Clonburris T2

Infrastructure Design Report

CLB-T2-ZZZ-SW-DTM-RP-DBFL-CE-0001

INFRASTRUCTURE

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DBFL CONSULTING ENGINEERS

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1 INTRODUCTION

1.1 Background

This infrastructure design report has been prepared by DBFL Consulting Engineers for the planning application for the Clonburris T2 development. The proposed development is part of the Clonburris Strategic Development Zone (SDZ) within the administrative area of South Dublin County Council (SDCC).

The proposed development is located within development area Clonburris Urban Centre (CUC – S3) and development area CSW-S3 within the SDZ, as indicated in Figure 1-1 below. The overall CUC comprises of sectors that will be mixed-use development areas with high density residential developments.

The proposed development will benefit from trunk infrastructure proposed as part of the Clonburris Infrastructure Development for which planning has been granted in August 2021 under planning reference SDZ20A/0021. The CSLS includes trunk road, drainage, watermain and utility infrastructure to serve the Clonburris Strategic Development Zone lands to the south of the Kildare/Cork Railway Line which includes the subject site.

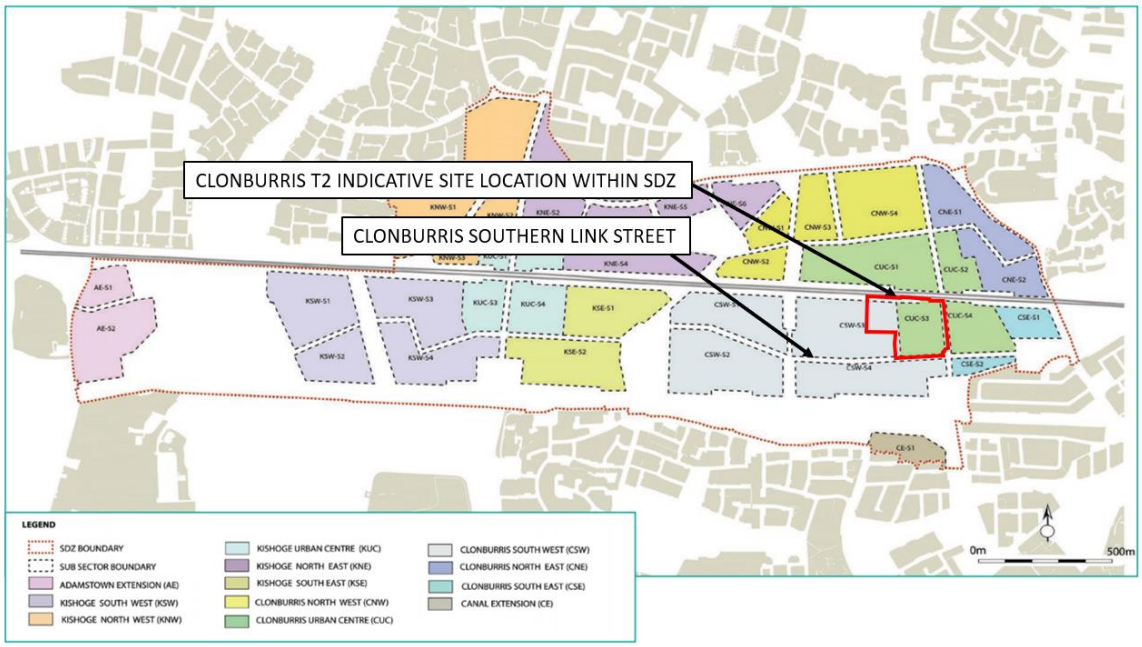

Figure 1-1 Subject site within SDZ (Boundary Indicative)

1.2 Objectives

This report aims to consider the proposed development main infrastructure elements, including the following;

- Road Layout/Site access
- Surface water strategy and servicing
- Foul sewer strategy and servicing
- Water supply and servicing
- Flood Risk

1.3 Location

The overall Clonburris SDZ lands, of approximately 280 Ha, is located to the west of Dublin City Centre and the M50, between the N4 and N7 national primary routes. The Kildare/Cork railway line bisects the lands centrally and the Grand Canal forms the southern boundary.

The subject site for this planning application is situated in the south eastern area of the SDZ lands to the south of the Kildare/Cork railway adjacent to the R113, the R136 Grange Castle Road is situated approximately 1.2km west of the subject site. The permitted Clonburris South Link Street which links the R113 to the R136 will provide access to the subject site. The Adamstown SDZ is located to the north-west of the Clonburris SDZ.

The permitted CSLS currently under construction borders the proposed development to the south. North of the CSLS, the site is within sub sector CUC S3 as shown in Figure 1-2 below.

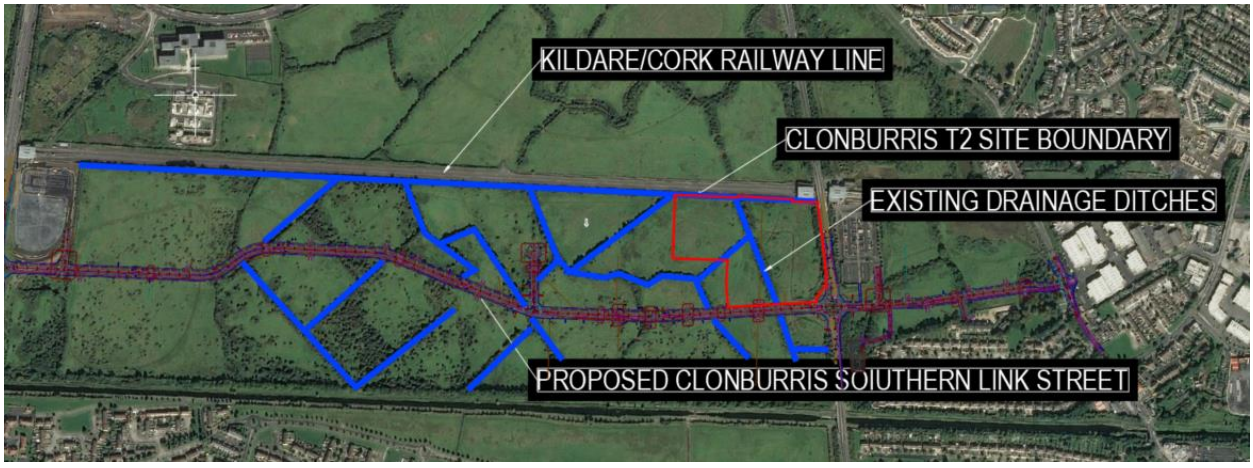

Figure 1-2 Site Location. (Boundary Indicative)

1.4 Site Topography

Overall, the topography of the site is relatively flat. There is a slight fall with a gradient of approximately 0.5% from localised high points to localised low points. A number of drainage ditches are located throughout the site. The site levels within the site generally range between 59.8 and 58.3. The general topography is shown in Figure 1-3. A topographical survey of the Site is provided as a background to the layout drawings issued with this report.

Figure 1-3 Site Topography (Boundary Indicative)

1.5 Proposed Development

The development will consist of the construction of a mixed-use development on a site of c. 5.18 hectares comprising 594 no. apartments (255 no. 1 bedroom apartments, 307 no. 2 bedroom apartments and 32 no. 3 bedroom apartments), as well as commercial office development in Block C, c. 4,516 sq.m), 1 no. retail unit at ground floor of Block B (c.147.5 sq. m) and 3 no. retail units at ground floor of Block E as follows (c.106.2 sq.m, c.141.6 sq.m and c.492.2 sq.m respectively) a creche (c. 609 sq. m) at ground floor and first floor of Block A. Car parking (396 no. spaces in a mixture of undercroft spaces Block A, Block B&D and Block E&F) and bicycle parking (1,232 no. spaces at undercroft and surface levels) along with all site development and landscaping works including public open space.

The application also includes infrastructure comprising a road layout, surface water drainage, foul drainage and the watermain infrastructure.

1.6 Existing Ground Conditions

- A site investigation was undertaken by Ground Investigations Ireland to ascertain the existing ground conditions. This detailed investigation is attached as **Appendix A** included the following.
    - Visit project site to observe existing conditions
    - 21 No. Trial Pits to a maximum depth of 3.10m BGL
    - 9 No. Soakaways to determine a soil infiltration value to BRE digest 365
    - 21 No. Dynamic Probes to determine soil strength/density characteristics
    - 9 No. Plate Bearing Test to ascertain constrained modulus and equivalent CBR
    - 22 No. Rotary Core Boreholes to a maximum depth of 6.80m BGL
    - 10 No. Groundwater monitoring wells
    - Geotechnical & Environmental Laboratory testing
    - Report with recommendations

The sequence of strata encountered were consistent across the site and are generally comprised;

- Topsoil: Topsoil was encountered in all the exploratory holes and was present to a maximum depth of 0.3m BGL.
- Made Ground : Made Ground deposits were encountered beneath the Topsoil and were present to a relatively consistent depth of between 0.50m and 0.90m BGL. These deposits were described generally as brown slightly sandy slightly gravelly CLAY with frequent cobbles and boulders and contained occasional fragments of concrete, red brick, glass, metal and plastic.
- Cohesive Deposits: Cohesive deposits were encountered beneath the Made Ground and were described typically as brown slightly sandy slightly gravelly CLAY or grey mottled brown slightly

sandy slightly gravelly CLAY with occasional cobbles overlying a grey slightly sandy slightly gravelly CLAY with occasional cobbles and boulders.

- Granular Deposits: Granular deposits were encountered below of the cohesive deposits and were typically described as Grey brown clayey sandy angular to sub angular fine to coarse GRAVEL with occasional cobbles and rare boulders.
- Weathered Bedrock: In the majority of exploratory holes weathered rock was encountered which was digable with the large excavator to a depth of up to 0.90m below the top of the stratum. The trial pits were terminated upon encountering the more competent bedrock, in which further excavation became more difficult.
- Bedrock: The rotary core boreholes recovered Weak to Medium strong to very strong dark grey fine to medium grained laminated LIMESTONE interbedded with weak black fine grained laminated Mudstone. This is typical of the Calp Formation, which is noted on the geological mapping to the east of the proposed site. Rare visible pyrite veins were noted during logging which are typically present within the Calp Limestone.

2 ACCESS AND ROADS

2.1 Overall Road and Access Layout

The overall road and access layout is accordance with the requirements defined in the Clonburris SDZ planning scheme. The proposed development will be accessed from the Clonburris Southern Link Street (CSLS) which has been granted permission in August 2021 under planning reference SDZ20A/0021 and is currently under construction.

The CSLS includes minor priority-controlled junctions along the street alignment to provide access to future development cells within the Clonburris SDZ including the subject site.

The subject site's internal road layout has been designed with a number of junctions throughout the development with appropriate traffic calming measures introduced to discourage "rat running" through the development.

Filtered priority junctions have been incorporated at key locations in order to prioritise pedestrian and cyclist movements.

The proposed road hierarchy and typologies are generally consistent with those set out in section 2.2.4 of Clonburris SDZ. Generally the proposed Local streets will be 6.0m wide with a 2m wide footpath on either side of the road. The homezone south of Blocks G and H is 4.8m wide, with minimum 2m footpaths on either side of the road. The development's internal layout has been designed flat top table ramps at strategic locations to calm traffic at junctions. Design speed limits of 30km/hr are applied throughout the development as per Design Manual for Urban Roads and Streets (DMURS).

It is intended that the roads and footpaths of the proposed development are designed to accommodate pedestrian and cycle links to future infrastructure to be constructed as part of the overall Clonburris SDZ. There are number of vehicular and pedestrian/cycle bridges proposed within the SDZ boundary. It is intended that the road, pedestrian and cycle infrastructure of the proposed development will be extended in the future to provide links to these locations as shown in Figure 2-1.

Figure 2-1 Indicative Future Bridge Location Plan

The proposed development's road layout is shown on drawings CLB-1B-95-SW-DTM-DR-DBFL-CE-1001. The standard road cross-sections and construction details are shown on drawings CLB-1B-95-SW-DTM-DR-DBFL-CE-5001 to 5003 comprise the following;

- Local Streets – typically 6m wide carriageway with 2m to 3m footpaths and intermittent 2.4m wide private parallel and perpendicular parking bays.
- Homezones - typically 5m wide carriageway with 2m to 3m footpaths and intermittent 2.4m wide private parallel parking bays.
- Maximum road corner radii of 4.5m are provided within the local streets, with the exception of certain turning heads which have corner radii 6m to accommodate refuse vehicles.

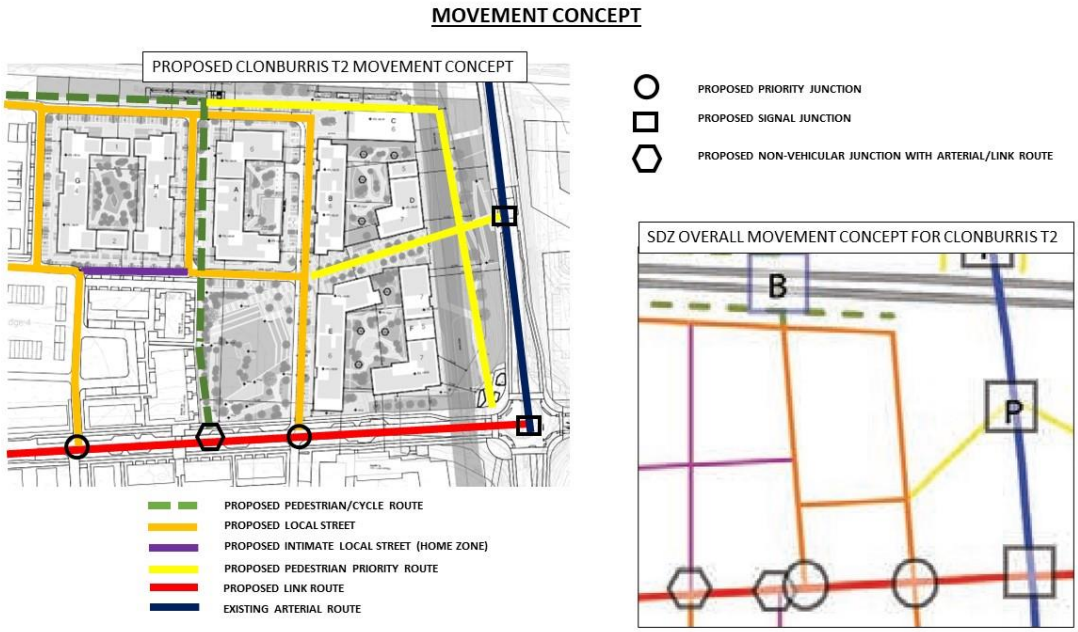

Figure 2-2: Schematic Roads Hierarchy

A Stage 1 Road Safety Audit has been completed and attached in **Appendix B**.

2.2 Pavement Design Standards

The main internal access roads are designed in accordance with the Design Manual for Urban Roads and Streets (DMURS) and Local Authority taking in charge requirements. A 100mm high kerb separation is proposed between typical roads and footpaths. Refer to drawings CLB-1B-95-SW-DTM-DR-DBFL-CE-5001 to 5003 for the proposed road construction details based on CBR tests done as part of the site investigation.

2.3 Vehicle Tracking

The proposed development has been tracked to show that the development's proposed streets and turning heads will accommodate a large refuse vehicle as shown on drawings CLB-1B-95-SW-DTM-DR-DBFL-CE-1001. Refuse staging areas have been indicated on these drawings indicatively and reflect the proposed refuse collection areas of the refuse collection strategy.

2.4 Parking Bays

Perpendicular parking bays are set to accommodate a targeted maximum 1:20 longitudinal gradient. Parallel parking bays have been designed to have a 1:40 crossfall towards the carriageway to allow for excess surface water runoff to be routed to the catchpits provided. All parking bays are permeable paving to be maintained by an appointed management company.

