MWP

STRATEGIC FLOOD RISK ASSESSMENT

South Ballincollig Drainage Study

Cork City Council

October 2021

Cork City Council

MWP

Contents

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Glossary of Acronyms and Terms

AEP	Annual Exceedance Probability
API	Antecedent Precipitation Index
CFRAMS	Catchment Flood Risk Assessment and Management Study
DEFRA	Department for Environment, Food and Rural Affairs
DTM	Digital Terrain Model
EPA	Environmental Protection Agency
FFL	Finished Floor Level
FRA	Flood Risk Assessment
FSR	Flood Studies Report
FSU	Flood Studies Update
GDSDS	Greater Dublin Strategic Drainage Study
HEP	Hydrological Estimation Point
HEFS	High End Future Scenario
LAP	Local Area Plan
mOD	Metres Above Ordnance Datum
MRFS	Mid Range Future Scenario
MWP	Malachy Walsh & Partners
OPW	Office of Public Works
PSFRM	The Planning System and Flood Risk Management Guidelines, November 2009
SAAR	Standard Average Annual Rainfall
SuDS	Sustainable Urban Drainage Systems

0. Non-Technical Summary

0.1 Introduction

In order to advance the planning objectives of the Cork City Draft Development Plan 2022 - 2028 and inform the overall infrastructure masterplan, a Strategic Flood Risk Assessment (SFRA) was completed to inform the design, layout and flood resilience of new development in the Ballincollig (Maglin) Urban Expansion Area. The subject lands cover approximately 220 ha and are zoned for mixed use including residential, educational, and recreational.

This report was prepared in accordance with *The Planning System and Flood Risk Management – Guidelines for Planning Authorities, November 2009 (PSFRM)*, published by the Office of Public Works and the Department of Environment, Heritage and Local Government.

0.2 Methodology & Report Structure

The methodology adopted for this study and the corresponding report structure is summarised as follows;

1. Complete a site characterisation and desktop study of the Plan Area and surrounds, as outlined in Section 2;
2. Review the flood history & existing flood risk information in the Plan Area based on available information, as described in Section 3;
3. Acquire and review all available data required to complete the SFRA, identify information gaps and obtain supplementary data where necessary, as summarised in Section 4;
4. Use the findings of the desktop studies to identify all possible sources of flooding that may affect the Plan Area, see Section 5
5. Complete a hydrological analysis and hydraulic model in order to quantify fluvial flood risk, as described in Sections 6, 7, and 8;
6. Consider the potential for pluvial and groundwater flooding in sufficient detail for the SFRA, as outlined in Sections 9 and 10;
7. Confirm the sources of flooding that pose a risk to the lands within the Plan Area, assess the risks from a strategic perspective, outline the required strategic flood risk management measures and provide recommendations for integrating the SFRA findings with the Development Plan. Refer to Sections 11 and 12.

0.3 The Planning System and Flood Risk

The Guidelines describe good flood risk practice in planning and development management and seek to integrate flood risk management into the planning process, thereby assisting in the delivery of sustainable development.

The core objectives are to:

- Avoid inappropriate development in areas at risk of flooding;
- Avoid new developments increasing flood risk elsewhere, including that which may arise from surface run-off;

- Ensure effective management of residual risks for development permitted in flood plains;
- Avoid unnecessary restriction of national, regional or local economic and social growth;
- Improve the understanding of flood risk among relevant stakeholders; and
- Ensure that the requirements of EU and national law in relation to the natural environment and nature conservation are complied with at all stages of flood risk management.

In the Planning System and Flood Risk Management Guidelines document, the likelihood of a flood occurring is established through the identification of Flood Zones which indicate a high, moderate or low risk of flooding from fluvial or tidal sources. The definition of Flood Zones as well as the implications for planning are summarised on Table 1.2.

0.4 Identification of Potential Flood Risk Sources

The existing flood risk information and flood history was reviewed in conjunction with observations from site walkovers and desktop studies in order to identify all possible sources of flooding which might affect the Plan Area. Based on this, the following flooding sources required further assessment in this SFRA:

1. Fluvial Flooding from the Maglin River and other artificial drains within the Plan Area;
2. Pluvial Flooding due to accumulation of surface water in topographical depressions;
3. Groundwater Flooding.

Comments on all potential sources of flooding are included on Table 5.1.

0.5 Hydrological Analysis

A detailed hydrological analysis was carried out to estimate the flood flows throughout the site and the surrounding areas. The results of this analysis were used as input parameters to the hydraulic modelling of the Study Area. Full details of the Hydrological Analysis are included in Section 6.

The hydrological analysis was extended to include an assessment of the likely extremity of the rainfall and the flood return period of the flood on the Maglin River on 19th February 2021. Flooding was observed at various locations within the Plan Area and photographic evidence of flood extents was collected. It was determined that the return period of the flood was most likely in the order to 2 to 3 years however given the potential spatial and temporal variations in rainfall and the limited hydrometric data available, the possibility of the flood return period being up to 5 years could not be ruled out.

Further information on the February 2021 flood event is provided in Section 7.

0.6 Fluvial Flood Risk & Baseline Hydraulic Modelling

A detailed hydraulic analysis was carried out which included modelling of approximately 3.2km of the Maglin North River, 2.1km of Maglin South River, circa 1km of the Curraheen River as well as the artificial drainage channel running through the western portion of the lands.

In order to ensure the model was performing to an acceptable level of accuracy, the model was calibrated using surveyed and estimated flood levels from the February 2021 event.

The model results were also compared to the results of the Lee CFRAM Study and it was found that the results from both models compare very well with predicted flood levels being generally within 50mm at all locations.

However, at the upstream (western) side of the lands the SFRA flood extents are larger because the Lee CFRAM study hydraulic model did not include all of the river network within the Plan Area.

It was identified that fluvial flooding from the Maglin River is the main source of flooding within the Plan Area and parts of the lands are located in Flood Zones A and B.

0.7 Flood Zones & Development Plan Land Zoning

Cork City Council reviewed the flood zones identified during the SFRA preparation and applied the sequential approach when making decisions on land zoning in the Plan Area.

