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Flood Risk Assessment

Issue | 23 July 2021

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Job number 280503-00

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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 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).

In order to assess the flood risk to the site, a 1D unsteady hydraulic model of the minor watercourse that flows through site was developed in order 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 has 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 approach and set the FFL of the Dub 15 data centre at 76.85mOD which is 0.55m higher than the recommended flood defence level. It is proposed to set the FFL of the Dub 16 data centre to 77.84mOD which is 1.54m higher than the recommended level. It is also proposed to set the FFL of the energy centre to 76.5mOD which is 1.4m higher than the recommended level.

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 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.

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 in order 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;
- 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.

1.4 Site Description

The subject site is located in Profile Park, Kilcarbery, Co. Dublin as indicated in **Figure 1** below. The site is situated southwest of Dublin City, approximately 4.65km outside the M50, between the N7 and N4 national primary routes.

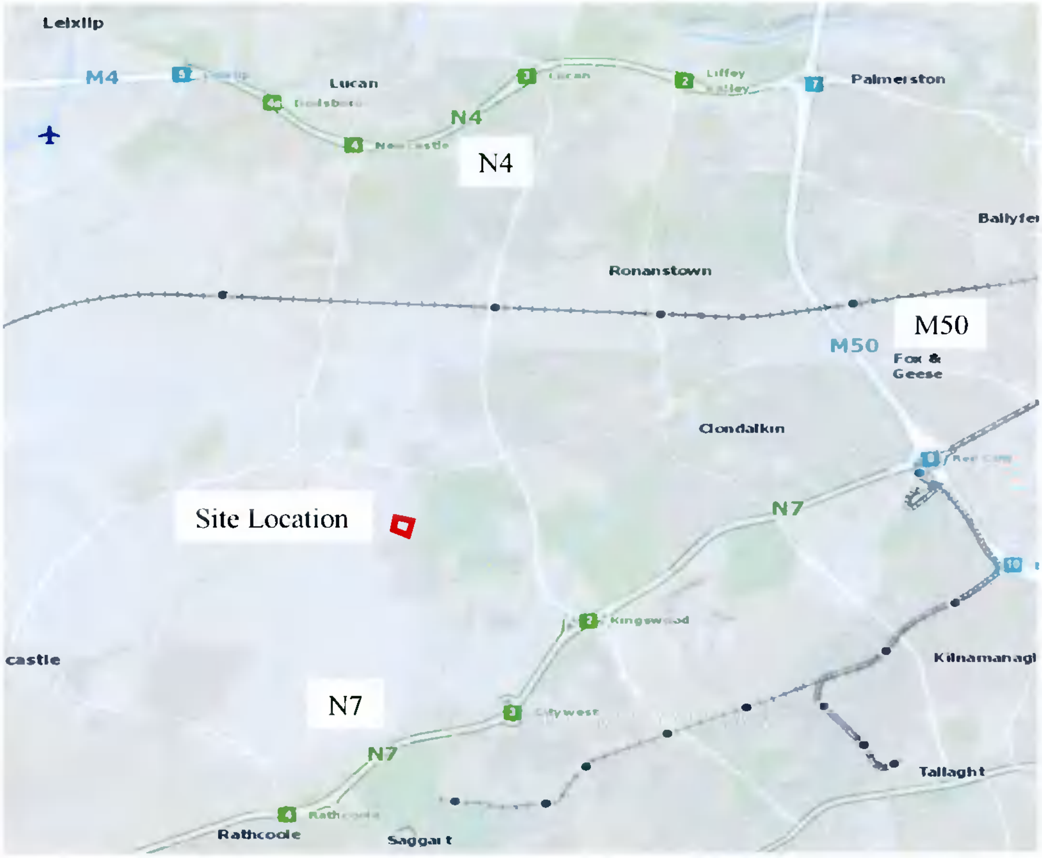

Figure 1: Profile Park location (Source: Google Maps)

There are 2 no. existing commercial buildings on the site (Figure 2) 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.

Figure 2: 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 3)**. This watercourse is an engineered 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 a number of culverts as described in Section 1.6 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 North West of the site as indicated in **Figure 3**. 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 3**. This structure is to be removed as part of the proposed development.

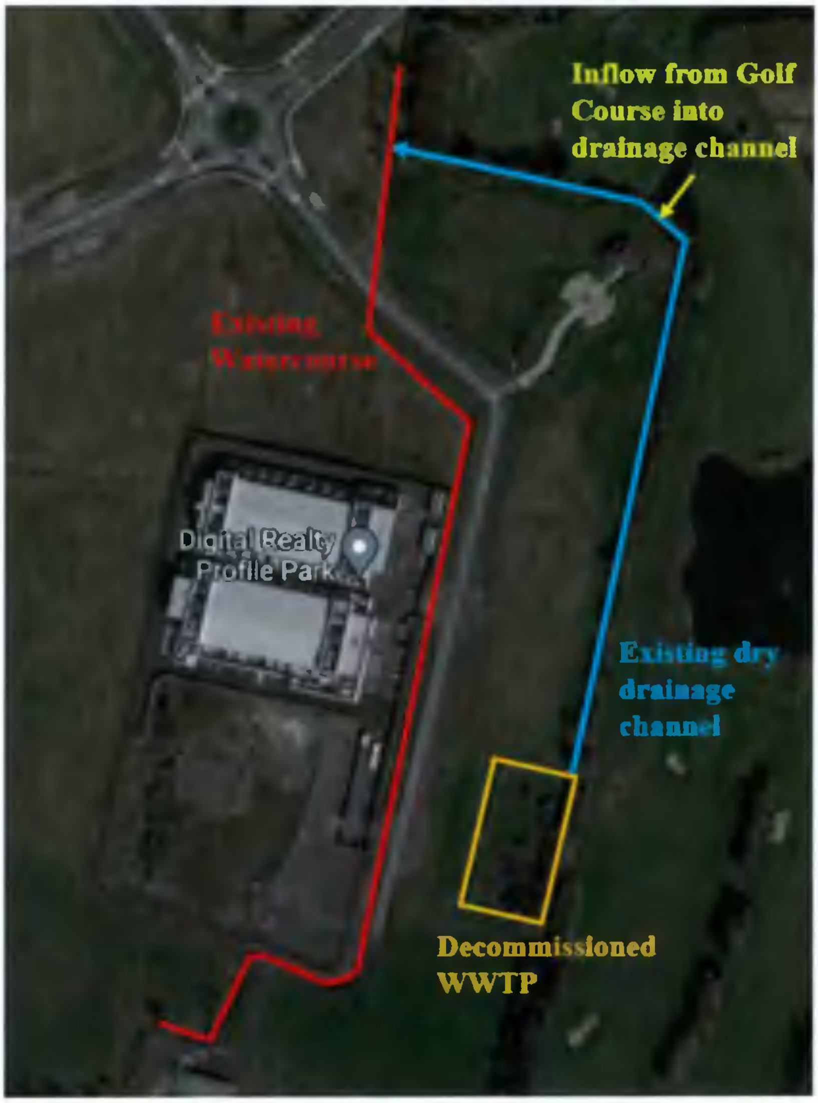

Figure 3: Figure showing water courses on the site

Figure 4 presents an extract from the Flood Studies Update (FSU) webportal and highlights the alignment of all the primary watercourses in the vicinity of the site. It can be seen that 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 webportal 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**.

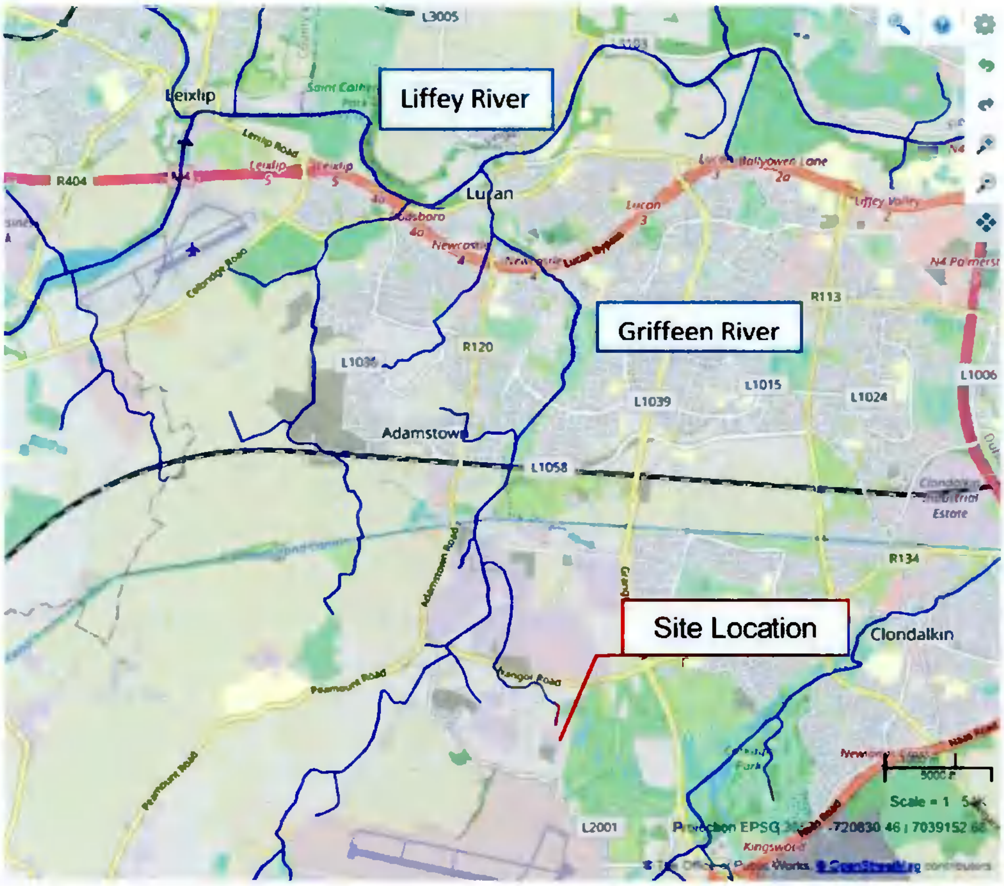

Figure 4: Watercourses in study area (Source: FSU Webportal)

1.6 Site visit

Arup visited the site on 28 May in order 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 5 presents a photograph of the existing channel upstream of the first culvert on the site. It can be seen that the channel is vee-shaped with a relatively high level of vegetation at the bottom of both the left and right bank.

Figure 5: 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 6**). These discharge points were fitted with non-return valves to ensure water cannot travel up the pipes.

Figure 6: Surface Water Discharge Points into the channel

Figure 7 presents a photograph of the confluence of the two minor watercourses circa 45m downstream of the site i.e. the confluence of the water course which runs through the site and the drainage channel which receives water from the Golf Course.

Figure 7: Confluence of both watercourses

1.7 Proposed Development

1.7.1 Buildings

The development will consist of the erection of two data centre buildings, gas powered energy generation compound, and all other associated ancillary buildings and works. The two data centre buildings, DUB 15 and DUB 16, will comprise a total floor area of c. 33,577m2 over two storeys. The first 2 storey data centre building (DUB15), located to the south west of the site, will comprise 16,865m2 data storage use, ancillary office use and associated electrical and mechanical plant rooms, loading bays, maintenance and storage space. A second 2 storey data centre building (DUB16), located to the south east of the site, will comprise 16,712m2 data storage areas, ancillary office use and associated electrical and mechanical plant rooms, loading bays, maintenance and storage space. Both data centre buildings will reach a height of 20m. Emergency generators and associated emission flues and plant are proposed in compounds adjacent to each data centre building. Gas powered energy generation is proposed to the north east corner of the site to provide electricity for the proposed development.

It is also proposed to remove an existing unused wastewater treatment facility on site.

1.7.2 Watercourses and landscaping

The application also proposes to re-route 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 aspect of the development is presented in **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. A Natura Impact Statement will be submitted to the planning authority with the application.

Figure 8 presents an outline of the development.