3 SURFACE WATER DRAINAGE

3.1 Existing Drainage

The existing site has a gradient towards the northeast as shown in Figure 3-1 below. There are a number of existing interconnected field drains on the site. These have extremely flat gradients but are understood to drain to existing drainage networks to the east of the site and ultimately to the Camac River. Additional detail on existing drainage within the SDZ is provided in the SWMP. Existing agricultural drainage on site will be maintained as required until it is replaced by drainage networks for the developed site

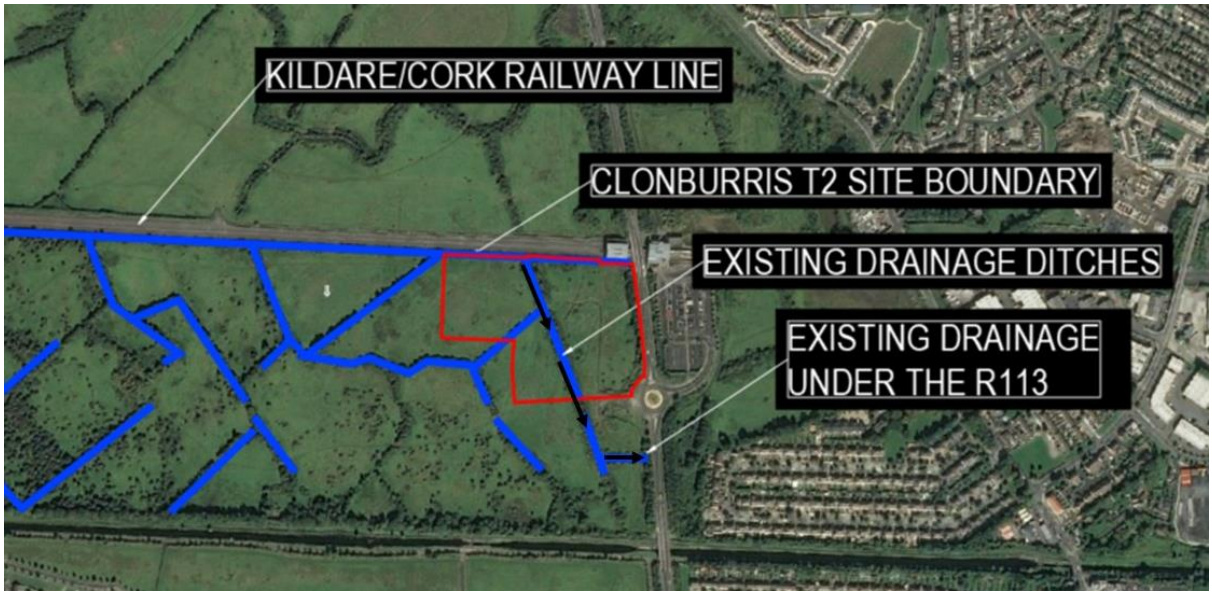

Figure 3-1 Existing Drainage (Boundary Indicative)

3.2 Surface Water Drainage Strategy

3.2.1 Compliance With SWMP

DBFL Consulting Engineers have undertaken a "Surface Water Management Plan" (SWMP) for the overall Clonburris Strategic Development Zone (SDZ). The SWMP for the SDZ been submitted to SDCC and agreed with SDCC. The SWMP outlines the surface water strategy for the overall SDZ lands and the requirements for each individual site within the SDZ which includes the subject site. The SWMP includes the strategy for attenuation design, SUDS features, run off rates and trunk infrastructure layout. The subject site has been designed in accordance with the strategy agreed upon in the SWMP.

The proposed site will benefit from trunk surface water infrastructure proposed as part of the Clonburris Infrastructure Development for which planning was granted under reference SDZ20A/0021. The planning application included trunk surface water sewers and regional attenuation to serve the subject site, this strategic infrastructure aligns with the SWMP proposals and allows for a treatment train of Suds measures within individual sites and within the regional features. The SDZ states that urban centres, which includes a part of the subject site, should attenuate surface water locally.

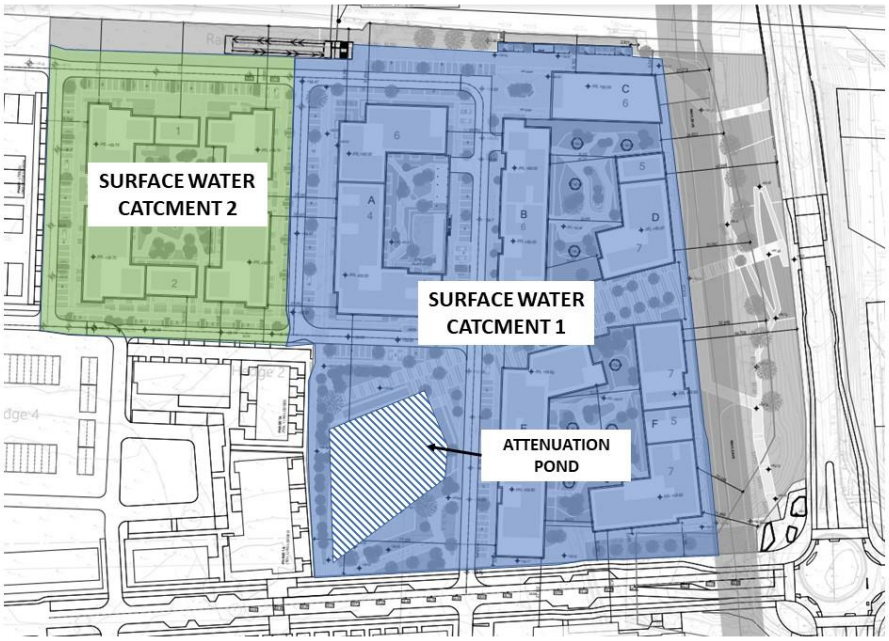

Figure 3-2 Surface Water Catchment Areas

Surface water runoff from the proposed development will be collected in a new gravity sewer. It is intended that the stormwater run-off generated from Catchment 1 as shown in Figure 3-2, will be attenuated in the local attenuation pond as this area of the site falls within urban centre CUC-S3. Controlled surface water discharged from the local attenuation pond is then routed to the regional attenuation basin ATN07 constructed as part of the separately permitted CSLS [SDZ20A/0021] to via the permitted Clonburris T1A [SDZ21A0022] surface water sewer.

Surface water generated from Catchment area 2 as shown in Figure 3-2 is routed west and discharges into the regional attenuation basin ATN07 of the separately permitted CSLS [SDZ20A/0021] via. surface water sewers constructed as part of the separately permitted Clonburris T1A [SDZ21A0022] surface water sewer.

The regional attenuation system ATN07 will consist of modular underground storage with over ground detention basins. Outflow from the attenuation structure within the SDZ limits flow to a rate of 3.1 l/s/ha as detailed in the SWMP for the SDZ.

The subject development application has been coordinated with the Clonburris CSLS application and therefore no significant alterations are proposed to the layout or design of the surface water infrastructure under planning reference SDZ20A/0021.

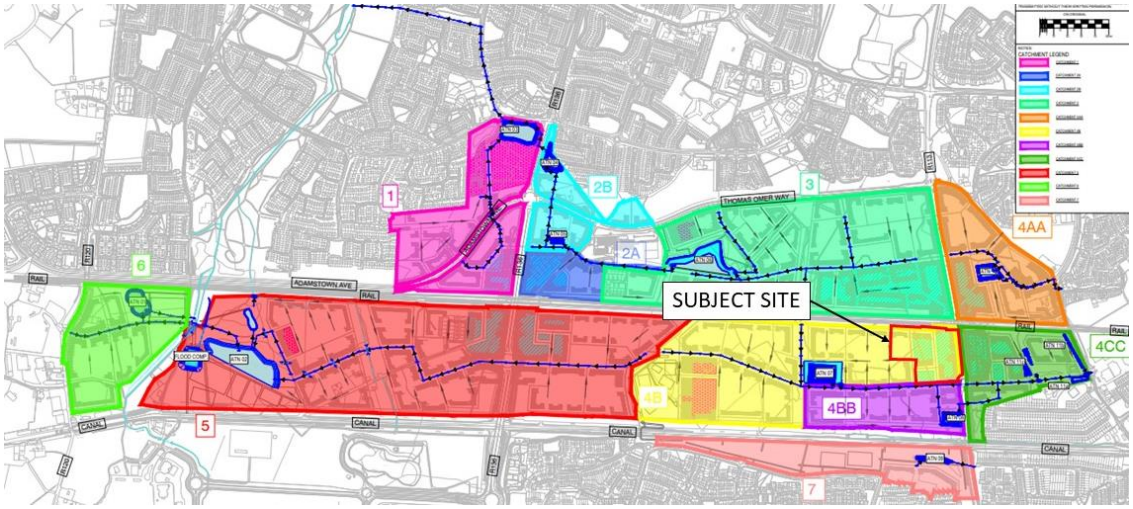

Figure 3-3 SDZ Catchment breakdown

The below table documents the site design compliance with the SWMP Requirements & Objectives

SDZ Requirements/ Objectives	Proposed Development Compliance
O1. It is an objective of the Surface Water Management Plan that proposals for all development cells include provision for at least two separate SuDS features	The proposed objective is met and exceeded in the subject design. Suds features in the site design (prior to discharge to regional SuDS features) include • Permeable paving • Bioretention areas • Swales • SuDS tree pits • Green Roofs on apartment blocks
O2. It is an objective of the Surface Water Management Plan that green roofs are provided to any suitable buildings with area >300m² within	The proposed site is within an Urban Centre sub sector therefore objective is applicable.

Urban Centre sub sectors. Green roof coverage should be minimum of 60% of building area	All apartment buildings with roof areas > 300m2 are designed to have green roofs
O3. It is an objective of the Surface Water Management Plan that runoff from roads adjacent to suitable parkland or landscape strips should be conveyed in vegetated open channel SuDS features	The proposed objective is met in the subject design. Bioretention areas are provided to collect and convey road runoff along roads adjacent to open space.
O4. It is an objective of the Surface Water Management Plan that new link streets incorporate drainage discharges from carriageway runoff to tree pits or similar features.	Link street design is provided separately to this development under planning reference SDZ20A/0021. Drainage discharges to suds features are noted to incorporated into this separate application
O5. It is an objective of the Surface Water Management Plan that all private parking areas are surfaced with pervious paving.	The proposed objective is met in the subject design. All parking areas are proposed to be surfaced with pervious paving.

3.2.2 Compliance with Surface Water Policy

Surface water management for the proposed development is designed to comply with the Greater Dublin Strategic Drainage Study (GDSDS) policies and guidelines and the requirements of South Dublin County Council. The guidelines require the following four main criteria to be provided by the development's surface water design;

- Criterion 1: River Water Quality Protection – satisfied by providing interception storage using permeable paving in driveways, treatment of run-off within the SUDS features e.g. permeable paving for driveways/parking bays, swales, bioretention areas and within the attenuation storage system and oil separators on the main surface water outfalls from the development. SuDS tree pits are also proposed to intercept road runoff.
- Criterion 2: River Regime Protection – satisfied by attenuating run-off with flow control devices prior to discharge to the outfall.
- Criterion 3: Level of Service (flooding) for the site – satisfied by the Site being outside the 1000 year coastal and fluvial flood zones, (See Flood Risk Assessment). Pluvial flood risk addressed by development designed to accommodate a 100 year storm as per GDSDS. Planned flood routing for storms greater than 100 year level, considered in design, the development has

been designed to provide an overland flood route from the development towards the surface water outfall.

- Criterion 4: River flood protection – attenuation and long term storage provided within the SUDS features e.g. permeable paving construction, swales, bioretention areas, tree pits and attenuation facilities.

3.3 SUDS

In accordance with the GDSDS it is proposed to use Sustainable Urban Drainage systems (SUDS) for managing storm-water for the proposed development. The aim of the SUDS strategy for the site will be to;

- Attenuate storm-water runoff.
- Reduce storm-water runoff.
- Reduce pollution impact.
- Replicate the natural characteristics of rainfall runoff for the site.
- Recharge the groundwater profile

The proposed layout of the drainage system and SUDS features is detailed on drawings CLB-1B-94-SW-DTM-DR-DBFL-CE-1001.

The Surface Water Management Plan agreed with SDCC includes a number of potential SUDs feature to be implemented on individual sites within the SDZ. The following SUDs features are incorporated into the design for the subject site:

3.3.1 Bioretention Areas / Rain Gardens

Where possible Bioretention areas have been implemented into the design as shown on drawings CLB-1B-94-SW-DTM-DR-DBFL-CE-1001. Surface water generated from the adjacent roads and footpaths will discharge directly to these SuDS features via inlet kerbs detailed on drawing CLB-1B-94-SW-DTM-DR-DBFL-CE-5004 or via road gullies feeding surface water from the road surface to the bioretention areas.

Water Quality - Surface water runoff from the adjacent roads/ roofs is conveyed to the bioretention area which routes any surface water that has not infiltrated naturally into the ground or absorbed by the vegetation, to the surface water pipe network to be attenuated in the regional attenuation basins.

The build-up of the bioretention consists of a filter medium, a transition layer and a drainage layer as detailed on drawing CLB-1B-94-SW-DTM-DR-DBFL-CE-5004. The 1200mm free draining filter medium will filter out pollutants and provides natural surface water flow control. The 300mm transition layer prevents fine filter medium from entering the drainage layer. The 200mm drainage layer is a surrounds the 150mm perforated pipe which collects the surface water after it has filtered through the bioretention area build-up and is then discharged into the surface water pipe network.

Storage Volume - The bioretention area build-up contributes to the local surface water storage volume, serving as a natural surface water source control.

Biodiversity - The bioretention areas will contribute to the biodiversity of the proposed development by adding habitat for wildlife. See the landscape architect drawings for further details on specific plants proposed in the bioretention areas.

Amenity - The bioretention areas are generally proposed along roads to receive runoff from adjacent roads and will contribute to aesthetics of the streetscape. Local bioretention areas are also proposed within the detention basins to receive surface water runoff from nearby roads to create local "wet" areas. This will encourage the growth of plants proposed in these areas, further adding to the biodiversity of the development and create enjoyable and aesthetically pleasing public open space areas.

3.3.2 Permeable Paving

The proposed design includes permeable finishes on all parking bays within the development as shown on drawings CLB-1B-95-SW-DTM-DR-DBFL-CE-1001. Surface water runoff from the paved areas is intercepted by the permeable build-up of the paved areas where it is intended to naturally infiltrate into the ground. If the porous build-up of the paving and the in-situ material beneath becomes saturated, surface water would drain overland to the adjacent road and into the surface water pipe network to be attenuated locally.

Water Quality - Permeable paving reduces pollutants such as petrol and diesel as it contributes to its biodegrading process. It also assists in filtering solid particles out of surface water runoff, providing filtration before discharge into the surface water pipe network and ultimately the receiving watercourse. The build-up of the permeable paving is shown on drawing CLB-1B-95-SW-DTM-DR-DBFL-CE-5001 to 5002

Storage Volume - The permeable pavement build-up of the parking areas for the development is 500mm thick and has a 0.3 void ratio. This adds a significant amount of local surface water storage to the development before the eventual discharge into the regional attenuation structures, while adding further opportunity for groundwater infiltration.

Biodiversity - Permeable paving does not directly contribute to the biodiversity of the development, but the surface water treatment it provides maximises the biodiversity in the downstream watercourses at the discharge point.

Amenity - Permeable provides amenity in its usefulness as it can be used for a range of activities, while also acting as a valuable component in the surface water treatment and storage train.

3.3.3 Tree Pits

Tree Pits are proposed to intercept road runoff throughout the entire development. Two types of SuDS tree pits have been proposed as shown on drawing CLB-1B-94-SW-DTM-DR-DBFL-CE-1001. Tree pits in local streets are generally placed behind the back of the footpath and intercept road runoff via a road gully with an overflow to the surface water pipe network. Once the tree pit is saturated, water would be routed to the surface water pipe network through an overflow as detailed on drawing CLB-1B-94-SW-DTM-DR-DBFL-CE-5004,

Water Quality - Surface water runoff will be filtered through the soil surrounding the tree root ball, removing pollutants. Pollutants are also naturally broken down during the transpiration process. This filtration process results in surface water with less pollutants being discharged into the surface water pipe network end the eventual receiving wasters.

Storage Volume - The base of the tree pit provides storage within the porous soil and drainage layers, allowing for natural infiltration into the ground and absorption by the tree roots, reducing the amount of surface water discharged into the surface water pipe network via overflow pipes.

Biodiversity - The addition of SuDS street trees will increase the habitat for a variety of animal species and insects and act as bridge for wildlife in the post-developed urban landscape. Filtered water passing through the tree pit and into the receiving watercourse would also add to the biodiversity downstream. See the landscape architect detail for proposed species of the street trees.

Amenity - The street trees will add significant amenity benefits such as improving the aesthetics of the urban landscape. The canopies of the trees will also provide a cooling effect in the post developed streetscape.

3.3.4 Green Roofs

Green roofs are proposed for Apartment Blocks A to H as shown on site drainage drawings CLB-1B-94-SW-DTM-DR-DBFL-CE-100. Details of the proposed build-up of the green roof are shown on drawing CLB-1B-94-SW-DTM-DR-DBFL-CE-5004.

Water Quality - Surface water runoff from the green roof areas would be filtered by the substate/growing medium and into the drainage layer before discharging in to the surface water pipe network. This process will reduce pollution from entering the surface water pipe network and ultimately the receiving water.

Storage Volume - The green roofs can be assumed to meet the interception requirements in the summer, based on their retention of the 5mm of rainfall as per the CIRIA SuDS Manual 2015, reducing runoff rates. The substrate is assumed to have 30% porosity and would be able to store a significant amount of rainwater before saturation is reached.

Biodiversity - The green roofs will provide further ecological benefits by attracting birds, bees, butterflies and other insects by creating pockets of habitat at high level for nesting and foraging.

Amenity - The use of green roofs will improve on a range of amenity principles such as improved climate resistance, air quality, noise levels and increased building services life due to aesthetic appeal, reduced anergy costs and increased association with sustainable design and social responsibility (CIRIA SuDS Manual 2015).