The flood zones were overlaid on the proposed Development Plan Zoning. Highly Vulnerable and Less Vulnerable development has been located in Flood Zone C and lands within Flood Zones A and B are restricted predominantly to Water-compatible Development. However, the following exceptions apply:

1. The proposed Spine Road will be located across Flood Zones A, B and C. The Spine Road is considered to be Less Vulnerable Development corresponding to Local Transport Infrastructure therefore a Development Plan Justification Test was carried out.
2. The boundaries of some of the lands zoned for development other than water compatible development encroach slightly into Flood Zones A or B. The extent of lands encroaching into Flood Zones A or B other than those zoned as ZO 16 (Public Open Space) are highlighted in Figure 11.2. Development of these lands for High Vulnerability uses is not appropriate without a Justification Test and furthermore development for Less Vulnerable uses is not appropriate without Justification in Flood Zone A. Recommendations for addressing this have been provided.

Further details and mapping are provided in Section 11.2.

0.8 Post-Development Situation

The hydraulic model was adjusted for the post-development situation which includes for the proposed new spine road which will cross the river and floodplain at a number of locations. It was assumed for the purpose of this SFRA that no other alterations will be made to the river or floodplains. An indicative road alignment and footprint was agreed with Cork City Council to enable the SFRA to be completed.

Three possible options were considered for the spine road construction, summarised as follows:

Option 1 – No Mitigation

Only the spine road embankment and three culverts at the crossing points of the Artificial Drain and Maglin North River were included. The analysis indicated that this would cause increases in flood levels within the Plan Area and downstream of it due to a reduction in floodplain storage and changes to the flow regime through the site. Although the increases outside of the Plan Area are minimal, the option was not considered appropriate in the context of the Guidelines which require that a precautionary response to development on floodplains is taken in response to the potential incremental impacts in the catchment.

Option 2 – Additional Culverts

Additional flood relief and balancing culverts through the proposed road embankment were added to the model. The analysis indicated that the flood levels within the Plan Area would be reduced to coincide closely with the existing situation however it was found that the increased conveyance combined with the reduced floodplain storage tended to increase flood levels and peak flow further downstream. This was not considered acceptable in the context of the Guidelines.

Option 3 – Flood Relief Culverts & Compensation Storage

The third option considered involved the provision of compensation storage in the Public Open Space on the western portion of the site together with a series of flood relief and balancing culverts through the embankment. Also, the flow through the embankment crossing the eastern/downstream floodplain of Maglin North River was limited so that additional storage is provided upstream of the embankment.

The concept layout of the proposed spine road, culvert system and compensation storage areas is shown on Figure 8.8.

The analysis confirmed that this approach is the most effective combination of measures in terms of maintaining existing flow paths, minimising flood risk increases within the plan area and ensuring flood risk outside of the site boundary is not increased. There would be a localised increase in flood level within the Plan Area upstream of the eastern embankment however this is justified because the solution would help to ensure the development does not have an adverse impact on flood risk downstream of the site. Figure 8.9 shows the extent of lands affected by these increases and Figure 8.10 shows the corresponding flood level increase.

The analysis carried out for this SFRA is sufficient to demonstrate that the road construction is acceptable in the context of flood risk and that suitable mitigation can be incorporated into the development design.

Full details of the post development hydraulic model are included in Section 8.3.3 and the predicted flood levels at specific locations are summarised on Table 8.3.

0.9 Pluvial Flood Risk

A distributed rainfall analysis was carried out to determine the risk of pluvial flooding or ponding of surface water within the lands. Indicative flood risk maps were provided to highlight potential areas at risk however it is considered that these flood risk issues are best dealt with on a site specific basis with the benefit of local data and that the management of such risks can generally be achieved by suitable surface water management and mitigation. Pluvial flood risk is discussed further in Section 9.

0.10 Groundwater Flood Risk

There are no identified incidences of groundwater flooding within the Plan Area. Based on the information available for this SFRA, groundwater flooding is unlikely to be a significant risk however the risk should be considered on a site specific basis using local site investigation data. Notwithstanding this, it is envisaged that any possible risk associated with groundwater flooding would inherently be mitigated by incorporating a suitable surface water and fluvial flood risk management plan. Groundwater flood risk is discussed further in Section 10.

0.11 Mitigation Measures

A number of mitigation measures are outlined on Table 11.2 which are recommended from a SFRA perspective to ensure that the flood risk to developments within the Plan Area is acceptable and to prevent an adverse impact upstream or downstream of the site.

0.12 Residual Risks

The main residual risks associated with development of this area are:

1. A blockage occurring in Maglin Road Bridge, leading to increased flood levels and extents upstream of the bridge. It was concluded that the risk to property within the Plan Area is acceptable given the freeboard to finished floors recommended in this SFRA and that the additional flood extent can be managed at project level stage.
2. An exceedance flow or bridge/culvert blockage in Maglin South could flow across the N22 Road and cause flooding within the Plan Area. It was concluded that these risks can be appropriately managed by incorporating drains adjacent to the N22 road to intercept the flow before it enters the site.

0.13 Justification Test

Parts of the proposed spine road would be located within Flood Zone A therefore a Development Plan Justification Test has been carried out which is summarised on Table 11.3. It was concluded that the development of the spine road is acceptable in the context of the Planning System and Flood Risk Management Guidelines.

Prior to finalising the Development Plan, a Plan-making Justification Test may be required in relation to localised areas of lands which encroach into Flood Zones A or B. The extent of these lands are highlighted in Figure 11.2.

0.14 Recommendations for Development Plan Objectives

It is recommended that the Development Plan Objectives address the key requirements for sustainable flood risk management of these lands. These are summarised on Table 12.1.

0.15 Quick Locator

A summary of the key figures and tables along with their location within this report is provided on Table 0.1 below.

