NOTICE OF GRANT OF PLANNING PERMISSION WITH CONDITIONS [*1]	☒

For OPW use only	*Date of Receipt*							
OPW Drainage Maintenance Region	East	☐	South East	☐	South West	☐	West	☐
Correspondence Number		OPW Register No:						
		Consent Issued		☐				

If the application form is not completed correctly, and in its entirety, the application may be deemed invalid and returned for correction.

ADDITIONAL INFORMATION

Hydrological Analysis				
Methodology Applied			Factors Applied	
Method Used	Tick box if used or state	Flow *2 (m³/sec)	Type of Factor	Value Used
			Climate Change	20%
6 – Variable Catchment characteristics	☐		Irish Growth Curve	2.61
			Factor for Standard Error	2.06 (FSU3) 1.69 (FSU4.2)
3 – Variable Catchment Characteristics	☒	0.589	Drained Channel	
			Other	
IH 124	☐			
Gauged Flow	☐			
Unit Hydrograph	☐		Tidal ☐	
Other – FSU4.2	☒	0.675	Comments;	
Other – Rational Method	☒	1.49		
FSR ☐	FSU ☒	Other ☒		
Comments				

Hydraulic/Structure Details	
Description of Structure*3	**Knockharley Stream Culvert**
The proposed culvert shall be installed at approx. Ch 5063. The proposed culvert is approx. 8 m in length. The culvert will consist of a concrete box culvert 2.5m wide x 2 m high with a slope of the culvert at 0.008. The proposed culvert and stream crossing has been designed for 1:100 year storm events plus 20% to cater for the effects of climate change.	
Effective Conveyance Area *4	2.4 m²
Upstream Invert Level; **59.52** mOD Upstream Soffit Level; **61.52** mOD	Downstream Invert Level; **59.46** mOD Downstream Soffit Level; **61.42** mOD
Upstream Design Flood Level; **60.56** mOD	Downstream Design Flood Level; **60.16** mOD

NOTES :

1. In line with OPW policy, section 50 approvals should be sought for bridges and culverts that are necessary for access or deemed acceptable by the planning authority. A copy of the notice of grant of planning permission with all conditions should be enclosed with all applications, that are not exempt development under the Planning and Development Act, 2000, as evidence that these factors have been considered.

2. Flow is the estimated flow from the catchment, without any factors applied.

3. The following details are to be included: the channel bed level, invert and soffit levels of the structure along with the width, length and total conveyance area. Any environmental considerations such as bed depression, baffles, mammal walkways etc. should be described.

4. Effective conveyance area is from channel bed level to design flood level.

5. All levels must be given to Ordnance Datum, Malin Head.

If the application form is not completed correctly, and in its entirety, the application may be deemed invalid and returned for correction.

Appendix B

Drawings

NOTE(S)

1. DO NOT SCALE FROM THIS DRAWING
2. ALL LEVELS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
3. ALL DIMENSIONS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
4. ALL LEVELS AND COORDINATES SHOWN ARE TO IRISH NATIONAL GRID

LEGEND

- PLANNING BOUNDARY
- OWNERSHIP BOUNDARY
- AREA WITHIN PLANNING BOUNDARY, NOT WITHIN SITE
- EXISTING OVERHEAD POWERLINES
- 20 KV OVERHEAD POWERLINES
- GAS MAIN
- EXISTING KNOCKHARLEY STREAM
- RESTORED LANDFILL
- EXISTING PLANNING BERM
- PROPOSED KNOCKHARLEY STREAM DIVERSION

0 160 320 480m
1:4,000

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2025-05-06	S50 OVERALL SITE PLAN	DC	POB	AL	AL

CLIENT
KNOCKHARLEY LANDFILL LTD.