3.4 Attenuation

As set out in the SWMP and the Infrastructure Design Report for SDZ20A/0021, attenuation volumes for the SDZ are generally provided on a regional basis, with the exception of urban centre and school sites. The subject site is partially within an urban centre and local attenuation is required and provided in the form of an open attenuation pond. Refer to the enclosed drawing CLB-1B-94-SW-DTM-DT-DBFL-CE-1002 for the surface water catchment plan.

The allowable outflow runoff rate of the subject site is 3.23l/s/h for the site. However, a conservative allowable outflow runoff rate of 3.1l/s/h has been used to calculate local attenuation volumes for the subject site as assumed for catchment area 4B in the SWMP. Outflow will be controlled using a 'hydrobrake optimum' or similar approved as a flow control device. A summary of the gross and net impermeable areas used for the calculation of the allowable outflow runoff is summarized in Table 3-1 below:

Table 3-1 Impermeable Area Calculations

	Hardstanding Type	Gross Area (ha)	Runoff Co-efficient	Impermeable Area (ha)
Catchment 1 (Locally attenuated)	Hard Surfacing (Roads, Footpaths)	1.233	1.00	1.233
	Permeable Paving	0.159	0.80	0.127
	Podium- soft landscaping	0.506	0.80	0.405
	Green Roof	0.571	0.85	0.486
	Roof (Buffer and PV Areas)	0.381	1.00	0.381
	Public Open Space & Grassed Areas	0.250	0.35	0.088
	Sub-Total	3.1		2.7
Catchment 2 (Regionally Attenuated)				
	Hard Surfacing (Roads, Footpaths)	0.343	1.00	0.343
	Permeable Paving	0.104	0.80	0.083
	Podium- soft landscaping	0.000	0.80	0.000
	Green Roof	0.221	0.85	0.188
	Roof (Buffer and PV Areas)	0.147	1.00	0.147
	Public Open Space & Grassed Areas	0.386	0.35	0.135
	Sub-Total	1.200		0.895
	TOTAL (ha)	**4.3**		**3.6**

Site discharge calculations are included in Appendix C. The required attenuation volume for the attenuation pond has been determined using the Micro Drainage Network model as attached in Appendix D.

3.4.1 Compliance with Attenuation Design

As required in the SWMP, the subject site will allow for local attenuation for surface water generated from Catchment 1 (refer to drawing CLB-1B-94-SW-DTM-DT-DBFL-CE-1002) through the introduction of an open attenuation pond due to the site located within an urban centre. The controlled discharge from the subject site attenuation pond will be routed to the regional attenuation areas ATN07 and ATN08 via the adjacent permitted Clonburris T1A development. Downstream surface water infrastructure has been designed to receive surface water outflow from the subject site. The regional attenuation downstream of the subject site is to be constructed as part of the Clonburris Southern Link Street (CSLS) in advance of the proposed development. As mentioned in Section 3.2.1, the proposed development spans across the SWMP Catchment 4B.

The downstream receiving attenuation systems for the CSLS have been approved under planning reference SDZ20A/0021.

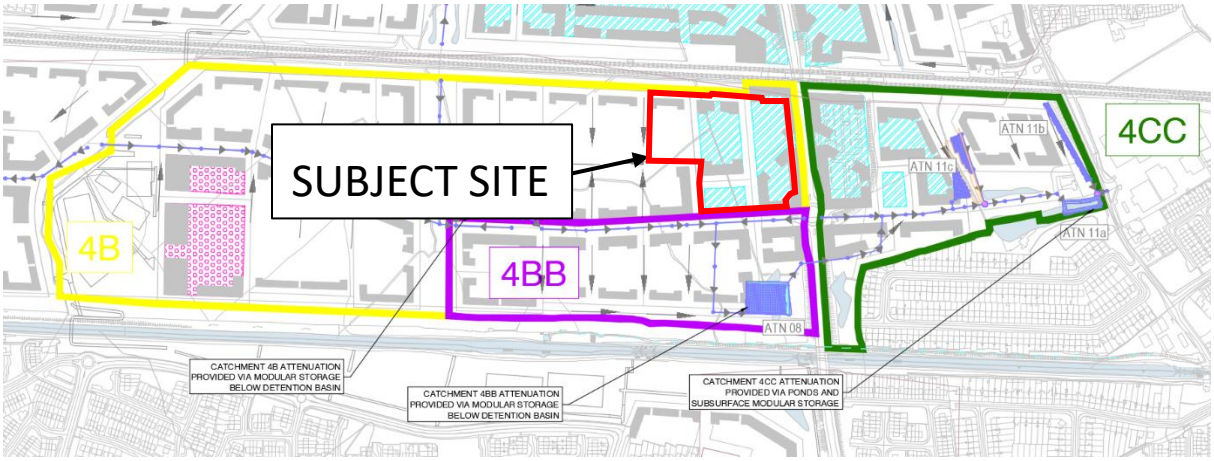

Figure 3-4: Regional Attenuation and Outfall

Calculation of regional attenuation volumes is included within the SWMP and within the CSLS infrastructure application. These regional features cater for the attenuation volumes required for individual developments such as the subject development and constricted surface water discharge from the subject site is limited to 3.1l/s as per the SWMP.

3.5 Design Standards

Drainage is designed in accordance with the Greater Dublin Regional Code of Practice for Drainage Works and the agreed Clonburris Joint Infrastructure Works. Surface water pipe-work was sized using the Microdrainage Windes drainage modelling software. The following parameters apply to the design:

- Qbar Allowable Attenuation Outflow (Calculated) 3.23 l/s/h
- Qbar Allowable Attenuation Outflow from SDZ 3.1 l/s/ha (Used)
- Time of entry: 4 minutes
- Pipe Friction (Ks): 0.6mm
- Minimum Velocity: 1.0 m/s
- Standard Average Annual Rainfall: 773mm (Met Eireann)
- M5-60: 17mm (Met Eireann)
- Ratio r (M5-60/M5-2D): 0.277 (Met Eireann)

- Storage System Storm Return Event GDSDS Volume 2, Criterion 3
- 30-year no flooding on site;
- 100-year check no internal property flooding. Flood routing plan. FFL freeboard above 100-year flood level.
- Climate Change 20% for rainfall intensities.

Surface water sewers have been designed in accordance with IS EN 752 and the recommendations of the 'Greater Dublin Strategic Drainage Study', (GDSDS).

Standard drainage details, as outlined on DBFL drawings CLB-1B-94-SW-DTM-DR-DBFL-CE-5001 to 5004 are in accordance with the Greater Dublin Regional Code of Practice for Drainage Works.

The minimum pipe diameter for public surface water sewers is 225mm. Private drains within the proposed development will comply with Irish Water/ GDSDS minimum requirements.

Surface water sewer network modelling results for the main drainage networks included in Appendix D.

3.6 Climate Change

Rainfall values for the proposed development are sourced from Met Eireann to calculate the FSR input hydrograph for the drainage design as required by the GDSDS. The design rainfall intensities were increased by a factor of 20% to take account of climate change, as required by the GDSDS for attenuation storage design.

3.7 Pluvial Flooding Provision

The surface water network, attenuation storage and site levels are designed to accommodate a 100 year storm event and includes climate change provision. Floor levels of houses are set above the 100 year flood levels by a minimum of 0.5m for protection. For storms in excess of 100 years, the development has been designed to provide overland flood routes along the various development roads without affecting building floor levels.

3.8 Surface Water Quality Impact

Run-off rates from the attenuation systems are controlled by Hydrobrake flow control devices. Surface water management proposals for the development also incorporate the following to reduce its impact;

- Designed in accordance with the 'Greater Dublin Strategic Drainage Study' GDSDS and the Clonburris joint infrastructure works surface water management plan requirements;
- Incorporates SUDS features e.g. permeable paving in high risk parking areas at the front of houses;
- On-line attenuation/infiltration facilities with an oil separator prior to discharge to a public surface water sewer has been included in the trunk infrastructure under planning reference SDZ20A/0021 via the djacent Clonburris T1 development, which has been designed to accommodate receiving surface water flows from the subject development.

3.9 Flood Risk Assessment

As part of the Clonburris SDZ Draft Planning Scheme, South Dublin Co Council commissioned a Strategic Flood Risk Assessment SFRA for the lands which was completed by JBA Consulting and is listed as a supporting document to the planning scheme (http://www.clonburris.ie/Documentation/Clonburris-SDZ-SFRA.pdf). The subject sites land was accounted for in the Clonburris SDZ Strategic Flood Risk Assessment. It was predicted that the subject site was at low risk of flooding (Flood Zone C) for events up to the Q1000 event. The study also found there is no existing development within the subject site that is at potential risk of flooding.

As part of the flood risk assessment, historic and predicted flood risk mapping published by the OPW on the Flood Hazard Mapping Website http://www.floodinfo.ie/ was reviewed.

Historical flood maps/data indicate there are no recorded flood events within the proposed site boundary. There are to recorded recurring flood events within 1km of the proposed site. The first is a recurring flood event at the Cappaghmore Culvert located approximately 500m to the east of the site. The Second is located at the Beech Row Bungalows approximately 380m to the east of the site.

The Eastern CFRAM (Catchment Flood Risk Assessment and Management) study details the predicted risk for a variety of fluvial and coastal flood scenarios. The mapping does not include the watercourse reaches affected by the proposed scheme and only maps downstream flooding. The proposed development is therefore outside of the Q100 and Q1000 flood extents and is therefore in within Flood Zone C (low risk of flooding).

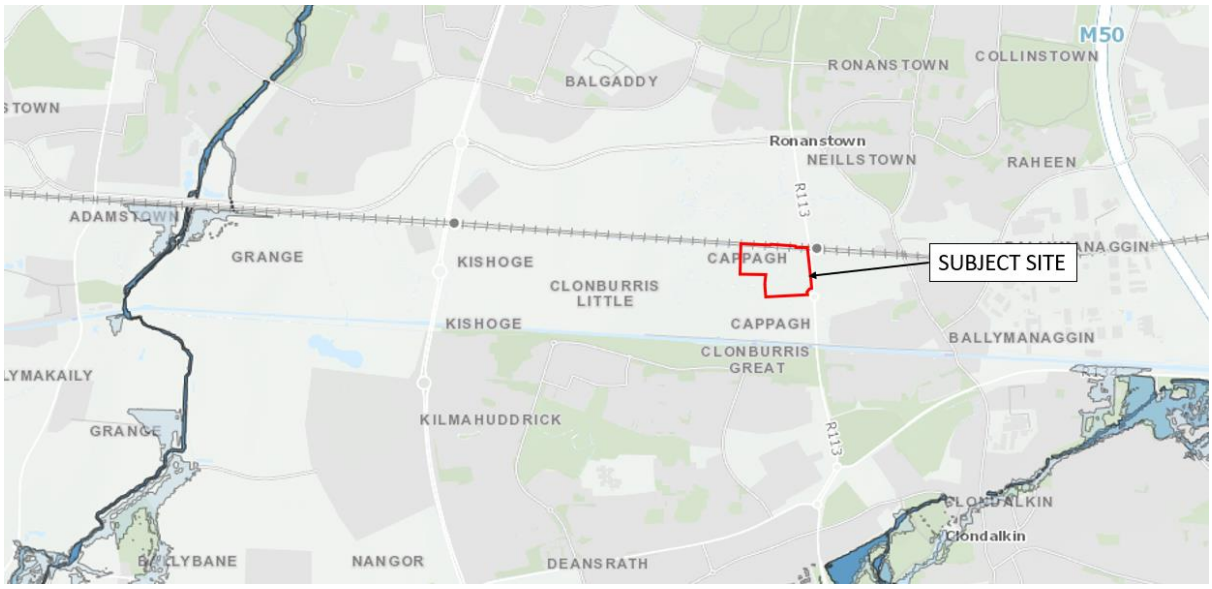

Figure 3-5 Extract of CFRAMS Data from OPW FloodInfo.ie

The OPW undertook an Irish Coastal Protection Strategy Study (ICPSS) which produced coastal/tidal flood extents maps for the Irish coastline for a 0.5% AEP tidal flood level. This map indicates that the Site is far outside the extents of the coastal/tidal flood zone.

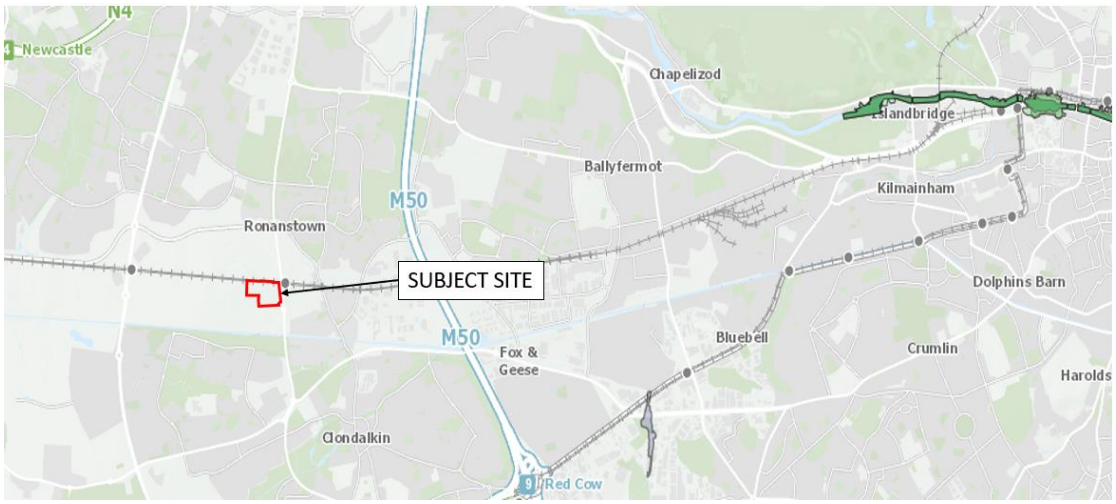

Figure 3-6 Extract of ICPSS Data from OPW FloodInfo.ie

3.10 Flood Exceedance

For storms greater than the 1%AEP pluvial event, the development's drainage network design may be exceeded and run-off may flow above ground along the main roads. The development has been designed without low areas/depressions where possible and run-off will generally make its way along the proposed roads south towards the local attenuation pond as shown in Figure 3-7. Apartment block floor levels have been set to make allowance for any possible areas of surface ponding during exceedance events.

Figure 3-7 Flood Exceedance Allowance

3.11 Receiving surface water network (Clonburris T1)

Certain portions of Stormwater infrastructure designed to be part of the permitted adjacent Clonburris T1 have been upsized so that they are suitable to receive surface water runoff from future development phases, including the subject development. The urban centre area will have its own local attenuation as per the SWMP and the restricted outflow has been allowed for.

Figure 3-8 indicates the surface water runoff from the subject site that would be received by the downstream Clonburris T1A development during the design of the Clonburris 1A phase.

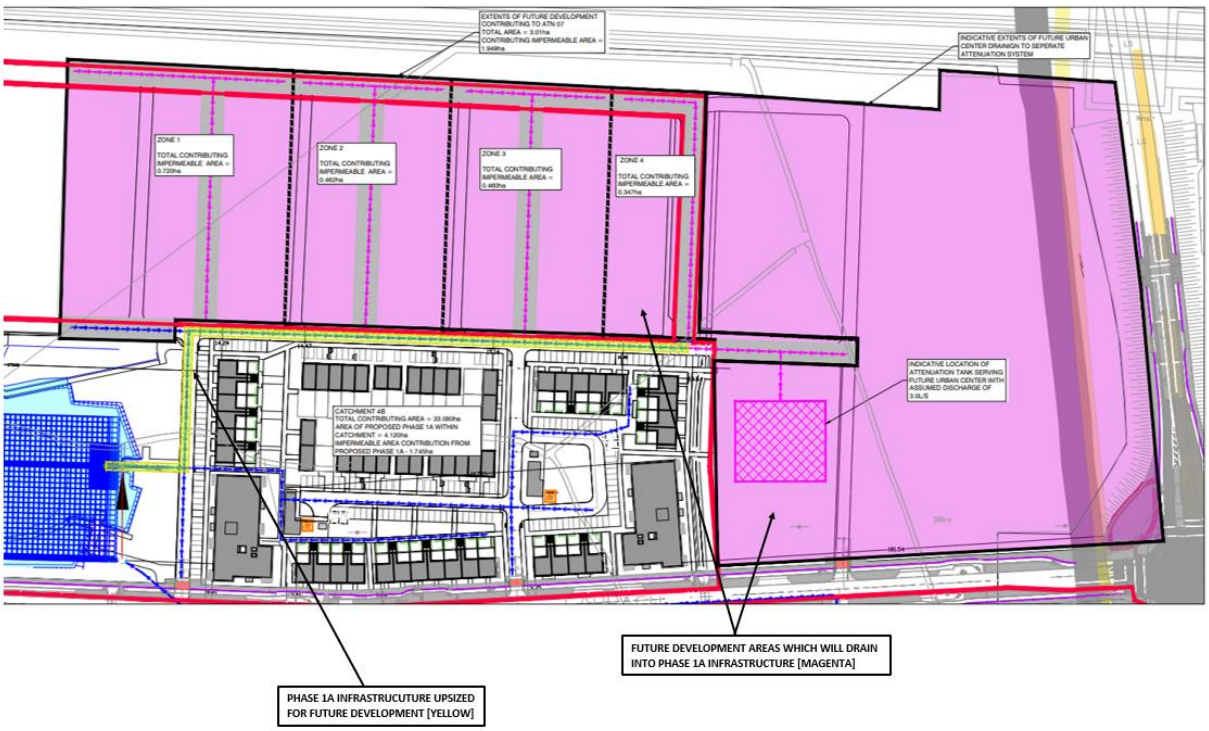

Figure 3-8 Surface Water Network Allowance for Subject Site to Drain to Regional Attenuation via T1

4 FOUL DRAINAGE

4.1 Existing Foul Drainage

The existing site is predominantly greenfield and therefore has no foul loading at present. The planning application SDZ20A/0021 includes the trunk foul sewers which the subject site will connect into at the southern boundary with the CSLS.