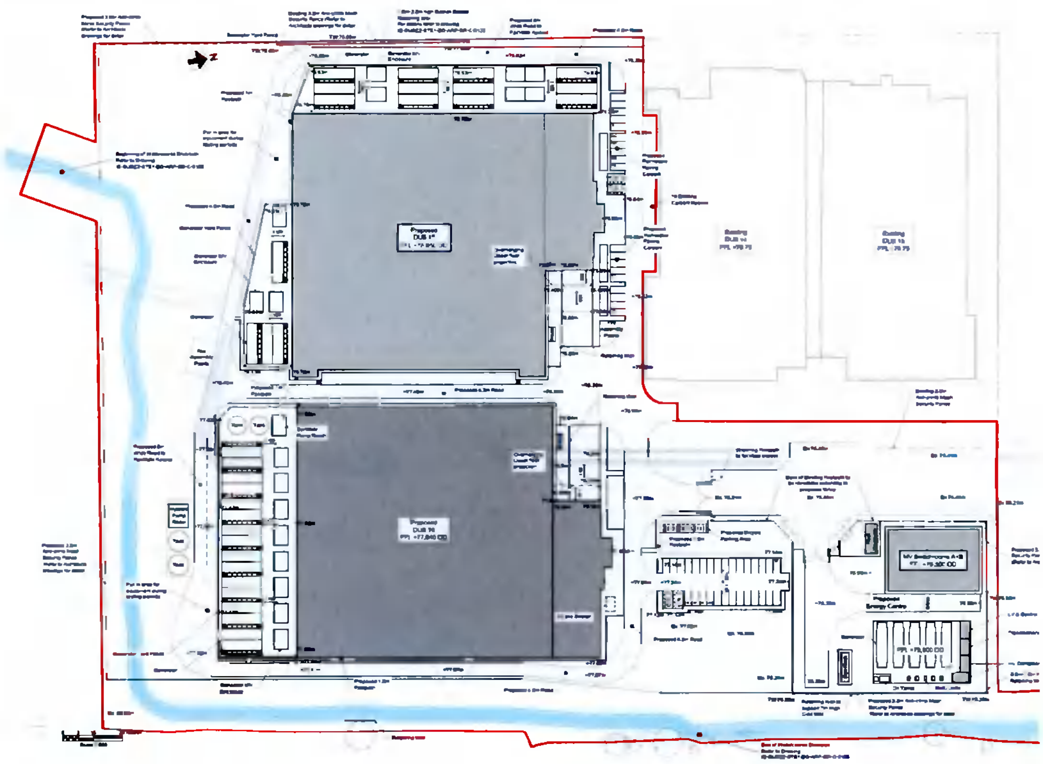

Figure 8: Proposed development

2 Overview of Flood Risk Policy

2.1 Overview

The two relevant planning policy documents are:

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

Both of these documents are now discussed.

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, Healthcares, 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. It can be seen that 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.

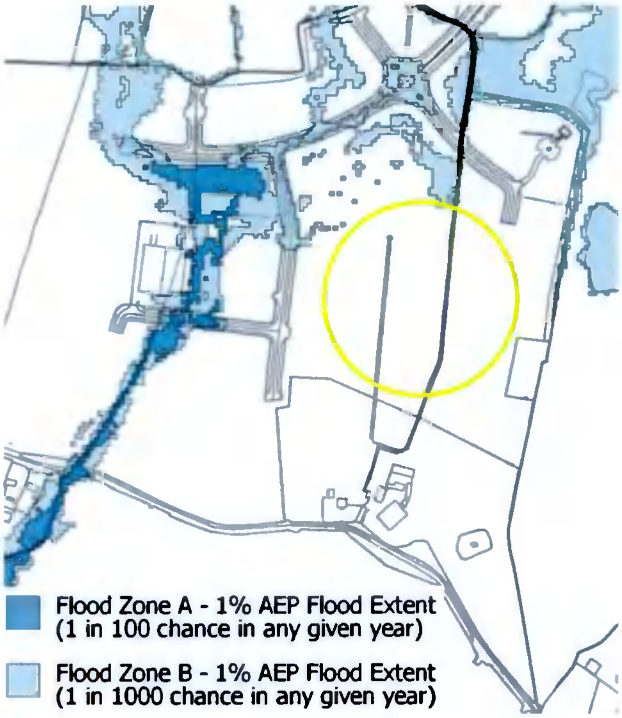

Figure 9: Flood risk map for the site as presented in the SDCC SFRA

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 can be categorised as:

- Fluvial (River) Flooding – Fluvial flooding occurs when excessive rainfall over an extended period of time 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 there is no risk of coastal flooding and it is therefore not considered further in the 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.

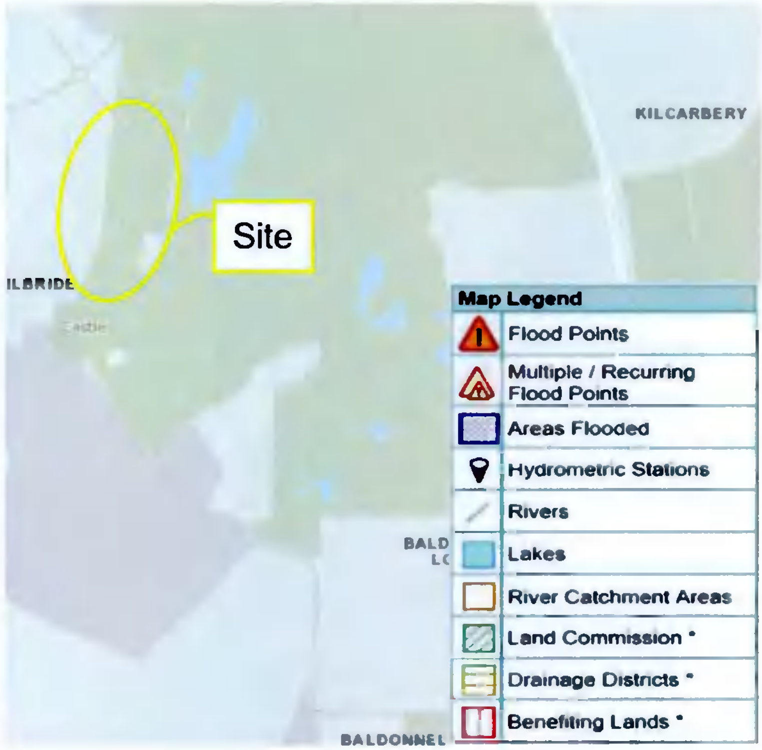

Figure 11: Extract from Floodinfo.ie

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 are indicated on the figure: 10 year, 100 year and 1000 year.

It can be seen that 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.

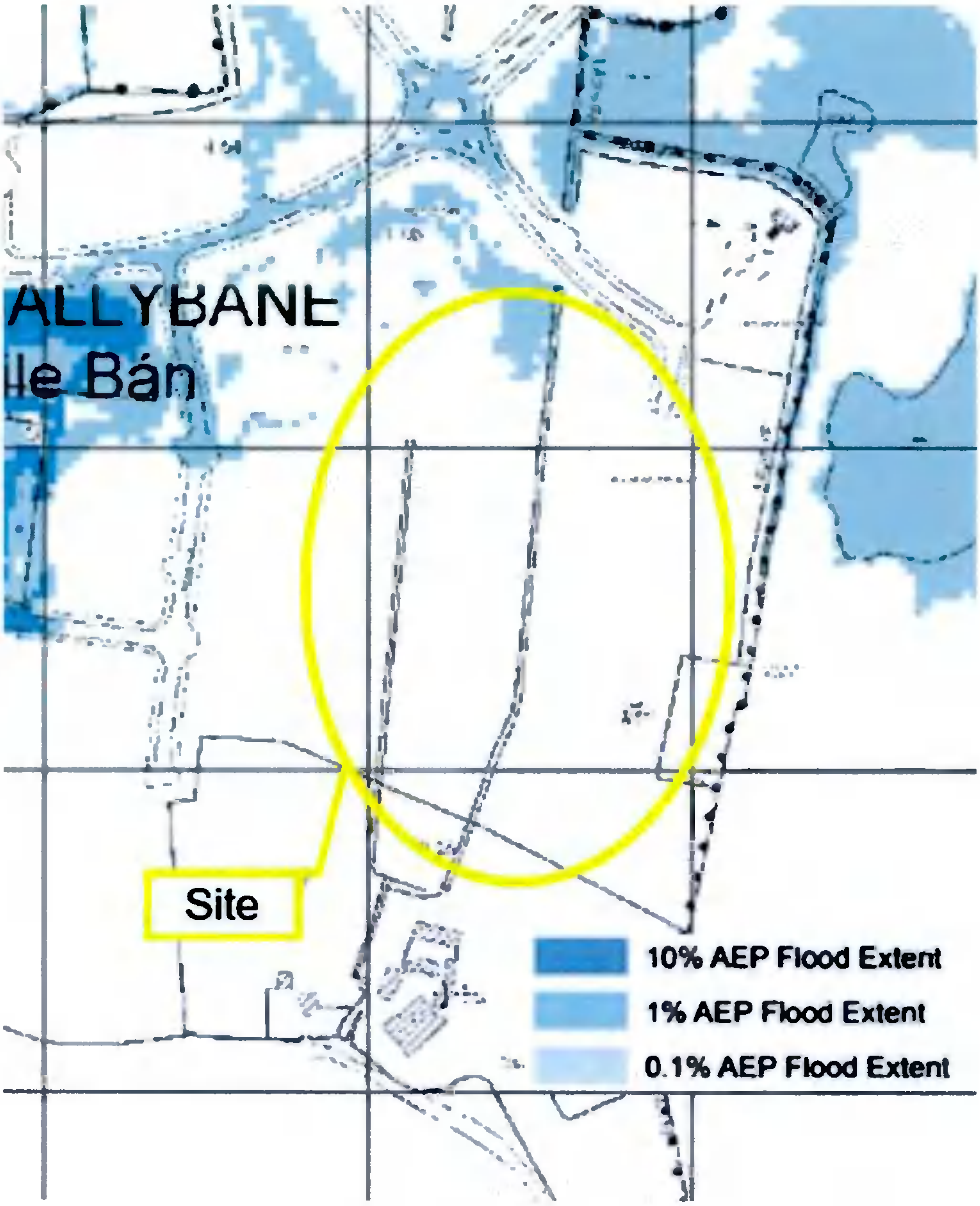

Figure 12: CFRAMS Fluvial flood extent map

It can also be seen from the floodmap that areas outside of the site boundary within the neighbouring Gold 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.