Item	Location
Identification of Potential Flood Risk Sources	Table 5.1, Page 30
Flood Zone Map (with Aerial Background)	Figure 8.5, Page 56
Comparison of SFRA and Lee CFRAM Study Predicted Flood Levels	Table 8.2, Page 57
Proposed Spine Road Alignment, Culverts and Compensation Storage Areas	Figure 8.8, Page 62
Flood extent and level increase Upstream of Eastern Spine Road Embankment	Figure 8.9, Figure 8.10, Page 63
Summary of Predicted Flood Levels (Post-Development)	Table 8.3, Page 64 Figure 8.11 and Figure 8.12, Page 65
Indicative Pluvial Flood Risk Maps	Figure 9.1 and Figure 9.2, Page 66
Flood Zone Map with Proposed Development Plan Zoning	Figure 11.1, Page 75
Extent of Lands in Flood Zones A or B which have a Zoning other than ZO 16 (Public Open Space)	Figure 11.2, Page 76
SFRA Mitigation Measures	Table 11.2, Page 79
Spine Road Justification Test	Table 11.3, Page 84
Recommendations for Development Plan Objectives	Table 12.1, Page 85

Table 0.1: Key Tables and Figures

1. Introduction to the Strategic Flood Risk Assessment

1.1 General

Malachy Walsh and Partners (MWP) Consulting Engineers have been commissioned by Cork City Council (CCC) to carry out a Strategic Flood Risk Assessment (SFRA) and Sustainable Drainage Strategy Study (SDS) in relation to the development of lands in Maglin, Ballincollig, Cork.

This report relates to the Strategic Flood Risk Assessment for the proposed Ballincollig (Maglin) Urban Expansion Area and has been prepared in accordance with *The Planning System and Flood Risk Management – Guidelines for Planning Authorities, November 2009 (PSFRM)*, published by the Office of Public Works and the Department of Environment, Heritage and Local Government.

1.2 Study Area Overview and Context

1.2.1 Current Site Overview

The subject lands for this study are located in Maglin, Ballincollig, Cork. The lands were formerly under the jurisdiction of Cork County Council but transferred to the jurisdiction of Cork City Council as part of the city boundary extension in 2019. Cork City contains five local electoral areas, and the subject lands lie within the Cork City South West Local Electoral Area. The subject lands are currently undeveloped. The current land use of the subject land is largely agricultural pastureland as evident by the most recent Corine Landcover Use map dated 2018.

The Maglin River flows into the western portion of the site via a culvert under the N22 road and continues to flow through the site until it discharges near the southeast corner. A series of artificial drainage channels extend through the agricultural lands to the west of the Maglin River. The site has been identified as at risk of fluvial flooding from the Maglin River and karst features may exist within the site which will need to be assessed and managed. There may also be a risk of high groundwater levels within the site which will need to be assessed further and managed.

There are a number of existing features on the site including Fulacht Fia's, Lime Kilns, a standing stone, Maglin House, Ballincollig cave, Ballincollig Castle which may provide restrictions.

1.2.2 Proposed Site Overview

The subject lands were identified under the Ballincollig-Carrigaline Municipal District Local Area Plan (Ballincollig-Carrigaline MD LAP, 2017) as the "Ballincollig (Maglin) Urban Expansion Area". Ballincollig has experienced a high level of growth since 2000. The vision for Ballincollig is that it will continue to grow as a major centre for population and employment. The Cork City Draft Development Plan 2022 - 2028 identifies suitable locations for residential and employment growth in the town and co-ordinates this growth with the upgrading of infrastructure services. The land south of Ballincollig town, Maglin, represents a major strategic housing development opportunity for Cork City. Cork City Draft Development Plan 2022 - 2028 facilitates the delivery of these lands for development.

The subject lands cover approximately 220 ha and are zoned for mixed use including residential, educational, and recreational. The zoning of these lands is illustrated in Figure 1.1 below. There are a total of six different land zones within the subject lands. The majority of the land has been zoned as Sustainable Residential Neighbourhoods (ZO 01), New Residential neighbourhoods (ZO 02), Tier 3 Residential Neighbourhoods (ZO 03) and Public Open Space (ZO 16). There is also areas zoned for Education (ZO 13) and an area zoned as the Urban Town Centre (ZO 07). The public open space land use has been largely influenced by this flood risk assessment which indicates these areas will be inundated during 0.1% and 1% AEP flood events. It is also proposed to construct a new spine road through the expansion area which would cross the Maglin River at a number of locations.

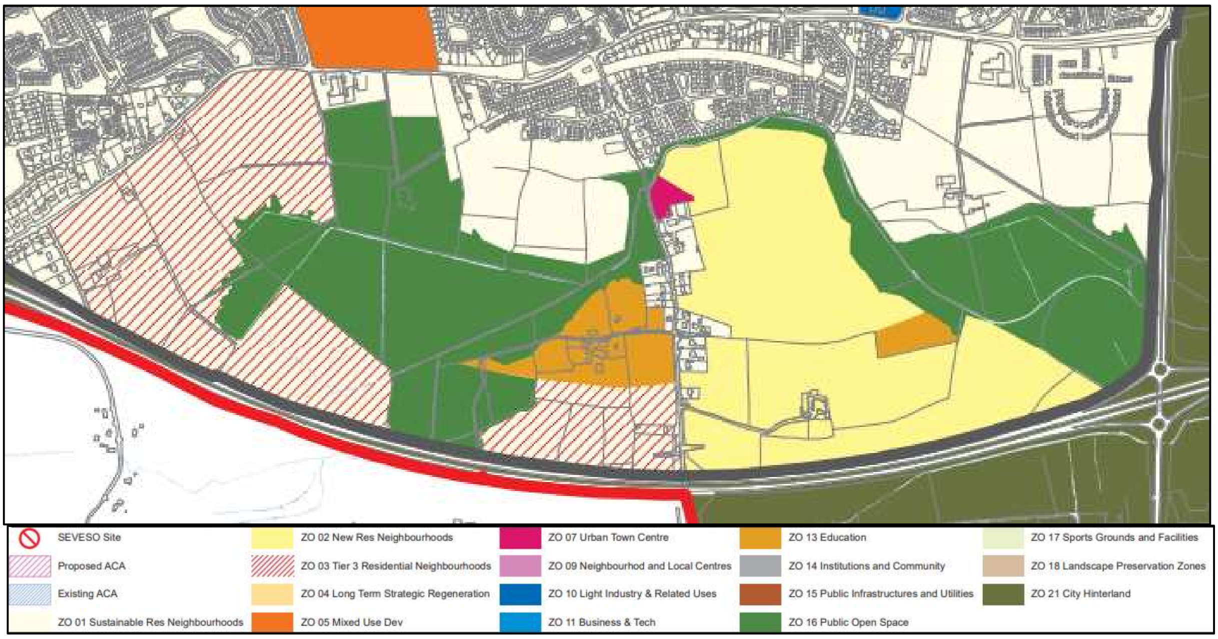


Figure 1.1: Ballincollig (Maglin) Urban Expansion Area Land Zoning Map (Cork City Development Plan 2022-2028)

1.3 Strategic Flood Risk Assessment Objectives

In order to advance the planning objectives of the Cork City Draft Development Plan 2022 - 2028 and inform the overall infrastructure masterplan, a Strategic Flood Risk Assessment (SFRA) is required to inform the design, layout and flood resilience of new development in the Ballincollig (Maglin) Urban Expansion Area.