4.2 Design Strategy

The overall SDZ site has been divided into 7 separate wastewater catchments (refer to Figure 4.1). The subject site falls within Catchment X (as identified in the Wastewater Management Plan). The proposed site will benefit from foul infrastructure proposed as part of the CSLS. Trunk Foul sewer network has been designed as part of the CSLS to serve the subject based on the average net density for catchment X, ranging from the "Low margin" to a "High Margin".

Table 4-1 Development Figures for Each Catchment

Catchment	Residential Dwellings Low Margin	Residential Dwellings Target	Residential Dwellings High Margin	Retail GFA (m^2)	Employment GFA (m^2)	Community/Civic Building GFA (m^2)	Number of Schools
Catchment 1	1236	1466	1705	600	4800	0	2 & 1 (Existing)
Catchment 5	0	0	0	0	0	0	0
Catchment U	2035	2615	3198	14370	9215	3100	2
Catchment Y	1521	1760	1991	850	2600	600	1
Catchment X	2680	3293	3896	5400	14100	1500	2
Catchment 7	148	162	175	0	0	0	
Catchment Z	110	121	133	0	0	0	

The overall SDZ lands are relatively flat therefore the pumping of wastewater is required. It is proposed that the wastewater generated from the new apartments for this application will be collected by new gravity sewers that discharges to the trunk sewer within the new Link Road via the adjacent permitted Clonburris T1 development. This in turn discharges to the permitted Irish Water pumping station (Pumping Station #1 as shown in Figure 4-1) adjacent to the R113 Fonthill Road. This future pumping station and its rising main connection to the existing 9B trunk sewer on Fonthill Road is being delivered by Irish Water as part of the Irish Water Clonburris Local Infrastructure Housing Activation Fund (LIHAF) Scheme. The foul water pump station has been separately granted under planning reference SDZ21A/0006 in November 2021.

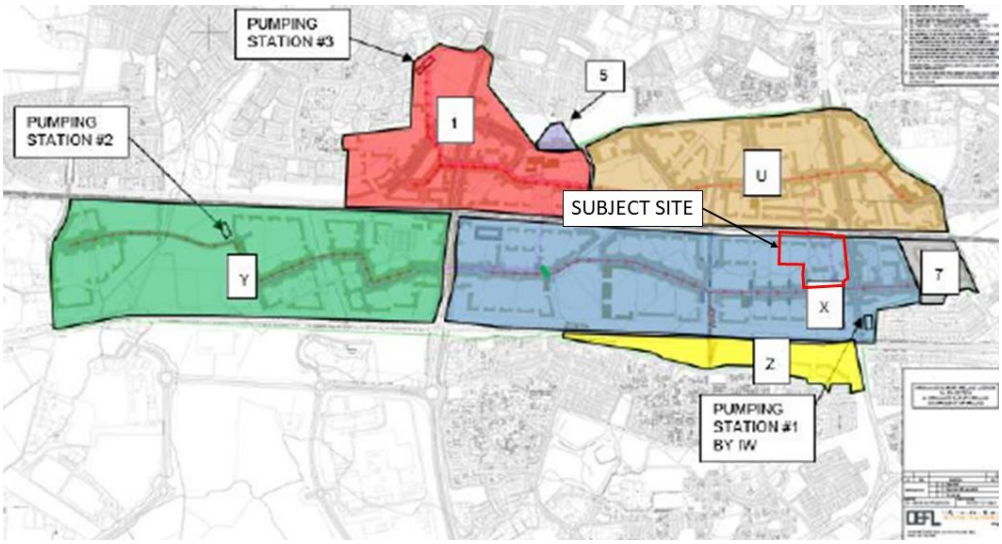

Figure 4-1 Wastewater Catchment Plan

4.3 Design Criteria

Foul sewers have been designed in accordance with the Building Regulations and specifically in accordance with the principles and methods set out in the Irish Water Design and Construction Requirements for Self-Lay Developments July 2020 (Revision 2) and the recommendations of the 'Greater Dublin Strategic Drainage Study', (GDSDS).

The following criteria have been applied:

Demand	446l/dwelling/day (based on 2.7 persons per house, a per capita wastewater flow of 150 litres per head per day and a 10% allowance for infiltration) 60l per person per day for Creche/civic building 50l per person per day for Commercial (office and retail)
Discharge units	14 units per house (as BS8301)
Pipe Friction (Ks)	1.5 mm
Minimum Velocity	0.75 m/s (self-cleansing velocity)
Maximum Velocity	2.5 m/s
Frequency Factor	0.5 for domestic use
Manhole Depths	< 5.0m

All foul sewers and manholes will be constructed in accordance with the Irish Water Standard Details and the Irish Water Code of Practice for Wastewater.

Longitudinal sections for the proposed foul sewers are detailed on drawings CLB-1B-94-SW-DTM-DR-DBFL-CE-3001 to 3002.

4.4 Compliance with Irish Water Standards

The proposed foul sewer design and layout is in accordance with the Irish Water Code of Practice for Wastewater Infrastructure and The Irish Water Wastewater Infrastructure Standard Details. The Irish Water Confirmation of Feasibility is attached in **Appendix E**.

4.5 Compliance with Clonburris Water and Wastewater Report

The proposed foul sewer design and layout complies with the Clonburris Water and Wastewater Report as agreed with SDCC and Irish Water.

4.6 Design Calculations

This application comprises 594 residential units, 609m^2 creche area, 646m2 amenity area, 4516m^2 office area and 887.5m^2 of retail area. The development will discharge to Pumping Station 1. The estimated average daily load from the development is 246m^3. See below for calculations in Table 4-2. Full network calculations are contained in **Appendix F**.

Table 4-2 Predicted Clonburris Phase 1A Foul Flow Calculations

TITLE Clonburris T2		Job Reference 210124	
SUBJECT Wastewater Demand for Irish Water		Calc. Sheet No. 1	
DRAWING NUMBER -	Calculations by DCB	Checked by JPC	Date 30/11/2022

RESIDENTIAL - PREDICTED DEVELOPMENT FOUL FLOWS						
Unit Type	**No.**	**Loading l/person/day**	**Occupancy person/unit**	**Occupancy**	**Daily Loading l/day**	**Daily Loading l/s**
Apartments	594	150	2.7	1,604	240,570	2.78
					Residential Daily Loading	**2.78**
					Growth Factor	1
					Infiltration @ 10% (as CoP App C 1.2.4)	0.28
					Dry Weather Flow l/s	**3.06**
					Residential Peak Factor (as CoP App C 1.2.5)	6.0
					Design Foul Flow l/s	**18.38**
***Flow rates calculated using IW CoP for Wastewater Infrastructure Appendix C**						

NON-RESIDENTIAL - PREDICTED DEVELOPMENT FOUL FLOWS						
Unit Type	**Floor Area m2**	**Occupancy Load m2 /person**	**Occupancy**	**Loading l/Person/day**	**Daily Loading l/day**	**Daily Loading l/day**
Creche	609	7	87	50	4,350	0.05
Commercial (Office)	4,516	50	90	12	1,084	0.01
Commercial (Retail)	887	50	18	12	213	0.00
					Non - Residential Daily Loading	**0.07**
					Growth Factor	1
					Infiltration @ 10% (as CoP App C 1.2.4)	0.01
					Dry Weather Flow l/s	**0.07**
					Commercial Peak Factor (as CoP App C 1.2.7)	6
					Design Foul Flow l/s	**0.43**

TOTAL PREDICTED DEVELOPMENT AVERAGE FOUL FLOWS l/s	**2.85**
TOTAL PREDICTED DEVELOPMENT PEAK FOUL FLOWS l/s	**18.81**
***Flow rates calculated using IW CoP for Wastewater Infrastructure Appendix D**	

5 WATER SUPPLY AND DISTRIBUTION

5.1 Existing Water supply

The proposed site will benefit from trunk watermain infrastructure proposed as part of the Clonburris Infrastructure Development for which was granted permission under planning reference SDZ20A/0021. The planning application includes a 400mm diameter watermain running along the Proposed CSLS at the north of the subject site as shown in Figure 5-1.

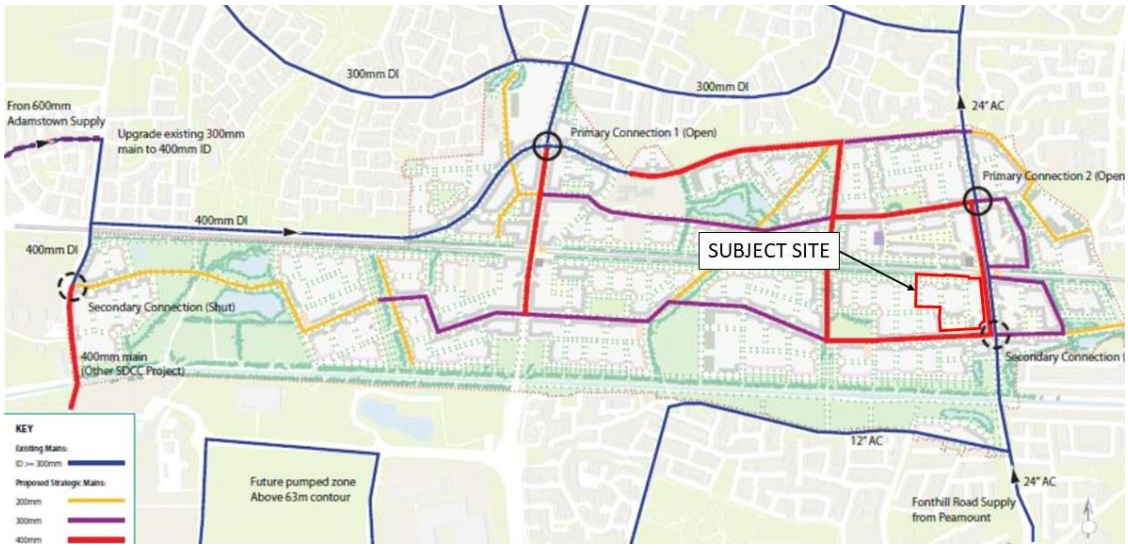

Figure 5-1 Irish Water Strategic Watermain Plan

5.2 Development Water Main Layout

The subject section of the site will connect into the CLSL trunk watermain infrastructure at the southern end of the site. The 180mm watermain loops around the apartment buildings and connects to each at the plant rooms with a 100mm connection.

The connection to the public water main will include a metered connection with sluice valve arrangement in accordance with the requirements of Irish Water.

Hydrants are provided for fire-fighting at locations to ensure that each dwelling is within the required Building Regulations distance of a hydrant. The development's proposed water-main distribution system is shown on drawing CLB-1B-93-SW-DTM-DR-DBFL-CE-1001 in Compliance with Irish Water Standards.

The proposed watermain design and layout is in accordance with the Irish Water Code of Practice for Water Infrastructure and The Irish Water, Water Infrastructure Standard Details. The Irish Water Confirmation of Feasibility is attached in **Appendix E**.

5.3 Compliance with Clonburris Water and Wastewater Report

A confirmation of feasibility for the overall SDZ lands has been received from Irish Water (ref: 2512559856). The proposed watermain design and layout complies with the Clonburris Water and Wastewater Report as agreed with SDCC and Irish Water. Refer to **Appendix E** for further details.

5.4 Design Calculations

The water demand is designed in accordance with the principles and methods set out in Irish Water's Code of Practice for Water Infrastructure Connections and Developer Services Design & Construction Requirements for Self-Lay Developments JULY 2020:

Overall water demand is calculated using IW CoP for Water Infrastructure section 3.7.2, as outlined below:

Per-capita consumption 150l/person/day

Average day/week demand factor 1.25

Peak demand factor 5.0

Average daily domestic demand = Total occupancy * Per-capita consumption

Average day/peak week demand = Average daily domestic demand * Average day/week demand factor

Peak hour water demand = Average day/peak week demand * Peak demand factor

Estimated water demand for the proposed development is provided in Table 5.1

Table 5-1 Predicted Water Calculations

TITLE Clonburris T2		Job Reference 210124	
SUBJECT Water Demand for Irish Water		Calc. Sheet No. 2	
DRAWING NUMBER -	Calculations by DCB	Checked by JPC	Date 30/11/2022

RESIDENTIAL - WATER DEMAND						
Unit Type	No. Dwellings	Occupancy Rate /dwelling	Occupancy	Per Capita Consumption l/Person/day	Average Daily Domestic Demand l/day	Average Daily Domestic Demand l/s
Apartments	594	2.7	1604	150	240,570	2.78
					Total Average Daily Loading l/s	**2.78**
					Average Day/Week Domestic Demand	1.25
					Average Day/Peak Week Demand l/s	**3.48**
					Peak Demand Factor	5
					Peak Hour Water Demand l/s	**17.40**
*Flow rates calculated using IW CoP for Water Infrastructure						

NON-RESIDENTIAL WATER DEMAND						
Unit Type	Floor Area m^2	Occupancy Rate m^2/person	Occupancy	Per Capita Consumption l/Person/day	Average Daily Demand l/day	Average Daily Demand l/s
Creche	609	7	87	150	13,050	0.15
Commercial (Office)	4,516	50	90	150	13,548	0.16
Commercial (Retail)	887	50	18	150	2,661	0.03
					Total Average Daily Loading l/s	**0.34**
					Average Day/Week Demand	1.25
					Average Day/Peak Week Demand l/s	**0.42**
					Peak Demand Factor	5
					Peak Hour Water Demand l/s	**2.12**
*Flow rates calculated using IW CoP for Water Infrastructure						

TOTAL AVERAGE DAILY LOADING l/s	**3.12**
AVERAGE DAY/PEAK WEEK DEMAND l/s	**3.90**
PEAK HOUR WATER DEMAND	**19.52**
*Flow rates calculated using IW CoP for Wastewater Infrastructure Appendix D	

Appendix A : SITE INVESTIGATION REPORT

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Ground Investigations Ireland

DBFL

Clonburris 1B

Ground Investigation Report

June 2022

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DOCUMENT CONTROL SHEET

Project Title	Clonburris 1B
Engineer	DBFL
Client	Cairn Homes
Project No	11542-03-22
Document Title	Ground Investigation Report

Rev.	Status	Author(s)	Reviewed By	Approved By	Office of Origin	Issue Date
A	Final	J McDowell	D MagLochlainn	C Finnerty	Dublin	12 July 2022

Ground Investigations Ireland Ltd. present the results of the fieldworks and laboratory testing in accordance with the specification and related documents provided by or on behalf of the client The possibility of variation in the ground and/or groundwater conditions between or below exploratory locations or due to the investigation techniques employed must be taken into account when this report and the appendices inform designs or decisions where such variation may be considered relevant. Ground and/or groundwater conditions may vary due to seasonal, man-made or other activities not apparent during the fieldworks and no responsibility can be taken for such variation. The data presented and the recommendations included in this report and associated appendices are intended for the use of the client and the client's geotechnical representative only and any duty of care to others is excluded unless approved in writing.

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APPENDICES

1.0 Preamble

On the instructions of DBFL Consulting Engineers, a site investigation was carried out by Ground Investigations Ireland Ltd., between April and June 2022 at the site of the proposed residential development in Clonburris, Cappagh, Dublin.

2.0 Overview

2.1. Background

It is proposed to construct a new residential development with apartments and associated services, access roads and car parking at the proposed site. The site is currently greenfield. The proposed construction is envisaged to consist of conventional foundations and pavement make up with some local excavations for services and plant. A basement is proposed as part of the proposed scheme which will require excavation of approximately 4.0m BGL.

