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Appendices

Executive Summary

Arup was commissioned to undertake a Flood Risk Assessment (FRA) to support the planning application for a large data centre development on a site in Profile Park, Co Dublin.

The planning application was submitted last year.

Further information was requested on the rerouting of the historic stream through the development. Further information was also requested on the storm water drainage with the recommendation that more natural water attenuation features be used.

The new design for the stream and storm drainage necessitated an update to the flood risk assessment report.

The new route and profiles of the stream are shown on the updated civil engineering master-planning drawings.

The purpose of the study is to identify and quantify the risk of flooding to the proposed development and identify measures, if required, to mitigate the risk to site. The FRA has been undertaken in accordance with 'The Planning System and Flood Risk Management' Guidelines for Planning Authorities published in November 2009, jointly by the Office of Public Works (OPW) and the then Department of Environment, Heritage and Local Government (DEHLG).

To assess the flood risk to the site, a 1D unsteady hydraulic model of the minor watercourse that flows through site was developed to determine design water levels for both the existing and proposed scenarios. The results of the modelling have demonstrated that water does not get out of bank within the site for the existing scenario. The risk of fluvial flooding risk to the site is therefore very low.

The results of the model have also indicated when conveyance improvements are considered as part of the proposed diversion channel, water is also kept within bank. Flood risk to the site in the proposed scenario is also therefore very low. The results of the hydraulic modelling have also clearly indicated that flood risk downstream of the site is not increased with the proposed development in place.

The risk of pluvial flooding to the site is very low. The risk of ground water flooding is also very low.

It is proposed to adopt a conservative FFL of the Dub 15 and Dub 16 data centres at 76.85mOD which is 0.55m higher than the recommended flood defence level. It is also proposed to set the FFL of the Switch Rooms to 76.56mOD which is 1.46m higher than the recommended level.

Drainage from the proposed development site shall be drained via a separate system which will now discharge into an attenuation pond. Discharge from the attenuation pond to the water course will be restricted to a greenfield runoff rate of 2 litres/second/hectare in line with South Dublin County Council (SDCC) Water Services requirements.

Access and egress routes are very unlikely to be compromised during flood events. Conveyance and floodplain storage will not be impacted by the proposed development given the low flood risk to the site.

The subject site is outside the 1000-year fluvial flood extent and is therefore classified as being within Flood Zone C. A Justification Test for the development was therefore not required and it was only necessary to identify mitigation measures for residual flood risks which have been described in this report.

1 Introduction

1.1 Background

Arup was commissioned to undertake a Flood Risk Assessment (FRA) to support the planning application for a large data centre development on a site in Profile Park, Co Dublin. The purpose of the study is to identify and quantify the risk of flooding to the proposed development and identify measures, if required, to mitigate the risk to site.

The FRA has been undertaken in accordance with 'The Planning System and Flood Risk Management' Guidelines for Planning Authorities published in November 2009, jointly by the Office of Public Works (OPW) and the then Department of Environment, Heritage and Local Government (DEHLG).

1.2 Scope of Study

The scope of the FRA is:

- Assess the risk of fluvial flooding to the site,
- Undertake a hydrological assessment for the two minor watercourses relevant to the site,
- Develop a 1D unsteady hydraulic model of the minor watercourse that flows through site to determine design water levels for both the existing and proposed scenarios,
- Assessment of the impact, if any, of the flood risk off site with the development in place,
- Assessment of the risk of Groundwater and pluvial flooding,
- Advise on the engineering measures that may be required to mitigate flood risk at the site,
- Preparation of a Flood Risk Assessment (FRA) Report which will be used to inform on the design of the development.

1.3 Summary of Data Used

In preparing this report, the following data was collated and utilised as part of the analysis:

- Two separate detailed topographic surveys of the site from 2021 and 2005,
- Master Plan Drawings of the proposed development Scheme (see Planning Drawings as part of this application).
- Catchment characteristics from the FSU web portal that is run and maintained by OPW,
- Eastern CFRAM flood mapping (produced by the Office of Public Works),

- Reports and maps from the OPW National Flood Hazard Mapping website,
- Ground investigation data from two separate geotechnical investigations in 2021 and 2011.
- Strategic Flood Risk Assessment (SFRA) for the South Dublin County Council Development Plan 2016-2022 which was undertaken by RPS.

All Ordnance Datum (OD) levels referred to in this report are to Malin Head Ordnance Datum unless otherwise stated.

Arup visited the site on 28 May 2021 to further develop our understanding of the mechanisms of flooding on the site and to assess possible overland flows routes. The findings of our site visit have informed the set up and development of the hydraulic models which are described later in the report.

Figure 1 presents a photograph of the existing channel upstream of the first culvert on the site. The channel is vee-shaped with a relatively high level of vegetation at the bottom of both the left and right bank.

Figure 1: Existing channel

A number of surface water discharge points were observed along the left bank of the existing channel adjacent to the developed site (**Figure 2**). These discharge points were fitted with non-return valves to ensure water cannot travel up the pipes.

Figure 2: Surface Water Discharge Points into the channel

Figure 3 presents a photograph of the confluence of the two minor watercourses, i.e., the water course which runs through the site and the drainage channel which receives water from the Golf Course is circa 70m downstream of the culvert under the Profile Park access road.

Figure 3: Confluence of both watercourses

1.4 Site Description

The subject site is in Profile Park, Kilcarbery, Co. Dublin as indicated in **Figure 4** below. The site is situated southwest of Dublin City, approximately 4.65km outside the M50, between the N7 and N4 national primary routes.

Figure 4: Profile Park location (Source: Google Maps)

There are 2 no. existing commercial buildings on the site with a combined area of 17,506m2. The land surrounding the site is primarily greenfield and commercial/industrial premises. Grange Castle Golf Club is located to the east of the site (see Planning Drawings).

Figure 5: Subject site

1.5 Watercourses in the Vicinity of the Site

There are two minor water courses within the site:

- A minor tributary of the Griffeen River flows through the site in the North South direction (**Figure 6**). This watercourse is an artificial channel and was set to its present alignment as part of the development of the two existing buildings on the site. The watercourse flows through several culverts as described in Section 1.3 of the report,
- An existing drainage channel is located to the East of the site. This channel is dry and receives no flow from the upstream catchment. An inflow from the Golf Course however discharges into the channel at the Northwest of the site as indicated in **Figure 6**. From this point the watercourse conveys the flows downstream to the Griffeen River.

It is noted that there is also a decommissioned WWTP on the site as indicated on **Figure 6**. This structure is to be removed as part of the proposed development.

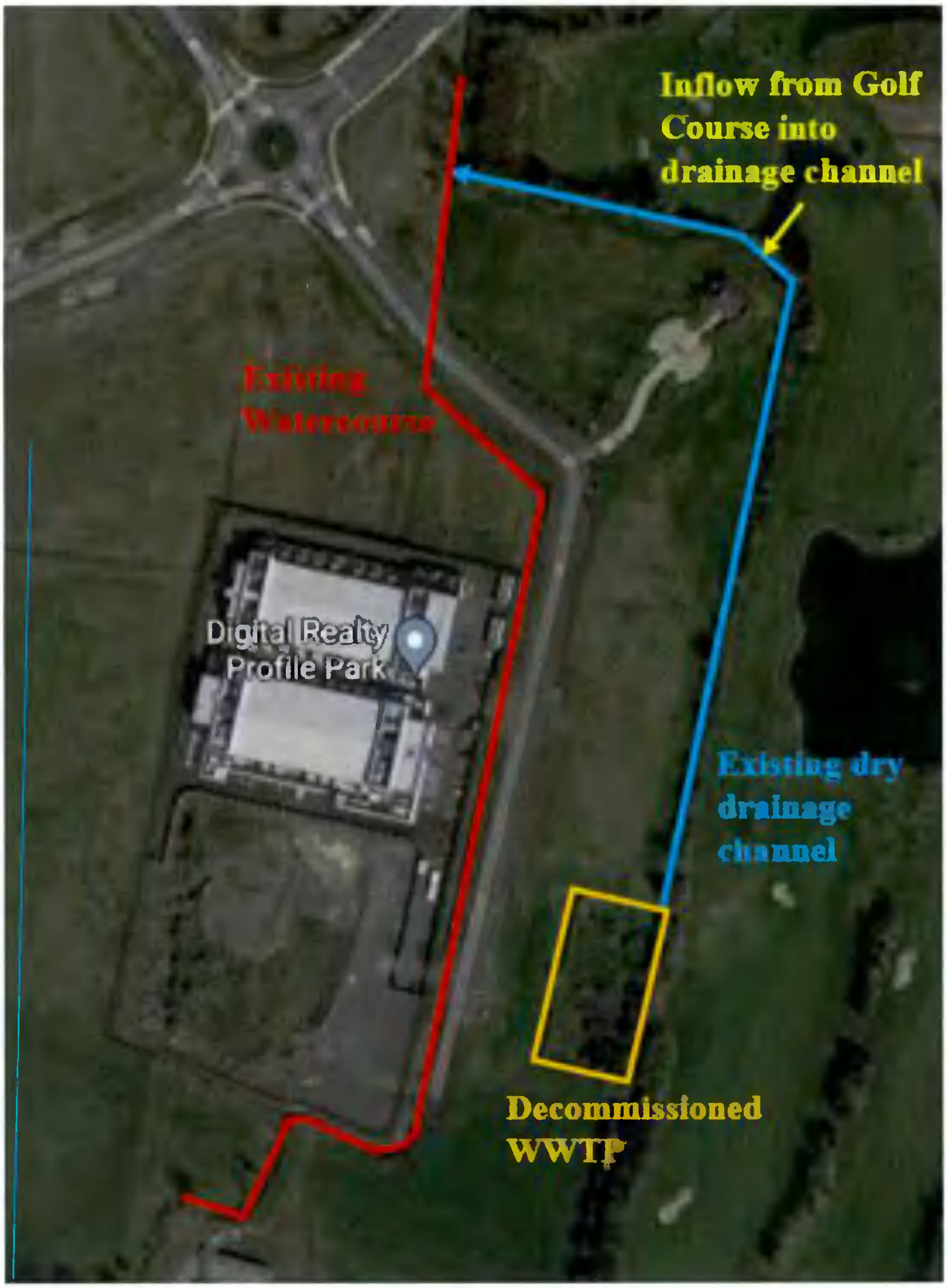

Figure 6: Figure showing water courses on the site

Figure 7 presents an extract from the Flood Studies Update (FSU) web portal and highlights the alignment of all the primary watercourses in the vicinity of the site. Neither of the two minor watercourses are indicated on the plot. The watercourse that runs through the site is shown to originate at the northern boundary of the site before joining the Griffeen River further downstream. The omission of the watercourses from within the site boundary is a function of the catchment sizes. The FSU web portal does not consider watercourses with catchment areas less than 1km^2 which is the case with both minor watercourses as discussed later in this report in **Section 4**.

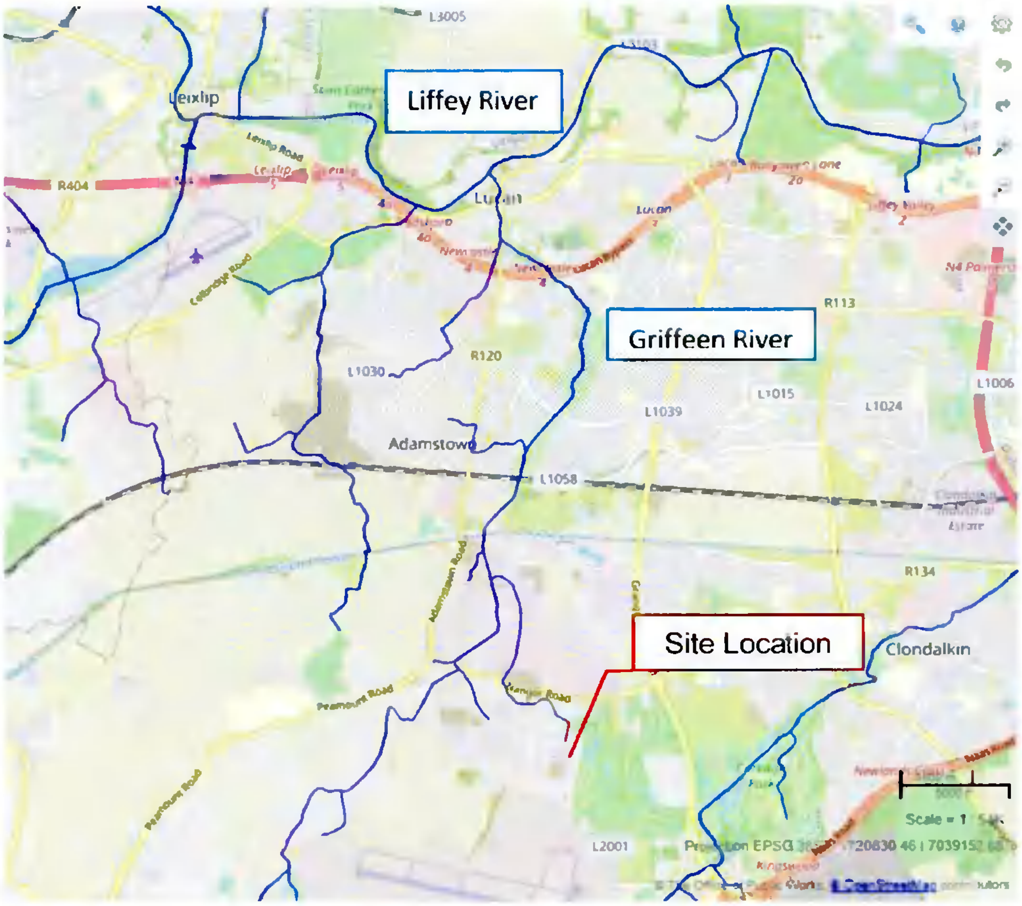

Figure 7: Watercourses near the development site (Source: FSU Web portal)

1.6 Proposed Development

1.6.1 Buildings

The development will consist of the erection of two data centre buildings and associated ancillary facilities. The proposal includes two data centre buildings, DUB 15 and DUB 16, with footprints of 8,310m^2 and 8,242m^2, respectively. There is a proposal to remove the existing disused wastewater treatment facility on the southwest corner of the site boundary. Other details of the development proposal are provided elsewhere within the planning application package.

1.6.2 Watercourses and landscaping

The development proposal also includes a plan to reroute and widen an existing watercourse constructed following an earlier planning permission. It is proposed to reroute this watercourse along the eastern and southern boundary of the site. Further details on this proposal is provided under **Section 5** of this report.

Landscaping is proposed to the south of the site to screen the buildings. Fencing and security gates are proposed around the site. New access roads within the site are proposed along with 71 car parking spaces and 26 cycle spaces, bin stores, site lighting, and all associated works including underground foul and storm water drainage attenuation and utility cables and all other ancillary works.

Figure 8 Master Plan of the Proposed development

2 Overview of Flood Risk Policy

2.1 Overview

The two relevant planning policy documents used were:

- The Planning System and Flood Risk Management guidelines (2009) form the OPW,
- Strategic Flood Risk Assessment (SFRA) for the South Dublin County Council Development Plan 2016-2022 which was undertaken by RPS.

Relevant sections of these documents are discussed below.

2.2 The Planning System and Flood Risk Management Guidelines

2.2.1 Introduction

In November 2009, the Department of Environment, Heritage and Local Government and the Office of Public works jointly published a Guidance Document for Planning Authorities entitled "the Planning System and Flood Risk Management".

The Guidelines are issued under Section 28 of the Planning and Development Act 2000 and Planning Authorities and An Bord Pleanála are therefore required to implement these Guidelines in carrying out their functions under the Planning Acts.

The aim of the guidelines is to ensure that flood risk is neither created nor increased by inappropriate development.

The Guidelines require the Planning system to avoid development in areas at risk of flooding unless the development can be justified on wider sustainability grounds and the risk can be reduced or managed to an acceptable level.

The guidelines require the adoption of a Sequential Approach (to Flood Risk Management) of Avoidance, Reduction, Justification and Mitigation and they require the incorporation of Flood Risk Assessment into the process of making decisions on Planning Applications and Planning Appeals.

Fundamental to the guidelines is the introduction of flood risk zoning and the classifications of different types of development having regard to their vulnerability.

The management of flood risk is now a key element of any development proposal in an area of potential flood risk and should therefore be addressed as early as possible in the site master planning stage.