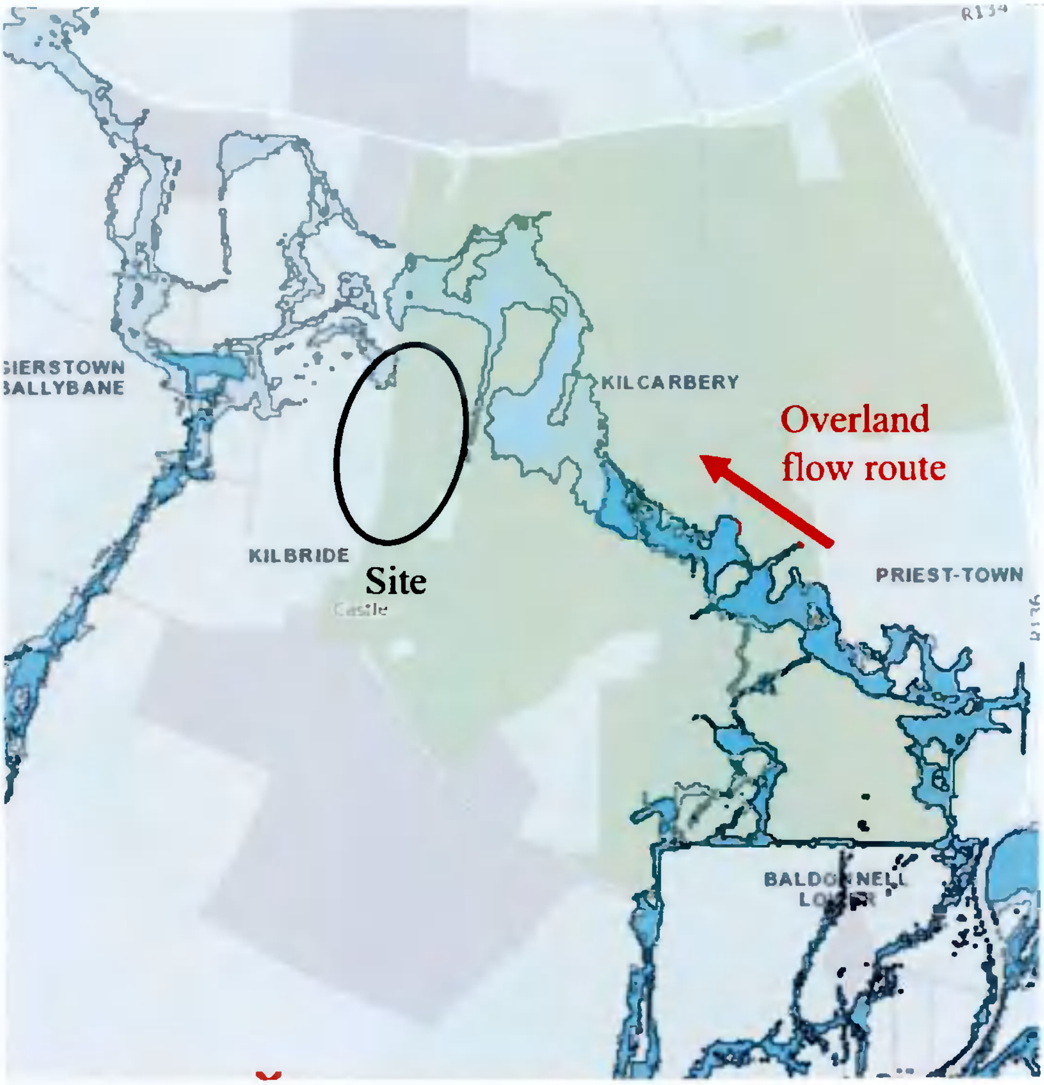

Figure 13: Overland flow route from the neighbouring river

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.

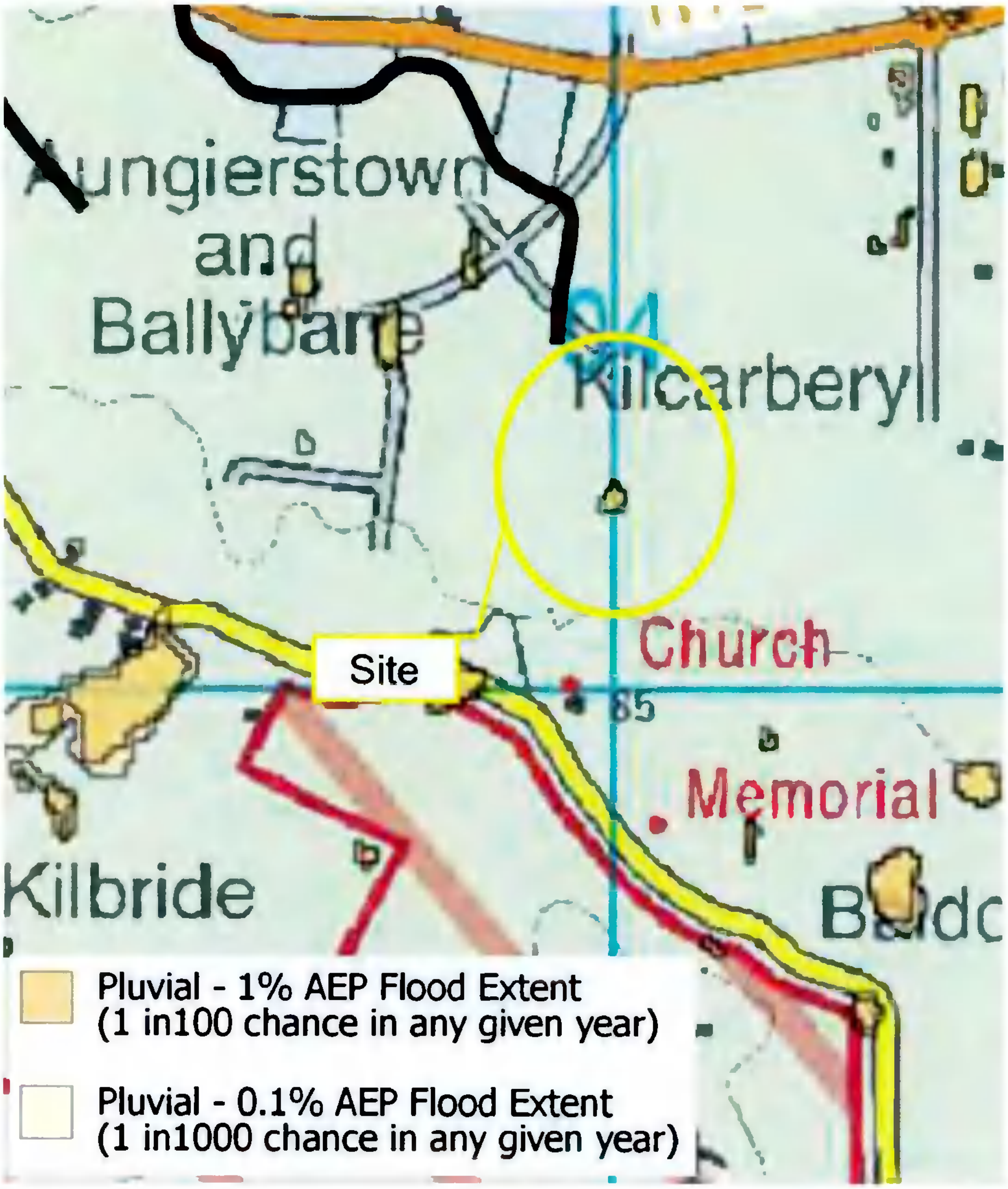

Figure 14: Pluvial flood map for the site

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 of the 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

In order to establish the existing flood risk and design water levels at the site associated with the minor water courses it is necessary to provide estimates of flood flows for the 1% AEP fluvial flood. This is 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 an allowance for climate change and freeboard.

As part of this study the Institute of Hydrology Report 124 (1H124) hydrological estimation method has been used to estimate the peak flows. Design hydrographs have been estimated using the Unit Hydrograph method which are then subsequently fitted to the peak flow estimates derived using the IH124 in order to generate the flood hydrographs for the site. This data is 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

In order to undertake the flood flow estimation, it is 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 is then carried out on the catchment contributing to each HEP in order to calculate the design flows at the HEP.

Two HEPs have been utilised in the study (**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 in the study

4.2.1 HEP 1 (US1)

Figure 17 presents an extract from the OPW's Flood Studies Update (FSU) portal which presents the catchment area and catchment characteristics of the FSU node closest to the site. It can be seen from the figure that the node is located downstream of the subject side. The total catchment area to this node is circa 1km2 such that the actual catchment area at the subject site is estimated to be circa 0.6km2.

In order to avoid an underestimation of the design flow at the subject site the catchment characteristics associated with FSU node downstream of the site have been used to inform the hydrological estimation for HEP 1. The design flows estimated by the study for this HEP are therefore conservative. This approach is however deemed prudent given the uncertainty associated with hydrological estimation for small catchments.

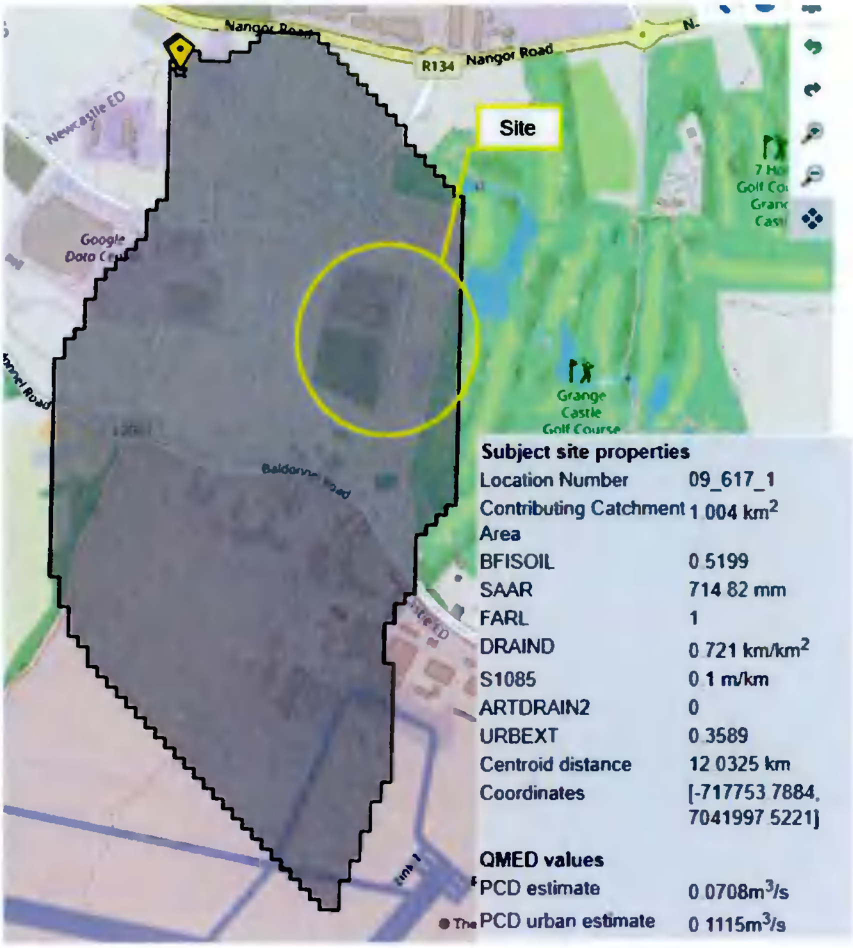

Figure 17: Catchment upstream of the site

4.2.2 HEP 2 (US2)

Figure 18 present the catchment area associated with the Golf Course inflow point. The catchment extent has been estimated using the data provided in the FSU database. The total area has been estimated as 0.36km2.

In order to avoid the underestimation of flow at the inflow point the full catchment area has been used as part of the hydrological 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) is applicable to small rural catchments (<25km2). The runoff estimate ($Qbar_{Rural}$) can be extended to estimate runoff from a partially urban catchment, $Qbar_{Urban}$.

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

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

Where

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

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

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

- 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 $NC = 0.92 - 0.00024SAAR$ for SAAR between 500mm and 1100mm

$NC = 0.74 - 0.000082SAAR$ for SAAR between 1100mm and 3000mm

The IH124 equation has a standard factorial error of approximately 1.65

The IH124 method has been used to determine the design flow rate for both of the HEPs in the study.

The Qbar is estimated as 0.25m3/s with the 95% confidence limit. As the Q100 growth curve is 1.96, the 95% confidence limit Q100 flow is estimated as 1.31m3/s for the current scenario which increases to 1.57m3/s for 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 results by the FSR Regional growth curve (1975) growth factor for the 100-year storm. The growth factor used for this event was 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 A**.

4.5 Overview of hydrology

The IH124 has been utilised to estimate design flood flows for the two HEP's relevant to the site. This method is suitable given its suitability for small rural catchments (i.e. less than 25km2 in size).

A conservative approach has been adopted by applying the flows estimated for a point downstream of the site at HEP 1. Conservatism is also ensured for the Golf course inflow by applying the flow for the whole sub catchment at the inflow point.

The 95%ile Q100 design flow with a 20% allowance for climate change has been adopted as the design flow for the study and will be used as the inflow boundary condition in the hydraulic modelling which is described in the following chapter.

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 in order to calculate design water levels for both the existing and proposed scenarios. This section of the report details the model development.

5.2 Topographic Survey

Two separate topographic surveys have been 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 water course and the culverts underneath the access road.