In line with the Guidelines, the objectives of the SFRA are:

- To provide for an improved understanding of flood risk issues within the Development Plan Area and development management process, and to communicate this to a wide range of stakeholders; to consider existing flood defence infrastructure and the consequences of failure of that infrastructure and also identification of areas of natural flood plain to be safeguarded;
- To produce a suitably detailed flood risk assessment, drawing on and extending existing data and information, leading to a suite of flood risk policies and objectives and, where appropriate, maps that support the application of the sequential approach;
- To inform, where necessary, the application of the Justification Test;
- To conclude whether measures to deal with flood risks to the area proposed for development can satisfactorily reduce the risks to an acceptable level while not increasing flood risk elsewhere, and
- To produce guidance on mitigation measures on how surface water should be managed and appropriate criteria to be used in the review of the Development Plan site specific flood risk assessments.

1.4 Disclaimer

This SFRA has been prepared in compliance with the Guidelines but the SFRA remains a living document and is based on the best available data at the time of preparation. It is subject to change based on more up to date and relevant flood risk information becoming available during the lifetime of the Plan. Accordingly, all information in relation to flood risk is provided for general policy guidance and may be updated in respect of emerging new data and analysis. Owners, occupiers, developers and any other interested bodies are advised to take all reasonable measures to assess the flooding vulnerability or risk of lands in which they have or may have an interest prior to making planning or development decisions. The aim of this SFRA is to provide an appraisal of all sources of flooding within the Study area and to set out a number of approaches in the plan making process to avoid, reduce and manage flood risk as part of a wider objective to ensure the protection of property, people and infrastructure. The SFRA does not contain advice for existing occupiers who currently live in areas at risk of flooding or those that may experience flooding.

1.5 The Planning System and Flood Risk

1.5.1 Overview

"The Planning System and Flood Risk Management: Guidelines for Planning Authorities", published in November 2009, describes flooding as a natural process that can occur at any time and in a wide variety of locations. The Guidelines describe good flood risk practice in planning and development management and seek to integrate flood risk management into the planning process, thereby assisting in the delivery of sustainable development. Planning authorities are directed to have regard to the Guidelines in the preparation of Development Plans and Local Area Plans, and for development management purposes. For this to be achieved, flood risk must be assessed as early as possible in the planning process.

Paragraph 1.6 of the guidelines states that the core objectives are to:

- Avoid inappropriate development in areas at risk of flooding;
- Avoid new developments increasing flood risk elsewhere, including that which may arise from surface run-off;
- Ensure effective management of residual risks for development permitted in flood plains;
- Avoid unnecessary restriction of national, regional or local economic and social growth;
- Improve the understanding of flood risk among relevant stakeholders; and
- Ensure that the requirements of EU and national law in relation to the natural environment and nature conservation are complied with at all stages of flood risk management.

The guidelines aim to facilitate "the transparent consideration of flood risk at all levels of the planning process, ensuring a consistency of approach throughout the country". The Guidelines work on a number of key principles, including:

- Adopting a staged and hierarchical approach to the assessment of flood risk;
- Adopting a sequential approach to the management of flood risk, based on the frequency of flooding (identified through Flood Zones) and the vulnerability of the proposed land use.

1.5.2 Flood Risk

In order to manage flood risk it is important to understand what the term "flood risk" implies and to define the components of flood risk in order to apply the principles of the DEHLG Flood Risk Management Guidelines.

Flood risk is generally accepted to be a combination of the likelihood of flooding and the potential consequences arising, and is normally expressed in terms of the following relationship:

Flood Risk = Probability of Flooding x Consequences of Flooding

Flood risk is assessed using the source – pathway – receptor model as illustrated on Figure 1.2.

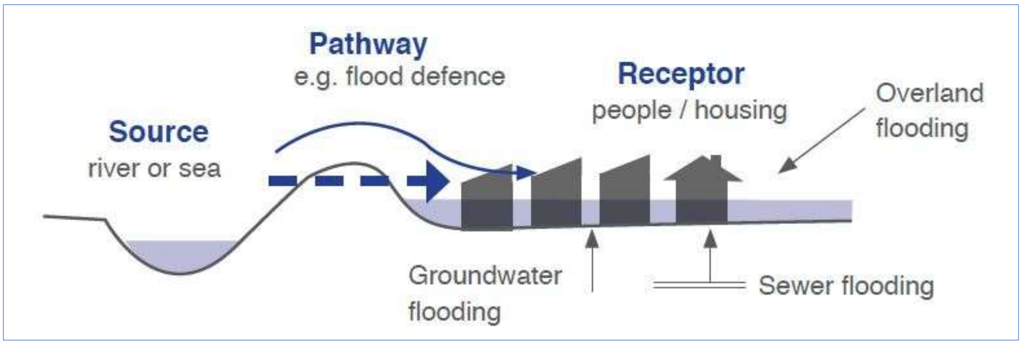


Figure 1.2: Source-Pathway-Receptor Model

Principal sources of flooding are intense or prolonged rainfall or higher than normal sea levels while the most common pathways are rivers, drains, sewers, overland flow and river and coastal flood plains and their defence assets. Receptors can include people, their property and the environment. All three elements must be present for flood risk to arise. Mitigation measures, such as defences or flood resilient construction, have little or no effect on sources of flooding but they can block or impede pathways or remove receptors.

Flood risk assessments require identification and assessment of all three components:

- The probability and magnitude of the source(s) (e.g. high river levels, sea levels and wave heights);
- The performance and response of pathways and barriers to pathways such as floodplain areas and flood defence systems, and
- The consequences to receptors such as people, properties and the environment.

The planning process is primarily concerned with the location of receptors, taking appropriate account of potential sources and pathways that might put those receptors at risk.