2.2. Purpose and Scope

The purpose of the site investigation was to investigate subsurface conditions utilising a variety of investigative methods in accordance with the project specification. The scope of the work undertaken for this project included the following:

- Visit project site to observe existing conditions
- Carry out 21 No. Trial Pits to a maximum depth of 3.0m BGL
- Carry out 9 No. Soakaways to determine a soil infiltration value to BRE digest 365
- Carry out 21 No. Dynamic Probes to determine soil strength/density characteristics
- Carry out 9 No. Plate bearing tests to ascertain the subgrade modulus
- Carry out 22 No. Rotary Core Boreholes to a maximum depth of 8.30m BGL
- Installation of 10 No. Groundwater monitoring wells
- Geotechnical & Environmental Laboratory testing
- Report with recommendations

3.0 Subsurface Exploration

3.1. General

During the ground investigation a programme of intrusive investigation specified by the Consulting Engineer was undertaken to determine the sub surface conditions at the proposed site. Regular sampling and in-situ testing was undertaken in the exploratory holes to facilitate the geotechnical descriptions and to enable laboratory testing to be carried out on the soil samples recovered during excavation and drilling.

The procedures used in this site investigation are in accordance with Eurocode 7 Part 2: Ground Investigation and testing (ISEN 1997 – 2:2007) and B.S. 5930:2015.

3.2. Trial Pits

The trial pits were excavated using a JCB 3CX excavator at the locations shown in the exploratory hole location plan in Appendix 1. The locations were checked using a CAT scan to minimise the potential for encountering services during the excavation. The trial pits were sampled, logged and photographed by a Engineering Geologist prior to backfilling with arisings. Notes were made of any services, inclusions, pit stability, groundwater encountered and the characteristics of the strata encountered and are presented on the trial pit logs which are provided in Appendix 2 of this Report.

3.3. Soakaway Testing

The soakaway testing was carried out in selected trial pits at the locations shown in the exploratory hole location plan in Appendix 1. These pits were carefully excavated and filled with water to assess the infiltration characteristics of the proposed site. The pits were allowed to drain and the drop in water level was recorded over time as required by BRE Digest 365. The pits were logged prior to completing the soakaway test and were backfilled with arising's upon completion. The soakaway test results are provided in Appendix 3 of this Report.

3.4. Dynamic Probing

The dynamic probe tests (DPH) were carried out at the locations shown in the location plan in Appendix 1 in accordance with B.S. 1377: Part 9 1990. The test consists of mechanically driving a cone with a 50kg weight in 100mm intervals and monitoring the number of blows required. An equivalent Standard Penetration Test (SPT) 'N' value may be calculated by dividing the total number of blows over a 300mm drive length by 1.5. The dynamic probe logs are provided in Appendix 4 of this Report.

3.5. Insitu Plate Bearing Test

The plate bearing tests were carried out using a 450mm diameter plate at the locations shown on the site plan in Appendix 1. The plate was loaded in increments using a hydraulic jack and an excavator to provide a reaction and the displacement was monitored in accordance with BS1377 Part 9 using independently mounted digital strain gauges. The constrained modulus and equivalent CBR are calculated in accordance with HD29/75 and are provided on the test reports in Appendix 5 of this Report.

3.6. Rotary Boreholes

The rotary coring was carried out by a track mounted T44 Beretta rig at the locations shown on the location plan in Appendix 1. The rotary boreholes were completed from the ground surface or alternatively, where noted on the individual borehole log, from the base of the cable percussion borehole where a temporary liner was installed to facilitate follow-on rotary coring.

The T44 Beretta is equipped with rubber tracks which allow for short travel on pavement surfaces avoiding any damage to the surface. The T44 Beretta utilises a triple tube core barrel system operated using a wireline drilling process. The outer barrel is rotated by the drill rods and at its lower end, carries the coring bit. The inner barrel is mounted on a swivel so that it does not rotate during the process. The third barrel or liner is placed within the second one to retain the core intact and to preserve as much as possible the fabric of the drilling stratum. The core is cut by the coring bit and passes to the inner liner. The core is brought up to the surface within the inner barrel on a small diameter wire rope or line attached to the "overshoot" recovery tool which is then placed into a core box in order of recovery. A drilling fluid, typically air mist or water flush is passed from the surface through hollow drill rods to the drill bit, and is used to cool the drill bit. Temporary casing is used in some situations to support unstable ground or to seal off fissures or voids. It should be noted that the rotary coring can only achieve limited recovery in overburden, particularly granular or weakly cemented strata due to the flushing medium washing away the cohesive fraction during coring. The recovery achieved, where required is noted on the borehole logs and core photographs are provided to allow assessment of the core recovered. The rotary borehole logs are provided in Appendix 6 of this Report.

3.7. Surveying

The exploratory hole locations have been recorded using a KQ GEO Technologies KQ-M8 System which records the coordinates and elevation of the locations to ITM as required by the project specification. The coordinates and elevations are provided on the exploratory hole logs in the appendices of this Report.

3.8. Groundwater/Gas Monitoring Installations

Groundwater and Gas Monitoring Installation were installed upon the completion of the boreholes to enable sampling and the determination of the equilibrium groundwater level. The typical groundwater monitoring installation consists of a 50mm uPVC/HDPE slotted pipe with a pea gravel response zone and bentonite seal installed to the Engineers specification. Where required the standpipe is sealed with a gas tap and finished with a durable steel cover fixed in place with a concrete surround. The installation details are provided on the exploratory hole logs in the appendices of this Report.

3.9. Laboratory Testing

Samples were selected from the exploratory holes for a range of geotechnical and environmental testing to assist in the classification of soils and to provide information for the proposed design.

Environmental & Chemical testing as required by the specification, including the Rilta Suite/Engineers Ireland Suite I, pH and sulphate testing was carried out by Element Materials Technology Laboratory in the UK. The Rilta suite testing includes both Solid Waste and Leachate Waste Acceptance Criteria.

Geotechnical testing consisting of moisture content, Atterberg limits, Particle Size Distribution (PSD), tests were carried out in NMTL's Geotechnical Laboratory in Carlow. Chemical testing to quantify the sulphate and sulphur present in the bedrock was carried out at Sandberg's laboratory in the UK.

Rock strength testing including Point Load (Is_{50}) and Unconfined Compressive Strength (UCS) testing was carried out in James Fishers Geotechnical Laboratory in Portlaois.
The results of the laboratory testing are included in Appendix 7 of this Report.

4.0 Ground Conditions

4.1. General

The ground conditions encountered during the investigation are summarised below with reference to insitu and laboratory test results. The full details of the strata encountered during the ground investigation are provided in the exploratory hole logs included in the appendices of this report.

The sequence of strata encountered were consistent across the site and generally comprised;

- Topsoil
- Made Ground
- Cohesive Deposits
- Granular Deposits
- Weathered Bedrock
- Bedrock

TOPSOIL: Topsoil was encountered in all the exploratory holes and was present to a maximum depth of 0.3m BGL.

MADE GROUND: Made Ground deposits were encountered beneath the Topsoil and were present to a relatively consistent depth of between 0.50m and 0.90m BGL. These deposits were described generally as *brown slightly sandy slightly gravelly CLAY with frequent cobbles and boulders and contained occasional fragments of concrete, red brick, glass, metal and plastic.*

COHESIVE DEPOSITS: Cohesive deposits were encountered beneath the Made Ground and were described typically as *brown slightly sandy slightly gravelly CLAY* or *grey mottled brown slightly sandy slightly gravelly CLAY with occasional cobbles* overlying a *grey slightly sandy slightly gravelly CLAY with occasional cobbles and boulders*. The secondary sand and gravel constituents varied across the site and with depth, with granular lenses occasionally present in the cohesive till matrix. The strength of the cohesive deposits typically increased with depth and was firm to stiff or stiff below 1.30m BGL in the majority of the exploratory holes. These deposits had some, occasional or frequent cobble and boulder content where noted on the exploratory hole logs.

GRANULAR DEPOSITS: Granular deposits were encountered below of the cohesive deposits and were typically described as G*rey brown clayey sandy angular to sub angular fine to coarse GRAVEL with occasional cobbles and rare boulders.* At location TP21 a *grey slightly clayey slightly gravelly fine to coarse SAND with rare cobbles* was encountered. The secondary sand/gravel and silt/clay constituents varied across the site and with depth while occasional or frequent cobble and boulder content also present where noted on the exploratory hole logs.

Based on the SPT N values the deposits are typically medium dense and become dense with depth. It should be noted that many of the trial pits where granular deposits or groundwater were encountered, experienced instability. This was described either as side wall spalling or as side wall collapse in the remarks section at the base of the trial pit logs. A significant groundwater strike was noted in the boreholes on encountering the granular deposits and the driller noted blowing sands or gravels during drilling.

WEATHERED BEDROCK: In the majority of exploratory holes weathered rock was encountered which was digable with the large excavator to a depth of up to 0.90m below the top of the stratum. The trial pits were terminated upon encountering the more competent bedrock, in which further excavation became more difficult. This material was recovered typically as angular gravel and cobbles of Limestone/Mudstone however there was some variability in the fracture spacing and the ease at which the excavator could progress. Some clay and sand were also present with the rock mass either from weathering or as infilling to fractures which were opened upon excavation.

BEDROCK: The rotary core boreholes recovered Weak to Medium strong to very strong dark grey fine to medium grained laminated LIMESTONE interbedded with weak black fine grained laminated Mudstone. This is typical of the Calp Formation, which is noted on the geological mapping to the east of the proposed site. Rare visible pyrite veins were noted during logging which are typically present within the Calp Limestone.

The depth to rock varies from 1.40m BGL in BH16 to a maximum of 3.10m BGL in BH22. The total core recovery is good, typically 100% with some of the uppermost runs dropping to 70% or 92%. The SCR and RQD both are relatively poor in the upper weathered zone, often recovered as non-intact, however both indices show an increase with depth in each of the boreholes.

4.2. Insitu Strength Testing

The correlated DPH blow counts indicate that the overburden deposits are soft or soft to firm to depth of 1.30m and become firm or firm to stiff with depth. DPH03, DPH19 and DPH21 had low blow counts in the soft to firm cohesive deposits depths of between 1.60m to 2.20m BGL which corresponds to the description on trial pit logs.

4.3. Groundwater

Groundwater strikes are noted on the exploratory hole logs where they occurred and where possible drilling was suspended for twenty minutes to allow the subsequent rise in groundwater to be recorded. We would point out that these exploratory holes did not remain open for sufficiently long periods of time to establish the hydrogeological regime and groundwater levels would be expected to vary with the tide, time of year, rainfall, nearby construction and other factors. For this reason, standpipes were installed in RCST01, RCST02, RCST03, RCST04, RCST05 and RCST06 to allow the equilibrium groundwater level to be determined. The groundwater monitoring is included in Appendix 6 of this Report.

4.4. Laboratory Testing

4.4.1. Geotechnical Laboratory Testing

The geotechnical testing carried out on soil samples recovered generally confirm the descriptions on the logs with the primary constituent of the cohesive deposits found to be a CLAY of low to intermediate plasticity. The Particle Size Distribution tests confirm that generally the cohesive deposits are well-graded with percentages of sands and gravels ranging between 16.2% and 57.2% generally with fines contents of 20.1% to 50.1%.
The Particle Size Distribution tests confirm that generally the granular deposits are well-graded/gap graded with percentages of sands/gravels and silt/clay typically of 6.6% with a gravel/sand content of typically 36.2% to 53.8%.

4.4.2. Chemical Laboratory Testing

The pH and sulphate testing carried out in overburden deposits indicate that pH results are near neutral and that the water soluble sulphate results is low when compared to the guideline values from BRE Special Digest 1:2005. The samples tested classify the soil as a Design Sulphate Level DS-1.

In samples testing in rock core the water soluble and acid soluble results are low however the total sulphur is elevated in each of the three samples tested at 1.2%. There is a risk of expansion if the appropriate conditions are met for pyrite to react with air/moisture, however this is considered very small given that construction is likely to of short duration, groundwater will be allowed to rise following basement construction and appropriate mitigation can be included in the basement construction.
Previously sealing with bitumen was advised, however this may not be practical, and this can be further discussed. Sealing all excavations as soon as they are completed with bitumen or 150mm of lean mix upon excavation is recommended.
Reuse of excavated stone below the floor slab is not recommended. Under slab drainage if required, which requires trenches or pits such as sumps or lift pits should not use excavated stone as backfill and should be sealed as soon as excavated. The conditions for differential movement should be considered, which are likely to be low due to the flat nature of the basement construction. The basement should be designed to include for a tanking membrane to prevent groundwater ingress which would also protect against sulphate attack from the groundwater or adjacent strata.
Further review of the results is recommended for specification of concrete where foundations are proposed to be on rock.

4.4.3. Environmental Laboratory Testing

A number of samples were analysed for a suite of parameters which allows for the assessment of the sampled material in terms of total pollutant content for classification of materials as *hazardous* or *non-hazardous*. The suite also allows for the assessment of the sampled material in terms of suitability for placement at licenced landfills (inert, stable non-reactive, hazardous etc.). The parameter list for the suite

includes analysis of the solid samples for arsenic, barium, cadmium, chromium, copper, cyanide, lead, nickel, mercury, zinc, speciated aliphatic and aromatic petroleum hydrocarbons, pH, sulphate, sulphide, moisture content, soil organic matter and an asbestos screen.

The suite also includes those parameters specified in the EU Council Decision establishing criteria for the acceptance of waste at Landfills (Council Decision 2003/33/EC), which for the solid samples are total organic carbon (TOC), speciated aliphatic and aromatic petroleum hydrocarbons, BTEX, phenol, polychlorinated biphenyls (PCB) and PAH.

As part of the suite a leachate is generated from the solid sample which is analysed for antimony, arsenic, barium, cadmium, chromium, copper, lead, mercury, molybdenum, nickel, selenium, zinc, chloride, fluoride, soluble sulphate, sulphide, phenols, dissolved organic carbon (DOC) and total dissolved solids (TDS).

While the laboratory report provides a comparison with the waste acceptance criteria limits it does not provide a waste classification of the material sampled nor does it comment on any potentially hazardous properties of the materials tested. The possibility for contamination, not revealed by the testing undertaken should be borne in mind particularly where Made Ground deposits are present or the previous site use or location indicate a risk of environmental variation. The waste classification report is included under the cover of a sperate report by Ground Investigations Ireland.

The results from the completed laboratory testing is included in Appendix 7 of this report.

5.0 Recommendations & Conclusions

5.1. General

The recommendations given and opinions expressed in this report are based on the findings as detailed in the exploratory hole records. Where an opinion is expressed on the material between exploratory hole locations, this is for guidance only and no liability can be accepted for its accuracy. No responsibility can be accepted for conditions which have not been revealed by the exploratory holes. Limited information has been provided at the ground investigation stage and any designs based on the recommendations or conclusions should be completed in accordance with the current design codes, taking into account the variation and the specific details contained within the exploratory hole logs.

5.2. Foundations

At the location of the proposed structure an allowable bearing capacity of 80 kN/m^2 and 125 kN/m^2 is recommended for conventional strip or pad foundations on the firm to stiff cohesive deposits at the depths outlined in the table below. Where the cohesive deposits are deeper, such as at the location DPH03, DPH13, DPH19 and DPH21, lean mix trench fill to a depth of 2.20m BGL is recommended to achieve the recommended allowable bearing capacity.

	Allowable Bearing Capacities (ABC) kN/m2						
DPH	**ABC**	**Depth**	**Comment**	**DPH**	**ABC**	**Depth**	**Comment**
No.	**kN/m2**	**m BGL**		**No.**	**kN/m2**	**m BGL**	
DPH01	125	1.00	**Cohesive**	**DPH12**	125	1.00	**Cohesive**
DPH02	100	1.00	**Cohesive**	**DPH13**	125	2.10	**Cohesive**
DPH03	125	2.10	**Cohesive**	**DPH14**	80	1.10	**Cohesive**
DPH04	125	1.00	**Cohesive**	**DPH15**	80	1.00	**Granular**
DPH05	125	1.00	**Granular**	**DPH16**	125	1.00	**Cohesive**
DPH06	125	1.40	**Cohesive**	**DPH17**	125	1.00	**Cohesive**
DPH07	80	1.20	**Cohesive**	**DPH18**	125	1.20	**Cohesive**
DPH08	125	1.00	**Cohesive**	**DPH19**	125	2.20	**Granular**
DPH09	100	1.10	**Cohesive**	**DPH20**	125	1.00	**Cohesive**
DPH10	100	1.00	**Cohesive**	**DPH21**	125	1.90	**Granular**
DPH11	100	1.00	**Cohesive**				

Where a higher bearing capacity is required of 500 kN/m2 is achievable on Rock. This bearing capacity may be increased once the point load and unconfined compression testing is completed.

If any part of the site, should part of the foundation be on rock we would recommend that all the foundations of the unit in question be lowered to the competent rock stratum to avoid differential settlement.
Further review of the sulphate test results is recommended for the specification of concrete where foundations are proposed to be on rock.

The possibility for variation in the depth of the made ground in the vicinity of these foundations should be considered and foundation inspections should be carried out. Any soft spots encountered at the proposed foundation depths should be excavated and replaced with lean mix concrete.