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

The survey data was processed in Civil 3D in order 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 in order to ensure that the channel geometry was described in high-definition in the model.

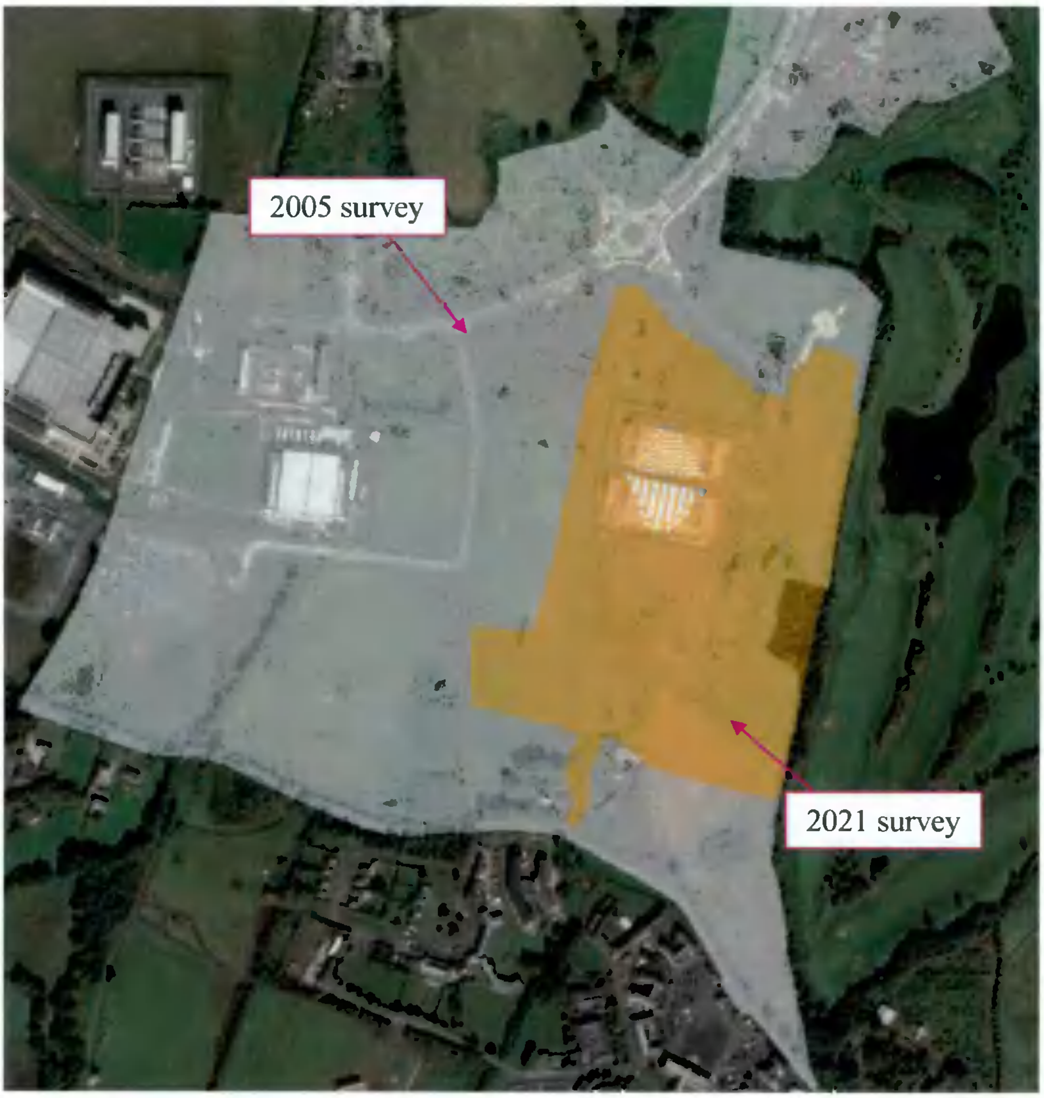

Figure 19: Topographic Survey

5.3 Model schematic – existing scenario

A schematic of the MIKE 11 existing scenario hydraulic model is presented in **Figure 20**. It can be seen that all the key hydraulic structures have been included in the model and that a large number of cross sections have been utilised in order to ensure that 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**.

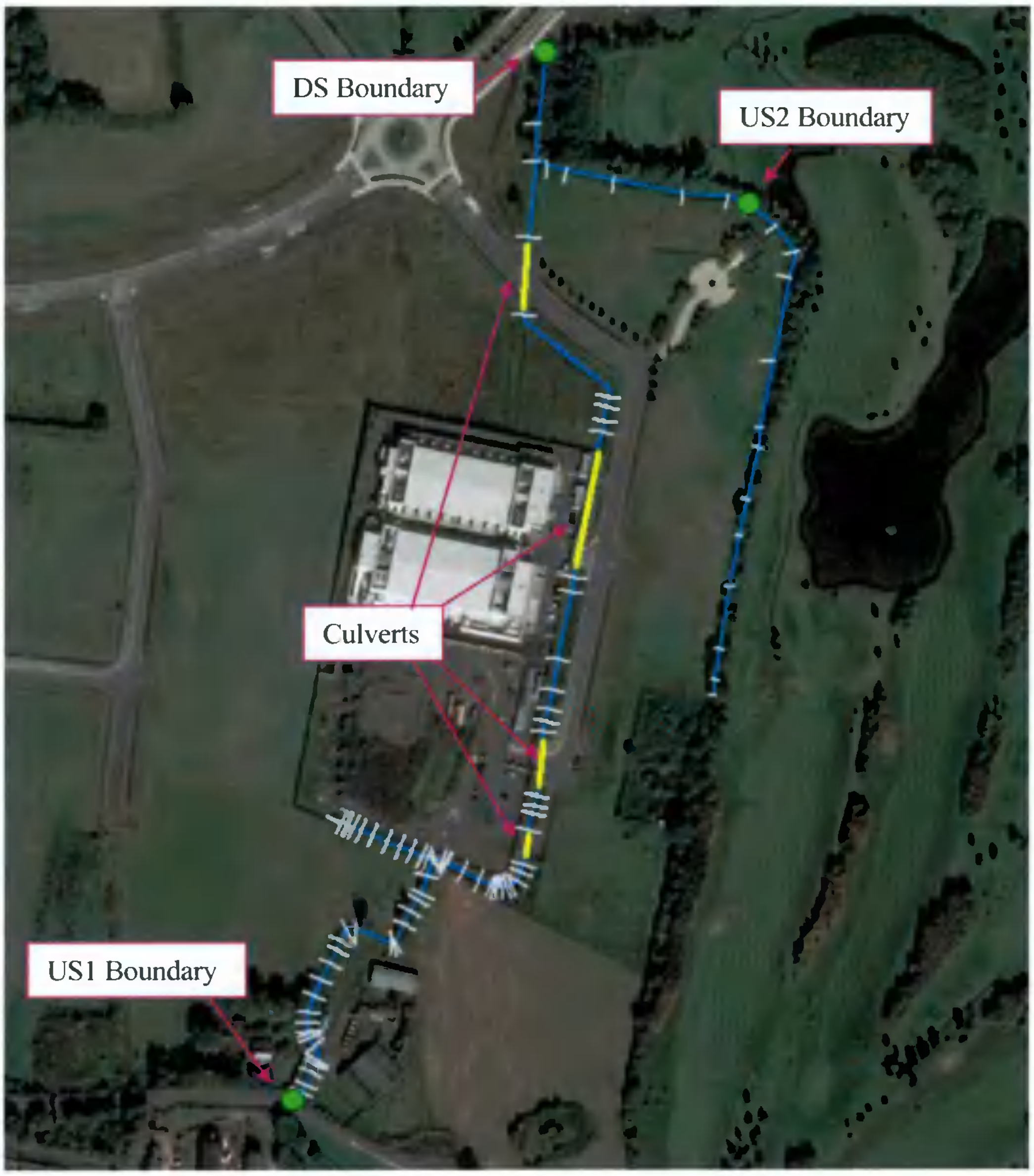

Figure 20: Hydraulic model schematic - Existing Scenario

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

It can be seen from the figure that the downstream boundary is located upstream of the bridge which forms part of the access road to site. From our site visit it was evident that the bridge opening is large enough to ensure there is sufficient capacity to pass a flood flow without any significant backwatering. The location of the downstream boundary is therefore deemed appropriate.

5.4 Model schematic – proposed scenario

A schematic of the proposed scenario hydraulic model is presented in **Figure 21**. It can be seen from the figure that the proposed channel involves constructing a new section of channel, utilising an existing section and upgrading an existing section.

In order to comply with the environmental requirements of SDCC, a 10m wide riparian zone will be incorporated as part of the proposed watercourse alignment.