1.5.3 The Staged Approach

The Guidelines recommend a staged approach to be adopted to ensure that only such an appraisal or assessment as is needed for the purposes of decision making at the various plan levels is undertaken. The stages include:

Stage 1 - Flood risk Identification: To identify whether there may be any flooding or surface water management issues related to the area of the regional planning guidelines, development plans or local area plans (LAPs) or a proposed development site that may warrant further investigation at the appropriate lower level plan or planning application levels. If the Planning Authority considers that there is potential flood risk issue, then stage 2 shall be entered into.

Stage 2 - Initial flood risk assessment: To confirm sources of flooding that may affect a plan area or proposed development site, to appraise the adequacy of existing information and to scope the extent of the risk of flooding which may involve preparing indicative flood zone maps. Where hydraulic models exist, the potential impact of a development on flooding elsewhere and of the scope of possible mitigation measures can be assessed. In addition, the requirement of the detailed assessment should be scoped; and

Stage 3 - Detailed flood risk assessment: To assess flood risk issues in sufficient detail and to provide quantitative appraisal of potential flood risk to a proposed or existing development or land to be zoned, of its potential impact on flood risk elsewhere and of the effectiveness of any proposed mitigation measures.

1.5.4 Climate Change

Climate change can be expected to generally increase flood risk and consequences of flooding. Due to the uncertainty associated with the potential effects of climate change, the Guidelines recommend that a precautionary approach to dealing with climate change is adopted and provide the following examples:

- Recognising that significant changes in the flood extent may result from an increase in rainfall or tide events and accordingly adopting a cautious approach to zoning land in these potential transitional areas;
- Ensuring that the levels of structures designed to protect against flooding, such as flood defences, land-raising or raised floor levels are sufficient to cope with the effects of climate change over the lifetime of the development they are designed to protect; and
- Ensuring that structures to protect against flooding and the development protected are capable of adaptation to the effects of climate change when there is more certainty about the effects and still time for such adaptation to be effective

1.5.5 Vulnerability of Developments

The Guidelines have outlined three Vulnerability Classifications for developments based on the proposed land use and type of development. These classifications and particular examples of development types which would be included in each classification are summarised as follows;

- Highly Vulnerable Development: This would include emergency services, hospitals, schools, residential institutions, dwelling houses, essential infrastructure, water & sewage treatment etc.
- Less Vulnerable Development: Retail, leisure, commercial, industrial buildings, local transport infrastructure.
- Water-compatible development: Docks, marinas and wharves. Amenity and open space, outdoor sports and recreation and essential facilities such as changing rooms.

The Guidelines also include a matrix of vulnerability versus flood zone to differentiate between developments which are appropriate in various flood zones and those which require a Justification Test. This table is reproduced as Table 1.1 below.

Vulnerability Classification	Flood Zone A	Flood Zone B	Flood Zone C
Highly Vulnerable Development	Justification Test	Justification Test	Appropriate
Less Vulnerable Development	Justification Test	Appropriate	Appropriate

Water-compatible Development	Appropriate	Appropriate	Appropriate

Table 1.1: Vulnerability Matrix

1.5.6 Flood Zones

In the Planning System and Flood Risk Management Guidelines document, the likelihood of a flood occurring is established through the identification of Flood Zones which indicate a high, moderate or low risk of flooding from fluvial or tidal sources. Table 1.2 below includes the definition of Flood Zones as well as the implications for planning.

It is important to note that the Flood Zones do not take other sources of flooding, such as groundwater or pluvial, into account, so an assessment of risk arising from such sources should also be made, where appropriate.

Flood Zone	Description & Summary of Planning Implications
Zone A High probability of flooding	More than 1% probability (1 in 100) for river flooding and more than 0.5% probability (1 in 200) for coastal flooding. Most types of development would be considered inappropriate in this zone.
Zone B Moderate probability of flooding	0.1% to 1% probability (between 1 in 100 and 1 in 1,000) for river flooding and 0.1% to 0.5% probability (between 1 in 200 and 1 in 1,000) for coastal flooding. Highly vulnerable development, such as hospitals, residential care homes, Garda, fire and ambulance stations, dwelling houses and primary strategic transport and utilities infrastructure, would generally be considered inappropriate in this zone.
Zone C Low probability of flooding	This zone defines areas with a low risk of flooding from rivers and the coast (i.e. less than 0.1% probability or less than 1 in 1,000). Development in this zone is appropriate from a flooding perspective (subject to assessment of flood hazard from sources other than rivers and the coast).

Table 1.2: Definitions of Flood Zones

1.5.7 The Sequential Approach

The sequential approach makes use of flood risk assessment and of prior identification of flood zones for river and coastal flooding and classification of the vulnerability to flooding of different types of development. The principle of the Sequential Approach mechanism is to:

- Avoid: Preferably choose lower risk flood zones for new development
- Substitute: Ensure the type of development proposed is not especially vulnerable to the adverse impacts of flooding
- Justify: Ensure that development is being considered for strategic reasons
- Mitigate: Ensure flood risk is reduced to acceptable levels
- Proceed: Only Justification Test is passed. Ensure emergency planning measures are in place.

The application of the Sequential Approach mechanism in the planning process is illustrated on Figure 1.3 which is an extract from the Planning System and Flood Risk Management Guidelines.