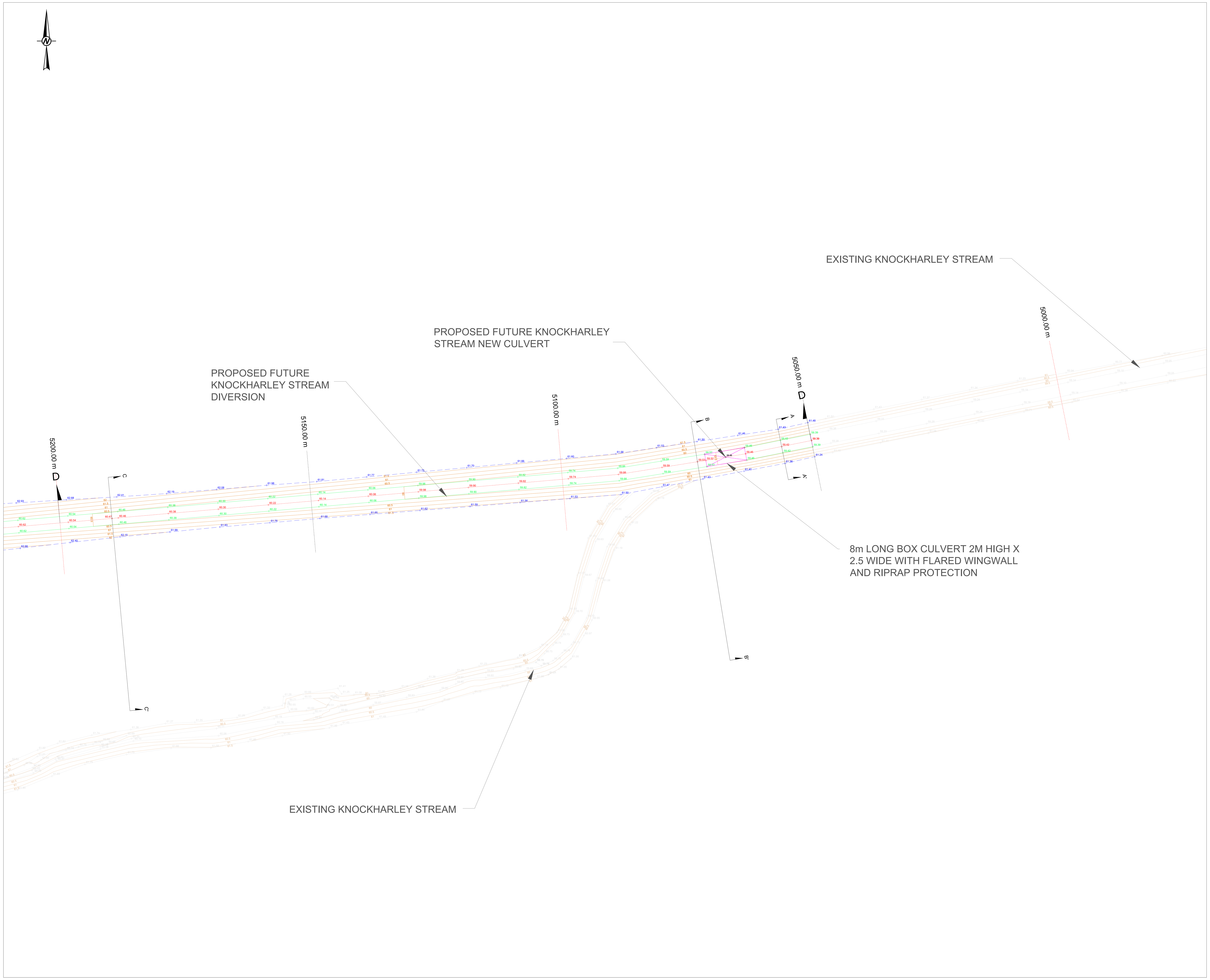
CONSULTANT

TOWN CENTRE HOUSE
DUBLIN ROAD
NAAS
REPUBLICOF IRELAND
[+353] 45 810200
www.wsp.com

PROJECT
KNOCKHARLEY LANDFILL DEVELOPMENT

TITLE
S50 - NEW CULVERT AND STREAM DIVERSION - SITE PLAN

PROJECT NO.	SCALE	REV.	of	DRAWING
297705	1/4000 @ A1	A		100



NOTE(S)

1. DO NOT SCALE FROM THIS DRAWING
2. ALL LEVELS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
3. ALL DIMENSIONS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
4. ALL LEVELS AND COORDINATES SHOWN ARE TO IRISH NATIONAL GRID

LEGEND

PROPOSED NEW CULVERT

NEW KNOCKHALREY STREAM TOE

NEW KNOCKHARLEY STREAM CREST

NEW KNOCKHARLEY STREAM CENTRE LINE

64 CONTOUR LINES

SECTION

0 16 36 48m

1:400

REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2025-05-06	S50 OVERALL CULVERT PLAN	DC	POB	AL	AL

CLIENT

KNOCKHARLEY LANDFILL LTD.

CONSULTANT

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REPUBLICOF IRELAND
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www.wsp.com