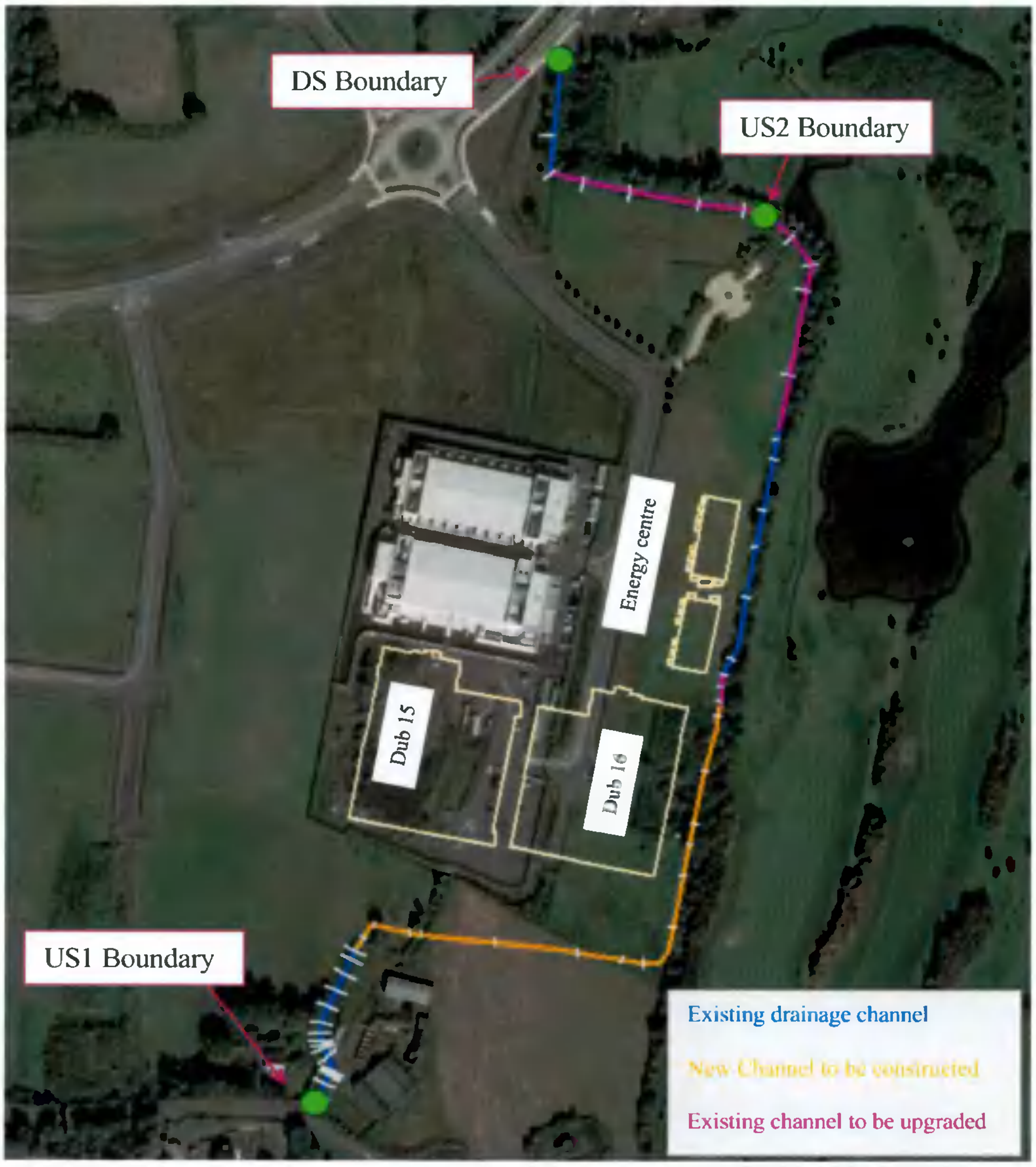

Figure 21: Hydraulic model schematic - Proposed Scenario

5.5 Boundary Conditions of the model

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

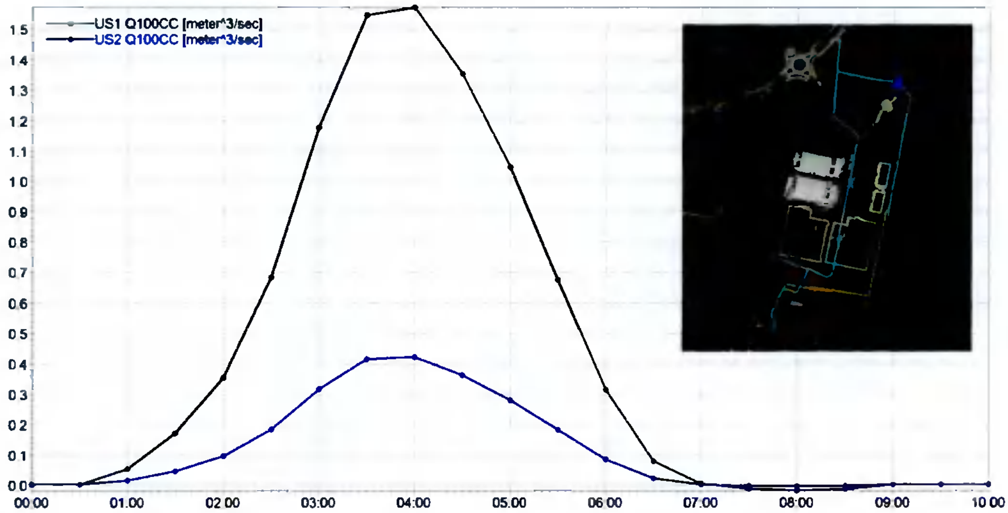

Figure 22: Upstream Boundary conditions

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

5.6 Model Calibration

The model was not calibrated against recorded data due to a lack of any suitable historic flood data from the site. The accuracy of the model however was therefore ensured by following best practice in the model build, adopting standard values of model parameters from the literature and utilising Arup extensive experience in hydraulic modelling.

5.7 Model Parameters

The Manning's n roughness values were selected based on our analysis of the existing vegetation along channel from our site visit. All other computational parameters were set based on best practice in modelling and our extensive experience in 1D river modelling.

5.8 Hydraulic structures

All of the culverts in the model have been modelled using the Culvert unit in MIKE as this is 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 have been taken from the surveyed data.

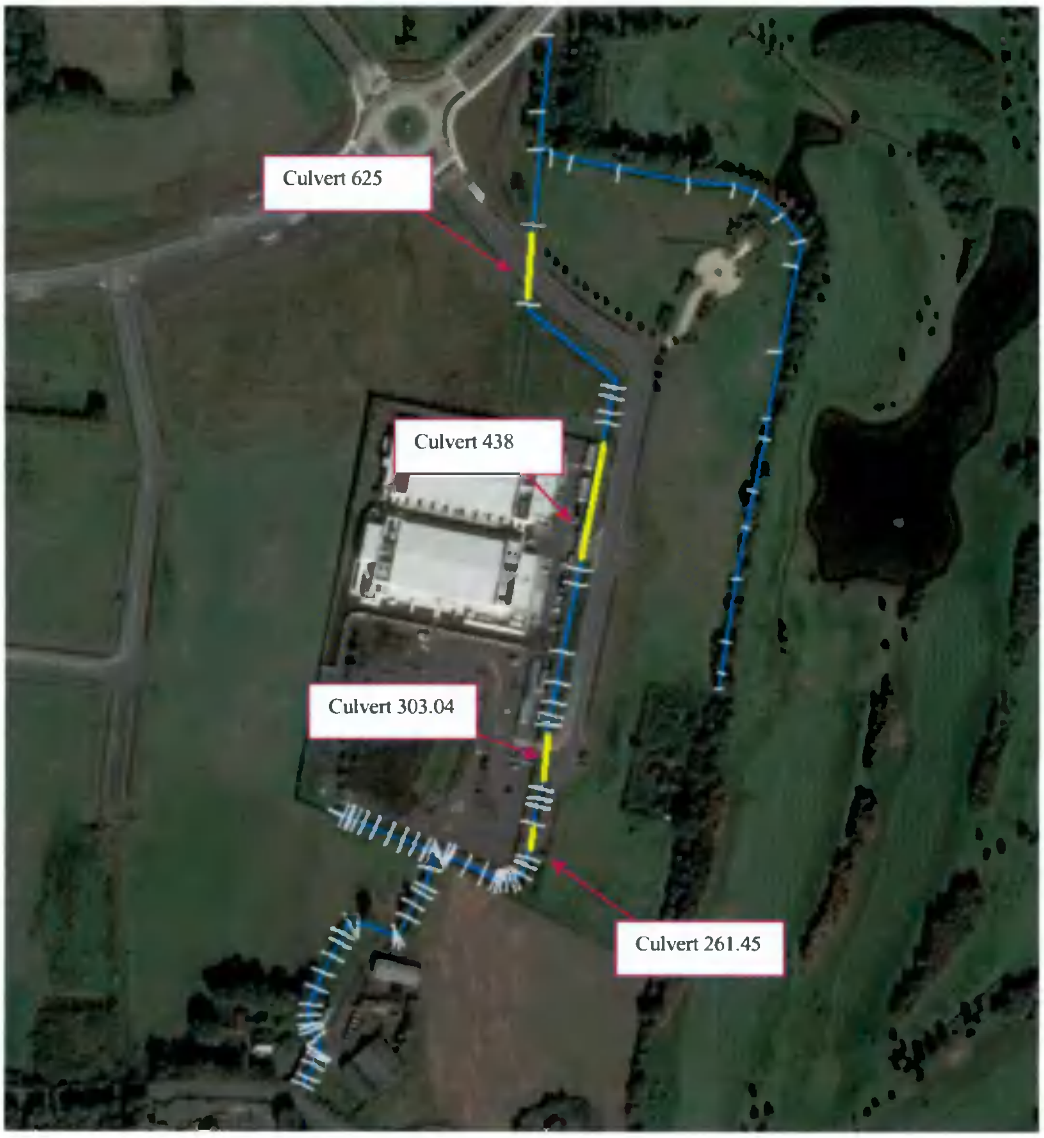

Figure 23: Culverts location – aerial view

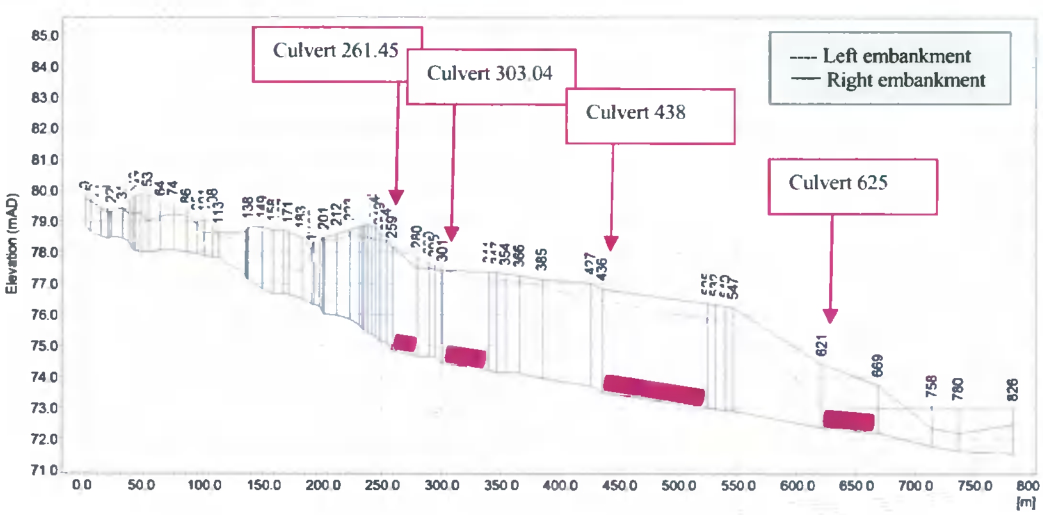

Figure 24: Culverts location – Longitudinal plot

6 Hydraulic modelling Results

6.1 Existing Scenario

The existing scenario 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 25**. It can be seen from the plot that the maximum water level varies along the reach from circa 79.4mOD at the upstream end of the reach to circa 72.2mOD at the downstream end. The water does not exit the channel at any location within the site boundary as there is sufficient capacity in the channel to accommodate the design flow.

Downstream of the site at cross sections 758 and 780 it can however be seen that the right hand bank of the water course is at risk of being overtopped.

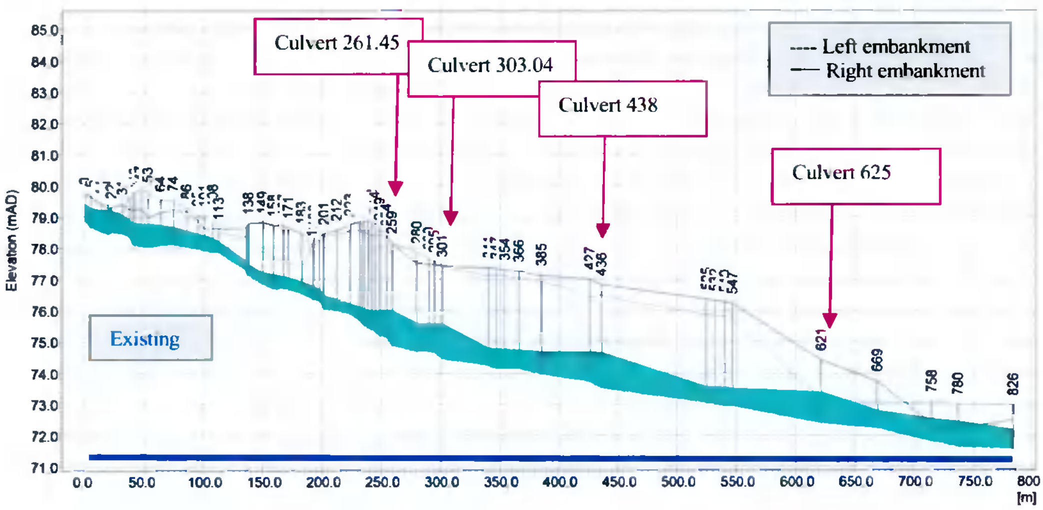

Figure 25: Maximum WL along the reach (95%ile Q100 +CC flow) - existing scenario

Figure 26 presents the maximum water level at cross section CRS 259 which is located immediately upstream of the site. It can be seen from the channel that the peak water level is great than 2m below the top of the bank at this location. A table of maximum water levels for this scenario is presented in the Appendices.