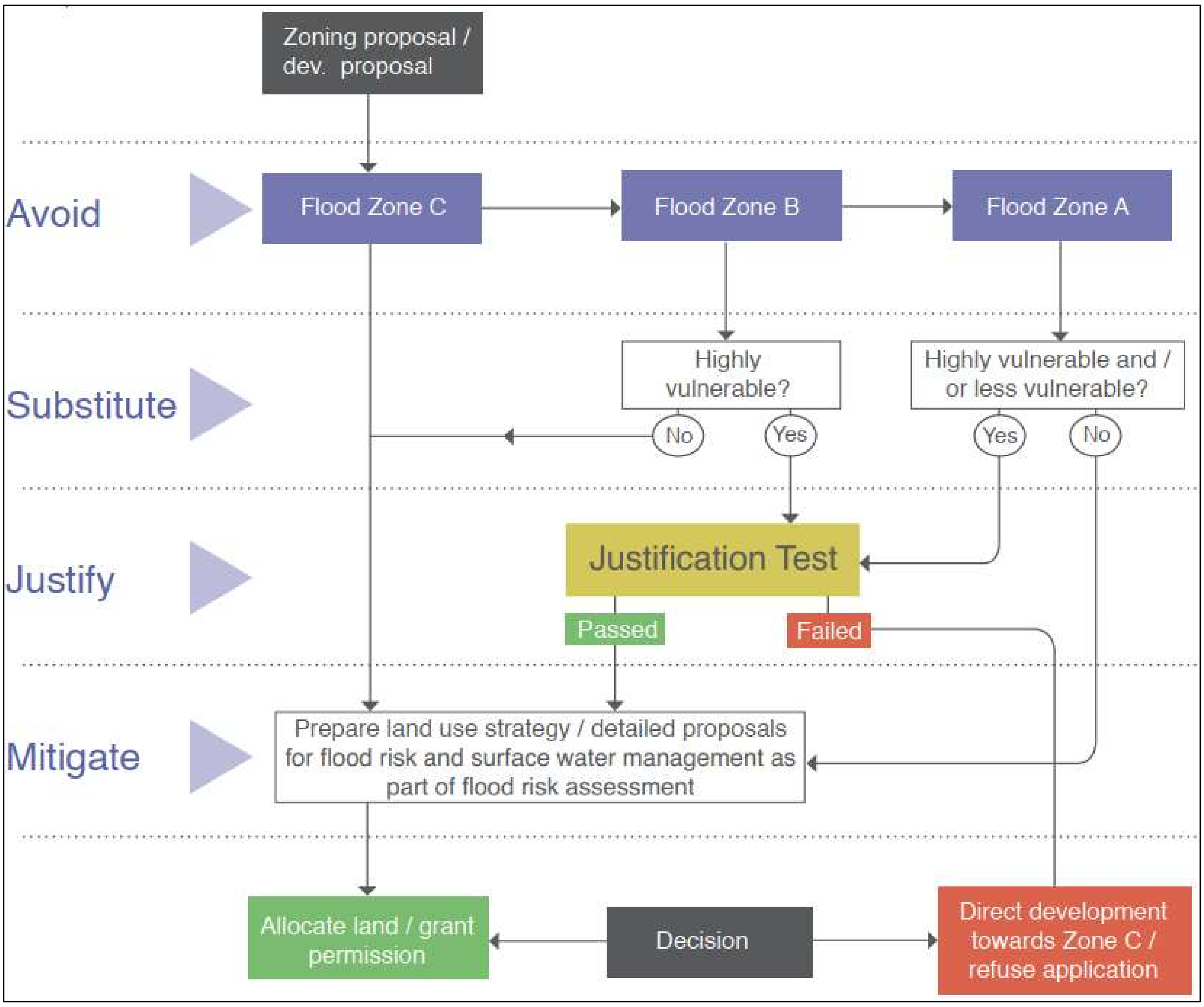


Figure 1.3: The Sequential Approach Mechanism in the Planning Process

1.6 Methodology & Report Structure

The methodology adopted for this study and the corresponding report structure is summarised as follows;

1. Complete a site characterisation and desktop study of the Plan Area and surrounds, as outlined in Section 2;
2. Review the flood history & existing flood risk information in the Plan Area based on available information, as described in Section 3;
3. Acquire and review all available data required to complete the SFRA, identify information gaps and obtain supplementary data where necessary, as summarised in Section 4;
4. Use the findings of the desktop studies to identify all possible sources of flooding that may affect the Plan Area, see Section 5
5. Complete a hydrological analysis and hydraulic model in order to quantify fluvial flood risk, as described in Sections 6, 7, and 8;
6. Consider the potential for pluvial and groundwater flooding in sufficient detail for the SFRA, as outlined in Sections 9 and 10;
7. Confirm the sources of flooding that pose a risk to the lands within the Plan Area, assess the risks from a strategic perspective, outline the required strategic flood risk management measures and provide recommendations for integrating the SFRA findings with the Development Plan. Refer to Sections 11 and 12.

2. Site Characterisation & Environmental Setting

2.1 Topography & Slope

The topography of South Ballincollig is relatively flat flanked on the north and south by small, east-west trending hills. The subject lands are situated south of the River Lee. The elevation within the site was found to be between 15 – 30m AOD. The site has a slight dip from west to east as ground level approaches sea-level within Cork City. Several rock outcrops have been identified within the site (Figure 2.1) which illustrates these outcrops have an average elevation of 22 – 25m AOD.

Figure 2.1: Topography of Plan Area (Geohive, 2021)

2.2 Existing Drainage Paths & Flow Routes

The existing drainage paths and basic flow routes within the Plan Area have been established and are illustrated on Figure 2.2 below. There are a number of existing natural and artificial flow paths which should be replicated insofar as possible in the detailed design of the overall plan area and on a site specific basis.

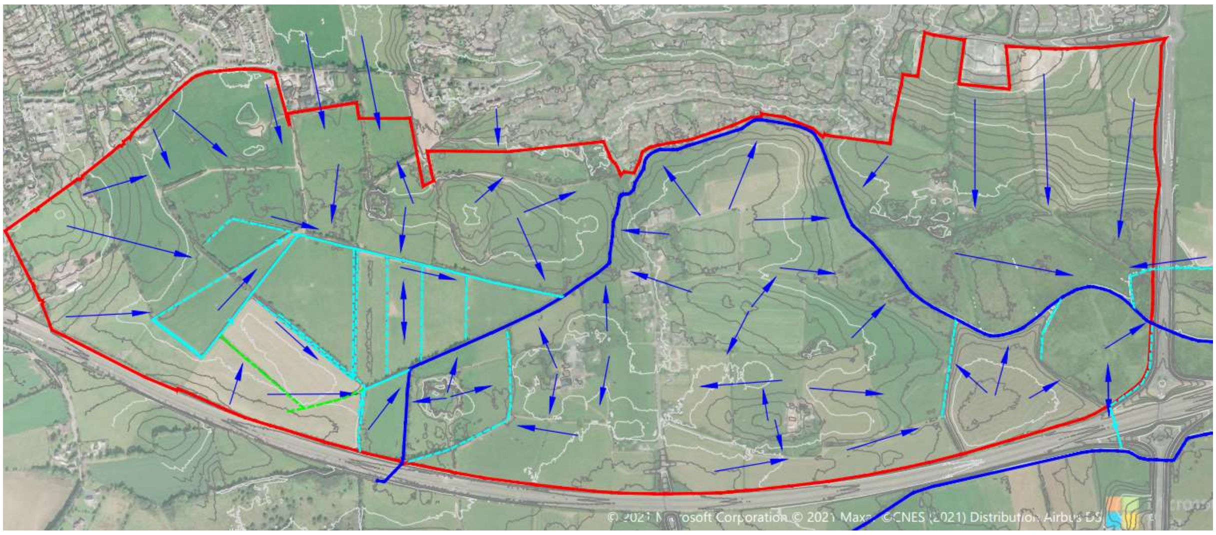


Figure 2.2: Existing Drainage Paths & Flow Routes

2.3 Historic Mapping

The historical and aerial mapping publicly available on the Geohive website and Google Earth were reviewed as part of this study to identify any noticeable items/ changes over time on the site.