PROJECT

KNOCKHARLEY LANDFILL DEVELOPMENT

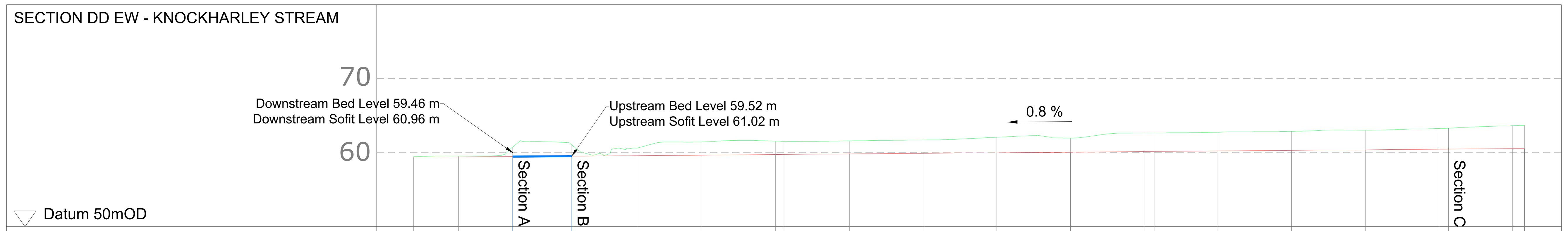
TITLE

S50 - NEW CULVERT PLAN

PROJECT NO.	SCALE	REV.	of	DRAWING
297705	1/400 @ A1	A		101

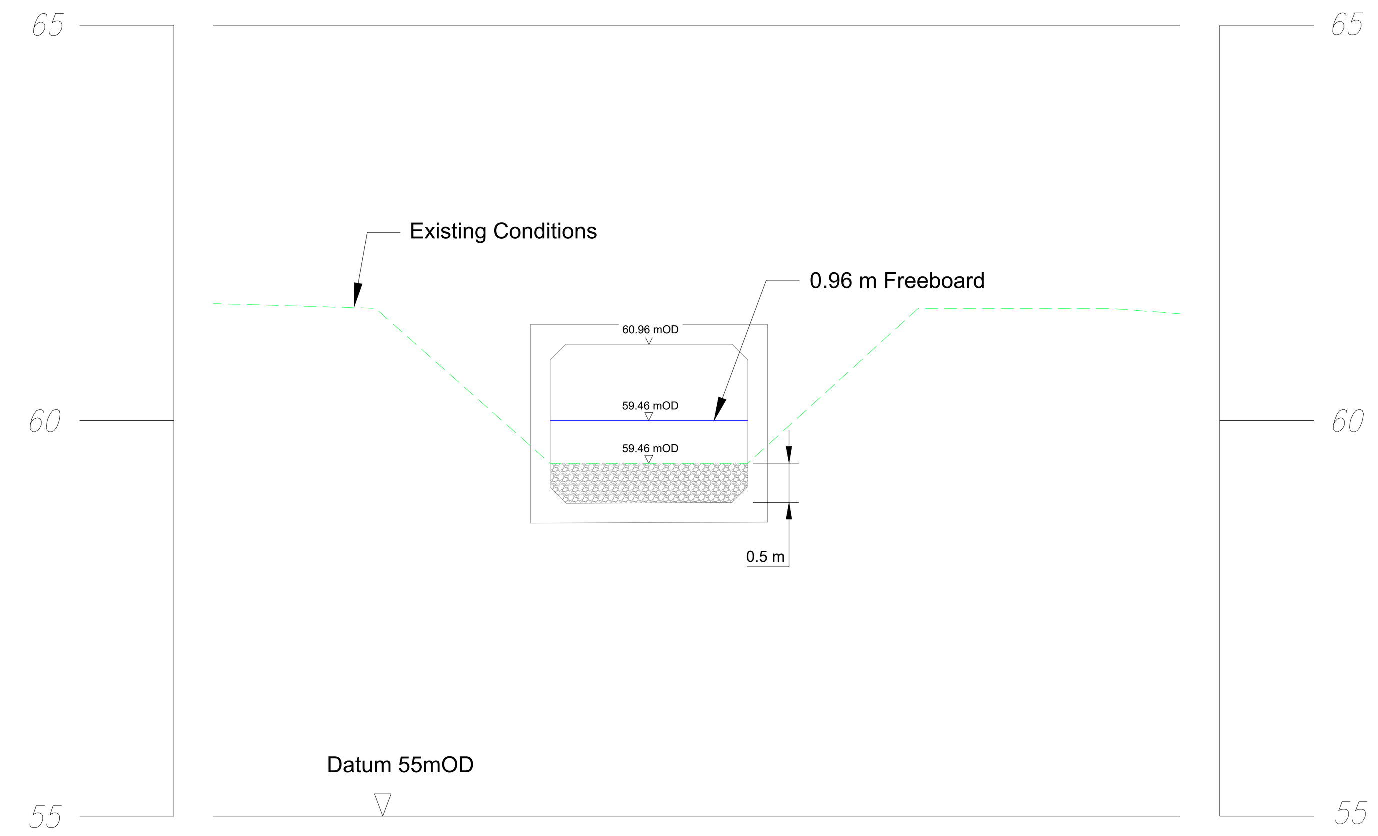
Proposed Design
00 - OGL - 11 Oct 2024

SECTION DD EW - KNOCKHARLEY STREAM

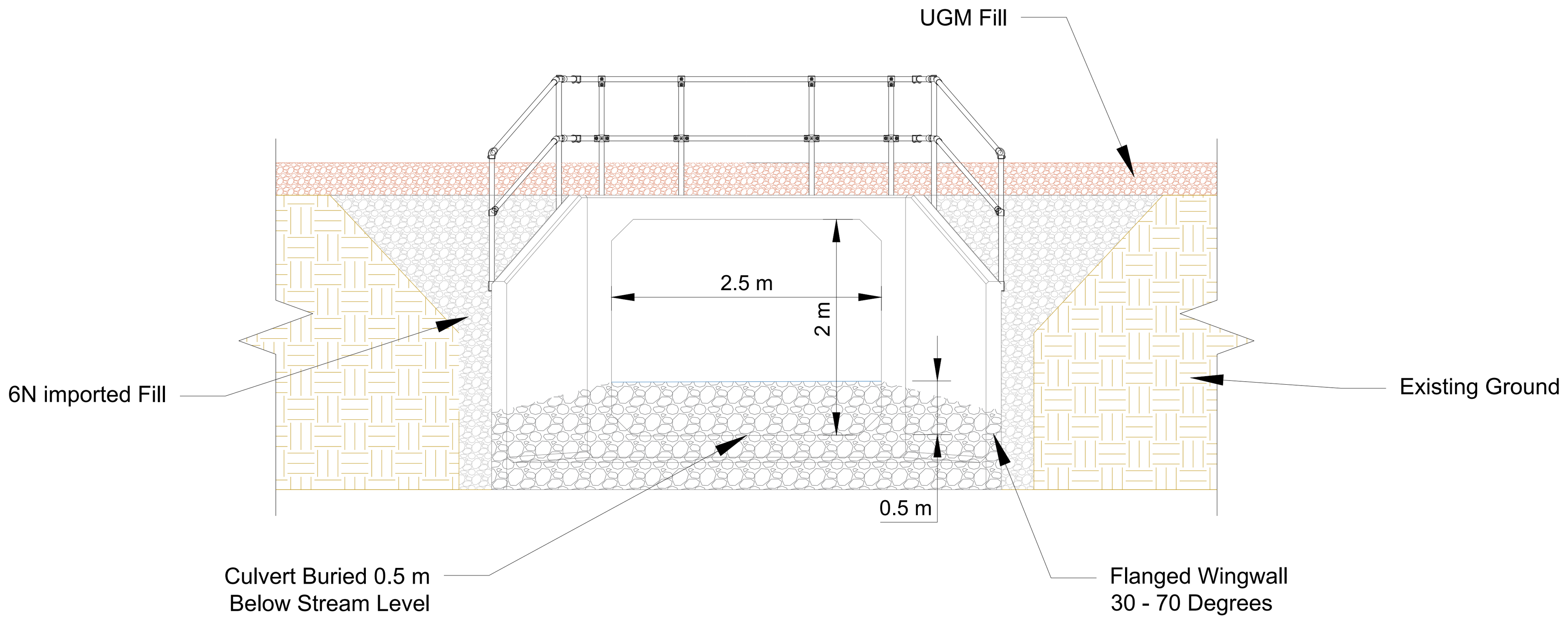


Level : Proposed Design	59.39	59.42	59.46	59.52	59.59	59.66	59.74	59.75	59.82	59.90	59.98	60.06	60.14	60.15	60.22	60.30	60.38	60.46	60.47	60.54	60.55