2.2.2 Definition of flood zones

Flood Zones are geographical areas within which the likelihood of flooding is in a particular range.

There are three types of flood zones defined in the Guidelines as follows:

Table 1: Definition of flood zones

Zone	Description
Flood Zone A	Probability of flooding from rivers and the sea is highest (greater than 1% or 1 in 100 for river flooding or 0.5% or 1 in 200 for coastal flooding).
Flood Zone B	Probability of flooding from rivers and the sea is moderate (between 0.1% or 1 in 1000 year and 1% or 1 in 100 for river flooding and between 0.1% or 1 in 1000 year and 0.5% or 1 in 200 for coastal flooding); and
Flood Zone C	Probability of flooding from rivers and the sea is low (less than 0.1% or 1 in 1000 for both river and coastal flooding). Flood Zone C covers all areas of the plan which are not in zones A or B.

2.2.3 Definition of vulnerability classes

The following table summarises the Vulnerability Classes defined in the Guidelines and provides a sample of the most common type of development applicable to each.

Table 2: Definition of vulnerability classes

Type of Vulnerability	Definition
Highly Vulnerable Development	Includes Garda, ambulance and fire stations, Healthcare, schools, residential dwellings, residential institutions, essential infrastructure, such as primary transport and utilities distribution and SEVESO and IPPC sites, etc.
Less Vulnerable Development	Includes retail, leisure, warehousing, commercial, industrial and non-residential institutions, etc.
Water Compatible Development	Includes Flood Control Infrastructure, docks, marinas, wharves, navigation facilities, water-based recreation facilities, amenity open spaces and outdoor sport and recreation facilities

2.2.4 Types of Vulnerability classes appropriate to each zone

The following table illustrates the different types of Vulnerability Class appropriate to each Zone and indicates where a Justification Test will be required.

Table 3: Vulnerability class and zones

	Flood Zone A	Flood Zone B	Flood Zone C
Highly Vulnerable	Justification Test	Justification Test	Appropriate
Less Vulnerable	Justification Test	Appropriate	Appropriate
Water Compatible	Appropriate	Appropriate	Appropriate

2.3 Strategic Flood Risk Assessment for SDCC Development Plan

The South Dublin County Council Development Plan 2016-2022 puts forward a vision for the future growth of the County over a six-year period. As part of the Development Plan, a Strategic Flood Risk Assessment (SFRA) was undertaken in accordance with the requirements of The Planning System and Flood Risk Assessment Guidelines for Planning Authorities (2009) and Circular PL02/2014 (August 2014).

The SFRA provided an assessment of all types of flood risk within the County and assisted SDCC to make informed strategic land-use planning decisions and formulate flood risk policies. The best available data at the time of the preparation of the SFRA was acquired from the OPW and used to inform on flood risk.

The subject site is not included as a flood risk Special Area of Interest in the SFRA due to the low risk of flooding at the site. An extract for the Development Plan Zoning maps for the subject site is shown in

Figure 9 below. The site is not indicated as being at risk of flooding. It is noted that the predicted flood extent shown in the figure is taken from the Eastern CFRAM study which is discussed in **Section 3.3** of this report.

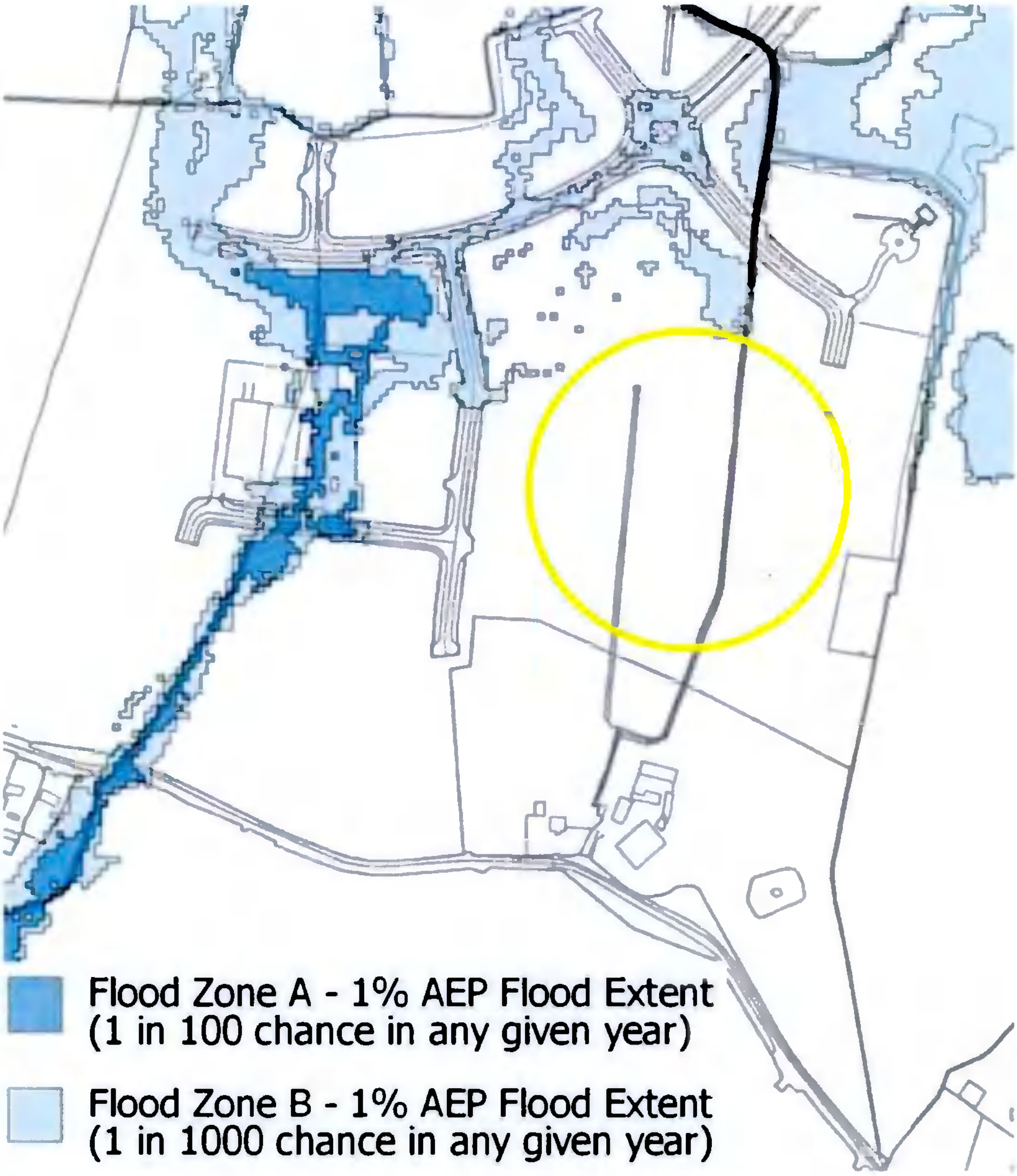

Figure 9: Flood risk map for the site as presented in the SDCC SFRA

The site is zoned as an area to provide 'enterprise and employment' in the SDCC development plan zones as indicated in **Figure 10**.

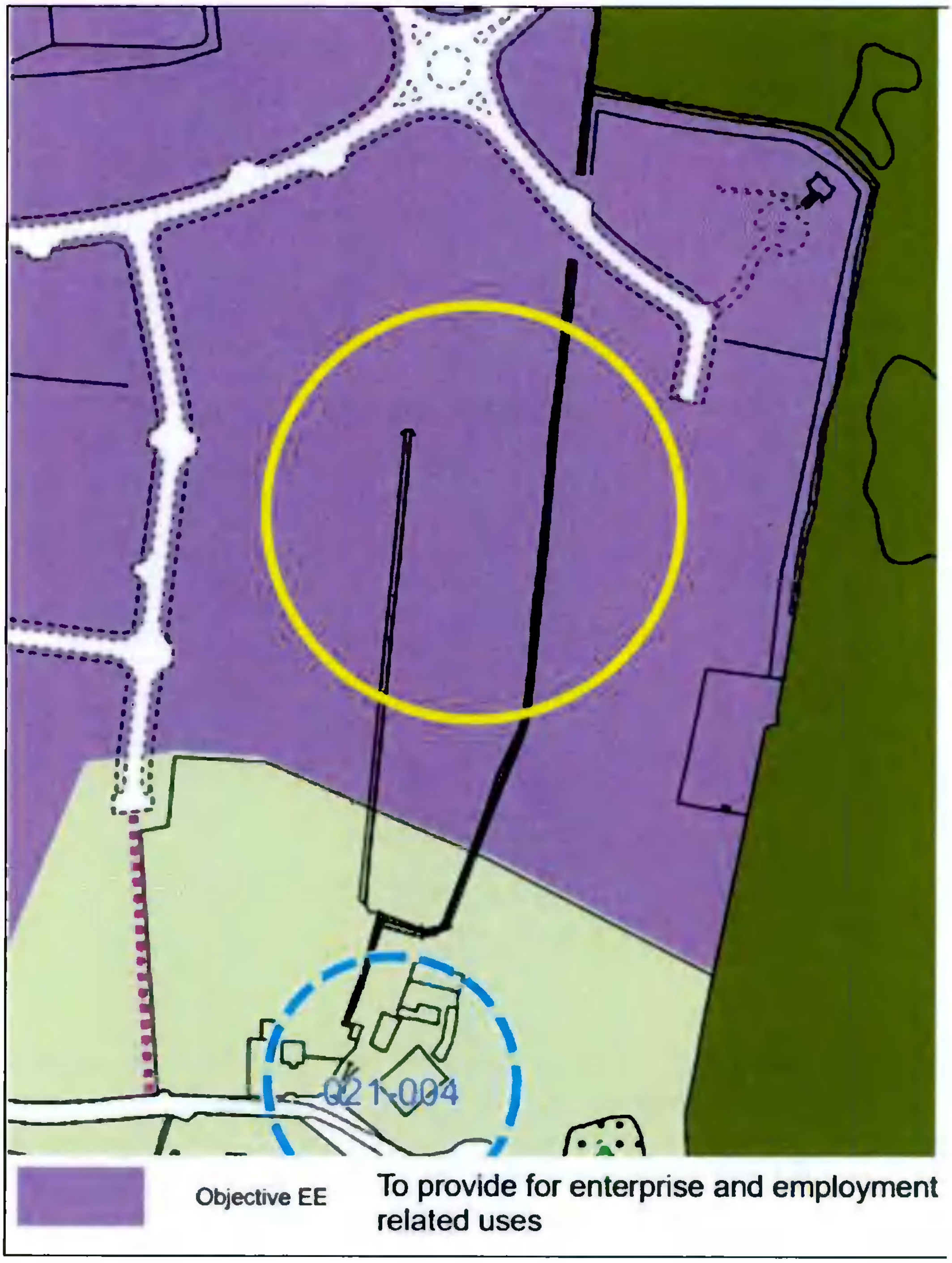

Figure 10: Zoning for the site

3 Overview of Flood Mechanisms at the Site

3.1 Overview of Mechanism of Flooding

In broad terms, the potential sources of flooding at the site were identified to be:

- Fluvial (River) Flooding – Fluvial flooding occurs when excessive rainfall over an extended period causes the capacity of the channel to be exceeded,
- Pluvial Flooding - Pluvial flooding occurs when the capacity of the local drainage network is exceeded during periods of intense rainfall. At these times, water can collect at low points in the topography and cause flooding,
- Groundwater Flooding – this can occur during lengthy periods of heavy rainfall, typically during late winter/early spring when the groundwater table is already high. If the groundwater level rises above ground level, it can pond at local low points and cause periods of flooding.

Each of these potential sources of flooding are considered in this FRA.

It is noted that given the distance of the site from the sea and its elevation above Mean Sea Level, the risk of coastal flooding is very low and therefore not considered further in this report.

3.2 Historic Flooding at the Site

Figure 11 presents an extract from floodinfo.ie showing past flood events at the site and its surrounds. It can be seen from the figure that there are no recorded flood events in and around the site location. It is noted however that while there is no record of past flooding on site, it is still possible that unrecorded flooding has occurred on the site in the past.

Figure 11: Extract from Floodinfo.ie

3.3 Fluvial Flood Risk

An extract from the Eastern CFRAM fluvial flood extent map for the site and its immediate vicinity is presented in **Figure 12**. The predicted flood extent for three separate return period events is indicated on the figure: 10-year, 100 year and 1000 year.

The site of the proposed development is not within the predicted flood extents for any of the events. Neither of the minor watercourses that flow within the site were however included as part of the CFRAM hydraulic model that was used to generate the flood map. The absence of a predicted flood extent for the site from the CFRAM does not therefore imply that the site is not as risk of flooding. A site-specific hydraulic model of the watercourses that run through the site is therefore required in order to accurately access design water levels through the site and hence to inform on flood risk to the site. Hydraulic modelling is also required to assess the flood risk associated with the proposed scenario.

As part of the FRA, we have therefore developed an unsteady MIKE 11 model of both minor watercourses and used to assess flood risk to the site for both the existing and proposed scenarios. This work is described in **Section 5** of the report.

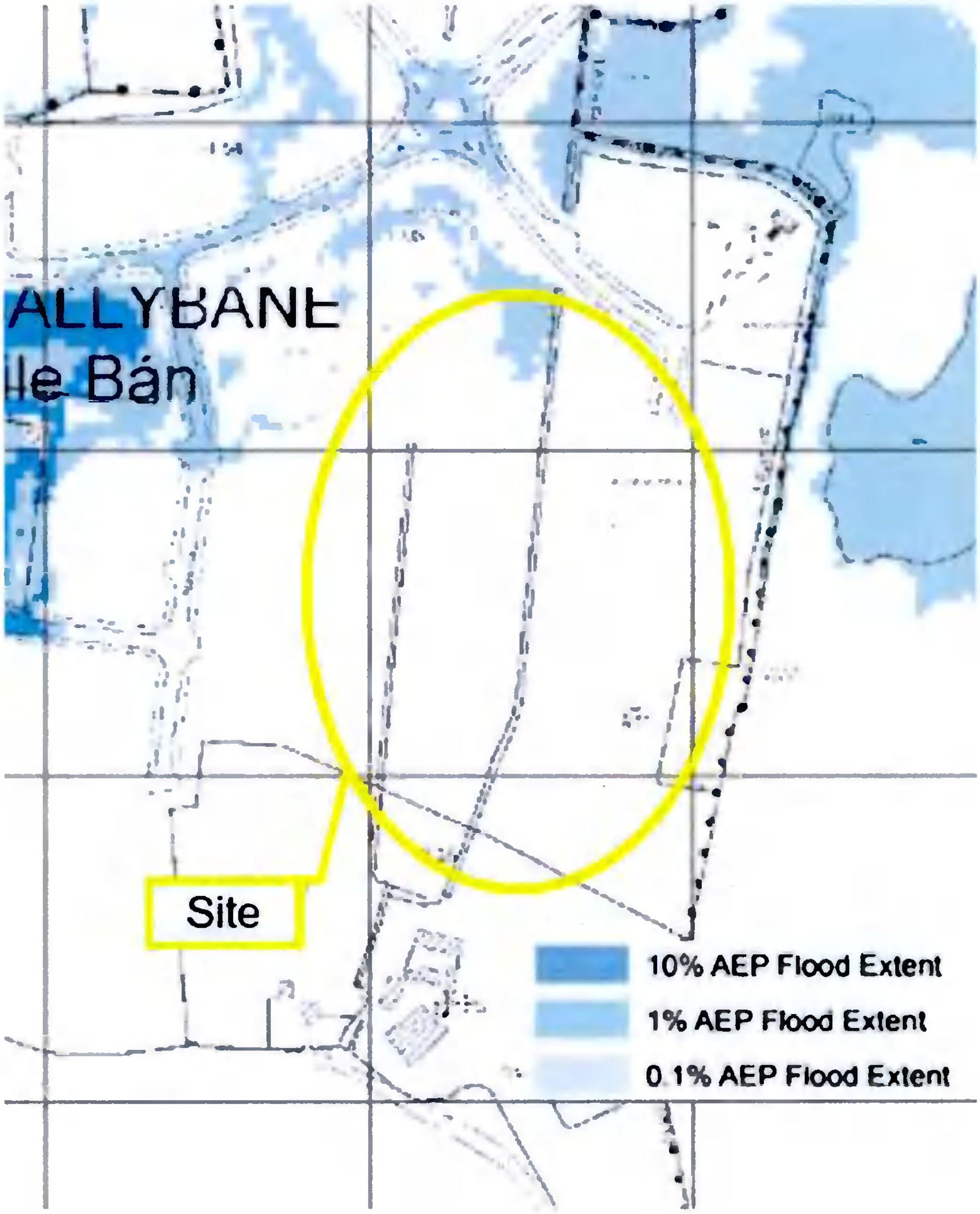

Figure 12: CFRAMS Fluvial flood extent map

It can also be seen from the flood map that areas outside of the site boundary within the neighbouring Golf Course are at risk of flooding in the Q1000 year event. The source of this inundation is an overland flow route from the neighbouring river to the east of the site (**Figure 13**). The inundation does not originate from either the minor watercourse that runs through the site or the Griffeen River.