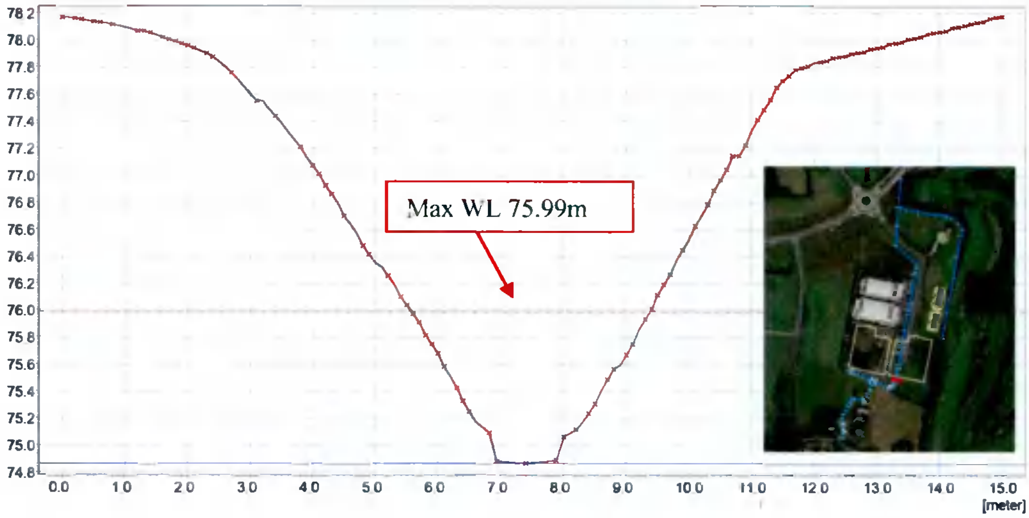

Figure 26: Maximum WL at CRS 259 (95%ile Q100 +CC flow) for existing scenario

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 of the reach in the vicinity of the confluence with the golf course. While no properties are at risk of inundation from this out of bank flow it is deemed unacceptable as is 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 on the water course to the East of the site). Engineering measures are therefore required to ensure flood risk to the golf course from the minor watercourse is addressed as part of the development.

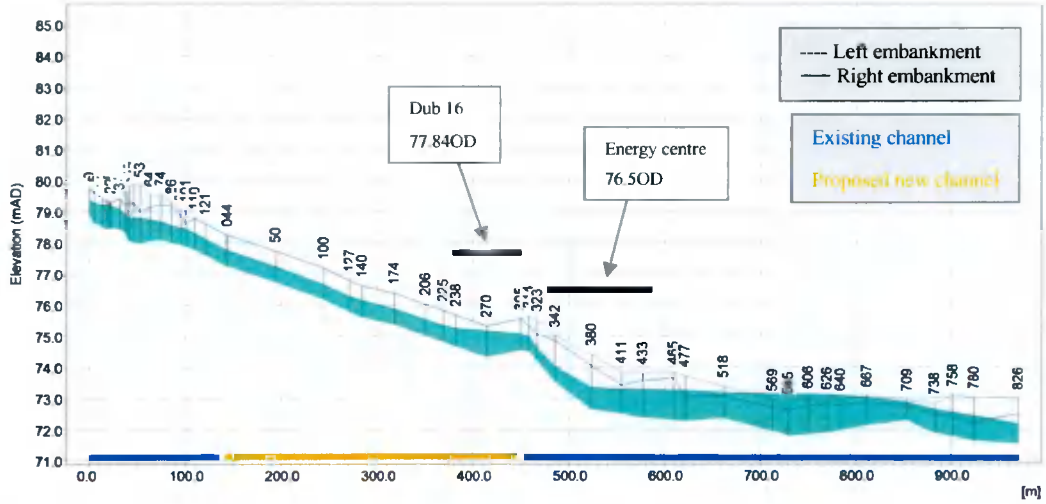

Figure 27: Maximum WL along the reach (95%ile Q100 +CC flow) – Proposed scenario with no conveyance improvement works

Modelled water levels through the reach for both the existing and proposed scenarios are presented in **Appendix A**.

6.3 Engineering measures to address flood risk at the site

A number of engineering measures were considered as part of the proposed development in order to manage flood risk downstream of the site. An overview of these measures are presented in the following table. Based on this assessment it was evident that conveyance improvement works offer the best solution to managing flood risk downstream of the site. A number of conveyance improvement works are therefore proposed for the downstream end of the reach in order to reduce the risk of flooding to both the site and the golf course. These are considered in the following section of the report.

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 in order 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 watercourse alignment. (Note: Due to a difference in elevation water cannot flow by gravity from the proposed to the existing channel)	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) 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	Neither of these options are deemed viable – the pumping option would be unfavourable to the 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 the channel in order to accommodate the excess flow in the watercourse	This option is deemed viable.

6.4 Proposed Scenario (with conveyance improvements)

It is proposed to implement a number of conveyance improvements along the existing drainage channel in order to reduce the flood risk along the watercourse with the development in place. These works consist of:

- Channel widening at the downstream end of the reach: it is proposed to widen the existing channel in the vicinity of the inflow from the golf course to 3m at the base in order to increase the capacity of the channel and reduce flood risk. A low flow section will be incorporated into the channel in order to eliminate very shallow water depths at low flows;
- Minor modifications to the bed levels at a number of discreet points in order to remove steep gradients from the channel (at cross section 314 and 709).

Further information on the conveyance improvements works are presented in Error! Reference source not found..

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 is kept in bank.

It can also be seen from **Figure 28** that the water level at the very downstream end of the reach at cross sections 758 and 780 (refer to **Figure 28** for a plan view location plot) exceeds the crest level of the right back. A small area of the golf course will therefore be at risk of flooding from the minor water course. As discussed in **Section 6.1** this area is however already at risk of flooding from the minor water course in the existing scenario.

The results of our analysis indicate that the proposed development does not increase the flood risk at this location or downstream of the site. This is discussed further in **Section 6.5**.

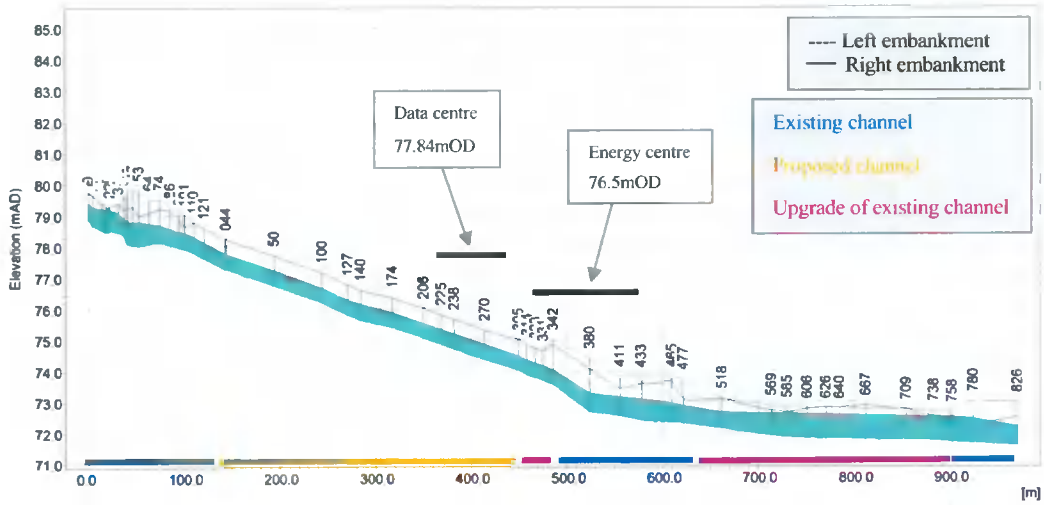

Figure 28: Maximum WL along the reach (95%ile Q100 +CC flow) – Proposed scenario with conveyance improvement works at the downstream end

Figure 29: Plan location of cross sections 758 and 780

Figure 30 presents the maximum water level at cross section 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.

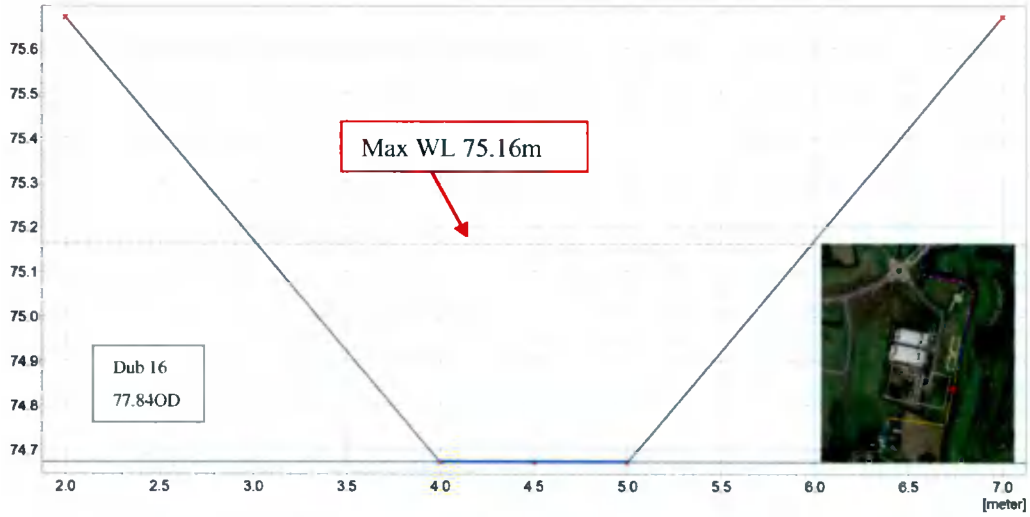

Figure 30: Maximum WL at CRS 238 (95%ile Q100 +CC flow) for existing scenario

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

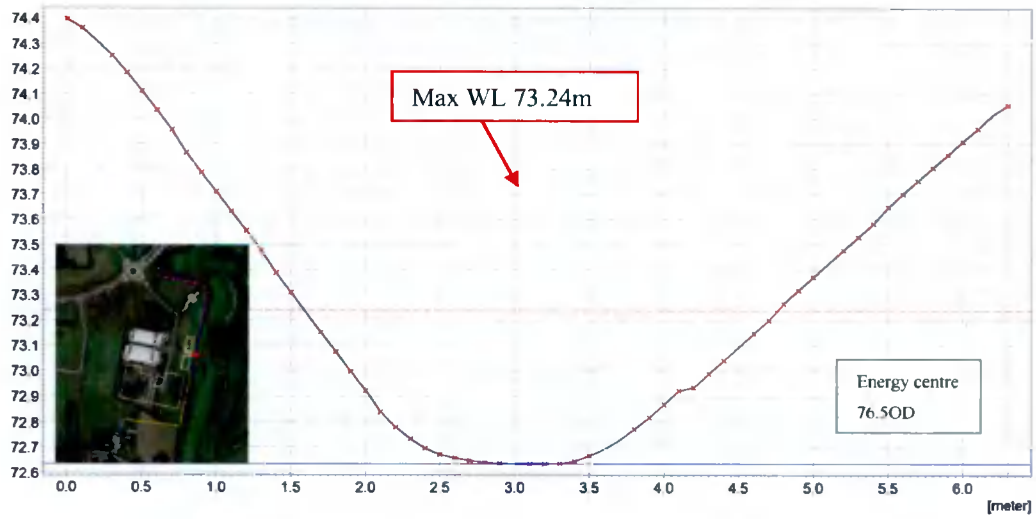

Figure 31: Maximum WL at CRS 380 (95%ile Q100 +CC flow) for existing scenario

6.5 Off-site impact

The impact of the development downstream of the site has been assessed by comparing the results of the existing scenario model with the results of the proposed scenario (with conveyance improvements) at the downstream end of the model at cross sections 826 and 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 is not increased with the proposed development in place.