Level : Original Ground Level Oct 2024	59.45	59.52	60.75	61.05	60.60	61.43	61.55	61.52	61.58	61.71	62.07	61.95	62.64	62.66	62.75	62.87	63.02	63.26	63.30	63.63	63.67
Chainage	5050	5056	5063	5071	5080	5089	5099	5100	5109	5119	5129	5139	5149	5150	5159	5169	5179	5188	5190	5198	5200

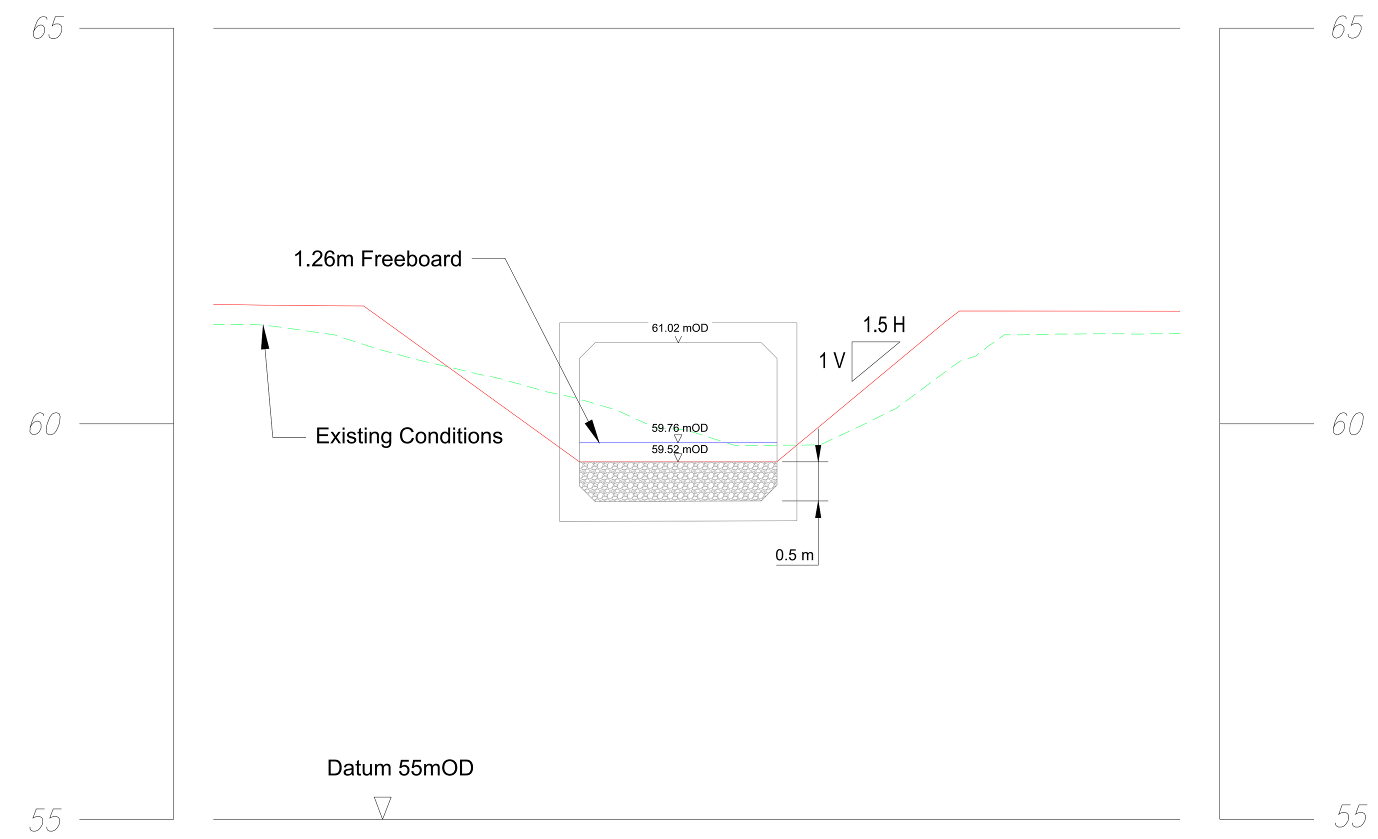
Section D:D' - East West Scale: 1/500 A1