Figure 13: Overland flow route from the neighbouring river

3.4 Pluvial Flooding

Pluvial flooding typically occurs when extreme rainfall overwhelms drainage systems or soil infiltration capacity, causing excess rainwater to pond above ground at low points in the topography.

The risk of pluvial flooding to the existing site was assessed by reviewing the pluvial flood maps included as part of the SFRA undertaken as part of the SDCC development plan. (It is noted that these maps were originally produced by the OPW as part of the Preliminary Flood Risk Assessment Study which assessed flood risk from multiple sources across the country). It can be seen from the map **(Figure 14)** that the risk of pluvial flooding to the site is low.

Figure 14: Pluvial flood map for the site

3.5 Groundwater Flooding

3.5.1 Overview

Groundwater flooding can occur during lengthy periods of heavy rainfall, typically during late winter/early spring when the groundwater table is already high. If the groundwater level rises above ground level, it can pond at local low points and cause periods of flooding.

According to the GSI online maps, the aquifer below the site is a locally important aquifer and is moderately productive. The groundwater vulnerability is defined as high by the same source and the groundwater recharge is indicated as 51-100 mm/year.

3.5.2 2011 Ground Investigation Data

Ground Investigation (GI) work was carried out on the subject site by IGSL Ltd. In April 2011. Sixteen trial pits were inspected at various locations on the subject site **(Figure 15)**, ranging in depth from 2.3m to 3.75m. Groundwater was typically encountered at the top of bedrock at depths of 2.3m and 3.75m bgl. It is most likely that local groundwater flows are likely to be influenced by topography and thus are also likely to flow from south to north.

GI data also identified poor natural permeability of the site due to the shallow depth of bedrock and underlying aquifer.

Figure 15: Approximate location of Trial Pits carried out by IGSL Ltd. In April 2011

3.5.3 2021 Ground Investigation Data

Groundwater monitoring standpipes were installed at a number of locations within the site as part of recent GI in May and June 2021. The level of groundwater ranged from 2.25mBGL to 3.24mBGL. The groundwater readings are presented in the table below. Groundwater strikes were noted in the trial pit logs (TP102 and TP112) ranging from 3.3 to 3.6mBGL in the Black Boulder Clay. Water ingress was reported as slow.

Table 4: Summary of static groundwater levels

BH no.	Slotted Standpipe (mBGL)	Res. Zone	18/05/21 (mBGL)	18/05/21 (mOD)	03/06/21 (mBGL)	03/06/21 (mOD)	10/06/21 (mBGL)	10/06/21 (mOD)
BH102	1 to 3.5	Made ground & Black Boulder Clay	Dry	Below 75.8 (Dry)	3.1	73.68	3.24	73.54
BH103	4.1 to 7.1	Bedrock	2.79	75.22	2.25	75.76	2.54	75.47
BH104	4.7 to 7.7	Bedrock	2.71	73.98	2.65	74.04	2.78	73.91

3.5.4 Groundwater flood risk conclusion

It is evident from both GI surveys that ground water levels at the site are relatively low. While groundwater levels can be subject to seasonal variations, the ground water levels are very likely to remain below ground level throughout the year. The risk of groundwater flooding to the site is therefore considered to be low.

4 Hydrological Assessment

4.1 Hydrological Estimation

To establish the existing flood risk and design water levels at the site associated with the minor water courses it was necessary to provide estimates of flood flows for the 1% AEP fluvial flood. This was achieved by calculating an index flood flow and scaling it up by a frequency growth curve.

We note that in line with OPW Planning Guidelines the required standard of protection of the development is the design water associated with the 1 in 100year fluvial event plus 20% allowance for climate change and a suitable freeboard.

As part of this study, the Institute of Hydrology Report 124 (IH124) hydrological estimation method was used to estimate the peak flows. Design hydrographs were estimated using the Unit Hydrograph method which were then subsequently fitted to the peak flow estimates derived using the IH124 to generate the flood hydrographs for the site. This data was then applied as the upstream boundary condition of the hydraulic model used to estimate design water levels across the site.

4.2 Hydrological Estimation Points

To undertake the flood flow estimation, it was necessary to establish a number of Hydrological Estimation Points (HEPs) at appropriate locations along the watercourses. HEPs are typically located at confluences, and at the upstream and downstream ends of modelled watercourses. Hydrological analysis was then carried out on the catchment contributing to each HEP to calculate the design flows at that location.

Two HEPs were utilised in the study (see **Figure 16**):

- HEP 1 – upstream catchment (referred to as US1 in the hydraulic modelling),
- HEP 2 – Inflow from the Golf Course (referred to as US2 in the hydraulic modelling).

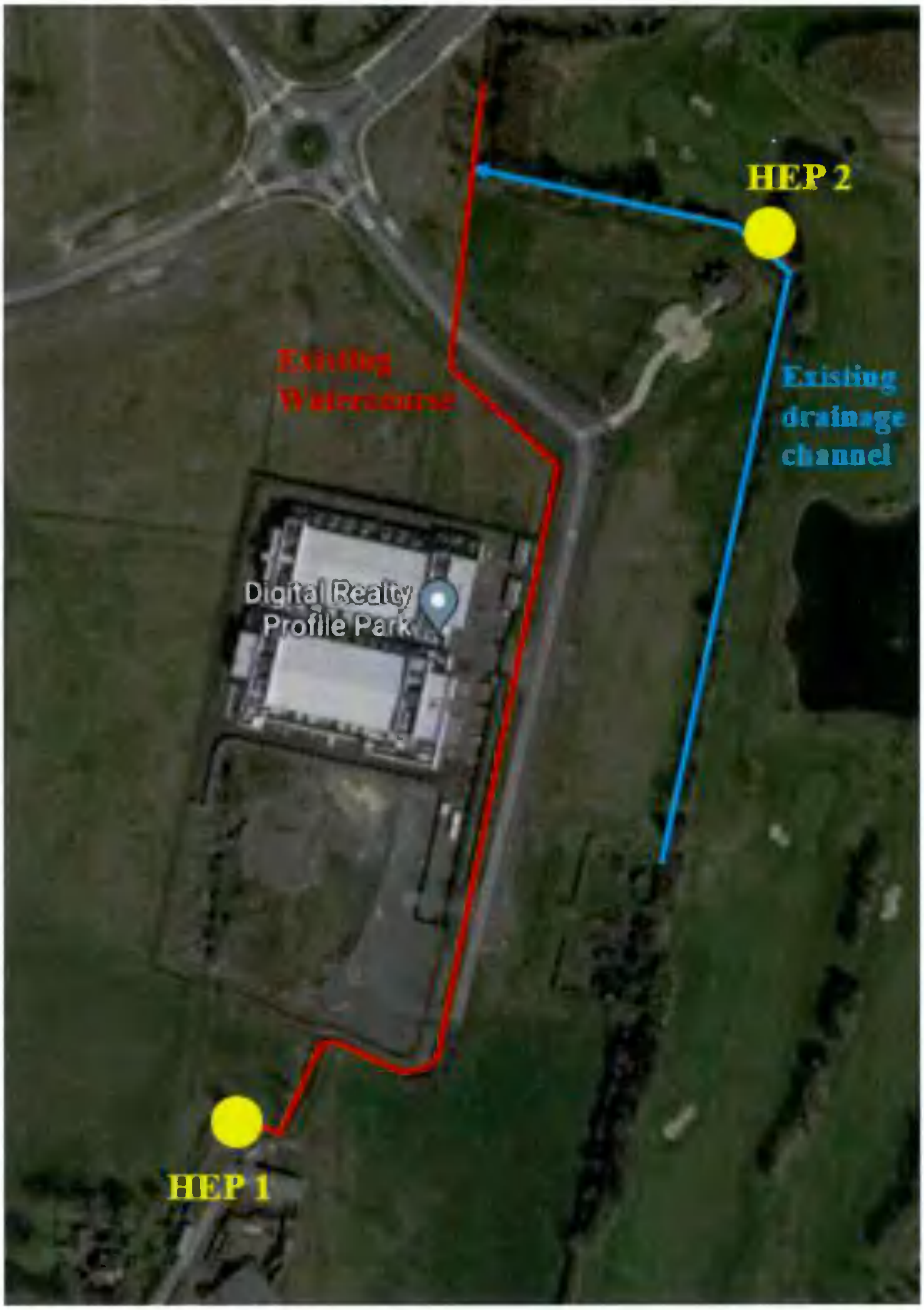

Figure 16: HEPs within the Model Extent

4.2.1 HEP 1 (US1)

Figure 17 shows an extract from the OPW's Flood Studies Update (FSU) web portal which provides the catchment area and catchment characteristics of the FSU node closest to the site. It can be seen from the figure that the outlet node is located downstream of the subject side. The total catchment area to this node is circa 1km^2 and the catchment area at the subject site was estimated to be circa 0.6km^2.

To avoid an underestimation of the design flow at the subject site, the catchment characteristics associated with the bigger FSU catchment node downstream of the site was used to inform the hydrological estimation for HEP 1. The design flows estimated by the study for this HEP are therefore conservative given the uncertainty associated with hydrological estimation for small catchments.

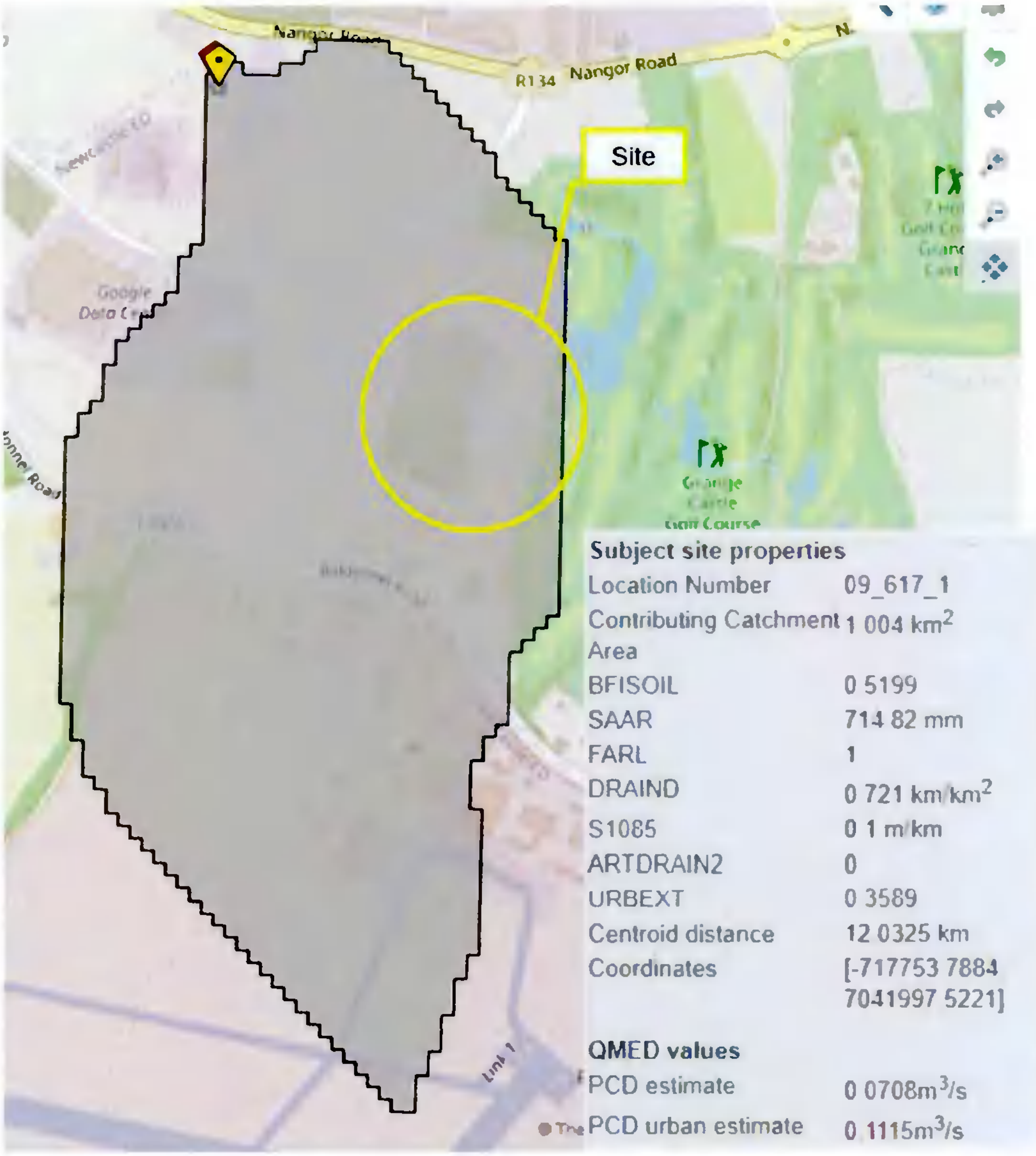

Figure 17: Catchment upstream of the site

4.2.2 HEP 2 (US2)

Figure 18 present the catchment area associated with the Golf Course inflow point. The catchment extent estimated using the data provided in the FSU database was 0.36km². This full catchment area was conservatively used for flow estimation calculation at HEP 2.

Figure 18: Golf Course inflow point – upstream catchment

4.3 Institute of Hydrology Report 124 – HEP 1

The Institute of Hydrology Report No. 124 (IH124) was developed for small rural catchments (<25km²). The runoff estimate ($Qbar_{Rural}$) can be modified to estimate runoff from a partially urban catchment, $Qbar_{Urban}$.

$$\mathrm{Qbar_{Rural}} = 0.00108\,\mathrm{xAREA^{0.89}SOIL^{12.17}SAAR^{1.17}}$$

$$\mathrm{Qbar_{Urban}} = \mathrm{Qbar_{Rural}}\,(1+\mathrm{URBAN})^{2NC}\quad\left[1+\mathrm{URBAN}\left(\frac{21}{\mathrm{CIND}}-0.3\right)\right]$$

Where,

- $\text{CIND} = 102.4\text{SOIL} + 0.28(\text{CWI} - 125)$
- AREA is the catchment area (km2)
- SOIL is an index of how the soil may accept infiltration and is a measure of the Winter Rainfall Acceptance Potential (WRAP).
  The index is based on five classifications (very high, high, moderate, low and very low WRAP). The fraction of catchment in each of the five soil classes is calculated, from this the SOIL index is calculated by the formula:

  $$\text{SOIL} = 0.15\ \text{SOIL1} + 0.3\ \text{SOIL2} + 0.4\ \text{SOIL3} + 0.45\ \text{SOIL4} + 0.5\ \text{SOIL5}$$

  where SOILn is the fraction of the catchment in WRAP class n
- SAAR is long-term mean annual rainfall amount in mm. Data from Met Éireann 1981-2010 was used
- URBAN is the proportion of urbanised area within the catchment
- CWI is the Catchment Wetness Index (mm)

And $\text{NC} = 0.92 - 0.00024\text{SAAR}$ for SAAR between 500mm and 1100mm

$\text{NC} = 0.74 - 0.000082\text{SAAR}$ for SAAR between 1100mm and 3000mm

The IH124 equation has a standard factorial error of approximately 1.65

The IH124 method is used to determine the design flow rate for both HEPs in the study area.

The Qbar was estimated as 0.25m^3/s with the 95% confidence limit. As the Q100 growth curve is 1.96, the 95% confidence limit of Q100 flow was estimated as 1.31m^3/s for the current scenario which was increased to 1.57m^3/s to allow the Q100 climate change scenario. The results of the analysis are summarised in the table below.