A ground bearing floor slab is recommended to be based on the firm to stiff cohesive deposits with an appropriate depth of compacted hardcore specified by the consulting engineer and in accordance with the limits and guidelines in SR21:2014 +A1:2016 and/or NRA SRW CL808 Type E granular stone fill. Where the depth of Made Ground/Soft deposits exceeds 0.9m then suspended floor slabs should be considered.

The pH and sulphate testing completed on samples recovered from the overburden deposits in exploratory holes indicates the pH results are near neutral and the sulphate results are low, when compared to the guideline values from BRE Special Digest 1:2005. No special precautions are required for concrete foundations to prevent sulphate attack. The samples tested were below the limits of DS1 in the BRE Special Digest 1:2005.

5.3. External Pavements

The proposed pavements are recommended to be designed in accordance with the CBR test results included in the Appendices of this Report. The low CBR test results indicate that a capping layer or a sufficient depth of crushed stone fill may be required. Plate bearing tests are recommended at the time of construction to verify the design assumptions for the proposed pavement make up and to verify adequate compaction has been achieved.
The use of a geogrid and separation membrane may improve the performance of the proposed pavement and enable a more economical pavement design to be achieved, a specialist supplier is recommended to advise of the required strength, depth and type of geotextile for the proposed design.

5.4. Excavations

Short term temporary excavations in the cohesive deposits will remain stable for a limited time only and will require to be appropriately battered or the sides supported if the excavation is below 1.25m BGL or is required to permit man entry.
Excavations in the Made Ground or soft Cohesive Deposits will require to be appropriately battered or the sides supported due to the low strength of these deposits.
Any excavations which penetrate the granular deposits will require to be appropriately battered or the sides supported and are likely to require dewatering due to the groundwater seepages noted in the exploratory hole logs in the Appendices of this Report.
The groundwater and stability noted on the trial pit logs should be consulted when determining the most appropriate construction methods for excavations.

Excavations in the upper cohesive and weathered rock deposits are expected to be excavatable with conventional excavation equipment, with zones of more intact bedrock below this depth requiring rock breaking techniques. Based on the fracture spacing, the rock strength testing and Pettifer & Fookes (1994)

Revised Excavatability Graph, the Calp Limestone ranges from hard digging to hard ripping, however the zones recovered as non-intact should be easy to hard digging. The 7T excavator was generally able to excavate to depths of 0.90m below the top of the weathered rock, and became difficult to excavate within the confines of the trial pit on encountering the more competent rock.
Any waste material to be removed off site should be disposed of to a suitably licenced landfill.
The environmental testing completed during the ground investigation is reported under the cover of a separate GII Waste Classification/Subsoil Assessment Report.

5.5. Soakaway Design

Infiltration rates of f=2.967 x 10^{-6} m/s ,3.38 x 10^{-6} m/s and 1.070 x 10^{-5} m/s respectively were calculated for the soakaway locations IT05, IT06 and IT07. At the locations of IT01, IT02, IT03, IT04, IT08 and IT09 the water level dropped too slowly to allow calculation of 'f' the soil infiltration rate. These locations are therefore not recommended as suitable for soakaway design and construction.

The recommendations provided in this report should be verified in the design of the proposed buildings, using the full details of the loading conditions and taking into consideration the allowable tolerable settlements/movements that the building can accommodate. The founding strata should be inspected and verified by a suitably qualified engineer prior to construction of the building foundations.

APPENDIX 1 - Site Location Plan

GROUND INVESTIGATIONS IRELAND
Geotechnical & Environmental

Ground Investigations Ireland Ltd.
Catherinstown House,
Hazelhatch Road,
Newcastle, Co. Dublin
www.gii.ie 01-6015175/5176

Client:
CAIRN PLC

Project Title:
Clonburris Phase 1B

Drawing Title:
Figure 1 Site Location Plan

GII Project Reference:
11543-03-22

Drawn By:
JMD

Date:
26/04/22

APPENDIX 2 – Trial Pit Records

Appendix B : SITE DISCHARGE CALCULATIONS

TITLE	Job Reference
Clonburris T2	210124
SUBJECT	**Calc. Sheet No.**
QBAR Calculation using IOH Report 124 for Sites < 25 km^2 Catchment 1	1

DRAWING NUMBER	Calculations by	Checked by	Date
	DCB	JPC	11/30/2022

Estimation of QBAR from IOH Report 124 for catchments less than 25 km^2 using the 3 variable equation

$$^{1}Q_{bar} = 0.00108 * (AREA)^{0.89}(SAAR)^{1.17}(SOIL)^{2.17}$$

^{2}Site Area = 4.300 Ha

Site area is less than 50 Ha, calculate Qbar for a 50 Ha Site then pro-rata

AREA =	0.043	km^2
SAAR =	773	mm
^{3}SOIL =	0.37	
Q_{bar} =	0.00006	cumecs/Ha
Q_{bar} =	3.23	l/s/Ha
9.95 (SDZ) $Q_{bar\ [rural]}$ =	13.88	l/s

Note to Institute of Hydrology Report No. 124 Eqn

Q_{bar}	The Mean Annual Flood (cumecs)
AREA	Area of the Catchment (km^2)
SAAR	Standard Annual Average Rainfall (mm) *NERC Flood Studies Report, 1975*
SOIL	Soil Index Values of Catchment *Winter Rain Acceptance Potential, (Supplementary Report No. 7)*

Soil Classification for Runoff Potential FSR Maps

Soil 1	0	%
Soil 2	0	%
Soil 3	100	%
Soil 4	0	%
Soil 5	0	%

^{4}QBar from Site with Factorial Error Allowance

r^2 =	0.847
n =	71
fse =	1.651

Q'_{bar} =	22.92	l/s

(With Allowance for the standard factorial error)

Is longterm storage provided? No

Permissible Outflow from Site using Growth Factor

Qbar growth for permitted outlows from site for given return period (assuming long term storage). (No allowance for standard factorial error)

Flood Return Event	^{5}Growth Factor	Permitted Flow (l/s)
1	0.85	11.8
QBAR	1	13.9
10	1.67	23.2
30	2.1	29.1
50	2.33	32.3
100	2.6	36.1
200	2.85	39.6
1000	3.5	48.6

Storm Return Period to be provided for =	30	Years *
QBAR (Growth) =	29.1	Litres/sec
^{6}Permissible Outflow from site =	13.88	Litres/sec
^{7}Maximum Allowable Outflow from site =	13.88	Litres/sec

(* 30, 50 or 100)

1 hectare = 10,000m^2 **1km^2 - 100 hectares**

Notes

1. Based on the Institute of Hydrology Report 124 for small catchments less than 25km^2.
2. For catchments smaller than 50 hectares in area, flow rates are linearly interpolated for smaller areas.
3. Soil index value (SPR) calculated from Flood Studies Report Vol V Fig I 4.18(1) - The Classification of Soils from Winter Rainfall Acceptance Rate .
4. Fse is the standard factoial error
5. QBAR multiplied by growth factors of 0.85 for 1 year, 2.1 for 30 year, 2.3 for 50 and 2.6 for 100 year return period events, from GDSDS Figure C2.
6. Total Permissible Outflow - $QBAR_{(RURAL)}$ calculated in accordance with GDSDS - Regional Drainage Policies (Volume 2 - Chapter 6), i.e. $QBAR(m3/s)=0.00108x(Area)^{0.89}(SAAR)^{1.17}(SOIL)^{2.17}$
7. Where Total Permissible Outflow is less than 2.0l/s and not achievable, use 2.0l/s.
8. Rainfall depth for 100 year return period, 6 hour duration with additional 10% for climate change. (Value from Dublin Airport)
9. Interception Volume Vt (m3) = Impermeable Area (ha) x 10mm x 10 (GDSDS, Vol 2, Section 6.3.1.2.1).

Appendix C : SURFACE WATER NETWORK CALCULATIONS FOR THE 100 YEAR ANNUAL EXCEEDANCE PROBABILITY

STORM SEWER DESIGN by the Modified Rational Method

Design Criteria for SW_1

Pipe Sizes STANDARD Manhole Sizes STANDARD

FSR Rainfall Model - Scotland and Ireland

Return Period (years)	100	PIMP (%)	100
M5-60 (mm)	17.000	Add Flow / Climate Change (%)	20
Ratio R	0.277	Minimum Backdrop Height (m)	0.200
Maximum Rainfall (mm/hr)	50	Maximum Backdrop Height (m)	1.500
Maximum Time of Concentration (mins)	30	Min Design Depth for Optimisation (m)	1.200
Foul Sewage (l/s/ha)	0.000	Min Vel for Auto Design only (m/s)	1.00
Volumetric Runoff Coeff.	0.750	Min Slope for Optimisation (1:X)	500

Designed with Level Inverts

Time Area Diagram for SW_1

Time (mins)	Area (ha)	Time (mins)	Area (ha)	Time (mins)	Area (ha)
0-4	1.038	4-8	2.445	8-12	0.118

Total Area Contributing (ha) = 3.600

Total Pipe Volume (m³) = 153.687

Network Design Table for SW_1

« - Indicates pipe capacity < flow

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
S1.000	46.012	0.161	285.8	0.174	4.00	0.0	0.600	o	300	Pipe/Conduit	
S1.001	44.624	0.161	277.2	0.174	0.00	0.0	0.600	o	375	Pipe/Conduit	
S2.000	38.566	0.193	199.8	0.050	4.00	0.0	0.600	o	300	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
S1.000	50.00	4.83	58.200	0.174	0.0	0.0	4.7	0.92	65.4	28.3
S1.001	50.00	5.52	58.039	0.348	0.0	0.0	9.4	1.08	119.7	56.5
S2.000	50.00	4.58	58.072	0.050	0.0	0.0	1.4	1.11	78.4	8.1

Network Design Table for SW_1

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
S1.002	6.340	0.023	275.7	0.174	0.00	0.0	0.600	o	600	Pipe/Conduit	
S3.000	31.867	0.159	200.4	0.174	4.00	0.0	0.600	o	300	Pipe/Conduit	
S3.001	54.380	0.272	199.9	0.174	0.00	0.0	0.600	o	375	Pipe/Conduit	
S1.003	48.791	0.175	278.8	0.174	0.00	0.0	0.600	o	600	Pipe/Conduit	
S4.000	25.995	0.130	200.0	0.174	4.00	0.0	0.600	o	450	Pipe/Conduit	
S1.004	23.381	0.081	288.7	0.174	0.00	0.0	0.600	o	600	Pipe/Conduit	
S1.005	39.249	0.200	196.2	0.438	0.00	0.0	0.600	o	1000	Pipe/Conduit	
S5.000	45.984	0.137	335.6	0.100	4.00	0.0	0.600	o	300	Pipe/Conduit	
S5.001	48.481	0.137	353.9	0.100	0.00	0.0	0.600	o	375	Pipe/Conduit	
S5.002	45.153	0.144	313.6	0.100	0.00	0.0	0.600	o	450	Pipe/Conduit	
S5.003	6.958	0.021	331.3	0.270	0.00	0.0	0.600	o	450	Pipe/Conduit	
S5.004	11.765	0.035	336.1	0.250	0.00	0.0	0.600	o	450	Pipe/Conduit	
S5.005	20.769	0.062	335.0	0.000	0.00	0.0	0.600	o	450	Pipe/Conduit	
S5.006	17.409	0.052	334.8	0.000	0.00	0.0	0.600	o	450	Pipe/Conduit	
S5.007	4.395	0.013	338.1	0.000	0.00	0.0	0.600	o	450	Pipe/Conduit	
S1.006	19.629	0.142	138.2	0.000	0.00	0.0	0.600	o	525	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
S1.002	50.00	5.59	57.879	0.572	0.0	0.0	15.5	1.46	413.3	92.9
S3.000	50.00	4.48	58.344	0.174	0.0	0.0	4.7	1.11	78.2	28.3
S3.001	50.00	5.19	58.128	0.348	0.0	0.0	9.4	1.28	141.1	56.5
S1.003	50.00	6.15	57.856	1.094	0.0	0.0	29.6	1.45	411.0	177.8
S4.000	50.00	4.30	57.811	0.174	0.0	0.0	4.7	1.43	228.1	28.3
S1.004	50.00	6.42	57.681	1.442	0.0	0.0	39.1	1.43	403.8	234.3
S1.005	50.00	6.69	57.600	1.880	0.0	0.0	50.9	2.38	1872.1	305.5
S5.000	50.00	4.90	58.000	0.100	0.0	0.0	2.7	0.85	60.3	16.2
S5.001	50.00	5.74	57.863	0.200	0.0	0.0	5.4	0.96	105.7	32.5
S5.002	50.00	6.40	57.726	0.300	0.0	0.0	8.1	1.14	181.7	48.7
S5.003	50.00	6.51	57.582	0.570	0.0	0.0	15.4	1.11	176.7	92.6
S5.004	50.00	6.68	57.561	0.820	0.0	0.0	22.2	1.10	175.5	133.2
S5.005	50.00	7.00	57.526	0.820	0.0	0.0	22.2	1.11	175.8	133.2
S5.006	50.00	7.26	57.464	0.820	0.0	0.0	22.2	1.11	175.8	133.2
S5.007	50.00	7.33	57.412	0.820	0.0	0.0	22.2	1.10	175.0	133.2
S1.006	50.00	7.50	57.400	2.700	0.0	0.0	73.1	1.90	412.0«	438.7

Network Design Table for SW_1

PN	Length (m)	Fall (m)	Slope (1:X)	I.Area (ha)	T.E. (mins)	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
S1.007	18.585	0.136	136.7	0.000	0.00	0.0	0.600	o	525	Pipe/Conduit	
S1.008	14.809	0.095	155.9	0.000	0.00	0.0	0.600	o	525	Pipe/Conduit	
S1.009	66.708	0.148	450.7	0.150	0.00	0.0	0.600	o	525	Pipe/Conduit	
S1.010	31.537	0.097	325.1	0.150	0.00	0.0	0.600	o	525	Pipe/Conduit	
S6.000	58.353	0.397	147.0	0.150	4.00	0.0	0.600	o	300	Pipe/Conduit	
S6.001	7.450	0.051	146.1	0.150	0.00	0.0	0.600	o	300	Pipe/Conduit	
S6.002	54.179	0.368	147.2	0.150	0.00	0.0	0.600	o	300	Pipe/Conduit	
S6.003	45.849	0.309	148.4	0.150	0.00	0.0	0.600	o	450	Pipe/Conduit	
S1.011	15.282	0.034	449.5	0.000	0.00	0.0	0.600	o	525	Pipe/Conduit	

Network Results Table

PN	Rain (mm/hr)	T.C. (mins)	US/IL (m)	Σ I.Area (ha)	Σ Base Flow (l/s)	Foul (l/s)	Add Flow (l/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
S1.007	50.00	7.66	57.258	2.700	0.0	0.0	73.1	1.91	414.4«	438.7
S1.008	50.00	7.80	57.122	2.700	0.0	0.0	73.1	1.79	387.8«	438.7
S1.009	50.00	8.86	57.028	2.850	0.0	0.0	77.2	1.05	227.0«	463.1
S1.010	50.00	9.28	56.880	3.000	0.0	0.0	81.2	1.24	267.7«	487.5
S6.000	50.00	4.75	57.906	0.150	0.0	0.0	4.1	1.29	91.5	24.4
S6.001	50.00	4.85	57.510	0.300	0.0	0.0	8.1	1.30	91.8	48.7
S6.002	50.00	5.54	57.460	0.450	0.0	0.0	12.2	1.29	91.4	73.1
S6.003	50.00	6.00	57.092	0.600	0.0	0.0	16.2	1.67	265.1	97.5
S1.011	50.00	9.53	56.783	3.600	0.0	0.0	97.5	1.05	227.3«	585.0