Section A:A' - North South Scale: 1/80 A1
Culvert Downstream



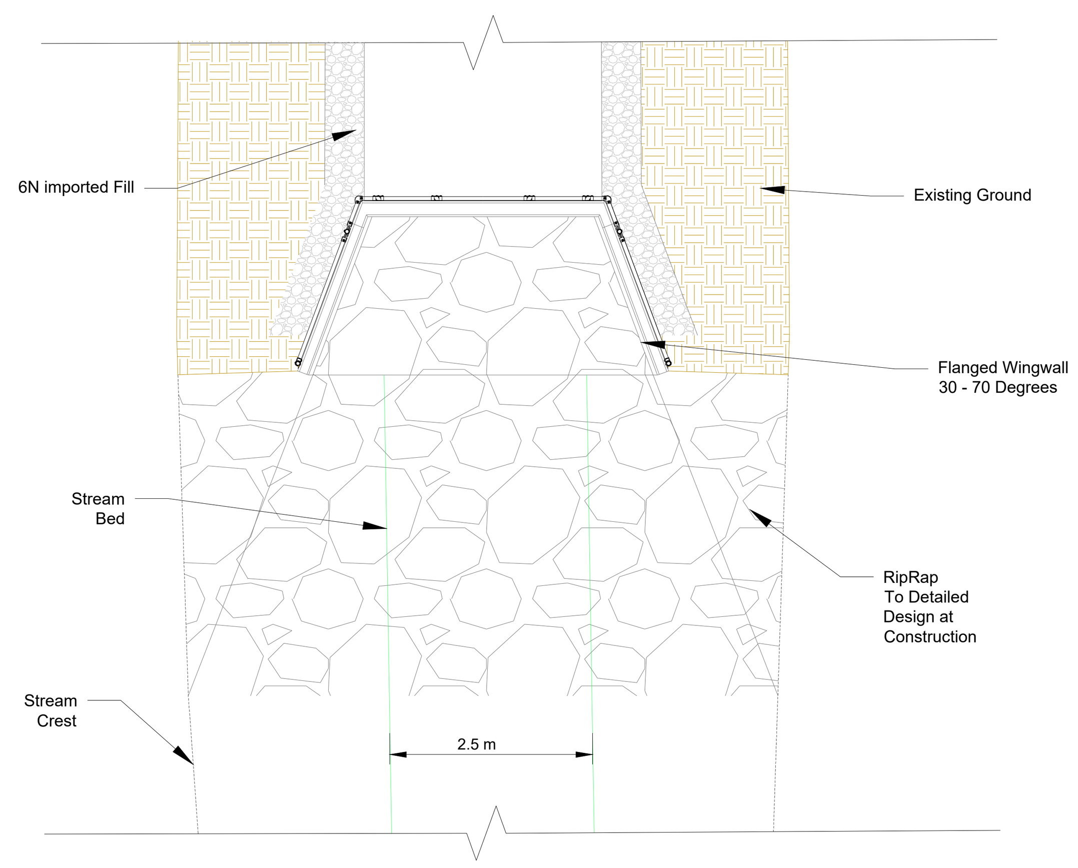
Typical Section Scale: 1/50 A1



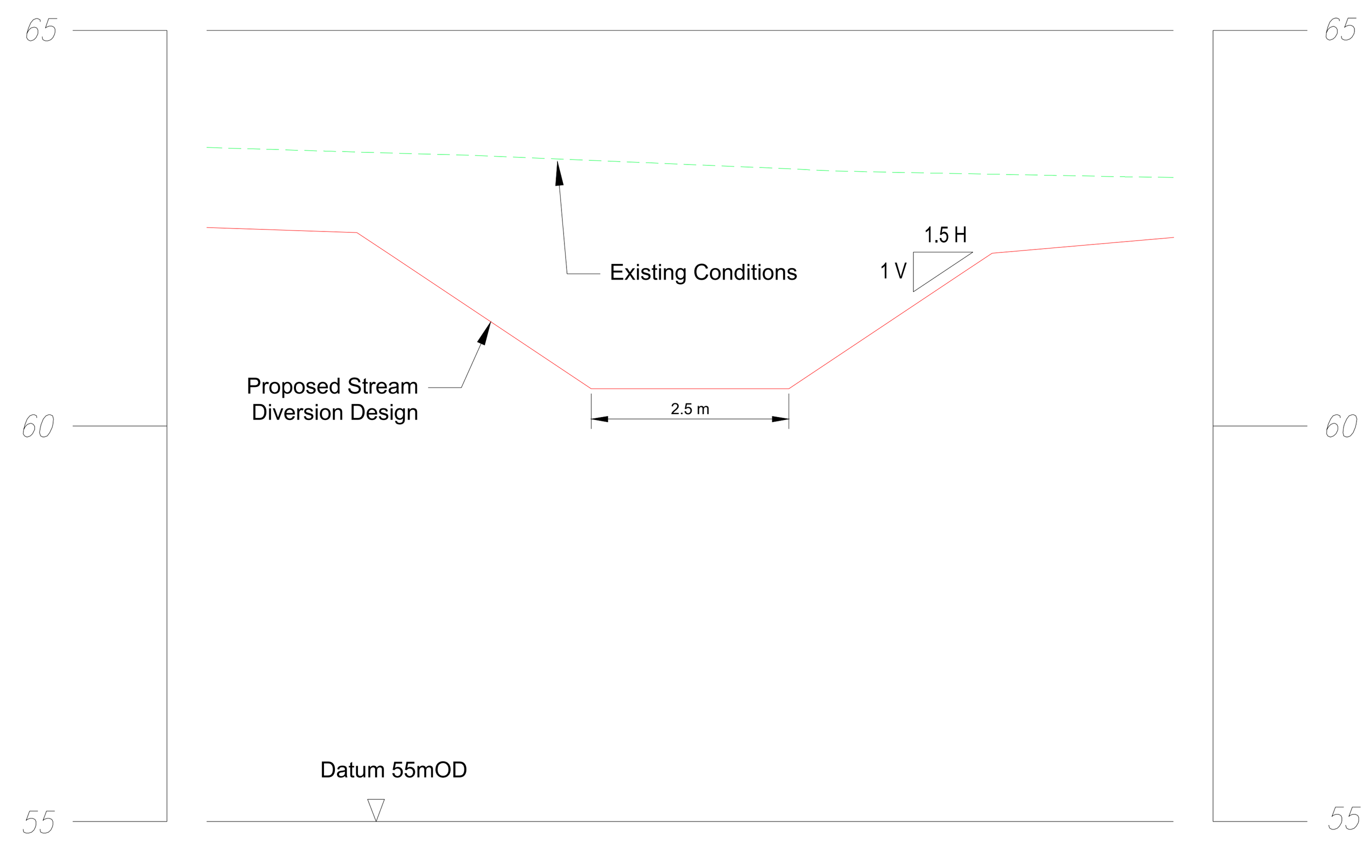
Section B:B' - North South Scale: 1/80 A1
Culvert Upstream

NOTE(S)

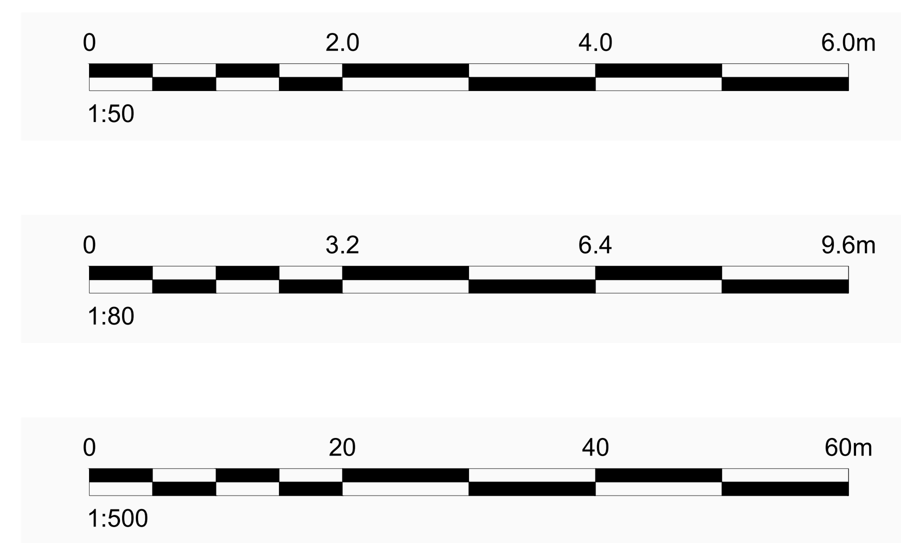
1. DO NOT SCALE FROM THIS DRAWING
2. ALL LEVELS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
3. ALL DIMENSIONS SHOWN ARE IN METERS (m) UNLESS STATED OTHERWISE
4. ALL LEVELS AND COORDINATES SHOWN ARE TO IRISH NATIONAL GRID MALIN HEAD



Typical Plan Scale: 1/80 A1



Section C:C' - North South Scale: 1/80 A1
Stream Diversion



REV.	YYYY-MM-DD	DESCRIPTION	DESIGNED	PREPARED	REVIEWED	APPROVED
A	2025-05-09	S50 OVERALL CULVERT PLAN	DC	POB	AL	AL

CLIENT
KNOCKHARLEY LANDFILL LTD.

CONSULTANT
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[+353] 45 810200
www.wsp.com

PROJECT
KNOCKHARLEY LANDFILL DEVELOPMENT

TITLE
S50 - NEW CULVERT SECTION & DETAILS

PROJECT NO.	SCALE	REV.	of	DRAWING
297705	Various	A		102

Appendix C

Pictures

Photo View 1: Stream channel directly downstream of proposed culvert location - April 2025.

Photo View 2: Stream channel directly upstream of proposed culvert location - April 2025.