Table 5 summarises the results from the above analysis for the un-factored scenario as well as the 68% and 95% confidence intervals.

Table 5: IH124 Method - Qbar urban results

	Qbar urban (m3/s)		
Site	Un-factored	68% Confidence	95% Confidence
HEP 1	0.25	0.41	0.67

Flow for the 1 in 100-year return period (Q100) was calculated by multiplying the above values by the FSR Regional growth curve (1975) growth factor for the 100-year storm (i.e., 1.96). A summary of these results can be seen in **Table 6** below.

Table 6: IH124 Method - Q100 results

	Q100 (m3/s)		
Site	Un-factored	68% Confidence	95% Confidence
HEP 1	0.48	0.79	1.31
HEP 1 (with CC)	0.58	0.95	1.57

4.4 Institute of Hydrology Report 124 – HEP 2

The following tables summarise the results for HEP 2 (Golf course inflow).

Table 7: IH124 Method - Qbar urban results

	Qbar urban (m3/s)		
Site	Un-factored	68% Confidence	95% Confidence
HEP 2	0.07	0.11	0.18

Table 8: IH124 Method - Q100 results

	Q100 (m3/s)		
Site	Un-factored	68% Confidence	95% Confidence
HEP 2	0.13	0.21	0.35
HEP 2 (with CC)	0.15	0.25	0.42

Full design calculations for both HEPs are included in Appendix B.

5 Hydraulic Modelling

5.1 Overview

An unsteady MIKE 11 hydraulic model of the primary watercourse on the site was developed as part of the study to calculate design water levels for both the existing and proposed watercourse. This section of the report details the model development.

5.2 Topographic Survey

Two separate topographic surveys were used to build the hydraulic model:

- 2005 survey of the site and water course as well as the adjacent lands,
- 2021 survey of the site, the minor watercourse and the culverts underneath the access road.

Both surveys have sufficient resolution to describe the ground elevations and the watercourse channel geometry throughout the site. The extent of both surveys is presented in **Figure 19**.

The survey data was processed in Civil 3D to generate cross sections of the watercourse and its floodplain for use in MIKE 11. The vertical resolution of the cross sections was set to 0.1m to ensure the channel geometry was represented as accurately as possible.

Figure 19: Topographic Survey

5.3 Model Schematic – Existing Scenario

A schematic of the MIKE 11 existing scenario hydraulic model is presented in **Figure 20**. All the key hydraulic structures were included in the model and as many of the cross sections were utilised to ensure the longitudinal profile of the watercourse is well represented in the model. The ING coordinates of the three boundary conditions of the model are presented in **Table 9**.

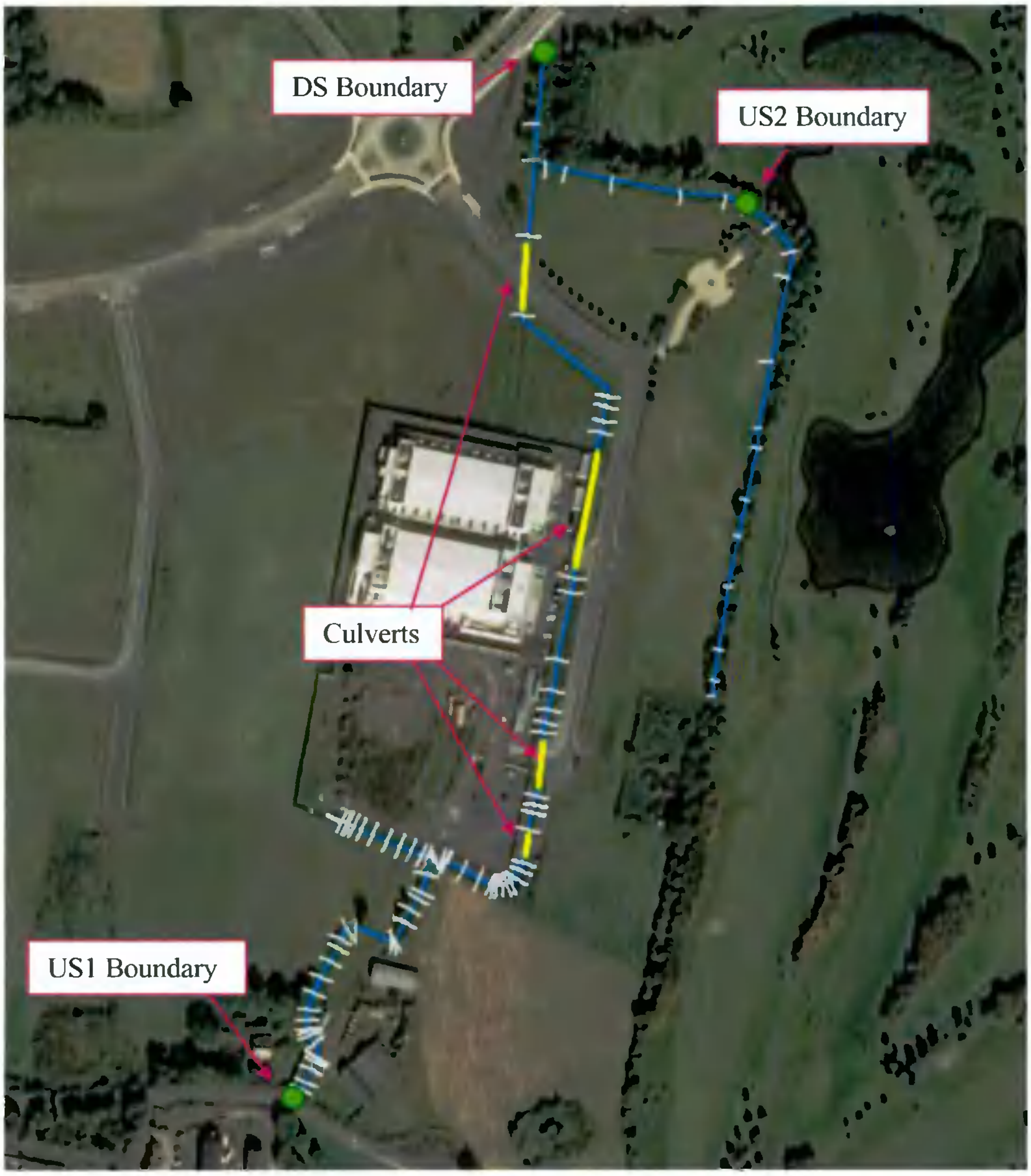

Figure 20: Hydraulic model schematic - Existing Scenario

Table 9: ING coordinates of the boundary conditions of the model

Boundary	X	Y
US1 Boundary	303784.90	230032.21
US2 Boundary	304055.16	230566.98
DS Boundary	303931.81	230660.48

The downstream boundary is located upstream of the bridge which forms part of the access road to the site. It was evident from our site visit that the bridge opening appears large enough to pass the design flood flow without any significant surcharge. The location was thus deemed appropriate for a downstream boundary for the model.

5.4 Model schematic – proposed scenario

A schematic of the proposed scenario hydraulic model is presented in **Figure 21**. The proposed channel involves constructing a new channel, partly utilising an existing channel and upgrading an existing section as shown in **Figure 21**.

A 10m wide riparian strip (between the top of the bank and the built development) was incorporated as part of the proposed realigned watercourse.

Figure 21: Hydraulic model schematic - Proposed Scenario

5.5 Boundary Conditions of the model

The upstream flow hydrographs applied to the model are presented in **Figure 22**.

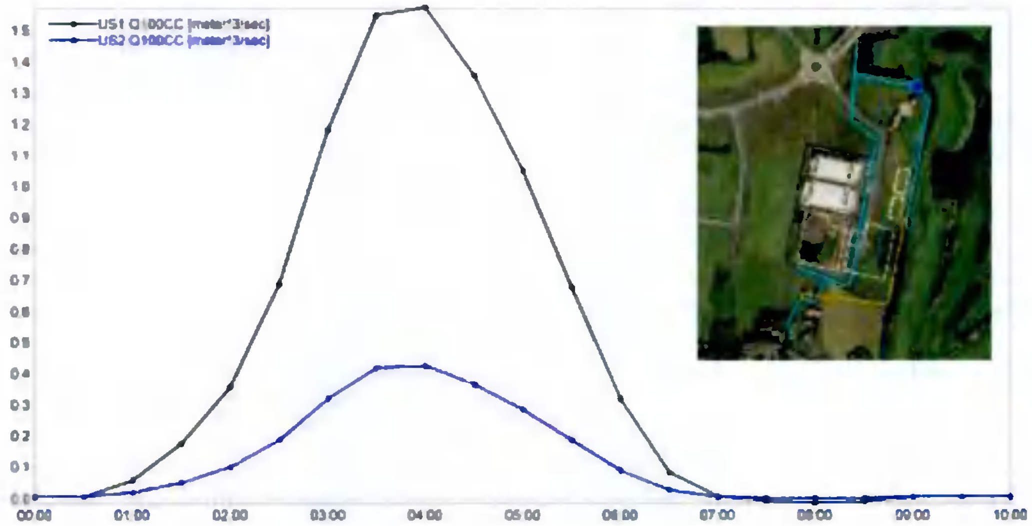

Figure 22: Upstream Boundary conditions

The downstream boundary was a Q-h relationship where the water levels were driven by the flows and geometry of the minor watercourse.

5.6 Model Calibration

The model was not calibrated due to a lack of recorded flow data from the site. The accuracy of the model however was enhanced by following best practice in hydraulic model building, adopting conservative values of the model parameters and utilising Arup's extensive experience in hydraulic modelling.

5.7 Model Parameters

The Manning's n roughness values were selected based on our observation of the existing vegetation along the channel from our site visit. A global manning's coefficient of 0.035 was selected to represent a grassed channel with some weed at the banks. All other computational parameters were set based on best practice in modelling and our extensive experience in 1D river modelling. A sensitivity analysis was carried out on the Manning's coefficient. The coefficient was varied between 0.030 and 0.045 to study its impact on the water level.

5.8 Hydraulic structures

All the culverts in the model were modelled using the Culvert unit in MIKE as this was the most suitable culvert model within MIKE for modelling the culverts in the study are due to their size relative to the river channel.

The dimensions of all the hydraulic structures were obtained from the surveyed conducted.

Figure 23: Culverts location – aerial view

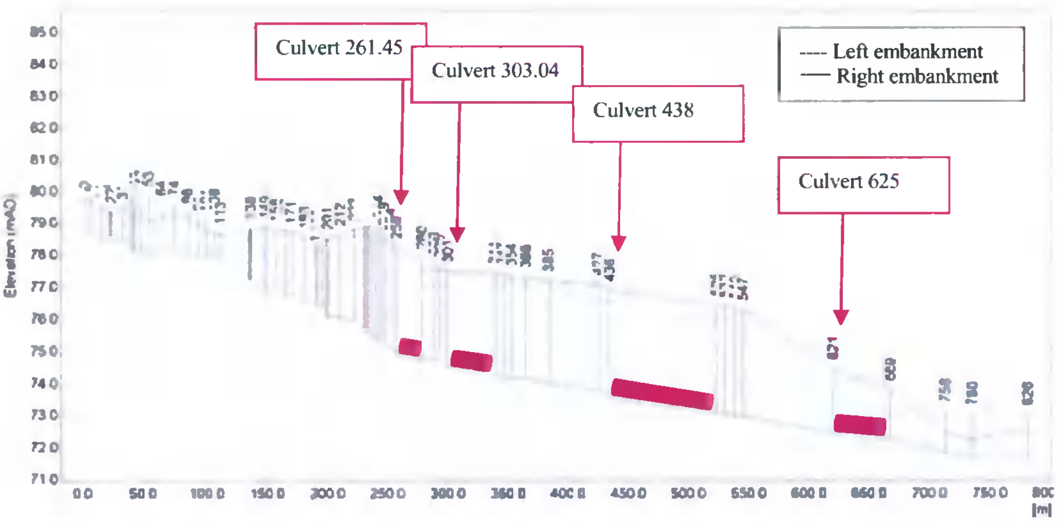

Figure 24: Culverts location – Longitudinal plot

6 Hydraulic Modelling Results

6.1 Existing Scenario

The existing scenario was simulated with the 95%ile Q100 +CC flows. The longitudinal plot of maximum water level along the reach from the simulation is presented in **Figure 26**. The maximum water level varied along the reach from circa 79.4mOD at the upstream end of the reach to circa 72.2mOD at the downstream end. The water did not overflow the channel at any location within the site boundary as there was sufficient capacity to accommodate the design flow.

For example, **Figure 25** presents the maximum water level at CRS 259 which is located at the upstream reach. The peak water level is approximately 2m below the top of the bank at this location which provides significant conveyance area for the flow. A table of maximum water levels for this scenario is presented in the Appendices.

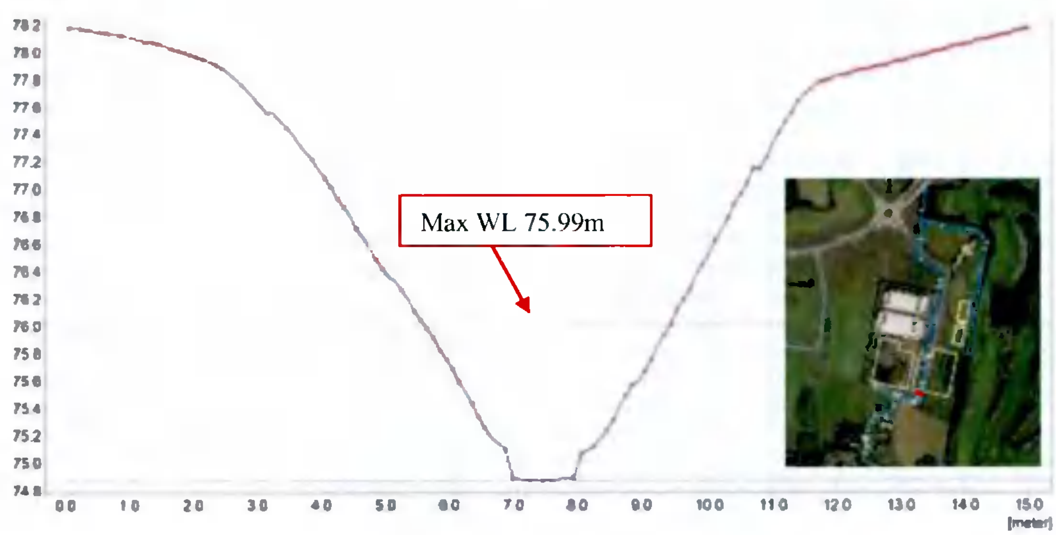

Figure 25: Maximum WL at CRS 259 (95%ile Q100 +CC flow) for existing scenario

It can however be seen that the right bank of the watercourse is at risk of being overtopped at CRS 758 and CRS 780 towards the downstream end of the reach.

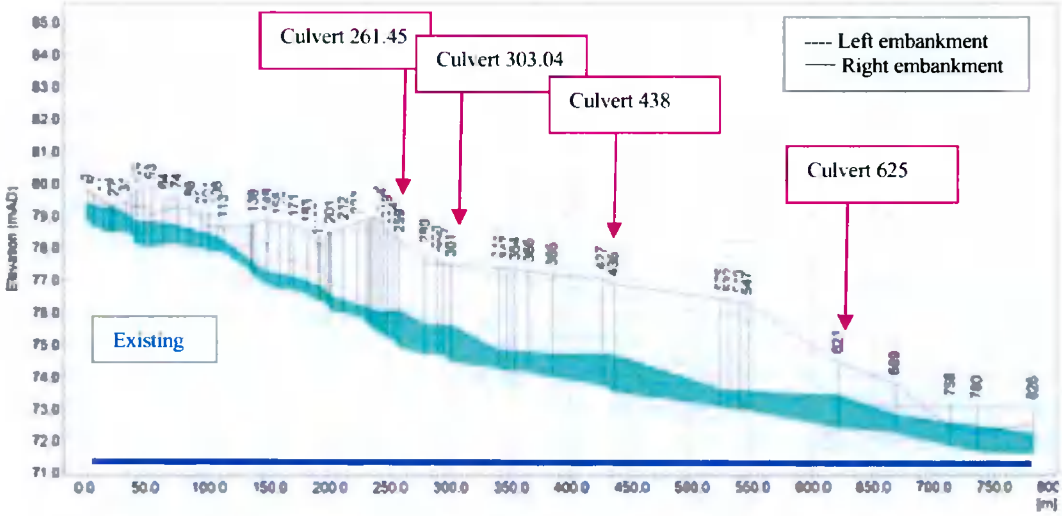

Figure 26: Maximum WL along the reach (95%ile Q100 +CC flow) - existing scenario

6.2 Proposed Scenario (with no engineering measures)

The proposed scenario model without any flood risk engineering measures implemented as part of the development was simulated with the 95%ile Q100 +CC flows. The longitudinal plot of maximum water level along the reach from the simulation is presented in **Figure 27**. It can be seen from the plot that the maximum water level varies along the reach from circa 79.4mOD at the upstream end to circa 72.2mOD at the downstream end. It can also be seen from the plot that there is out of bank flooding at the downstream end north of the site boundary in the vicinity of the confluence with the golf course.