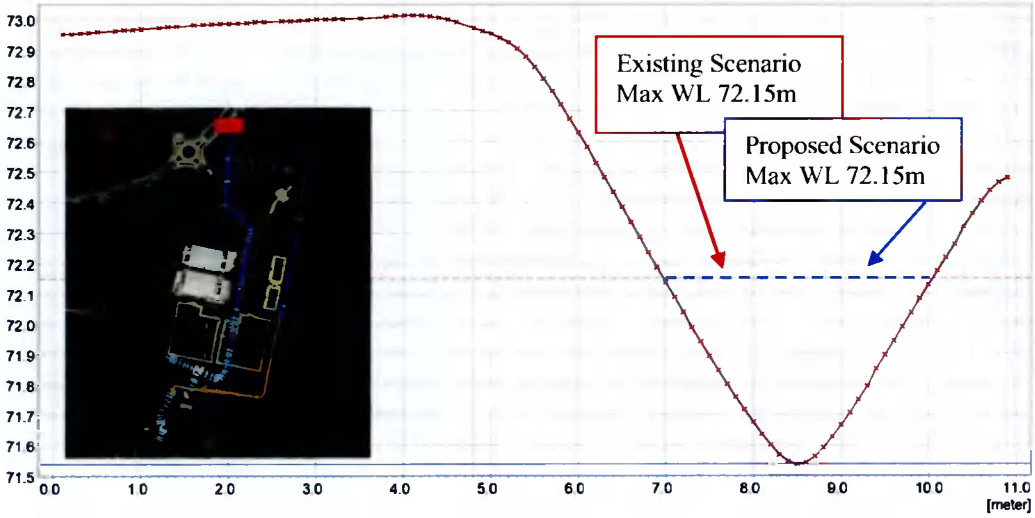

Figure 32: Maximum WL at CRS 826 (95%ile Q100 +CC flow) for existing and proposed scenarios

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.

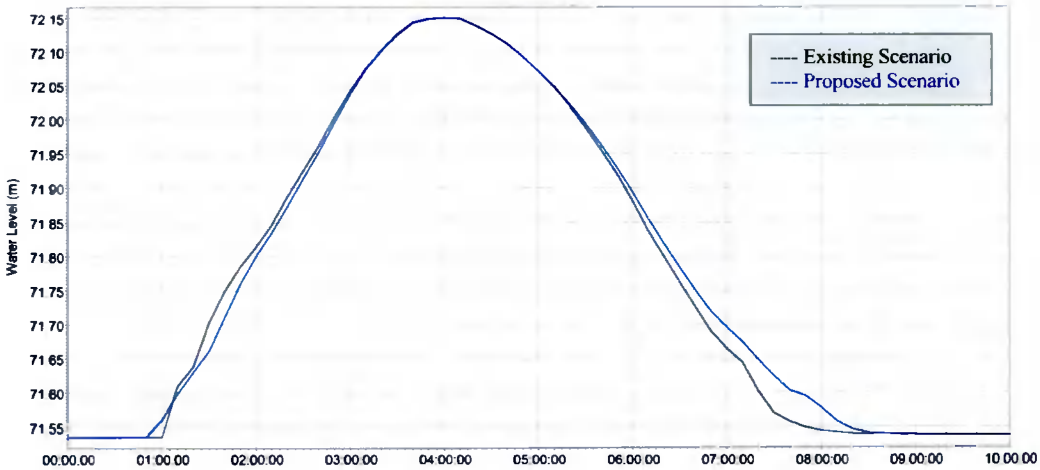

Figure 33: WL timeseries at CRS 826 (95%ile Q100 +CC flow) for existing and proposed scenarios

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.

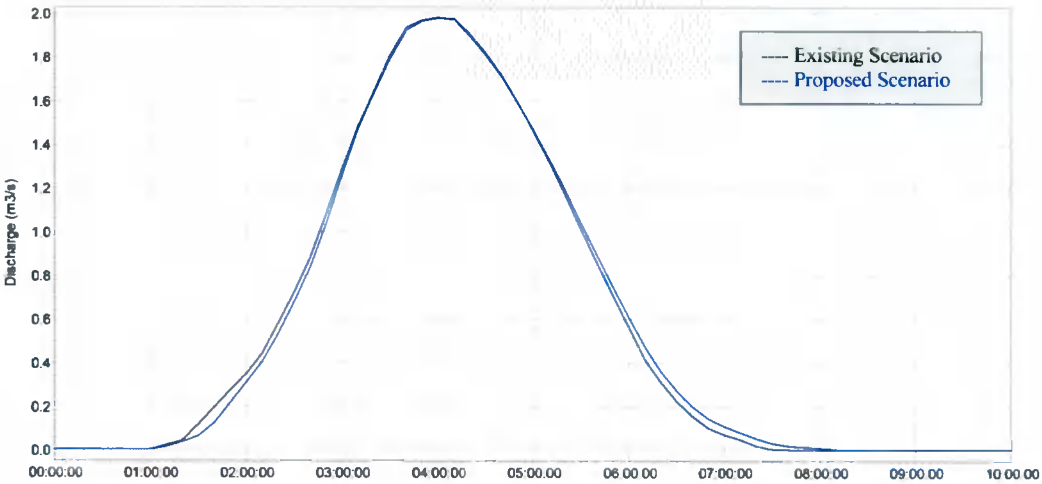

Figure 34: Discharge timeseries at CRS 826 (95%ile Q100 +CC flow) for existing and proposed scenarios

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.

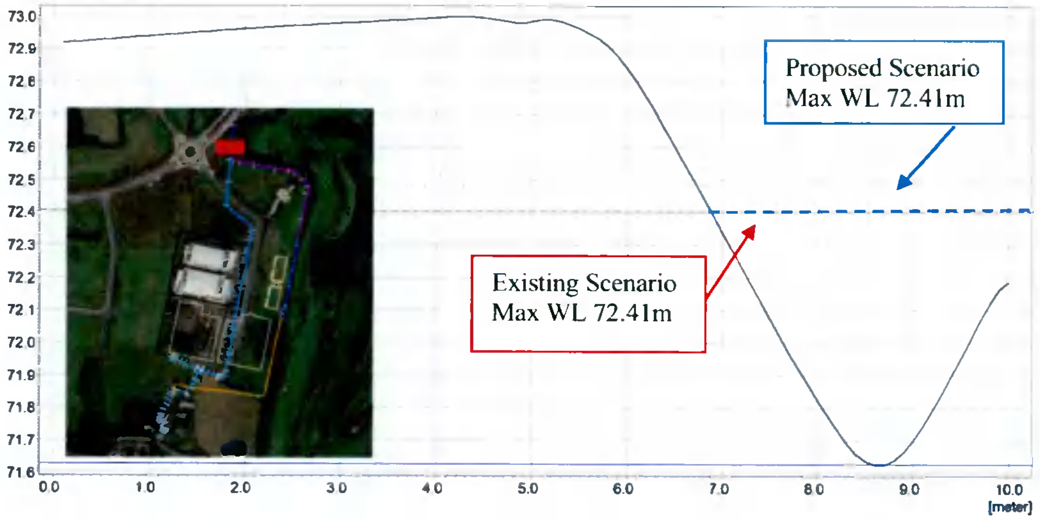

Figure 35: Maximum WL at CRS 780 (95%ile Q100 +CC flow) for existing and proposed scenarios

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.

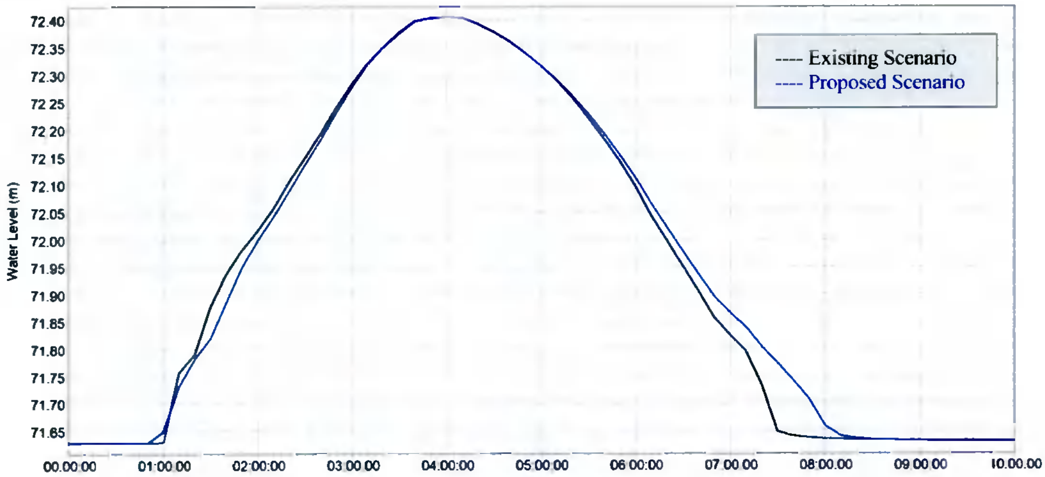

Figure 36: WL timeseries at CRS 780 (95%ile Q100 +CC flow) for existing and proposed scenarios

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.

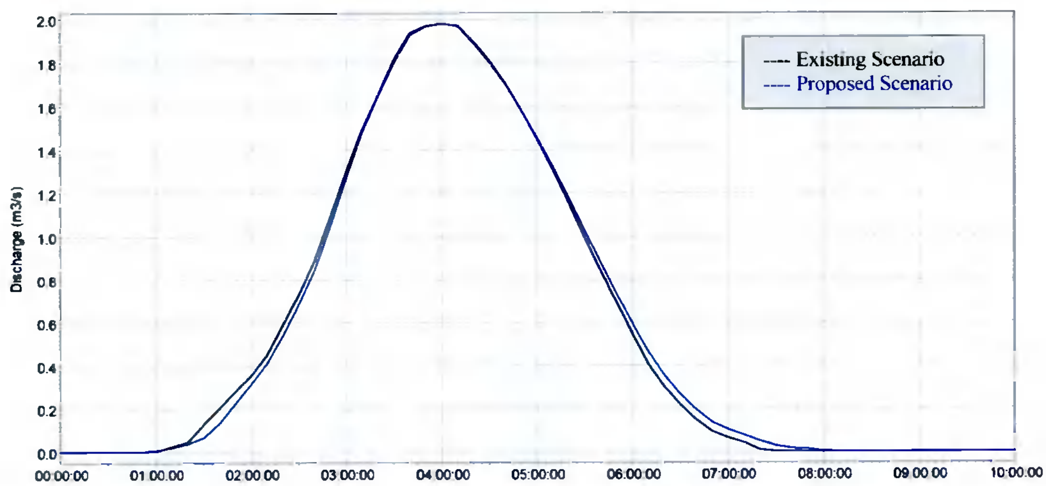

Figure 37: Discharge timeseries at CRS 780 (95%ile Q100 +CC flow) for existing and proposed scenarios

6.5.3 Conclusion of the off-site impact assessment

It is evident from the analysis that flood risk downstream of the site is not increased with the proposed development in place.

6.6 Conclusions of the Hydraulic modelling

The results of the hydraulic modelling have demonstrated that water does not get out of bank within the site for the existing scenario. Flood risk to the existing site is therefore very low. There is however risk of flooding outside of the site boundary at the downstream end of the modelled reach.

The results of the model have also indicated when conveyance improvements are considered as part of the proposed channel, water is kept in bank through the site. Flood risk to the site in the proposed scenario is also therefore very low. The risk of flooding at the very downstream end of the reach out site the site boundary will however remain.

The results of the hydraulic modelling has also clearly indicated that flood risk downstream of the site is not increased with the proposed development in place.

7 Management of flood risk at the Site

7.1 Finished Floor Levels of the Buildings on the site

7.1.1 Design water level

The 100 year plus allowance for climate change maximum design fluvial water level at our site of interest varies 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 energy centre the peak water level is circa 74.8mOD.

7.1.2 Freeboard

A detailed freeboard analysis has not been undertaken as part of this study as it is not within the scope of the work. 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 can therefore be calculated as:

- **Data centres:** 76.0mOD (100 year fluvial level + 20% increase in flow due to climate change) + 0.3mOD (freeboard) = 76.3mOD.
- **Energy centre:** 74.8mOD (100 year fluvial level + 20% increase in flow due to climate change) + 0.3mOD (freeboard) = 75.1mOD.

It is however proposed to adopt a conservative approach and set the FFL of the Dub 15 data centre at 76.85mOD which is 0.55m higher than the recommended level. It is also proposed to set the FFL of the Dub 16 data centre to 77.84mOD which is 1.54m higher than the recommended level.

It is also proposed to set the FFL of the energy centre to 76.5mOD which is 1.4m 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 Energy Centre development shall be drained by a completely separate system, with separate foul and surface water drains. The outfall of the proposed surface water system will discharge into the existing watercourse, which after completion of the proposed development will cater exclusively for surface water run-off coming from the proposed DUB 15, DUB 16, Energy Centre and existing DUB 13 and DUB 14 Data Centres.