Appendix D

Hydrologic Calculations

Project	Knockharley 2025 Culvert		
Subject	OPW Section 50_Knockharley 2025 Calculation of Flow Estimation		
Prepared by:	Beth O'Connor	Job No	1
Checked by:	Neil Verwood	Date	05/08/2025
Approved by:	Eoghan Hayes	Revision	1

Key:
- Input Data
- *Calculated*
- **Output**

1.0 PHYSICAL CATCHMENT DESCRIPTORS (PCD'S):

1.1 Hydrological PCD's

S1085 - Mainstream Slope	*12.444*	*m/km*

1.2 Spatial PCD's

AREA - Catchment Area	1.908	km^2	
SAAR - Standard Annual Average Rainfall	842	mm	*
FARL - Flood Attenuation by Rivers and Lakes	1		*

1.3 Spatial PCD's Representing Soil, Subsoil & Aquifer Types

BFI_{SOIL}	0.539		*
URBEXT	0		*
SOIL	0.3		
DRAIND	*N/A*	*km/km²*	
ARTDRAIN2	0		*

1.4 Catchment Rainfall Characteristics

Catchment/Watercourse Length	*3.253*	*km*
Area Equivalent Diameter	*1.559*	*km*
T_c - Time of Concentration	**1.124**	**hr**
C_{av} - Runoff Coefficient (Average)	*0.184*	
C_{max} - Runoff Coefficient (Maximum)	*0.260*	
ARF - Areal Reduction Factor @ T_c	0.953	
Rainfall Depth - 2 year @T_c	12.22	mm
Rainfall Depth - 100 year @T_c	34.66	mm
Rainfall Depth ARF - 2 year @T_c	*11.65*	*mm*
Rainfall Depth ARF - 100 year @T_c	*33.03*	*mm*
i - Rainfall Intensity - 2 year	*10.361*	*mm*
i - Rainfall Intensity - 100 year	*29.391*	*mm*

2.0 FEH - STATISTICAL EQUATION

QMED - Index Flood	*0.636*	*m^3/s*
QBAR	**0.662**	**m^3/s**

3.0 INSTITUTE OF HYDROLOGY REPORT 124 (IoH 124)

$QBAR_{RURAL}$	**0.372**	**m^3/s**

4.0 FSU - 3 VARIABLES EQUATION

QMED	*0.566*	*m^3/s*
QBAR	**0.589**	**m^3/s**

5.0 FSU 4.2A REGRESSION MODEL

QMED	*0.648*	*m^3/s*
QBAR	**0.675**	**m^3/s**

6.0 FSU - 7 VARIABLE EQUATION (only if catchment greater than 25km²)

$QMED_{RURAL}$	*N/A*	*m^3/s*
QMED	*N/A*	*m^3/s*
QBAR	**N/A**	**m^3/s**

7.0 ADAS (only if catchment less than 0.4km²)

Q 75 year	*N/A*	*m^3/s*

8.0 Rational Method (only if catchment less than 5km²)

$QMED_{av}$	*1.01*	*m^3/s*
$QMED_{max}$	**1.43**	**m^3/s**
Q 100 year $_{av}$	*2.87*	*m^3/s*
Q 100 year $_{max}$	**4.05**	**m^3/s**

Is the watercourse in an Arterial Drainage Scheme or Drainage District? (Y/N)	N

Calculation of Flow Estimation

Method	QBAR	FSE	$QBAR_{FSE}$ (95%)	QBAR (ADS or DD)	100 Year Flow 95% Con Limits*	100yr + CC 95% Con Limits
FSU 3-Variable Method	0.589 m3/s	2.06	1.214 m3/s	N/A	3.17 m3/s	**3.80 m3/s**
FSU 4.2A Regression	0.675 m3/s	1.69	1.141 m3/s	N/A	2.98 m3/s	**3.57 m3/s**
FSU - 7 Variable Equation**	N/A	N/A	N/A	N/A	N/A	**N/A**

*Growth curve factor taken as 2.61 which aligns with national growth factor for 100 year return period and 1.88 for the 75 year return period
**FSU 7 Variable method not suitable for catchments less than 25km2

Selected Method:	**FSU 3-Variable Method**	**100yr + CC Flow:**	**3.80 m3/s**

CHECK METHODS

Method	*QBAR*	*FSE*	*$QBAR_{FSE}$ (95%)*	*QBAR (ADS or DD)*	*100 Year Flow 95% Con Limits*	*100yr + CC 95% Con Limits*
FEH - Statistical	*0.662 m3/s*	*1.43*	*0.948 m3/s*	*N/A*	*1.86 m3/s*	***2.23 m3/s***
IoH 124	*0.372 m3/s*	*1.65*	*0.615 m3/s*	*N/A*	*1.20 m3/s*	***1.45 m3/s***

If catchment less than 0.4km²

Method	*QBAR*	*Q 75 Year*	*100 Year Flow*	*Climate Change*
ADAS	*N/A*	*N/A*	*N/A*	*N/A*

If catchment less than 5km²

Method	*QBAR*	*QMED*	*100 Year Flow*	*Climate Change*
Rational Method	*1.49 m3/s*	*1.43 m3/s*	*4.05 m3/s*	*4.86 m3/s*

Notes:
QMED = 0.96QBAR as per "Flood Estimation in Small and Urbanised Catchments in Ireland" by the OPW
Climate change allowance is 20%
Growth curve factor taken as 1.96 which aligns with national growth factor for 100 year return period and 1.88 for the 75 year return period
For the ADAS and Rational Method the time of concentration has been calculated using the Bransby/Williams formula Tc = (L/D) A^0.4 S^0.2