While no properties are at risk of inundation it was deemed unacceptable as this would increase the risk of flooding to the golf course. As noted in Section 3.3 of this report, the golf course is however at risk from overland flow from the Q1000 event to the East of the site. Several engineering measures were therefore proposed to ensure flood risk to the golf course from the minor watercourse is addressed as part of the development.

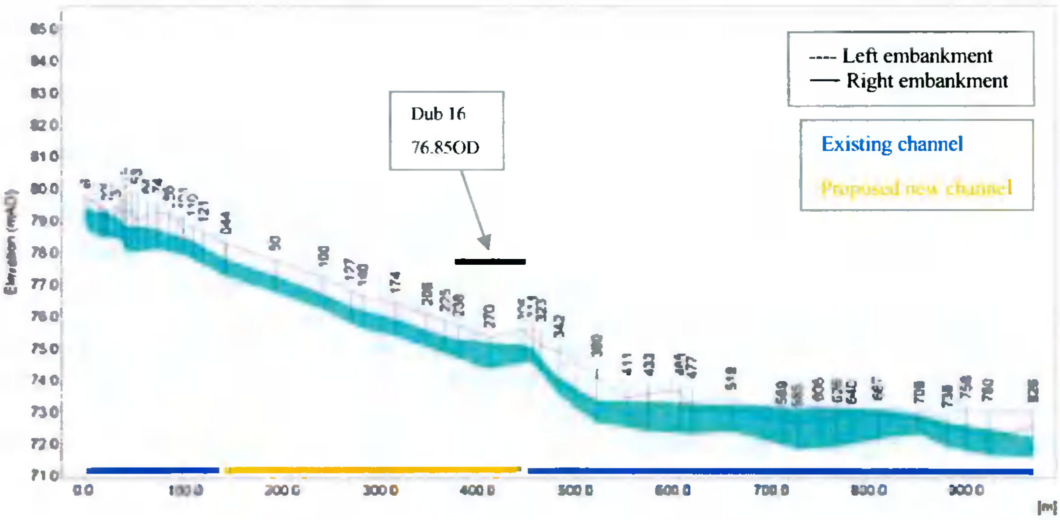

Figure 27: Maximum WL along the reach (95%ile Q100 +CC flow) – Proposed scenario with no engineering measures

6.3 Engineering Measures to Address Flood Risk on Site

Several engineering measures were considered to manage the flood risk downstream of the site. An overview of these measures is presented in the **Table 10**. Based on this assessment, it became apparent that conveyance improvement works offer the best solution to managing flood risk downstream of the site. Conveyance improvement works are therefore the preferred option to reduce the risk of flooding to the development site and the golf course.

Table 10: Overview of optioneering for the site

Option	Operation	Comment
Attenuation tank on site	The tank would allow for any excess water to be stored on site in an underground tank and thereby reduce the flood risk to the Golf course.	This option is not deemed viable due to the lack of available space within the site
Direct defences along the downstream reach	The direct defences would be constructed to keep flood flows in bank and thereby prevent flooding of the golf course.	This option is not deemed viable due to environmental constraints and the need to develop a 10m wide riparian zone adjacent to the proposed watercourse
Diverting excess flood water from the proposed watercourse back to the existing	This option would involve either (1) pumping excess water from the proposed watercourse in times of flood to the existing water course on site, or (2)	Neither of these options are deemed viable – the pumping option would be unfavourable to the

watercourse alignment. (Note: Due to a difference in elevation water cannot flow by gravity from the proposed to the existing channel)	constructing a perched channel along the alignment of the proposed channel in order to divert the excess water in times of flood to the existing watercourse. This option would avoid the need for pumping	environment and would involve a residual risk of pump failure while the perched channel option would be difficult to implement due to space constraints within the site
Channel conveyance improvement works	Widening and/or deepening, creating flood plains in the channel to accommodate the excess flow in the watercourse	This option is deemed viable.

6.4 Proposed Scenario (with conveyance improvements)

It was proposed to implement a variety of conveyance improvements measures along the existing drainage channel to reduce flood risk along the watercourse with the development in place. These consisted of:

- Creating floodplain at the upstream end of the model extent (southern boundary of the development site) with the main channel sized to convey the 1:2-year return period flow otherwise known as the bankfull discharge.
- Minor modifications to the bed levels at several discreet points to remove steep gradients from the channel.
- Channel widening at the downstream end of the model extent (north of the development site): it is proposed to widen the existing channel in the vicinity of the inflow from the golf course to 3m at the base to increase the capacity of the channel and reduce flood risk.

The proposed scenario model with each of these conveyance improvements accounted for in the model was simulated with the 95%ile Q100 +CC flows. The longitudinal plot of maximum water level along the reach from the simulation is presented in **Figure 27**. It can be seen from the plot that with the conveyance improvements in place the maximum water levels at the downstream end (from Cross section 465 to 738) are significantly reduced and the design flood flow remained in bank.

It can also be seen from **Figure 28**Error! Reference source not found. that the water level at the very downstream end of the reach at cross sections 758 and 780 exceeds the crest level of the right bank and flooding will encroach to the golf course. At this section, the ground slopes up to the golf course and hence a small area of the golf course will be affected. As discussed in **Section 6.1,** this area is already at risk of flooding from the minor watercourse under the existing scenario.

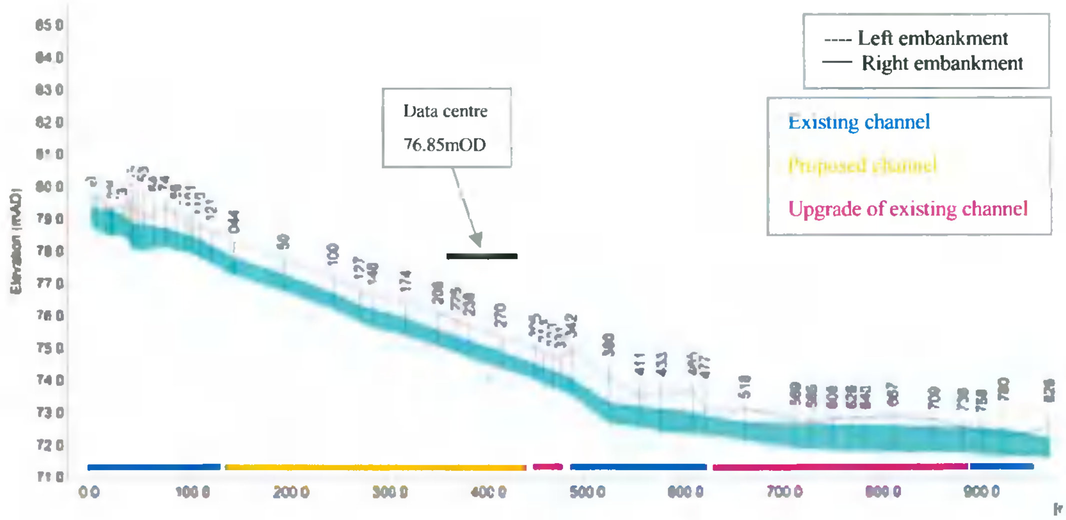

Figure 28 Maximum WL along the reach (95%ile Q100 +CC flow) – Proposed scenario with conveyance improvement works

Figure 29: Plan location of cross sections 758 and 780

Figure 30 presents the maximum water level at CRS 238 which is located immediately adjacent to the proposed data centre. It can be seen from the plot that the peak water level in the channel is circa 0.5m below the top of the bank at this location.

Figure 30: Maximum WL at CRS 238 (95%ile Q100 +CC flow) under existing scenario

Figure 31 presents the maximum water level at cross section CRS 380 which is located immediately adjacent to the proposed Switch Room (indicated on the figure). It can be seen from the plot that the peak water level in the channel is below the top of the bank by more than 1m at this location.

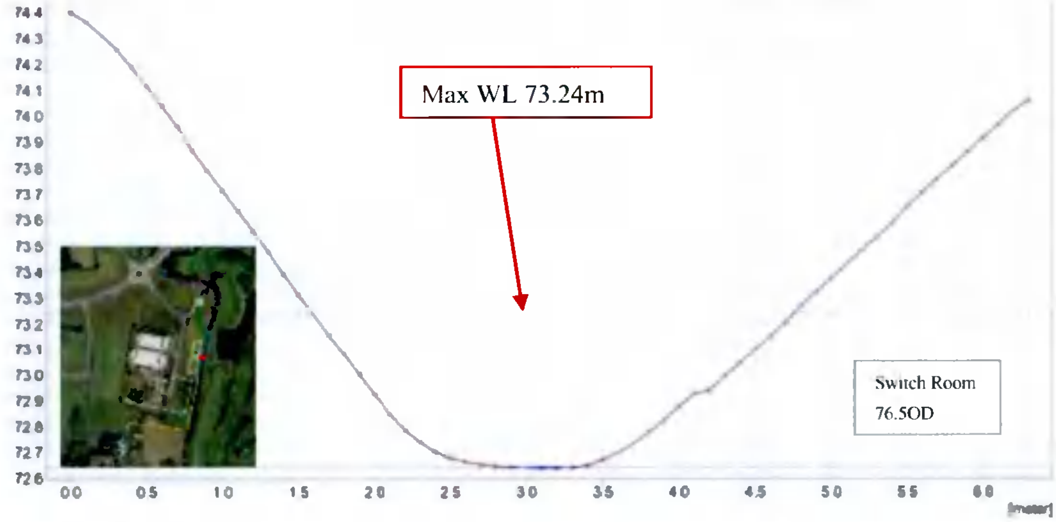

Figure 31: Maximum WL at CRS 380 (95%ile Q100 +CC flow) for existing scenario

6.5 Off-Site Impact

The impact of the development downstream of the site was assessed by comparing the results of the existing scenario with that of the proposed scenario (with conveyance improvements) at the downstream end of the model at CRS 826 and CRS 780. The findings are presented in the following sections of the report.

6.5.1 Differences at model cross section 826

Figure 32 presents both the existing and proposed peak water levels at cross section 826 which is located at the downstream section of the site (indicated with the red box in the plan view plot on the diagram). It can be seen from the plot that the peak water level at this location did not increase with the proposed development in place.

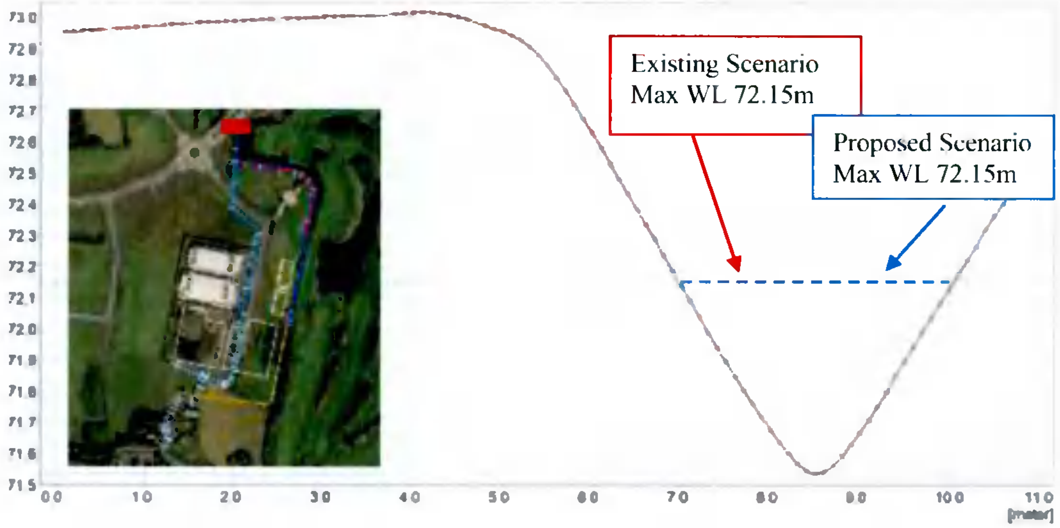

Figure 32: Maximum WL at CRS 826 (95%ile Q100 +CC flow) for existing and proposed scenarios

Figure 33 presents both the existing and proposed water level timeseries at cross section 826. It can be seen from the plot that there are minor differences in the water level on both the rising and falling limb of the hydrograph. At the peak of the hydrograph however there is no difference in the results between the two scenarios, which is more important.

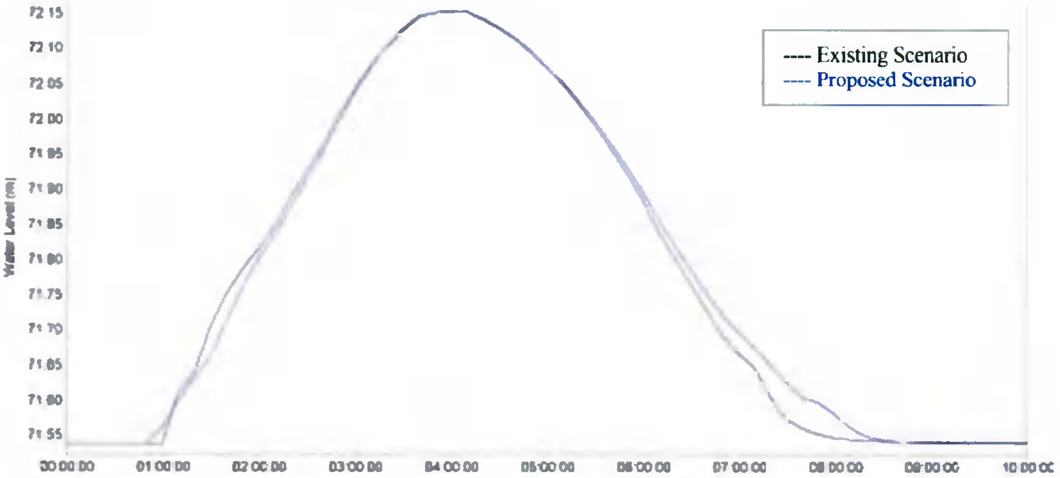

Figure 33: WL timeseries at CRS 826 (95%ile Q100 +CC flow) for existing and proposed scenarios

Figure 34 presents both the existing and proposed discharge timeseries at cross section 826. It can be seen from the plot that there are very minor differences on both the rising and falling limb of the hydrograph. At the peak of the hydrograph however there is no difference discharge through the reach.

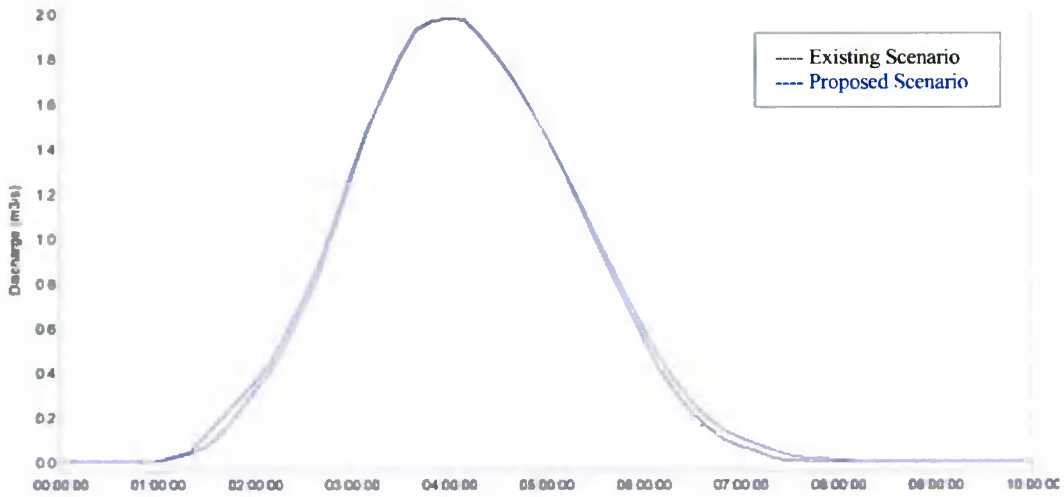

Figure 34: Discharge timeseries at CRS 826 (95%ile Q100 +CC flow) for existing and proposed scenarios

6.5.2 Differences at model cross section 780

Figure 35 presents both the existing and proposed peak water levels at cross section 780 which is located upstream of the cross section 826 (indicated with the red box in the plan view plot on the diagram). It can be seen from the plot that the peak water level at this location is not increased with the proposed development in place.