PIPELINE SCHEDULES for SW_1

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S1.000	o	300	SM2	59.668	58.200	1.168	Open Manhole	1200
S1.001	o	375	SM2	59.759	58.039	1.345	Open Manhole	1200
S2.000	o	300	SM3	59.707	58.072	1.335	Open Manhole	1200
S1.002	o	600	SM3	59.517	57.879	1.038	Open Manhole	1500
S3.000	o	300	SM9	59.600	58.344	0.956	Open Manhole	1200
S3.001	o	375	SM10	59.600	58.128	1.097	Open Manhole	1200
S1.003	o	600	SM4	59.485	57.856	1.029	Open Manhole	1500
S4.000	o	450	SM12	59.020	57.811	0.759	Open Manhole	1200
S1.004	o	600	SM5	59.503	57.681	1.222	Open Manhole	1500
S1.005	o	1000	SM6	59.458	57.600	0.858	Open Manhole	1900
S5.000	o	300	SM11	59.660	58.000	1.360	Open Manhole	1200
S5.001	o	375	SM12	59.240	57.863	1.002	Open Manhole	1200
S5.002	o	450	SM13	58.966	57.726	0.790	Open Manhole	1200
S5.003	o	450	SM14	59.512	57.582	1.480	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S1.000	46.012	285.8	SM2	59.759	58.039	1.420	Open Manhole	1200
S1.001	44.624	277.2	SM3	59.517	57.878	1.264	Open Manhole	1500
S2.000	38.566	199.8	SM3	59.517	57.879	1.338	Open Manhole	1500
S1.002	6.340	275.7	SM4	59.485	57.856	1.029	Open Manhole	1500
S3.000	31.867	200.4	SM10	59.600	58.185	1.115	Open Manhole	1200
S3.001	54.380	199.9	SM4	59.485	57.856	1.254	Open Manhole	1500
S1.003	48.791	278.8	SM5	59.503	57.681	1.222	Open Manhole	1500
S4.000	25.995	200.0	SM5	59.503	57.681	1.372	Open Manhole	1500
S1.004	23.381	288.7	SM6	59.458	57.600	1.258	Open Manhole	1900
S1.005	39.249	196.2	SM7	59.316	57.400	0.916	Open Manhole	1900
S5.000	45.984	335.6	SM12	59.240	57.863	1.077	Open Manhole	1200
S5.001	48.481	353.9	SM13	58.966	57.726	0.865	Open Manhole	1200
S5.002	45.153	313.6	SM14	59.512	57.582	1.480	Open Manhole	1200
S5.003	6.958	331.3	SM15	59.556	57.561	1.545	Open Manhole	1200

PIPELINE SCHEDULES for SW_1

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S5.004	o	450	SM15	59.556	57.561	1.545	Open Manhole	1200
S5.005	o	450	SM16	59.618	57.526	1.642	Open Manhole	1200
S5.006	o	450	SM17	59.477	57.464	1.563	Open Manhole	1200
S5.007	o	450	SM18	59.269	57.412	1.407	Open Manhole	1200
S1.006	o	525	SM7	59.316	57.400	1.391	Open Manhole	1900
S1.007	o	525	SM8	59.397	57.258	1.614	Open Manhole	1500
S1.008	o	525	SM9	59.560	57.122	1.913	Open Manhole	1500
S1.009	o	525	SM10	58.788	57.028	1.235	Open Manhole	1500
S1.010	o	525	SM23	59.240	56.880	1.835	Open Manhole	1500
S6.000	o	300	SM24	59.191	57.906	0.985	Open Manhole	1200
S6.001	o	300	SM25	59.214	57.510	1.404	Open Manhole	1200
S6.002	o	300	SM26	59.500	57.460	1.740	Open Manhole	1200
S6.003	o	450	SM27	59.463	57.092	1.921	Open Manhole	1200
S1.011	o	525	SM24	59.130	56.783	1.822	Open Manhole	1500

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
S5.004	11.765	336.1	SM16	59.618	57.526	1.642	Open Manhole	1200
S5.005	20.769	335.0	SM17	59.477	57.464	1.563	Open Manhole	1200
S5.006	17.409	334.8	SM18	59.269	57.412	1.407	Open Manhole	1200
S5.007	4.395	338.1	SM7	59.316	57.399	1.467	Open Manhole	1900
S1.006	19.629	138.2	SM8	59.397	57.258	1.614	Open Manhole	1500
S1.007	18.585	136.7	SM9	59.560	57.122	1.913	Open Manhole	1500
S1.008	14.809	155.9	SM10	58.788	57.027	1.236	Open Manhole	1500
S1.009	66.708	450.7	SM23	59.240	56.880	1.835	Open Manhole	1500
S1.010	31.537	325.1	SM24	59.130	56.783	1.822	Open Manhole	1500
S6.000	58.353	147.0	SM25	59.214	57.509	1.405	Open Manhole	1200
S6.001	7.450	146.1	SM26	59.500	57.459	1.741	Open Manhole	1200
S6.002	54.179	147.2	SM27	59.463	57.092	2.071	Open Manhole	1200
S6.003	45.849	148.4	SM24	59.130	56.783	1.897	Open Manhole	1500
S1.011	15.282	449.5	SM	59.265	56.749	1.991	Open Manhole	0

Area Summary for SW_1

Pipe Number	PIMP Type	PIMP Name	PIMP (%)	Gross Area (ha)	Imp. Area (ha)	Pipe Total (ha)
1.000	-	-	100	0.174	0.174	0.174
1.001	-	-	100	0.174	0.174	0.174
2.000	-	-	100	0.050	0.050	0.050
1.002	-	-	100	0.174	0.174	0.174
3.000	-	-	100	0.174	0.174	0.174
3.001	-	-	100	0.174	0.174	0.174
1.003	-	-	100	0.174	0.174	0.174
4.000	-	-	100	0.174	0.174	0.174
1.004	-	-	100	0.174	0.174	0.174
1.005	-	-	100	0.438	0.438	0.438
5.000	-	-	100	0.100	0.100	0.100
5.001	-	-	100	0.100	0.100	0.100
5.002	-	-	100	0.100	0.100	0.100
5.003	-	-	100	0.270	0.270	0.270
5.004	-	-	100	0.250	0.250	0.250
5.005	-	-	100	0.000	0.000	0.000
5.006	-	-	100	0.000	0.000	0.000
5.007	-	-	100	0.000	0.000	0.000
1.006	-	-	100	0.000	0.000	0.000
1.007	-	-	100	0.000	0.000	0.000
1.008	-	-	100	0.000	0.000	0.000
1.009	-	-	100	0.150	0.150	0.150
1.010	-	-	100	0.150	0.150	0.150
6.000	-	-	100	0.150	0.150	0.150
6.001	-	-	100	0.150	0.150	0.150
6.002	-	-	100	0.150	0.150	0.150
6.003	-	-	100	0.150	0.150	0.150
1.011	-	-	100	0.000	0.000	0.000
				Total	Total	Total
				3.600	3.600	3.600

Free Flowing Outfall Details for SW_1

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
S1.011	SM	59.265	56.749	0.000	0	0

Simulation Criteria for SW_1

Volumetric Runoff Coeff	0.750	Additional Flow - % of Total Flow	0.000
Areal Reduction Factor	1.000	MADD Factor * 10m³/ha Storage	2.000
Hot Start (mins)	0	Inlet Coeffiecient	0.800
Hot Start Level (mm)	0	Flow per Person per Day (l/per/day)	0.000
Manhole Headloss Coeff (Global)	0.500	Run Time (mins)	60
Foul Sewage per hectare (l/s)	0.000	Output Interval (mins)	1

Number of Input Hydrographs	0	Number of Storage Structures	1
Number of Online Controls	1	Number of Time/Area Diagrams	0
Number of Offline Controls	0	Number of Real Time Controls	0

Synthetic Rainfall Details

Rainfall Model	FSR	Profile Type	Summer
Return Period (years)	100	Cv (Summer)	0.750
Region	Scotland and Ireland	Cv (Winter)	0.840
M5-60 (mm)	17.000	Storm Duration (mins)	30
Ratio R	0.277		

Online Controls for SW_1

Hydro-Brake® Optimum Manhole: SM7, DS/PN: S1.006, Volume (m³): 35.2

Unit Reference	MD-SHE-0164-1330-1100-1330
Design Head (m)	1.100
Design Flow (l/s)	13.3
Flush-Flo™	Calculated
Objective	Minimise upstream storage
Application	Surface
Sump Available	Yes
Diameter (mm)	164
Invert Level (m)	57.400
Minimum Outlet Pipe Diameter (mm)	225
Suggested Manhole Diameter (mm)	1200

Control Points	Head (m)	Flow (l/s)
Design Point (Calculated)	1.100	13.3
Flush-Flo™	0.339	13.3
Kick-Flo®	0.746	11.1
Mean Flow over Head Range	-	11.4

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

Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)	Depth (m)	Flow (l/s)
0.100	5.9	1.200	13.8	3.000	21.4	7.000	32.2
0.200	12.7	1.400	14.9	3.500	23.1	7.500	33.3
0.300	13.2	1.600	15.9	4.000	24.6	8.000	34.3
0.400	13.2	1.800	16.8	4.500	26.0	8.500	35.4
0.500	13.0	2.000	17.6	5.000	27.4	9.000	36.3
0.600	12.6	2.200	18.5	5.500	28.7	9.500	37.3
0.800	11.4	2.400	19.3	6.000	29.9		
1.000	12.7	2.600	20.0	6.500	31.1		

Storage Structures for SW_1

Tank or Pond Manhole: SM7, DS/PN: S1.006

Invert Level (m) 57.400

Depth (m)	Area (m²)	Depth (m)	Area (m²)	Depth (m)	Area (m²)
0.000	1400.0	1.110	2250.0	1.410	0.0
1.100	1400.0	1.400	2250.0		

Summary of Critical Results by Maximum Level (Rank 1) for SW_1

Simulation Criteria

Areal Reduction Factor	1.000	Additional Flow - % of Total Flow	0.000
Hot Start (mins)	0	MADD Factor * 10m³/ha Storage	2.000
Hot Start Level (mm)	0	Inlet Coeffiecient	0.800
Manhole Headloss Coeff (Global)	0.500	Flow per Person per Day (l/per/day)	0.000
Foul Sewage per hectare (l/s)	0.000		

Number of Input Hydrographs	0	Number of Storage Structures	1
Number of Online Controls	1	Number of Time/Area Diagrams	0
Number of Offline Controls	0	Number of Real Time Controls	0

Synthetic Rainfall Details

Rainfall Model		FSR	Ratio R 0.303
Region	Scotland and Ireland	Cv (Summer)	0.750
M5-60 (mm)	17.200	Cv (Winter)	0.840

Margin for Flood Risk Warning (mm)	300.0	DVD Status	ON
Analysis Timestep	Fine	Inertia Status	ON
DTS Status	OFF		

Profile(s)	Summer and Winter
Duration(s) (mins)	15, 30, 60, 120, 180, 240, 360, 720, 1440, 2880, 7200, 10080
Return Period(s) (years)	30, 100
Climate Change (%)	20, 20

PN	US/MH Name	Event	Duration (mins)	US/CL (m)	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)
S1.000	SM2	15 minute 100 year Winter I+20%	15	59.668	59.074	0.574	0.000
S1.001	SM2	15 minute 100 year Winter I+20%	15	59.759	58.898	0.484	0.000
S2.000	SM3	15 minute 100 year Winter I+20%	15	59.707	58.701	0.329	0.000
S1.002	SM3	15 minute 100 year Winter I+20%	15	59.517	58.676	0.197	0.000
S3.000	SM9	15 minute 100 year Winter I+20%	15	59.600	59.042	0.398	0.000
S3.001	SM10	15 minute 100 year Winter I+20%	15	59.600	58.916	0.413	0.000
S1.003	SM4	15 minute 100 year Winter I+20%	15	59.485	58.655	0.199	0.000
S4.000	SM12	15 minute 100 year Winter I+20%	15	59.020	58.552	0.291	0.000
S1.004	SM5	15 minute 100 year Winter I+20%	15	59.503	58.438	0.157	0.000
S1.005	SM6	720 minute 100 year Winter I+20%	720	59.458	58.371	-0.229	0.000
S5.000	SM11	15 minute 100 year Winter I+20%	15	59.660	59.088	0.788	0.000
S5.001	SM12	15 minute 100 year Winter I+20%	15	59.240	59.028	0.790	0.000
S5.002	SM13	15 minute 100 year Winter I+20%	15	58.966	58.958	0.782	0.000
S5.003	SM14	15 minute 100 year Winter I+20%	15	59.512	58.863	0.831	0.000
S5.004	SM15	15 minute 100 year Winter I+20%	15	59.556	58.753	0.742	0.000
S5.005	SM16	15 minute 100 year Winter I+20%	15	59.618	58.512	0.536	0.000
S5.006	SM17	720 minute 100 year Winter I+20%	720	59.477	58.368	0.454	0.000
S5.007	SM18	720 minute 100 year Winter I+20%	720	59.269	58.366	0.504	0.000
S1.006	SM7	720 minute 100 year Winter I+20%	720	59.316	58.366	0.441	0.000
S1.007	SM8	120 minute 100 year Summer I+20%	120	59.397	57.499	-0.284	0.000
S1.008	SM9	120 minute 100 year Summer I+20%	120	59.560	57.495	-0.152	0.000

Summary of Critical Results by Maximum Level (Rank 1) for SW_1

PN	US/MH Name	Flow / Cap.	Overflow (l/s)	Maximum Vol (m³)	Pipe Flow (l/s)	Status
S1.000	SM2	1.04		0.983	63.6	SURCHARGED
S1.001	SM2	1.14		4.133	124.8	SURCHARGED
S2.000	SM3	0.25		0.706	17.9	SURCHARGED
S1.002	SM3	0.92		8.811	206.5	SURCHARGED
S3.000	SM9	0.90		0.784	64.1	SURCHARGED
S3.001	SM10	0.95		3.053	124.6	SURCHARGED
S1.003	SM4	1.08		8.629	386.7	SURCHARGED
S4.000	SM12	0.34		0.832	65.3	SURCHARGED
S1.004	SM5	1.60		18.540	502.7	SURCHARGED
S1.005	SM6	0.06		8.301	71.1	OK
S5.000	SM11	0.55		1.225	31.3	SURCHARGED
S5.001	SM12	0.63		4.477	61.7	FLOOD RISK
S5.002	SM13	0.57		6.610	93.8	FLOOD RISK
S5.003	SM14	1.70		8.434	179.9	SURCHARGED
S5.004	SM15	2.25		2.258	270.6	SURCHARGED
S5.005	SM16	1.86		2.790	265.8	SURCHARGED
S5.006	SM17	0.24		4.129	33.1	SURCHARGED
S5.007	SM18	0.28		3.652	33.0	SURCHARGED
S1.006	SM7	0.05		1381.796	13.2	SURCHARGED
S1.007	SM8	0.06		1.487	15.8	OK
S1.008	SM9	0.09		2.823	21.6	OK

REQUIRED ATTENUATION VOLUME WITHIN ATTENUATION POND

Summary of Critical Results by Maximum Level (Rank 1) for SW_1

PN	US/MH Name	Event	Duration (mins)	US/CL (m)	Water Level (m)	Surcharged Depth (m)	Flooded Volume (m³)
S1.009	SM10	60 minute 100 year Summer I+20%	60	58.788	57.491	-0.062	0.000
S1.010	SM23	30 minute 100 year Summer I+20%	30	59.240	57.409	0.004	0.000
S6.000	SM24	15 minute 100 year Winter I+20%	15	59.191	59.027	0.821	0.000
S6.001	SM25	15 minute 100 year Winter I+20%	15	59.214	58.892	1.082	0.000
S6.002	SM26	15 minute 100 year Winter I+20%	15	59.500	58.731	0.971	0.000
S6.003	SM27	15 minute 100 year Winter I+20%	15	59.463	57.588	0.046	0.000
S1.011	SM24	15 minute 100 year Winter I+20%	15	59.130	57.380	0.072	0.000

PN	US/MH Name	Flow / Cap.	Overflow (l/s)	Maximum Vol (m³)	Pipe Flow (l/s)	Status
S1.009	SM10	0.20		3.174	42.2	OK
S1.010	SM23	0.36		13.503	80.6	SURCHARGED
S6.000	SM24	0.56		1.263	49.1	FLOOD RISK
S6.001	SM25	1.62		5.599	99.6	SURCHARGED
S6.002	SM26	1.73		1.874	149.4	SURCHARGED
S6.003	SM27	0.81		3.802	192.9	SURCHARGED
S1.011	SM24	1.98		13.999	273.2	SURCHARGED

Appendix D : IRISH WATER CONFIRMATION OF FEASIBILITY

Carin Homes PLC C/O John Carr
DBFL Consulting,
Ormond House,
Upper Ormond Quay,
Dublin 7,
Dublin

Uisce Éireann
Bosca OP 6000
Baile Átha Cliath 1
Éire

Irish Water
PO Box 6000
Dublin 1
Ireland

T: +353 1 89 25000
F: +353 1 89 25001
www.water.ie

04 February 2020

Dear Sir/Madam,

Re: Customer Reference No 2512559856 pre-connection enquiry - Subject to contract | Contract denied
Connection for Housing Development of 1500 Units at Clonburris Little, Clondalkin, Co Dublin

Irish Water has reviewed your pre-connection enquiry in relation to water and wastewater connections at Clonburris Little, Clondalkin, Co Dublin
. Based upon the details that you have provided with your pre-connection enquiry and on the capacity currently available in the network(s), as assessed by Irish Water, we wish to advise you that, subject to a valid connection agreement being put in place, your proposed connection to the Irish Water network(s) can be facilitated.

In the case of wastewater connections this assessment does not confirm that a gravity connection is achievable. Therefore a suitably sized pumping station may be required to be installed on your site. All infrastructure should be designed and installed in accordance with the Irish Water Code of Practice.

All infrastructure should be designed and installed in accordance with
- the Clonburris Master Plan approved by Irish Water
- the Development phasing and connection timelines of each phase agreed with Irish Water
- the Irish Water Codes of Practice and Standard Details.