SOIL Value

SOIL Type	SOIL
1	0.15
2	0.3
3	0.4
4	0.45
5	0.5

Key:

Input Data
Calculated
Output

Slope (S1085) Calculation

Option 1 Full FSU catchment is suitable

Option 2 Catchment less than 2km2 and/or where length of available survey upstream of crossing is less than 500m - Use length of full catchment - US level from LiDAR at upstream end of catchment and DS level from Survey at culvert inlet cross section

Option 3 Available watercourse survey upstream of crossing point is greater than 500m - Use length from survey data upstream of crossing - US taken at approximately 10% along the survey length and DS Level taken at approximately 85% along the survey length respectively

Which option is appropriate for this catchment? Option 2

Option 1 S1085 from FSU Portal 9.247

Main Watercourse Length (m)

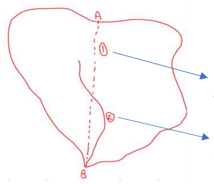
	Length (m)	Type	10%	85%	DS Level	US Level	Slope (m/km)
Option 2	3253	Catchment	*N/A*	*N/A*	59.52	100	*12.444*
Option 3		Watercourse	*0.000*	*0.000*			*#DIV/0!*

Catchment Length (m) *3253.000*

Main Watercourse Length (m)

Catchment/Watercourse Length **3.253**

Final Selected Slope **12.444**



Diagram

Option 2

1) Represents Catchment Length from A to B. This is most appropriate for small catchments where we don't necessarily have a watercourse length.

Option 3

2) Represents Watercourse Length, which is more appropriate for larger catchments where we're interested in the general routing along that length, so it's more important to include it in slope calculations

DRAIND Calculation

Option 1 DRAIND from FSU Portal 1

Runoff Coefficient

Total Area *1.908* km2

Group	Feature Detail	Runoff Coefficient Average	Runoff Coefficient Maximum	Area (km2)	Percentage of Catchment	Coefficient (Maximum)	Coefficient (Average)
Forestry	N/A	0.15	0.25	0	*0.000*	*0.000*	*0.000*
Roads	N/A	0.825	0.95	0.02	*0.010*	*0.010*	*0.009*
Open Spaces	N/A	0.175	0.25	*1.868*	*0.979*	*0.245*	*0.171*
Buildings	Dwellling House	0.4	0.5	0.02	*0.010*	*0.005*	*0.004*
	General Buildings	0.85	0.95			0	0
Other	Land Parcel Urban	0.3	0.35			0	0
	Land Parcel Rural	0.35	0.5			0	0
					Total	**0.260**	**0.184**

Rainfall Depth

Duration (hr)	DDF Rainfall Depth 2 year	DDF Rainfall Depth 100 year
0.5	9.1	27.8
1	11.6	33.3

Tc *1.124* hr

Duration (hr)	DDF Rainfall Depth 2 year	DDF Rainfall Depth 100 year
1.124	**12.22**	**34.66**

Select duration either side of the Time to Concentration (Example below)

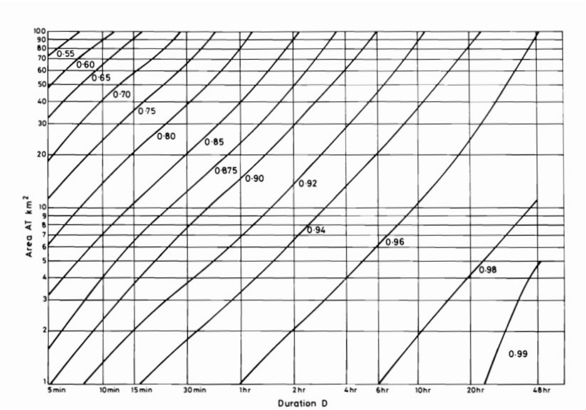
Met Eireann
Return Period Rainfall Depths for sliding Durations
Irish Grid: Easting: 297089, Northing: 267724,

DURATION	Interval 6months,	1year,	Years 2,	3,	4,	5,	10,	20,	30,	50,	75,	100,	120,
5 mins	2.7,	3.8,	4.4,	5.2,	5.8,	6.2,	7.7,	9.3,	10.4,	11.9,	13.2,	14.2,	14.9,
10 mins	3.8,	5.3,	6.1,	7.3,	8.1,	8.7,	10.7,	13.0,	14.4,	16.5,	18.4,	19.8,	20.7,
15 mins	4.5,	6.2,	7.2,	8.6,	9.5,	10.2,	12.6,	15.3,	17.0,	19.4,	21.6,	23.3,	24.4,
30 mins	5.9,	8.0,	9.1,	10.8,	11.9,	12.8,	15.6,	18.6,	20.7,	23.5,	25.9,	27.8,	29.1,
1 hours	7.7,	10.3,	11.6,	13.6,	15.0,	16.0,	19.2,	22.8,	25.1,	28.3,	31.1,	33.3,	34.7,
2 hours	10.0,	13.2,	14.8,	17.2,	18.8,	20.0,	23.8,	27.9,	30.6,	34.2,	37.4,	39.8,	41.4,
3 hours	11.7,	15.2,	17.1,	19.7,	21.4,	22.7,	26.9,	31.4,	34.3,	38.2,	41.6,	44.2,	45.9,

Areal Reduction Factor

Based on catchment area and duration Tc

If area is less than 1km2, assume area is 1km2



Areal reduction factor ARF related to Area AT and Duration D

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