The subject lands consist of many low-lying fields, farms and private dwellings. Between the 1800's and the early 2000's, there have been significant changes, such as a railway line that cut through the site that no longer exists. Several quarries and lime kilns were also present and had caused small flood plains in the centre of the site. A small lough is noted by Maglin house that is not seen in more recent maps and photography. Historic water pumps noted from the 1830's and 1870's can also be found across the site.

In the 2000's, the railway has since been dismantled, quarries disused and kilns abandoned. The N22 was built from 2004 which forms part of the southern and western boundaries of the site. The alignment of the Maglin River south and north tributaries were altered to facilitate the construction of this road.

Other specific features noted from historic mapping that may be relevant to flood risk are illustrated on Figure 2.5 to Figure 2.8 below.

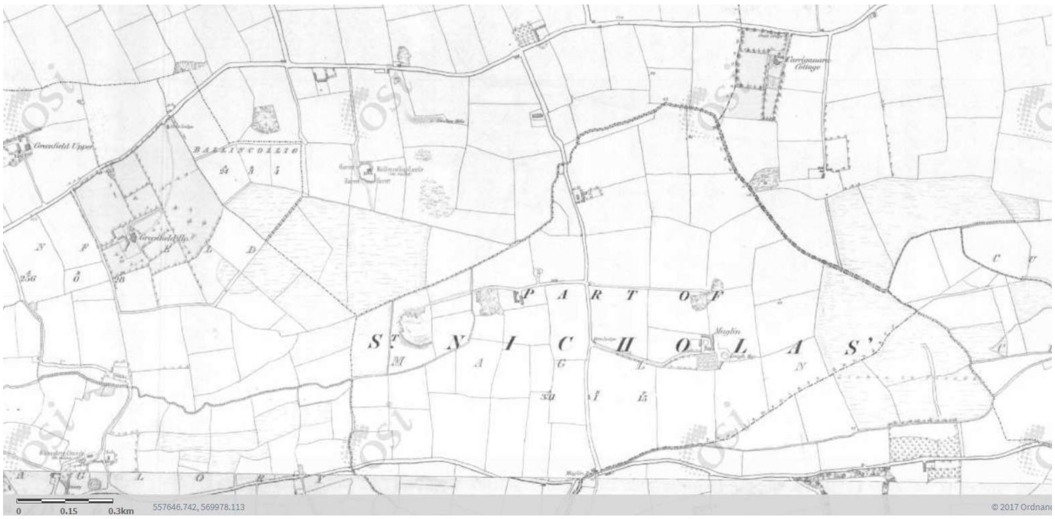


Figure 2.3: Cassini 6-inch historical map of the site (Geohive, 2021)

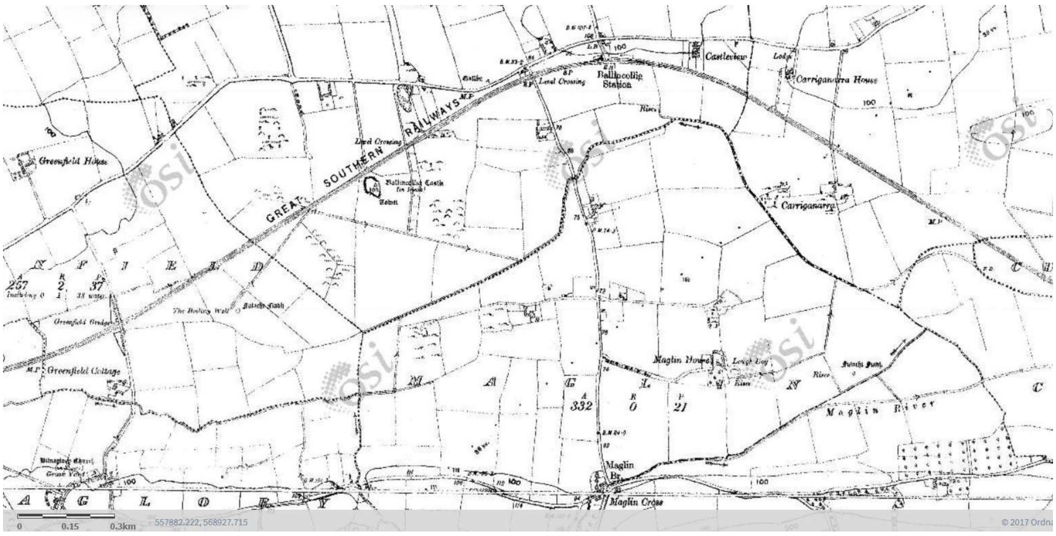


Figure 2.4: Historic Map 6 inch Black & White, 1837-1842. (Geohive, 2021)

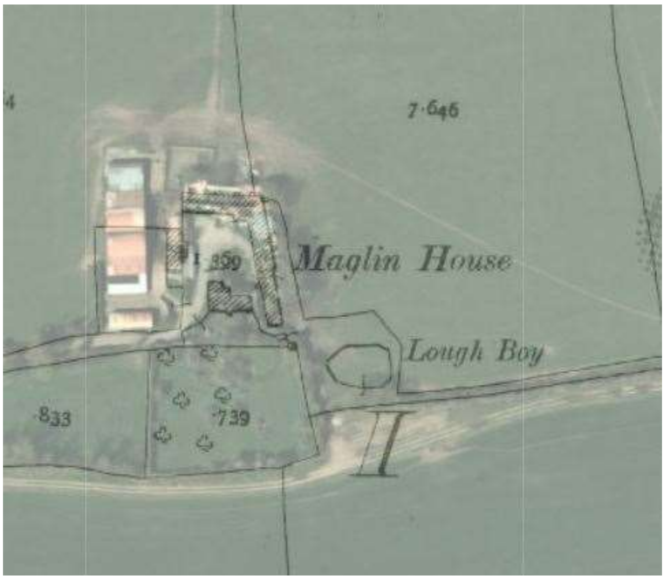


Figure 2.5: Maglin House & Lough Boy (Geohive, 2021).