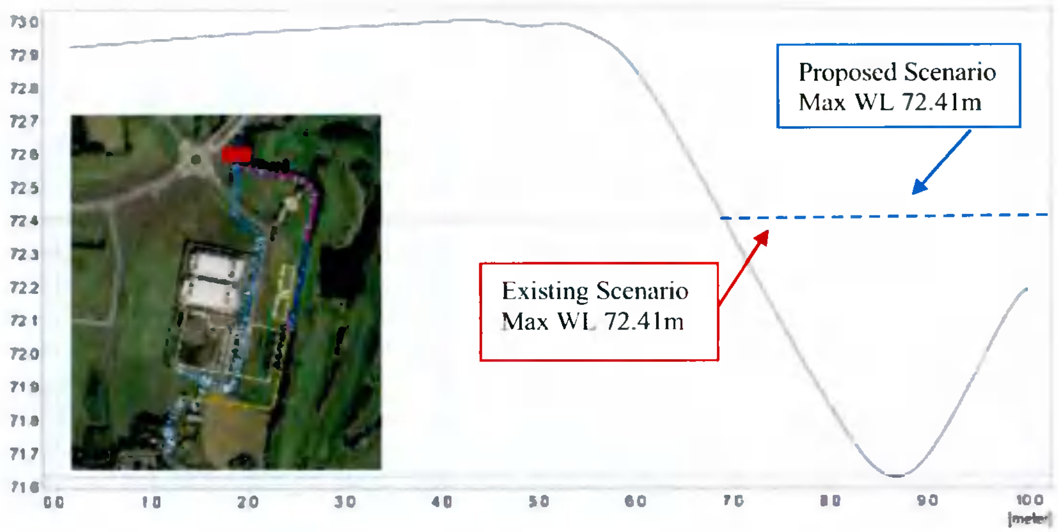

Figure 35: Maximum WL at CRS 780 (95%ile Q100 +CC flow) for existing and proposed scenarios

Figure 36 presents both the existing and proposed water level timeseries at cross section 780. It can be seen from the plot that there are minor differences in the water level on both the rising and falling limb of the hydrograph.

At the peak of the hydrograph however there is no difference in the results between the two scenarios.

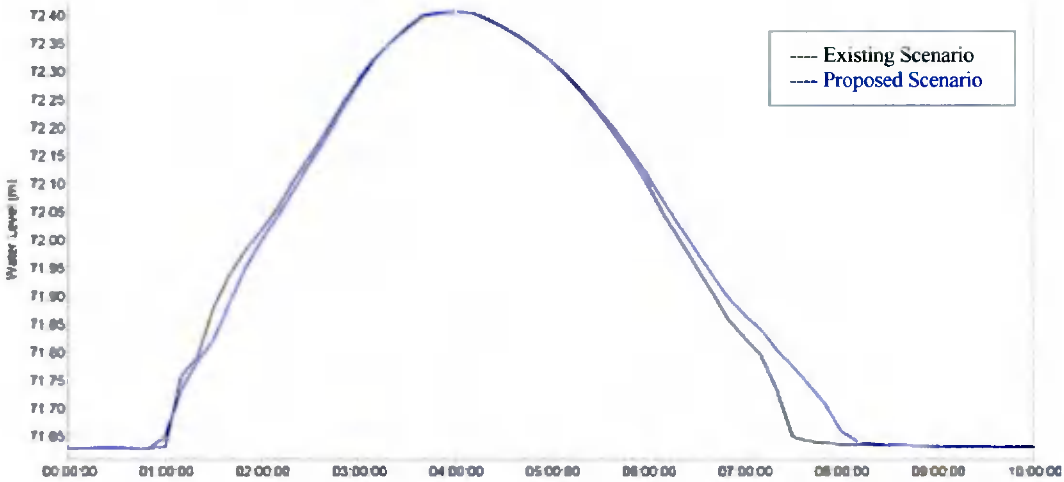

Figure 36: WL timeseries at CRS 780 (95%ile Q100 +CC flow) for existing and proposed scenarios

Figure 37 presents both the existing and proposed discharge timeseries at cross section 780. It can be seen from the plot that there are very minor differences on both the rising and falling limb of the hydrograph. At the peak of the hydrograph however there is no difference discharge through the reach. It is evident from the analysis that flood risk downstream of the site is not increased with the proposed development in place.

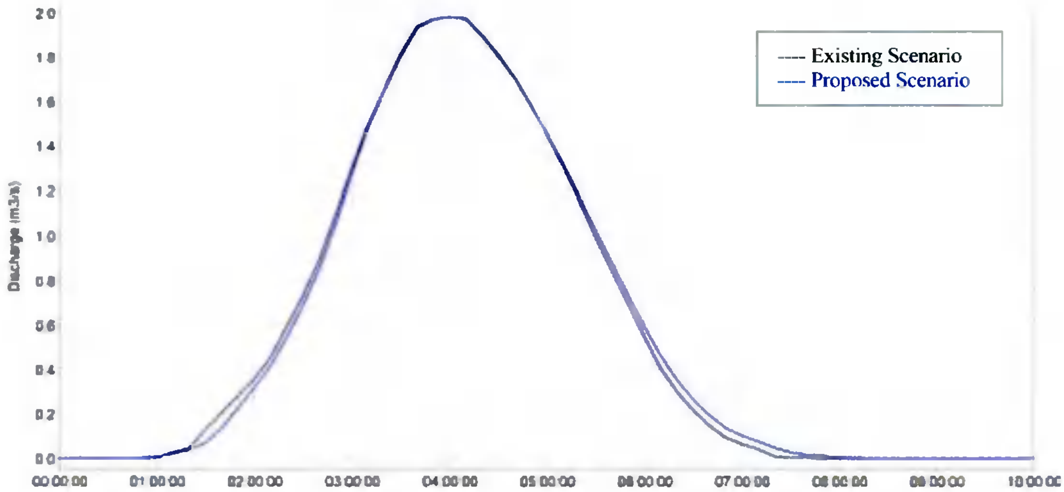

Figure 37: Discharge timeseries at CRS 780 (95%ile Q100 +CC flow) for existing and proposed scenarios

6.6 Hydraulic Modelling Conclusion

The results of the hydraulic modelling have demonstrated that the proposed development site is at a very low risk of flooding at the current scenario without the development proposal being in place. There is however risk of overbank flow at the downstream end of the modelled reach north of the site (near the bridge under Profile Park Road).

When the stream was realigned was realigned to conform with the requirements of the development proposal, there was no increase in the risk of flooding to the site. However, out of bank flooding at the downstream side north of the site boundary near its confluence with the golf course. This was deemed unacceptable as this would increase the risk of flooding to the golf course.

Conveyance improvements works were proposed to reduce the risk by keeping the water in the channel at the specified location. This was accomplished by widening and/or deepening the channel to accommodate the in the watercourse. This conveyance improvement helped to keep the flow within the channel. However, the results of the hydraulic modelling have indicated that flood risk downstream of the site (near the culvert under Profile Park Road) did not change and remains in its original state of stream alignment.

7 Management of Flood Risk at the Site

7.1 Finished Floor Levels of the Buildings

To protect the development proposal from flooding, appropriate finished floor levels (FFL) levels must be set with reference to the design flood level observed in the stream adjacent to the realigned stream.

7.1.1 Design water level

The 100-year plus allowance for climate change maximum design fluvial water level at our site of interest varied along the reach due to the longitudinal gradient of the channel. Upstream of the proposed data centre, the existing scenario peak water level is circa 76mODM. Upstream of the Switch Room the peak water level is circa 74.8mOD.

7.1.2 Freeboard

A detailed freeboard analysis was not undertaken as part of this study. However, it is generally recognised and accepted in Ireland, that a minimum freeboard of 300mm be used with a higher freeboard where this is justified.

7.1.3 Recommended FFL of the development

Allowing for freeboard the recommended site flood defence level for the proposed development was therefore calculated as:

- **Data centres:** 76.0mOD (100-year fluvial level) + 20% increase in flow due to climate change + 0.3mOD (freeboard) = 76.3mOD.

It is however proposed to adopt a conservative approach and set the FFL of the Data Centres at 76.85mOD which is 0.55m higher than the recommended level.

- **Switch Rooms:** 74.8mOD (100-year fluvial level + 20% increase in flow due to climate change) + 0.3mOD (freeboard) = 75.1mOD.

It is also proposed to set the FFL of the Switch Rooms to 76.56mOD which is 1.46m higher than the recommended level.

The flood risk to the proposed development is therefore remote.

7.2 Drainage System Design

Drainage from the proposed DUB 15, DUB 16 and Switch Room shall be drained by a separate system, with separate foul and surface water drains. The outfall of the proposed surface water system will discharge into an attenuation pond, which after completion of the proposed development will cater exclusively for surface water run-off coming from the proposed development sites before discharging to the watercourse.

Surface water discharges from the attenuation pond will be restricted to a greenfield runoff rate of 2 litres/second/hectare in line with South Dublin County Council (SDCC) Water Services requirements.

Foul water drainage will outfall and discharge into the existing Profile Business Park private foul drainage system along The Fairways estate road which subsequently discharges into the existing Irish Water Foul sewerage network.

Further detail regarding surface drainage is contained in the drainage design engineering report contained in this planning application.

7.3 Access and egress routes

Given the absence of significant risk of flooding of the site and the access road, access and egress routes are unlikely to be compromised during flood events.

7.4 Off-site impact

It is demonstrated in this report that the proposed development will not increase flood risk upstream or downstream of the site.

7.5 Risk of pluvial flooding

In a design exceedance rainfall event, there is a minor risk of surface water collecting on the site. The risk of ingress to the proposed development on the site will however be remote given that the FFL of the buildings will be elevated above ground levels external to the site.

7.6 Impact of climate change

The impact of climate change was considered as part of the hydraulic analysis by increasing the design flows by 20%.

7.7 Justification Test Requirement

The subject site is outside the 1000-year fluvial flood extent and is therefore classified as being within Flood Zone C. A Justification Test for the development is therefore not required and it is necessary only to identify mitigation measures for any residual flood risk which has been described in this report.

8 Conclusion and Recommendations

This FRA was completed to support the planning application for a large data centre development on a site in Profile Park, Co Dublin. It was completed in accordance with the 'The Planning System and Flood Risk Management' Guidelines for Planning Authorities published in November 2009.

The development proposal included realigning of the existing stream satisfy the requirements of the data centre. A 1D hydraulic model was developed to assess the flood risk to the site with and without the development proposal. It was determined that the existing site with or without the development proposal is at low risk of flooding from all sources. However, with realigned water course the area north (part of the golf course) of the site boundary will be at increased risk flooding from overtopping of the stream bank for a 1% AEP + 20% Climate change allowance.

To mitigate against this increased risk, a conveyance improvement in the form of widening and deepening of the channel at the said section was proposed for inclusion as part of the development proposal. With this it was possible to eliminate the risk of overtopping and hence flooding of the golf course. However, the flood will overtop the channel at the downstream of the model same as it did without the development proposal. Further survey data of this section of the stream must be obtained to further assess the extent of this flooding.

To manage any risk of flooding to the development site, it is recommended that the FFL of the two buildings (Dub 15 and Dub 16) is set at 76.85mOD. It is also proposed to set the FFL of the Switch Rooms to be set at 76.56mOD.

The site drainage system will be designed to collect runoff into the attenuation pond (adjacent to the Switch Room) which will discharge to the realigned watercourse at a controlled rate of 2 litres/second/hectare in line with South Dublin County Council (SDCC) Water Services requirements.

None of the access and egress routes are impacted by flood events. The site is in flood zone C and no justification test is required to be completed for the development proposal.

Appendix A – Maximum and Bankfull Discharge Water Levels

Table of water levels at model nodes through the reach.

Table 11: Maximum and Bankfull WL at all cross sections for the Proposed Scenario (95%ile Q100 +CC flow)

River Station	Chainage (m)	Invert Level (m OD)	W.S. Elev. (m OD) (Q100)	W.S. Elev. (m OD) (Q2)
180	0.00	77.71	78.23	78.06
179	10.00	77.58	78.06	77.96
178	20.00	77.47	77.95	77.87
177	30.00	77.36	77.85	77.77
176	40.00	77.25	77.74	77.66
175	50.00	77.14	77.63	77.55
174	60.00	77.03	77.53	77.44
173	70.00	76.92	77.44	77.33
172	80.00	76.81	77.32	77.21
171	90.00	76.70	77.22	77.11
170	100.00	76.59	77.14	76.99
169	110.00	76.48	77.02	76.86
168	120.00	76.37	76.9	76.74
167	130.00	76.26	76.79	76.63
166	140.00	76.15	76.68	76.52
165	150.00	76.04	76.58	76.42
164	160.00	75.93	76.42	76.28

River Station	Chainage (m)	Invert Level (m OD)	W.S. Elev. (m OD) (Q100)	W.S. Elev. (m OD) (Q2)
163	170.00	75.82	76.28	76.14
162	180.00	75.71	76.17	76.02
161	190.00	75.6	76.06	75.92
160	200.00	75.49	75.97	75.85
159	210.00	75.38	75.89	75.77
158	220.00	75.27	75.78	75.67
157	230.00	75.16	75.67	75.56
156	240.00	75.05	75.56	75.45
155	250.00	74.94	75.45	75.34
154	260.00	74.83	75.34	75.23
153	270.00	74.72	75.23	75.12
152	280.00	74.61	75.12	75.01
151	290.00	74.5	75.01	74.9
150	300.00	74.39	74.9	74.79
149	310.00	74.28	74.79	74.68
148	320.00	74.17	74.68	74.57
147	330.00	74.06	74.57	74.46
146	340.00	73.95	74.48	74.35
145	350.00	73.84	74.4	74.24
144	360.00	73.73	74.2	74.05

River Station	Chainage (m)	Invert Level (m OD)	W.S. Elev. (m OD) (Q100)	W.S. Elev. (m OD) (Q2)
143	370.00	73.63	74.17	74
142	380.00	73.5	73.97	73.83
141	390.00	73.34	73.76	73.62
140	400.00	73.16	73.53	73.41
139	410.00	72.97	73.38	73.21
138	420.00	72.83	73.35	73.17
137	430.00	72.74	73.28	73.1
136	440.00	72.66	73.23	73.05
135	450.00	72.6	73.19	73
134	460.00	72.54	73.1	72.93
133	470.00	72.47	73.01	72.82
132	480.00	72.4	73.01	72.81
131	490.00	72.36	72.99	72.79
130	500.00	72.33	72.95	72.76
129	510.00	72.29	72.91	72.72
128	520.00	72.29	72.79	72.63
127	530.00	72.23	72.73	72.57
126	540.00	72.18	72.63	72.46
125	550.00	72.12	72.63	72.44
124	560.00	72.08	72.61	72.41

River Station	Chainage (m)	Invert Level (m OD)	W.S. Elev. (m OD) (Q100)	W.S. Elev. (m OD) (Q2)
123	570.00	72.05	72.58	72.38
122	580.00	72.01	72.57	72.36
121	590.00	71.97	72.55	72.34
120	600.00	71.94	72.54	72.32
119	610.00	71.9	72.53	72.31
118	620.00	71.87	72.51	72.3
117	630.00	71.83	72.51	72.29
116	640.00	71.8	72.49	72.28
115	650.00	71.79	72.47	72.26
114	660.00	71.78	72.46	72.25
113	670.00	71.77	72.44	72.24
112	680.00	71.77	72.42	72.22
111	690.00	71.76	72.41	72.21
110	700.00	71.75	72.39	72.19
109	710.00	71.74	72.37	72.18
108	720.00	71.73	72.35	72.16
107	730.00	71.72	72.32	72.14
106	740.00	71.7	72.3	72.12
105	750.00	71.69	72.27	72.1
104	760.00	71.68	72.24	72.07