All infrastructure should be designed and installed in accordance with the Irish Water Codes of Practice and Standard Details. A design proposal for the water and/or wastewater infrastructure should be submitted to Irish Water for assessment. Prior to submitting your planning application, you are required to submit these detailed design proposals to Irish Water for review.

You are advised that this correspondence does not constitute an offer in whole or in part to provide a connection to any Irish Water infrastructure and is provided subject to a connection agreement being signed at a later date.

A connection agreement can be applied for by completing the connection application form available at **www.water.ie/connections**. Irish Water's current charges for water and wastewater connections are set out in the Water Charges Plan as approved by the Commission for Regulation of Utilities.

If you have any further questions, please contact James O'Sullivan from the design team on 022 52269 or email jameosull@water.ie. For further information, visit **www.water.ie/connections**

Yours sincerely,

Maria O'Dwyer
Connections and Developer Services

CONFIRMATION OF FEASIBILITY

Uisce Éireann
Bosca OP 448
Oifig Sheachadta na
Cathrach Theas
Cathair Chorcaí

Irish Water
PO Box 448,
South City
Delivery Office,
Cork City.

www.water.ie

Dieter Bester
DBFL Consulting Engineers Ltd.
Ormond House
Ormond Quay Upper
Dublin

23 September 2022

Our Ref: CDS22005723 Pre-Connection Enquiry
Clonburris Little, Clondalkin, Dublin

Dear Applicant/Agent,

We have completed the review of the Pre-Connection Enquiry.

Irish Water has reviewed the pre-connection enquiry in relation to a Water & Wastewater connection for a Housing Development of 468 unit(s) at Clonburris Little, Clondalkin, Dublin, (the **Development)**.

Based upon the details provided we can advise the following regarding connecting to the networks;

- **Water Connection**
  - Feasible Subject to upgrades.
  -
  - The Development is within Clonburris Strategic Development Zone. All relevant core water infrastructure within the Zone has to be completed, of adequate capacity and integrity, connected to the Irish Water networks and in operation prior the connection. The infrastructure will be delivered by Clonburris Infrastructure Limited developers.
  - A bulk meter will be required on the connection main.

• **Wastewater Connection**	- Feasible Subject to upgrades. - The Development is within Clonburris Strategic Development Zone. All relevant core wastewater infrastructures within the Zone has to be completed, of adequate capacity and integrity, connected to the Irish Water networks and in operation prior the connection. The infrastructure will be delivered by Clonburris Infrastructure Limited developers only. Pumping Station PS01 will be delivered by Irish Water, and it is scheduled to be completed by Q3/2025 (this may be subject to change)

This letter does not constitute an offer, in whole or in part, to provide a connection to any Irish Water infrastructure. Before the Development can be connected to our network(s) you must submit a connection application and be granted and sign a connection agreement with Irish Water.

As the network capacity changes constantly, this review is only valid at the time of its completion. As soon as planning permission has been granted for the Development, a completed connection application should be submitted. The connection application is available at www.water.ie/connections/get-connected/

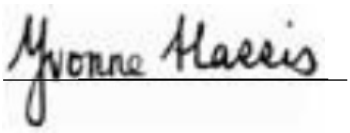
Where can you find more information?

- **Section A -** What is important to know?
- **Section B -** Details of Irish Water's Network(s)

This letter is issued to provide information about the current feasibility of the proposed connection(s) to Irish Water's network(s). This is not a connection offer and capacity in Irish Water's network(s) may only be secured by entering into a connection agreement with Irish Water.

For any further information, visit **www.water.ie/connections**, **email** **newconnections@water.ie** or contact 1800 278 278**.**

Yours sincerely,



Yvonne Harris
Head of Customer Operations

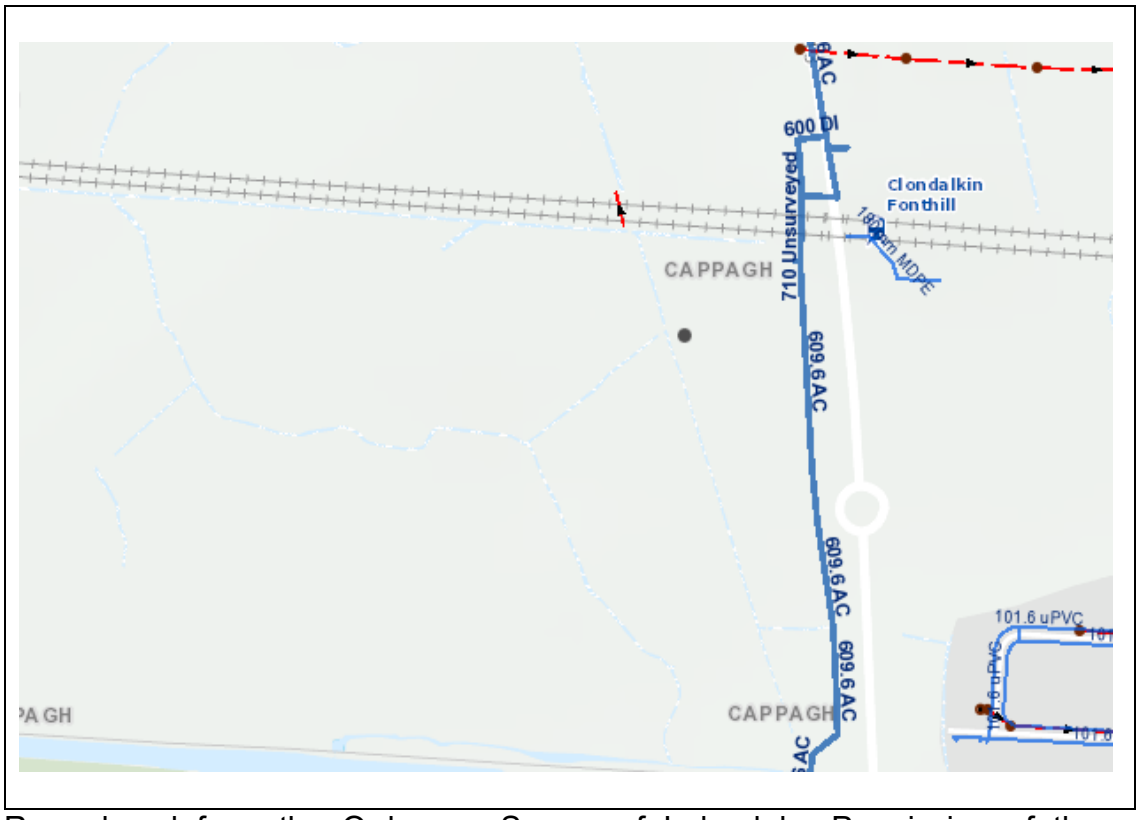
Section A - What is important to know?

What is important to know?	Why is this important?
Do you need a contract to connect?	• Yes, a contract is required to connect. This letter does not constitute a contract or an offer in whole or in part to provide a connection to Irish Water's network(s). • Before the Development can connect to Irish Water's network(s), you must submit a connection application and be granted and sign a connection agreement with Irish Water.
When should I submit a Connection Application?	• A connection application should only be submitted after planning permission has been granted.
Where can I find information on connection charges?	• Irish Water connection charges can be found at: https://www.water.ie/connections/information/charges/
Who will carry out the connection work?	• All works to Irish Water's network(s), including works in the public space, must be carried out by Irish Water*. *Where a Developer has been granted specific permission and has been issued a connection offer for Self-Lay in the Public Road/Area, they may complete the relevant connection works
Fire flow Requirements	• The Confirmation of Feasibility does not extend to fire flow requirements for the Development. Fire flow requirements are a matter for the Developer to determine. • **What to do?** - Contact the relevant Local Fire Authority
Plan for disposal of storm water	• The Confirmation of Feasibility does not extend to the management or disposal of storm water or ground waters. • **What to do?** - Contact the relevant Local Authority to discuss the management or disposal of proposed storm water or ground water discharges.
Where do I find details of Irish Water's network(s)?	• Requests for maps showing Irish Water's network(s) can be submitted to: datarequests@water.ie

What are the design requirements for the connection(s)?	• The design and construction of the Water & Wastewater pipes and related infrastructure to be installed in this Development shall comply with ***the Irish Water Connections and Developer Services Standard Details and Codes of Practice,*** available at www.water.ie/connections
Trade Effluent Licensing	• Any person discharging trade effluent** to a sewer, must have a Trade Effluent Licence issued pursuant to section 16 of the Local Government (Water Pollution) Act, 1977 (as amended). • More information and an application form for a Trade Effluent License can be found at the following link: https://www.water.ie/business/trade-effluent/about/ **trade effluent is defined in the Local Government (Water Pollution) Act, 1977 (as amended)

Section B – Details of Irish Water's Network(s)

The map included below outlines the current Irish Water infrastructure adjacent the Development: To access Irish Water Maps email datarequests@water.ie



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Note: The information provided on the included maps as to the position of Irish Water's underground network(s) is provided as a general guide only. The information is based on the best available information provided by each Local Authority in Ireland to Irish Water.

Whilst every care has been taken in respect of the information on Irish Water's network(s), Irish Water assumes no responsibility for and gives no guarantees, undertakings or warranties concerning the accuracy, completeness or up to date nature of the information provided, nor does it accept any liability whatsoever arising from or out of any errors or omissions. This information should not be solely relied upon in the event of excavations or any other works being carried out in the vicinity of Irish Water's underground network(s). The onus is on the parties carrying out excavations or any other works to ensure the exact location of Irish Water's underground network(s) is identified prior to excavations or any other works being carried out. Service connection pipes are not generally shown but their presence should be anticipated.

Appendix E : FOUL NETWORK CALCULATIONS [SITE NETWORK DRAINAGE]

FOUL SEWERAGE DESIGN

Design Criteria for FS_1

Pipe Sizes STANDARD Manhole Sizes STANDARD

Industrial Flow (l/s/ha)	0.00	Add Flow / Climate Change (%)	0
Industrial Peak Flow Factor	0.00	Minimum Backdrop Height (m)	0.800
Flow Per Person (l/per/day)	222.00	Maximum Backdrop Height (m)	1.500
Persons per House	3.00	Min Design Depth for Optimisation (m)	1.200
Domestic (l/s/ha)	0.00	Min Vel for Auto Design only (m/s)	0.75
Domestic Peak Flow Factor	6.00	Min Slope for Optimisation (1:X)	300

Designed with Level Inverts

Network Design Table for FS_1

PN	Length (m)	Fall (m)	Slope (1:X)	Area (ha)	Houses	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
F1.000	92.400	1.155	80.0	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F1.001	4.961	0.025	198.4	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F1.002	70.039	0.350	200.1	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F2.000	59.225	0.296	200.1	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F2.001	50.318	0.252	199.7	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F2.002	52.436	0.262	200.1	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F3.000	21.880	0.146	149.9	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F3.001	49.030	0.236	207.8	0.000	43	0.0	1.500	o	225	Pipe/Conduit	

Network Results Table

PN	US/IL (m)	Σ Area (ha)	Σ Base Flow (l/s)	Σ Hse	Add Flow (l/s)	P.Dep (mm)	P.Vel (m/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
F1.000	57.655	0.000	0.0	43	0.0	30	0.62	1.28	51.1	2.0
F1.001	56.500	0.000	0.0	86	0.0	53	0.55	0.81	32.3	4.0
F1.002	56.475	0.000	0.0	129	0.0	66	0.62	0.81	32.2	6.0
F2.000	57.800	0.000	0.0	43	0.0	38	0.45	0.81	32.2	2.0
F2.001	57.504	0.000	0.0	86	0.0	53	0.55	0.81	32.2	4.0
F2.002	57.252	0.000	0.0	129	0.0	66	0.62	0.81	32.2	6.0
F3.000	57.800	0.000	0.0	43	0.0	36	0.49	0.94	37.2	2.0
F3.001	57.654	0.000	0.0	86	0.0	54	0.54	0.79	31.6	4.0

Network Design Table for FS_1

PN	Length (m)	Fall (m)	Slope (1:X)	Area (ha)	Houses	Base Flow (l/s)	k (mm)	HYD SECT	DIA (mm)	Section Type	Auto Design
F1.003	79.592	0.398	200.0	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F4.000	56.532	0.283	199.8	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F4.001	48.417	0.242	200.1	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F4.002	52.176	0.261	199.9	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F4.003	30.879	0.549	56.2	0.000	43	0.0	1.500	o	225	Pipe/Conduit	
F5.000	65.747	0.341	192.8	0.000	35	0.0	1.500	o	225	Pipe/Conduit	
F4.004	39.380	0.555	71.0	0.000	0	0.0	1.500	o	225	Pipe/Conduit	

Network Results Table

PN	US/IL (m)	Σ Area (ha)	Σ Base Flow (l/s)	Σ Hse	Add Flow (l/s)	P.Dep (mm)	P.Vel (m/s)	Vel (m/s)	Cap (l/s)	Flow (l/s)
F1.003	56.125	0.000	0.0	387	0.0	120	0.83	0.81	32.2	17.9
F4.000	58.393	0.000	0.0	43	0.0	38	0.45	0.81	32.2	2.0
F4.001	58.110	0.000	0.0	86	0.0	53	0.55	0.81	32.2	4.0
F4.002	57.868	0.000	0.0	129	0.0	66	0.62	0.81	32.2	6.0
F4.003	57.607	0.000	0.0	172	0.0	55	1.06	1.53	60.9	8.0
F5.000	57.400	0.000	0.0	35	0.0	34	0.42	0.83	32.8	1.6
F4.004	57.059	0.000	0.0	207	0.0	64	1.03	1.36	54.2	9.6

PIPELINE SCHEDULES for FS_1

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
F1.000	o	225	F1	59.086	57.655	1.206	Open Manhole	1200
F1.001	o	225	F2	58.794	56.500	2.069	Open Manhole	1200
F1.002	o	225	F3	58.742	56.475	2.042	Open Manhole	1200
F2.000	o	225	F4	59.678	57.800	1.653	Open Manhole	1200
F2.001	o	225	F5	59.400	57.504	1.671	Open Manhole	1200
F2.002	o	225	F6	59.751	57.252	2.274	Open Manhole	1200
F3.000	o	225	F7	0.000	57.800		Open Manhole	1200
F3.001	o	225	F8	0.000	57.654		Open Manhole	1200
F1.003	o	225	F4	59.552	56.125	3.202	Open Manhole	1200
F4.000	o	225	F10	59.280	58.393	0.662	Open Manhole	1200
F4.001	o	225	F11	59.050	58.110	0.715	Open Manhole	1200
F4.002	o	225	F12	59.398	57.868	1.305	Open Manhole	1200
F4.003	o	225	F13	59.140	57.607	1.308	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
F1.000	92.400	80.0	F2	58.794	56.500	2.069	Open Manhole	1200
F1.001	4.961	198.4	F3	58.742	56.475	2.042	Open Manhole	1200
F1.002	70.039	200.1	F4	59.552	56.125	3.202	Open Manhole	1200
F2.000	59.225	200.1	F5	59.400	57.504	1.671	Open Manhole	1200
F2.001	50.318	199.7	F6	59.751	57.252	2.274	Open Manhole	1200
F2.002	52.436	200.1	F4	59.552	56.990	2.337	Open Manhole	1200
F3.000	21.880	149.9	F8	0.000	57.654		Open Manhole	1200
F3.001	49.030	207.8	F4	59.552	57.418	1.909	Open Manhole	1200
F1.003	79.592	200.0	F5	59.009	55.727	3.057	Open Manhole	1200
F4.000	56.532	199.8	F11	59.050	58.110	0.715	Open Manhole	1200
F4.001	48.417	200.1	F12	59.398	57.868	1.305	Open Manhole	1200
F4.002	52.176	199.9	F13	59.140	57.607	1.308	Open Manhole	1200
F4.003	30.879	56.2	F14	59.173	57.058	1.890	Open Manhole	1200

PIPELINE SCHEDULES for FS_1

Upstream Manhole

PN	Hyd Sect	Diam (mm)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
F5.000	o	225	F14	58.778	57.400	1.153	Open Manhole	1200
F4.004	o	225	F14	59.173	57.059	1.889	Open Manhole	1200

Downstream Manhole

PN	Length (m)	Slope (1:X)	MH Name	C.Level (m)	I.Level (m)	D.Depth (m)	MH Connection	MH DIAM., L*W (mm)
F5.000	65.747	192.8	F14	59.173	57.059	1.889	Open Manhole	1200
F4.004	39.380	71.0	F	58.848	56.504	2.119	Open Manhole	0

Free Flowing Outfall Details for FS_1

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
F1.003	F5	59.009	55.727	0.000	1200	0

Free Flowing Outfall Details for FS_1

Outfall Pipe Number	Outfall Name	C. Level (m)	I. Level (m)	Min I. Level (m)	D,L (mm)	W (mm)
F4.004	F	58.848	56.504	0.000	0	0

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