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.

Surface water discharges from the proposed development will be restricted in line with South Dublin County Council (SDCC) Water Services requirements. Surface water discharges from the site will be restricted to 2 litres/second/hectare with flows in excess of the allowable discharge rate being retained on site in underground attenuation facilities for storms up to and including the 1 in 100 year event + 20% climate change allowance. The proposed surface water drainage strategy is divided into three separate online attenuation systems which will serve buildings DUB 15, DUB 16 and the Energy Centre separately.

The drainage systems shall be designed in accordance with Part H Building Regulations, BSEN 752 Drain and Sewer Systems outside Buildings, the Greater Dublin Regional Code of Practice for Drainage Works, the Greater Dublin Strategic Drainage Study (GDSDS) and to the requirements of South Dublin County Council and Irish Water.

The reader is referred to the drainage design engineering report that forms part of this planning application for further detail.

7.3 Access and egress routes

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

7.4 Conveyance and floodplain storage

Conveyance and floodplain storage will not be impacted by the proposed development given the low flood risk to the site.

7.5 Off-site impact

It has been demonstrated in Section 7.5 of this report that the proposed development will not increase flood risk downstream of the site.

7.6 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.

7.7 Impact of climate change

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

8 Application of Flood Risk Management Guidelines

8.1 Flood zones at the subject Site

The subject site is outside the 1000-year fluvial flood extent and is therefore classified as being within Flood Zone C.

8.2 Vulnerability Classification

It is considered that the proposed development should be classified as a "Less Vulnerable Development" as per the vulnerability classification in **Table 11**.

Table 11: Vulnerability classification as per the planning guidelines

Vulnerability class	Land uses and types of development which include*:
Highly Vulnerable development (include essential infrastructure)	Garda, ambulance and fire stations and command centres require to be operational during flooding; Healthcares; Emergency access and aggress points; Schools; Dwelling houses, student half of residence and hostels; Residential institutions such as residential care homes, children's homes and social service homes; Caravans and mobile home parks; Dwelling houses designed, constructed or adapted for the elderly or, other people with impaired mobility; and Essential infrastructure, such as primary transport and utilities distribution including electricity generated power stations and sub-stations, water and sewage treatment, and potential significant sources of pollution (SECESO sites, IPPC sites, etc.) in the event of flooding.
Less vulnerable development	Buildings used for: retail, leisure, warehousing, commercial, industrial and non-residential institutions; Land and buildings used for holiday or sort-let caravans and camping, subject to specific warning and evacuation plans; Land and buildings used for agriculture and forestry; Water treatment (except landfill and hazardous waste); Mineral working and processing; and

Vulnerability class	Land uses and types of development which include*:
	Local transport and infrastructure.
Water- compatible development	Flood control infrastructure; Docks, marinas and wharves; Navigation facilities; Ship building, repair and dismantling, dockside fish processing and refrigeration and compatible activities requiring a water side location; Water based recreation and tourism (excluding sleeping accommodation); Lifeguard and coastguard stations; Amenity open space, outdoor sports and recreation and essential facilities such as changing rooms; and Essential ancillary sleeping or residential accommodation for staff required by uses in this category (subject to a specific warning and evacuation plan).

8.3 Sequential Approach

The figure below illustrates the sequential approach to be adopted under the 'Planning Systems and Flood Risk Management' guidelines.

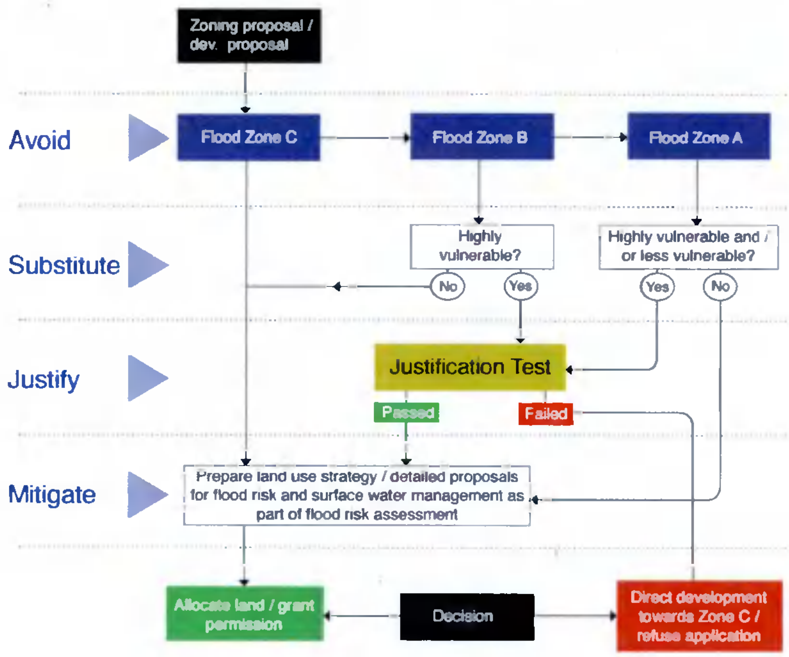

Figure 38: Sequential approach

As the proposed development lies within Flood Zones C, a Justification Test is not required, and it is necessary only to identify mitigation measures for any residual flood risk.

These have been identified as:

- Setting the FFL of all the buildings above the site flood defence level;
- Implementing conveyance improvements along the existing drainage channel on the site in order to minimise flood risk to the golf course;
- Minimising the risk of pluvial flooding to the development by ensuring that the FFL of the buildings are elevated above the ground levels surrounding the building.

9 Discussion

9.1 Overview

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.

As part of the study a 1D unsteady hydraulic model of the minor watercourse that flows through site was developed in order 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 has 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 approach and set the FFL of the Dub 15 data centre at 76.85mOD which is 0.55m higher than the recommended flood defence level. It is proposed to set the FFL of the Dub 16 data centre to 77.84mOD which is 1.54m higher than the recommended level. It is also proposed to set the FFL of the energy centre to 76.5mOD which is 1.4m higher than the recommended level.

Access and egress routes are 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 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.

Appendix A

Design Water Levels

A1 Existing Scenario

Table of peak water levels at model nodes through the reach.

Table 12: Maximum WL at all cross sections for existing scenario (95%ile Q100 +CC flow)

ID	Cross Section ID	Maximum WL(m)
EXISTING	0	79.39
EXISTING	5	79.27
EXISTING	14	79.21
EXISTING	19	79.2
EXISTING	21	79.19
EXISTING	22	79.18
EXISTING	31	79.06
EXISTING	37	78.91
EXISTING	40	78.79
EXISTING	42	78.78
EXISTING	47	78.77
EXISTING	53	78.78
EXISTING	64	78.73
EXISTING	74	78.66
EXISTING	86	78.54
EXISTING	95	78.45
EXISTING	101	78.39

EXISTING	108	78.29
EXISTING	113	78.18
EXISTING	136	77.57
EXISTING	137	77.51
EXISTING	138	77.46
EXISTING	149	77.2
EXISTING	158	77.15
EXISTING	167	77.08
EXISTING	171	77.03
EXISTING	183	76.9
EXISTING	191	76.78
EXISTING	191	76.78
EXISTING	193	76.75
EXISTING	198	76.65
EXISTING	200	76.49
EXISTING	201	76.45
EXISTING	212	76.34
EXISTING	223	76.19
EXISTING	231	76.09
EXISTING	234	76.07
EXISTING	236	76.03
EXISTING	238	76

EXISTING	242	75.97
EXISTING	244	75.96
EXISTING	249	76
EXISTING	254	75.98
EXISTING	259	75.99
EXISTING	280	75.58
EXISTING	290	75.58
EXISTING	295	75.56
EXISTING	301	75.56
EXISTING	341	74.77
EXISTING	347	74.76
EXISTING	354	74.75
EXISTING	366	74.72
EXISTING	385	74.66
EXISTING	427	74.64
EXISTING	436	74.64
EXISTING	525	73.55
EXISTING	532	73.55
EXISTING	540	73.53
EXISTING	547	73.49
EXISTING	621	73.37
EXISTING	669	72.77

EXISTING	758	72.51
EXISTING	780	72.41
EXISTING	826	72.15

A2 Proposed Scenario

Table of peak water levels at model nodes through the reach.

Table 13: Proposed Scenario – the 'do nothing scenario' – max WL – Q100 plus climate change

ID	Station	Max WL(m)
EXISTING	0	79.39
EXISTING	5	79.27
EXISTING	14	79.21
EXISTING	19	79.2
EXISTING	21	79.19
EXISTING	22	79.18
EXISTING	31	79.06
EXISTING	37	78.91
EXISTING	40	78.8
EXISTING	42	78.78
EXISTING	47	78.78
EXISTING	53	78.78
EXISTING	64	78.74
EXISTING	74	78.67
EXISTING	86	78.55
EXISTING	95	78.47
EXISTING	101	78.42
PROPOSED	110	78.31

PROPOSED	121	78.12
PROPOSED	143	77.76
PROPOSED	144	77.75
PROPOSED	0	77.75
PROPOSED	50	77.22
PROPOSED	100	76.64
PROPOSED	127	76.27
PROPOSED	140	76.14
PROPOSED	174	75.86
PROPOSED	206	75.51
PROPOSED	225	75.33
PROPOSED	238	75.23
PROPOSED	270	75.14
PROPOSED	305	75.06
EXISTING	314	74.97
EXISTING	323	74.6
EXISTING	342	74.01
EXISTING	380	73.33
EXISTING	411	73.28
EXISTING	433	73.26
EXISTING	465	73.24
EXISTING	477	73.23

EXISTING	518	73.16
EXISTING	569	73.12
EXISTING	585	73.12
EXISTING	606	73.11
EXISTING	626	73.09
EXISTING	640	73.07
EXISTING	667	73.01
EXISTING	709	72.85
EXISTING	738	72.6
EXISTING	758	72.51
EXISTING	780	72.41
EXISTING	826	72.15

Table 14: Proposed Scenario – updated sections – max WL – Q100 plus climate change

ID	Station	Max WL(m)
EXISTING	0	79.39
EXISTING	5	79.27
EXISTING	14	79.21
EXISTING	19	79.2
EXISTING	21	79.19
EXISTING	22	79.18
EXISTING	31	79.06
EXISTING	37	78.91

EXISTING	40	78.8
EXISTING	42	78.78
EXISTING	47	78.78
EXISTING	53	78.78
EXISTING	64	78.74
EXISTING	74	78.67
EXISTING	86	78.55
EXISTING	95	78.47
EXISTING	101	78.42
PROPOSED	110	78.31
PROPOSED	121	78.12
PROPOSED	143	77.76
PROPOSED	144	77.75
PROPOSED	0	77.75
PROPOSED	50	77.22
PROPOSED	100	76.64
PROPOSED	127	76.27
PROPOSED	140	76.14
PROPOSED	174	75.86
PROPOSED	206	75.51
PROPOSED	225	75.3
PROPOSED	238	75.16

PROPOSED	270	74.82
PROPOSED	305	74.45
UPDATED	314	74.32
UPDATED	323	74.22
UPDATED	331	74.13
EXISTING	342	74
EXISTING	380	73.24
EXISTING	411	73.12
EXISTING	433	73.05
EXISTING	465	72.92
EXISTING	477	72.83
PROPOSED	518	72.63
PROPOSED	569	72.58
PROPOSED	585	72.58
PROPOSED	606	72.57
PROPOSED	626	72.55
PROPOSED	640	72.54
PROPOSED	667	72.52
PROPOSED	709	72.49
PROPOSED	738	72.47
PROPOSED	758	72.46
EXISTING	780	72.41

EXISTING	826	72.15

Appendix B

Hydrological Estimation

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 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 = 0.17 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 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 x $AREA^{0.89}$ $SAAR^{1.17}$ $SOIL^{2.17}$

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

Appendix C

Master planning drawings

C1