River Station	Chainage (m)	Invert Level (m OD)	W.S. Elev. (m OD) (Q100)	W.S. Elev. (m OD) (Q2)
103	770.00	71.67	72.21	72.04
102	780.00	71.66	72.16	72
101	790.00	71.64	72.08	71.93

Appendix B – Hydrological Calculations

B1 HEP 1 (Minor watercourse)

ARUP	Job No	Sheet No	Rev
	Member/Location Cork		
Job Title Dub 15	Drg Ref		
Calculation Institute of Hydrology Report No. 124	Made by KB	Date 16/07/2021	Chd

1.0 Subcatchment: HEP_01

2.0 Flood Studies Report Catchment Characteristics:

AREA = 1.00 km^2 Contributing catchment area

SAAR = 715 mm Standard annual average rainfall

Area		WRAP Class (FSR, fig i 4.18(i))
0	km^2	1
1	km^2	2
0	km^2	3
0	km^2	4
0	km^2	5

Area check (sum) = 1 km^2

SOIL = 0.15 SOIL1 + 0.3 SOIL2 + 0.4 SOIL3 + 0.45 SOIL4 + 0.5 SOIL5

where SOIL n is the fraction of the catchment in Wrap class n

SOIL = 0.30

3.0 Mean Annual Flood (Rural)

Qbar (rural) = $0.00108 \times AREA^{0.89}\ SAAR^{1.17}\ SOIL^{2.17}$

IH124 Calcs

Qbar_rural = 0.17 m^3/s

4.0 Adjustment for Urbanisation

CWI = 120.00 Catchment Wetness Index (FSR, 1975)

CIND = $102.4SOIL + 0.28(CWI - 125)$

CIND = 29.32

Urban Area 0.35 km^2

URBAN = 0.35 Fraction of urbanised area in the catcment

Qu bar/Qr bar = $(1 + URBAN)^{2NC}\,[1 + URBAN((21/CIND) - 0.3)]$

Nc = $0.92 - 0.00024\ SAAR$ for $500 \le SAAR \le 1100mm$

or

Nc = $0.74 - 0.000082\ SAAR$ for $1100 \le SAAR \le 3000mm$

Nc = 0.68

Qu bar/Qr bar = 1.42

Qbar_urban = 0.25 m^3/s

ARUP	Job No.	Sheet No.	Rev.
	Member/Location Cork		
Job Title Dub 15	Drg. Ref.		
Calculation Institute of Hydrology Report No. 124	Made by KB	Date 16/07/2021	Chd.

5.0 Standard Error

Standard Factorial Error = 1.65

$Qbar_{Urban}$ (68% Confidence) = 0.41 m^3/s with standard factorial error applied

$Qbar_{Urban}$ (95% Confidence) = 0.67 m^3/s

6.0 Growth Curve

Growth Curve used = FSR Regional (1975)

Return period (years)	FSR (1975) FSR Regional
2	0.95
5	1.20
10	1.37
25	1.60
50	1.77
100	1.96
200	2.14
1000	2.60

7.0 Flood Frequencies

Return period (years)	Current Scenario Flows (m^3/s)			Mid-Range Future Scenario (m^3/s)		
	Un-factored	68% Confidence	95% Confidence	Un-factored	68% Confidence	95% Confidence
2	0.23	0.38	0.64	0.28	0.46	0.76
5	0.29	0.49	0.80	0.35	0.58	0.96
10	0.34	0.56	0.92	0.40	0.67	1.10
25	0.39	0.65	1.07	0.47	0.78	1.29
50	0.44	0.72	1.19	0.52	0.87	1.43
100	0.48	0.79	1.31	0.58	0.95	1.57
200	0.53	0.87	1.44	0.63	1.04	1.72
1000	0.64	1.06	1.74	0.77	1.27	2.09

B2 HEP 2 (Golf Course inflow)

ARUP	Job No	Sheet No	Rev
	Member/Location Cork		
Job Title Dub 15	Drg Ref		
Calculation Institute of Hydrology Report No. 124	Made by KB	Date 16/07/2021	Chd

1.0 Subcatchment: HEP_02

2.0 Flood Studies Report Catchment Characteristics:

AREA = 0.36 km^2 Contributing catchment area

SAAR = 715 mm Standard annual average rainfall

Area		WRAP Class (FSR, fig i 4.18(i))
0	km^2	1
0.36	km^2	2
0	km^2	3
0	km^2	4
0	km^2	5

Area check (sum) = 0.36 km^2

SOIL = 0.15 SOIL1 + 0.3 SOIL2 + 0.4 SOIL3 + 0.45 SOIL4 + 0.5 SOIL5

where SOILn is the fraction of the catchment in Wrap class n

SOIL = 0.30

3.0 Mean Annual Flood (Rural)

Qbar (rural) = $0.00108 \times AREA^{0.89}\ SAAR^{1.17}\ SOIL^{2.17}$

IH124 Calcs

Qbar_rural = 0.07 m^3/s

4.0 Adjustment for Urbanisation

CWI = 120.00 Catchment Wetness Index (FSR, 1975)

CIND = 102.4SOIL + 0.28(CWI - 125)

CIND = 29.32

Urban Area 0.05 km^2

URBAN = 0.14 Fraction of urbanised area in the catcment

Qu bar/Qr bar = $(1 + URBAN)^{2NC}\,[1 + URBAN((21/CIND) - 0.3)]$

Nc = 0.92-0.00024 SAAR for 500 ≤ SAAR ≤ 1100mm

or

Nc = 0.74-0.000082 SAAR for 1100 ≤ SAAR ≤ 3000mm

Nc = 0.68

Qu bar/Qr bar = 0.93

Qbar_urban = 0.07 m^3/s

ARUP	Job No.	Sheet No	Rev.
	Member/Location Cork		
Job Title Dub 15	Drg Ref.		
Calculation Institute of Hydrology Report No. 124	Made by KB	Date 16/07/2021	Chd.

5.0 Standard Error

Standard Factorial Error = 1.65

$Qbar_{Urban}$ (68% Confidence) = 0.11 m^3/s with standard factorial error applied

$Qbar_{Urban}$ (95% Confidence) = 0.18 m^3/s

6.0 Growth Curve

Growth Curve used = FSR Regional (1975)

	FSR (1975)
Return period (years)	FSR Regional
2	0.95
5	1.20
10	1.37
25	1.60
50	1.77
100	1.96
200	2.14
1000	2.60

7.0 Flood Frequencies

Return period (years)	Current Scenario Flows (m^3/s)			Mid-Range Future Scenario (m^3/s)		
	Un-factored	68% Confidence	95% Confidence	Un-factored	68% Confidence	95% Confidence
2	0.06	0.10	0.17	0.07	0.12	0.20
5	0.08	0.13	0.21	0.09	0.15	0.25
10	0.09	0.15	0.24	0.11	0.18	0.29
25	0.10	0.17	0.28	0.12	0.21	0.34
50	0.12	0.19	0.31	0.14	0.23	0.38
100	0.13	0.21	0.35	0.15	0.25	0.42
200	0.14	0.23	0.38	0.17	0.28	0.46
1000	0.17	0.28	0.46	0.20	0.34	0.55

Digital Netherlands VIII B. V.

INXN DUB15/16

Removal of Derelict Wastewater Treatment Plant

280503-GE-RFI-RP001

Issue 1 | 3 February 2022

Job number 280503

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Executive Summary

This report has been prepared in response to the request for additional information received from South Dublin County Council on 28 September 2021 in respect of the application for permission by Digital Netherlands V11 B.V. (Netherlands) for the expansion of their data centre facilities on their site in Profile Park, Nangor Road, Dublin 22 ('the proposed development'), which reads as follows:

12. The Planning Authority notes the proposal to remove the existing wastewater treatment from the site. The Planning Authority requests additional information on the works that this will entail and how the materials will be disposed of.

This report provides background information on the existing derelict wastewater treatment plant (WwTP), a summary of the environmental and site investigations undertaken, our analysis of the data and the proposed removal of the wastewater treatment plant.

Historic mapping and aerial photographs indicate that the WwTP was constructed sometime between 1913 and the 1930s, likely at the same time as the nearby Casement Aerodrome was developed. The site appears disused by 1995 but has remained in place since.

The WwTP covers an area of approximately 75m x 40m and is surrounded by mesh fencing. It contains overgrown vegetation/shrubs, two partially buried disused circular open-top sedimentation tanks both approximately 13m in diameter, a buried aeration tank approximately 12m x 4m, a buried clear water tank approximately 7m x 3m and a stone hut approximately 2.5m x 4.5m in area.

The sedimentation tanks were constructed of brick, with two metal agitators still in place. The sedimentation tanks have been infilled with clinker. It is presumed that the sedimentation tanks were connected through an underground system of pipelines from the aeration tank. For a proper operation, each of the sedimentation tanks had a discharge pipeline from one side of the tank to the clear water tank and one sludge pipeline from the centre of the sedimentation tank to a different facility on site (possible a platform). The site is surrounded by chain link fencing and concrete posts.

Enabling works across the proposed development will include for removal of the WwTP and excavation of the associated buried structures and foundations. The area comprising of the sewage treatment works will be excavated to an average depth of approximately 2m BGL. These soils will be replaced with suitable engineering fill to the proposed earthworks formation level to allow for the construction of the Dub16 building, the proposed new water course at this location and associated site security fencing.

Ground investigation and laboratory testing in and around the WwTP site show that the soils there are described as Made Ground which will require disposal to licenced landfills. The excavation will be monitored for any evidence of further contamination and validation sampling and testing will be carried out.

All demolished and removed material from the structures present on site will be delivered for reuse and recycling where feasible and where such facilities exist.

Where material is reused it will be undertaken in accordance with “Article 27” of the European Union (Waste Directive) Regulations, 2011 to 2020.

Where possible natural soils will be retained on site for reuse and the excavation will be backfilled to the required levels with suitable materials, leaving the site in a suitable environmental condition for the proposed development.

1 Introduction

This report has been prepared in response to the request for additional information received from South Dublin County Council on 28 September 2021 in respect of the application for permission by Digital Netherlands V11 B.V. (Netherlands) for the expansion of their data centre facilities on their site in Profile Park, Nangor Road, Dublin 22 ('the proposed development').

Arup has been commissioned to prepare a report in response to the following additional information request:

12. The Planning Authority notes the proposal to remove the existing wastewater treatment from the site. The Planning Authority requests additional information on the works that this will entail and how the materials will be disposed of.

This report provides background information on the existing derelict wastewater treatment plant (WwTP) including a summary of the environmental and site investigations undertaken and the proposed removal of the existing defunct wastewater treatment plant. It should be read in conjunction with the full planning application and in particular the following reports:

- INXN DUB1516 Construction Waste Management Plan
- INXN DUB1516 Construction Management Plan
- INXN DUB1516 Engineering Report
- INXN DUB1516 Soils and Geology

2 Background

Available historical mapping and aerial photographs of the entire site and the WwTP were reviewed to provide an overview of the site history (see Report INXN DUB1516 Soils and Geology, Appendix A) as follows:

- The site was used for agricultural purposes with a river/drainage channel running through the centre of the site from south to north.
- Historic mapping and aerial photographs indicate that the WwTP was constructed sometime between 1913 and the 1930s, likely at the same time as the nearby Aerodrome was developed.
- The WwTP was likely disused by 1995 but has remained in place since.

The WwTP covers an area of approximately 75m x 40m and is surrounded by mesh fencing as shown on Figure 1 (this figure is also attached to this report in A3 for clarity).

Figure 1 Wastewater Treatment Plant

It contains overgrown vegetation/shrubs, two partially buried disused circular open-top sedimentation tanks both approximately 13m in diameter, a buried aeration tank approximately 12m x 4m, a buried clear water tank approximately 7m x 3m and a stone hut approximately 2.5m x 4.5m in area.

The sedimentation tanks were constructed of brick, with two metal agitators. The sedimentation tanks have been infilled with clinker. It is presumed that the sedimentation tanks were connected through an underground system of pipelines from the aeration tank. For a proper operation, each of the sedimentation tanks had a discharge pipeline from one side of the tank to the clear water tank and one sludge pipeline from the centre of the sedimentation tank to a different facility on site (possible a platform). The site is surrounded by chain link fencing and concrete posts.

3 Ground Investigation

An intrusive ground investigation was undertaken by Site Investigations Limited (SIL) from April to May 2021 (see Report INXN DUB1516 Soils and Geology, Appendix C). The ground investigation took place across the entirety of the site, but the following took place within and surrounding the Waste-Water Treatment Plant as shown on Figure 1.

- 3 No. Cable Percussive boreholes to a maximum depth of 4.7mBGL (BH102, BH103 and BH104);
- 3 No. Rotary Coreholes to a maximum depth of 7.10mBGL (BH102, BH103 and BH104);
- Groundwater and Gas monitoring standpipes were installed in BH102, BH103 and BH104;
- 3 No. trial pits to a maximum depth of 2.6mBGL (TP103, TP105 and TP106); and
- 3 No. foundation inspection pits to a maximum depth of 2.6mBGL (FIP101, FIP102 and FIP103).

3.1 Stratigraphy

The ground conditions encountered during the 2021 ground investigation are summarised in Table 1 below.

Table 1: Stratigraphy surrounding the WWTP

Strata	Description	Top of strata (mBGL)	Top of strata (mOD)	Thickness (m)
Topsoil	Topsoil	0	79.0 - 76.0	0.1 - 0.2
Made Ground	Firm brown to grey silty slightly sandy to sandy, slightly gravelly to gravelly clay with anthropogenic material *or* brown to grey silty slightly sandy to sandy gravel with anthropogenic material	0 - 0.2	79.0 – 73.8	1.4 - 2.9
Brown Boulder Clay	Brown slightly sandy slightly gravelly to gravelly silty CLAY with low cobble content	0 - 0.2	78.8 – 76.0	1.0 – 1.2
Fluvioglacial Sand and Gravel lens	Grey silty slightly sandy fine to coarse, subangular to subrounded GRAVEL of limestone with medium cobble content	1.0 - 4.1	75.0 - 72.8	0.4 – 0.5
Black Boulder Clay	Firm to very stiff dark grey to black slightly sandy, slightly gravelly to very gravelly, silty Clay with medium cobble and low boulder content	1.3 – 5.4	78.6 – 71.0	0.7 – 4.0

Strata	Description	Top of strata (mBGL)	Top of strata (mOD)	Thickness (m)
Calp Limestone	strong to very strong light grey fine-grained muddy Limestone interbedded with a moderately strong to strong dark grey calcareous Mudstone and occasional calcite veins (<20mm) and pyrite crystals	3.6 - 5.4	71.0 - 74.3	Unproven

3.2 Foundation Inspection pits

Foundation inspection pits were undertaken at the WwTP tank structures to determine the dimensions of the tanks and foundations and to obtain soil samples for laboratory testing.

The findings of these foundation inspection pits include:

- The foundation observed in FP101 was a red brick wall to 1.1mBGL, the foundation extended 0.1m and was 1.2m thick;
- The foundation observed in FP102 was composed of concrete, this was 0.25m thick at ground level this continued to 0.9mBGL. At 0.9mBGL the foundation extended another 0.15m, this continued to 1.9mBGL.
- The foundation observed in FP103 was composed of concrete, this extended to 1.7mBGL and tapered out to 0.22m at 1.7mBGL.

3.3 Geo-Environmental Testing

3.3.1 Soils Assessment

Please refer to INXN Soils and Geology report, Section 9 for the Geo-Environmental Assessment Methodology. For the purpose of this report, the soils beneath the WwTP are described in terms of Waste Disposal Categories.

The soils beneath the site are described by the following European Waste Code as outlined in the EPA 2015, Waste Classification.

- 17 05 04 – Soil and Stones excluding those included in 17 05 03*

Based on the outcomes of these screening exercises the soils are described by the categories below:

- Soils suitable for disposal to an inert licensed landfill; and
- Soils suitable for disposal to a non-hazardous licenced landfill.
- Soils requiring disposal to a hazardous licenced landfill.

As part of the ground investigation, 20 no. samples were geo-environmentally tested surrounding the WwTP. The following is the split of performed tests in terms of stratigraphy:

- 10 no. samples from Made Ground;
- 9 no. samples from the Glacial Till; and
- 1 no. sample from the Gravel lens.

Table 2 provides the breakdown of the geo-environmental test results into the following WAC categories: suitable for disposal to an inert, non-hazardous, or hazardous licenced landfill.

Table 2: Tested sample categorisation (from GI locations surrounding the WWTP)

Soil Type	No. of samples	No. of Inert samples	No. of Non-Haz samples	No. of Haz samples
Made Ground	10	5	4	1
Glacial Till	9	6	2	1
Gravel	1	1	0	0

Made Ground

Based on the results shown in Table 2 the Made Ground is categorised as follows:

- 50% suitable for disposal to an inert licenced landfill
- 40% suitable for disposal to a non-hazardous licenced landfill
- 10% suitable for disposal to a hazardous licenced landfill.

A sample of Made Ground from BH104 at 0.5mBGL was classified as requiring disposal to a Hazardous licenced landfill due to an exceedance of Dissolved Organic Carbon when compared to the Waste Acceptance Criteria for landfills.

The samples identified as requiring disposal to a non-hazardous licenced landfill were due to the presence of the following parameters elevated above the limits for inert licenced landfills:

- Mineral Oil
- Chromium
- Nickel
- Dissolved Organic Carbon

Glacial Till

Based on the results shown in Table 2, the Glacial Till is categorised as follows:

- 66% suitable for disposal to an inert licenced landfill;
- 22% suitable for disposal to a non-hazardous licenced landfill; and
- 11% suitable for disposal to a hazardous licenced landfill.

The samples from the Glacial Till which were classified as requiring disposal to a non-hazardous licenced landfill were due to elevated concentrations of naturally occurring selenium, along with elevated concentrations of Dissolved Organic Carbon.

The samples from the Glacial Till which were classified as requiring disposal to a hazardous licenced landfill were due to elevated concentrations of Dissolved Organic Carbon.

Given the detection of potentially hazardous samples, further investigation will be undertaken as part of detailed design to determine the presence and extent of any such materials.

All excavated materials determined as waste will be removed and deposited at suitably permitted or licensed facilities in accordance with current waste legislation. Considering the site history and uncontrolled nature of the filling there is always the possibility of encountering hotspots of further contamination.

3.3.2 Groundwater

As noted in Section 8, INXN DUB1516 Soils and Geology report, groundwater samples were recovered from BH102 at 2.8mBGL and BH103 at 2.7mBGL. The results of the were screened against the Threshold Values listed in the following documents:

- Column 4 of Schedule 5 of the EU Groundwater Regulations 2016;
- Column 4 of Schedule 5 of the EU Groundwater Regulations 2010; and
- Petroleum Hydrocarbons in Groundwater Guidance - CL:AIRE

There have been no recorded exceedances but there has been a high concentration of Iron recorded above the detection limit with a concentration range of 1800 to 1100ug/l.

3.3.3 Ground Gas

Gas Screening Values (GSV) have been calculated for Carbon dioxide and Methane readings across the site. This has been calculated using the guidance provided in the CIRIA guidance (C665) on assessing risks posed by hazardous ground gases to buildings with the following formula:

- Gas screening value (litres of gas per hour) = max borehole flow rate (l/hr) × max gas concentration (%).

The results are summarised in Table 3 below.

Table 3: Gas Screening Values across the site

Parameter	Unit	Standpipe		
		BH102	BH103	BH104
CO2	GSV	0.01	0.01	0.04
CH4	GSV	0.01	0.01	0.08

The results can be classified as very low risk to low risk when compared to the Gas Screening Values (GSV) in the CIRIA guidance (C665) on Assessing Risks posed by hazardous ground gases to buildings.

Using Table 2 from the BS 8485:2015 - Code of practice for the design of protective measures for methane and carbon dioxide ground gases for new buildings (+A1:2019) the GSV for Carbon dioxide and Methane across site has been calculated as CS1 with one exception for BH104 where the GSV for CH4 reflects a CS2.

The results suggest it is unlikely that there will be a requirement for mitigation of ground gas for the proposed works.

4 Proposed Removal Works

4.1 Enabling and Demolition Works

Enabling works across the proposed development will include for removal of the WwTP and excavation of the associated buried structures and foundations.

The area around the sewage treatment works will be excavated to an average depth of approximately 2m BGL and replaced with suitable engineering fill to the proposed earthworks formation level to allow for the construction of the Dub 16 building, road, the proposed new water course at this location and associated site security fencing.

Demolition works of the WwTP will include the following:

- The fence shall be demolished including its foundations and all paving shall be grubbed up. The hut shall be demolished including excavation of ground floor and foundations.
- All tanks and buried structures shall be excavated and demolished, including the clinker in the sedimentation tanks.
- All underground services, including any pipes, chambers etc shall be excavated.
- Demolition waste shall be segregated and removed to appropriate waste handling facilities.

The appointed contractor will be responsible for regular testing of excavated soils to monitor the suitability of the soil for recovery or disposal. A minimum frequency of compliance testing will be agreed with the appointed contractor and spot samples of any soils identified on site exhibiting gross contamination shall be tested during the enabling works.

Throughout the works, while good construction practices will be undertaken as outlined in Section 4.2 below, continuous monitoring of the work by a qualified engineer will be undertaken to determine if further validation testing is required (i.e., testing of the soils beneath the former tanks will be undertaken if necessary).

Based on the results of on-site testing, ground excavated from the WwTP site will be disposed of to a suitably licensed or permitted site in accordance with current Irish waste management legislation.

While it is unlikely that groundwater will be encountered during construction, any dewatering in areas of contaminated ground shall be designed by the appointed contractor to minimise the mobilisation of contaminants into the surrounding environment.

Following the demolition, the area will be backfilled to the required earthworks formation levels with suitable fill material to be prescribed in the construction earthworks specification.

4.2 Construction management

As outlined in INXN DUB1516 Soils and Geology, good construction management practices shall be employed to minimise the risk of transmission of hazardous materials as well as pollution of adjacent watercourses and groundwater.

The construction management of the site will take account of the recommendations of the CIRIA guidance Control of Water Pollution from Construction Sites – Guidance for consultants and contractors (Masters-Williams et al., 2001) to minimise as far as possible the risk of soil, groundwater and surface water contamination.

Measures to be implemented to minimise the risk of spills and contamination of soils and waters should include:

- Employing only competent and experienced workforce, and site-specific training of site managers, foremen and workforce, including all subcontractors, in pollution risks and preventative measures;
- Ensure that all areas where liquids (including fuel) are stored, or cleaning is carried out, are in designated impermeable areas that are isolated from the surrounding area and within a secondary containment system, e.g., by a roll-over bund, raised kerb, ramps or stepped access;
- The location of any fuel storage facilities shall be considered in the design of all construction compounds. These are to be designed in accordance with relevant guidelines and codes of best practice and will be fully bunded;
- Good housekeeping at the site (daily site clean-ups, use of disposal bins, etc.) during the entire construction phase;
- All concrete mixing and batching activities will be located in areas away from watercourses and drains;
- Potential pollutants to be adequately secured against vandalism;
- Provision of proper containment of potential pollutants according to codes of best practice;
- Thorough control during the entire construction stage to ensure that any spillage is identified at early stage and subsequently effectively contained and managed; and
- Spill kit to be provided and to be kept close to the storage area. Staff to be trained on how to use spill kits correctly.

A contingency plan for pollution emergencies should also be developed by the appointed contractor prior to the commencement of works and regularly updated, which would identify the actions to be taken in the event of a pollution incident. It shall address, between others, containment measures, emergency discharge routes, a list of appropriate equipment and clean-up materials and notification procedures to inform the relevant environmental protection authority.

4.3 Materials Disposal

As discussed in Section 4.1, the enabling works across the proposed development will include for removal of the WwTP and excavation of the associated buried structures and foundations which will comprise:

- The disused sewage treatment plant area is approximately 75m x 40m and surrounded in mesh fencing. It contains overgrown vegetation/shrubs, two partially buried disused open-top sedimentation tanks both approximately 13m in diameter, a buried aeration tank approximately 12m x 4m, a clean water tank approximately 7m x 3m and a single stone hut approximately 2.5m x4.5m in area.
- The sedimentation tanks consist of brick and two metal agitators. The sedimentation tanks have been infilled with clinker which will be removed prior to construction.
- The aeration tank consists of brick and a metal mesh top.
- The hut consists of stone brick and small sheet of metal. The demolition of the stone hut will consist of approximately 20 tonnes of stone brick.

All demolished and removed material will be delivered for reuse and recycling where feasible and where such facilities exist. Where material is reused it will be undertaken in accordance with "Article 27" of the European Union (Waste Directive) Regulations, 2011 to 2020.

Where wastes from the site are generated and can be reused, where feasible these will be delivered for recycling to a facility authorised in accordance with the Waste Management (Facility Permit and Registration) Regulations, 2007 as amended or an EPA licence.

Where hazardous wastes are generated this will be delivered to an EPA licenced waste facility or shipped to an authorised hazardous waste treatment, recovery or disposal facility in accordance with the transfrontier shipment regulations (Waste Management (Shipments of Waste) Regulations, 2007 as amended)

Typical construction and demolition waste types which are likely to arise during the proposed demolition and construction works are set out **Appendix 1**, including EPA List of Wastes (LOW) codes.

5 Conclusion

The WwTP was decommissioned at some time prior to 1995. The proposed remediation will include the demolition and removal of all remaining WwTP structures on site and the excavation of the existing soils to an approximate level of 2mBGL. This is due to take place during the enabling works phase of the project.

Ground investigation and laboratory testing in and around the site show that there are soils which will require disposal to inert, non-hazardous and hazardous licenced landfills. The excavation will be monitored for any evidence of further contamination, and validation sampling and testing will be carried out.

All demolished and removed material will be delivered for reuse and recycling where feasible and where such facilities exist. Where material is reused it will be undertaken in accordance with "Article 27" of the European Union (Waste Directive) Regulations, 2011 to 2020.

The excavation will be backfilled to the required level with suitable materials to allow for the safe construction of the Dub 16 building, the road, proposed new water course at this location and associated site security fencing, leaving the site in a suitable environmental condition for the proposed development.

Figures

Figure 2 Wastewater Treatment Plant (A3)

Appendix A

EPA EWC Codes - C&D Waste

Relevant Waste EWC Codes and Corresponding Waste Descriptions

03	**WASTES FROM WOOD PROCESSING AND THE PRODUCTION OF PANELS AND FURNITURE, PULP, PAPER AND CARDBOARD**
03 02	**wastes from wood preservation**
03 02 01*	non-halogenated organic wood preservatives
03 02 02*	organochlorinated wood preservatives
03 02 03*	organometallic wood preservatives
03 02 04*	inorganic wood preservatives
03 02 05*	other wood preservatives containing hazardous substances
03 02 99	wood preservatives not otherwise specified

13	**OIL WASTES AND WASTES OF LIQUID FUELS (except edible oils, and those in chapters 05, 12 and 19)**
13 07	**wastes of liquid fuels**
13 07 01*	fuel oil and diesel
13 07 02*	petrol
13 07 03*	other fuels (including mixtures)

15	**WASTE PACKAGING; ABSORBENTS, WIPING CLOTHS, FILTER MATERIALS AND PROTECTIVE CLOTHING NOT OTHERWISE SPECIFIED**
15 01	**packaging (including separately collected municipal packaging waste)**
15 01 01	paper and cardboard packaging
15 01 02	plastic packaging
15 01 03	wooden packaging
15 01 04	metallic packaging
15 01 05	composite packaging
15 01 06	mixed packaging
15 01 07	glass packaging
15 01 09	textile packaging

16	WASTES NOT OTHERWISE SPECIFIED IN THE LIST
16 02	**wastes from electrical and electronic equipment**
16 02 09*	transformers and capacitors containing PCBs
16 02 10*	discarded equipment containing or contaminated by PCBs other than those mentioned in 16 02 09
16 02 11*	discarded equipment containing chlorofluorocarbons, HCFC, HFC
16 02 12*	discarded equipment containing free asbestos
16 02 13*	discarded equipment containing hazardous components' other than those mentioned in 16 02 09 to 16 02 12
16 02 14	discarded equipment other than those mentioned in 16 02 09 to 16 02 13
16 02 15*	hazardous components removed from discarded equipment
16 02 16	components removed from discarded equipment other than those mentioned in 16 02 15
16 06	**batteries and accumulators**
16 06 01*	lead batteries
16 06 02*	Ni-Cd batteries
16 06 03*	mercury-containing batteries
16 06 04	alkaline batteries (except 16 06 03)
16 06 05	other batteries and accumulators
16 06 06*	separately collected electrolyte from batteries and accumulators

17	CONSTRUCTION AND DEMOLITION WASTES (INCLUDING EXCAVATED SOIL FROM CONTAMINATED SITES)
17 01	**concrete, bricks, tiles and ceramics**
17 01 01	concrete
17 01 02	bricks
17 01 03	tiles and ceramics
17 01 06*	mixtures of, or separate fractions of concrete, bricks, tiles and ceramics containing hazardous substances
17 01 07	mixtures of concrete, bricks, tiles and ceramics other than those mentioned in 17 01 06
17 02	**wood, glass and plastic**
17 02 01	wood
17 02 02	glass
17 02 03	plastic

17 02 04*	glass, plastic and wood containing or contaminated with hazardous substances
17 03	**bituminous mixtures, coal tar and tarred products**
17 03 01*	bituminous mixtures containing coal tar
17 03 02	bituminous mixtures other than those mentioned in 17 03 01
17 03 03*	coal tar and tarred products
17 04	**metals (including their alloys)**
17 04 01	copper, bronze, brass
17 04 02	aluminium
17 04 03	lead
17 04 04	zinc
17 04 05	iron and steel
17 04 06	tin
17 04 07	mixed metals
17 04 09*	metal waste contaminated with hazardous substances
17 04 10*	cables containing oil, coal tar and other hazardous substances
17 04 11	cables other than those mentioned in 17 04 10
17 05	**soil (including excavated soil from contaminated sites), stones and dredging spoil**
17 05 03*	soil and stones containing hazardous substances
17 05 04	soil and stones other than those mentioned in 17 05 03
17 05 05*	dredging spoil containing hazardous substances
17 05 06	dredging spoil other than those mentioned in 17 05 05
17 05 07*	track ballast containing hazardous substances
17 05 08	track ballast other than those mentioned in 17 05 07
17 06	**insulation materials and asbestos-containing construction materials**
17 06 01*	insulation materials containing asbestos
17 06 03*	other insulation materials consisting of or containing hazardous substances
17 06 04	insulation materials other than those mentioned in 17 06 01 and 17 06 03
17 06 05*	construction materials containing asbestos
17 08	**gypsum-based construction material**
17 08 01*	gypsum-based construction materials contaminated with hazardous substances
17 08 02	gypsum-based construction materials other than those mentioned in 17 08 01
17 09	**other construction and demolition wastes**
17 09 01*	construction and demolition wastes containing mercury

20	**MUNICIPAL WASTES (HOUSEHOLD WASTE AND SIMILAR COMMERCIAL, INDUSTRIAL AND INSTITUTIONAL WASTES) INCLUDING SEPARATELY COLLECTED FRACTIONS**
20 01	**separately collected fractions (except 15 01)**
20 01 01	paper and cardboard
20 01 02	glass
20 01 08	biodegradable kitchen and canteen waste
20 01 11	textiles
20 01 21*	fluorescent tubes and other mercury-containing waste
20 01 25	edible oil and fat
20 01 27*	paint, inks, adhesives and resins containing hazardous substances
20 01 33*	batteries and accumulators included in 16 06 01, 16 06 02 or 16 06 03 and unsorted batteries and accumulators containing these batteries
20 01 36	discarded electrical and electronic equipment other than those mentioned in 20 01 21, 20 01 23 and 20 01 35
20 01 39	plastics
20 01 40	metals
20 03	**other municipal wastes**
20 03 01	mixed municipal waste
20 03 07	bulky waste

A3

Legend

- WWTP Tanks
- Site Boundary
- Borehole with Rotary follow on locations
- Wastewater Treatment Plant boundary
- Foundation Inspection Pit locations
- Soak Away locations
- Trial Pit locations

Metres 0 1,800 3,600 7,200

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Drawing Title
WWTP features

Client
Confidental

Scale at A3
1:400

Job No
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Figure number
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