Figure 2.6: Water pumps located in the Western Portion of the Site (Geohive, 2021)

Figure 2.7: Possible "Boiling Spring" Location (Geohive, 2021)

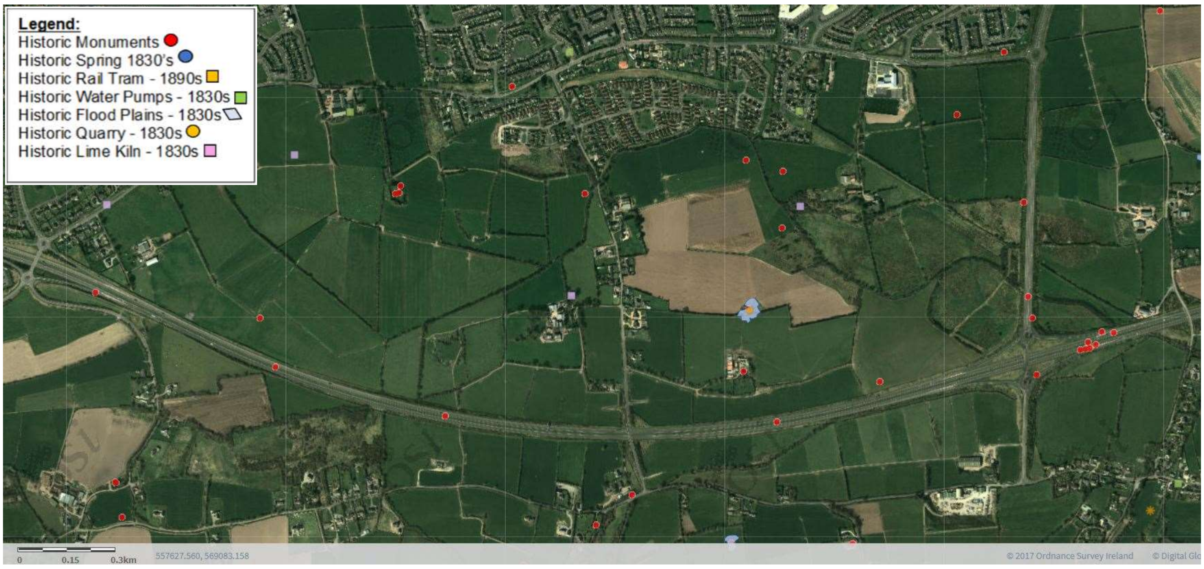


Figure 2.8: Historical Features (Geohive, 2021)

2.4 Geology & Soils

2.4.1 Solid & Structural Geology

The bedrock geology of the site is primarily composed of Carboniferous Limestones. Located within the Cork Syncline of the Munster Basin, the site lies within a limestone valley flanked by the north and south by sandstone ridges. This area of the Munster Basin has been heavily folded and subsequently eroded which has produced a series of east-west trending anticlines and synclines that have influenced the local topography. Sandstones are found at higher-elevation anticlines while limestones are found at lower-elevations synclines due to differential erosion as the limestone was eroded at a quicker rate than the sandstone.

Bedding on either side of the Cork Syncline is steeply dipping at an average of 55° to the south on the northern limb, and 60° to the north on the southern limb. The Cork Syncline axis runs directly through the northern boundary of the site.

There are two sets of cross-cutting faults that have been identified by GSI mapping: the first strikes east to west which appears to follow strike of bedding, the second strikes north to south.

The bedrock found within and immediately adjacent to the site are described from literature below with the symbol for each formation given in brackets for cross-reference purposes with the GSI 1:100,000 scale bedrock geology map.

- Little Island Formation (CDLITT). Described as Carboniferous massive calcilutite limestones (mudbank facies) and crinoidal calcilutites. The formation is a uniform succession of crinoidal biomicrite limestones (wackestones) and massive unbedded calcilutite limestones of mudbank facies. The top of the formation is highly crinoidal and passes up to poorly bedded wackestones. The formation is estimated to be up to 500m thick.
- Waulsortian Limestones (CDWAUL). Described as Carboniferous massive, unbedded limes mudstones. Sometimes informally called "reef" limestones, although inaccurate. Dominantly pale grey, crudely bedded or massive limestone. Known to be moderately to intensely karstified. Typically, 300 - 500 m thick.

- Cuskinny Member (CDKINS2). Described as Carboniferous flaser-bedded sandstone & mudstone. The member is sand dominant and characterised by alternations of flaser-bedded sandstones, lenticular-bedded (linsen) mudstones, massive sandstones, and nodular mudstones. Thin quartz-pebbly sandstones and conglomerates also occur throughout the member. Thickness can vary between 199-235m.
- Castle Slate Member of Kinsale Formation (KNcs). Described as Carboniferous uniform well-cleaved dark-grey slaty mudstones. The member consists of uniform dark grey, well cleaved massive mudstones. Comminuted crinoidal debris is common in some beds as are phosphatic nodules and disseminated pyrite. Rhythmic upward grading from sediment of medium silt size to fine silt and mud. Thicknesses of 5 – 62m have been reported.
- Old Head Sandstone Formation (DUOHSF). Described as Devonian grey flaser-bedded sandstones, fine grained sandstones and minor mudstones and lenticular bedded mudstones. The formation is dominated by lithologies belonging to the Heterolithic Facies (mainly flaser-bedded sandstones), wavy bedded fine-grained sandstones and minor mudstones. Thickness has been reported as between 899-1098m.

The bedrock geology within the subject area is dominated by Carboniferous limestones in both the northern and southern half of the subject area with carboniferous sandstones and mudstones located at the very southern edge of the subject area. There are also a number of bedrock outcrops located within the subject area.

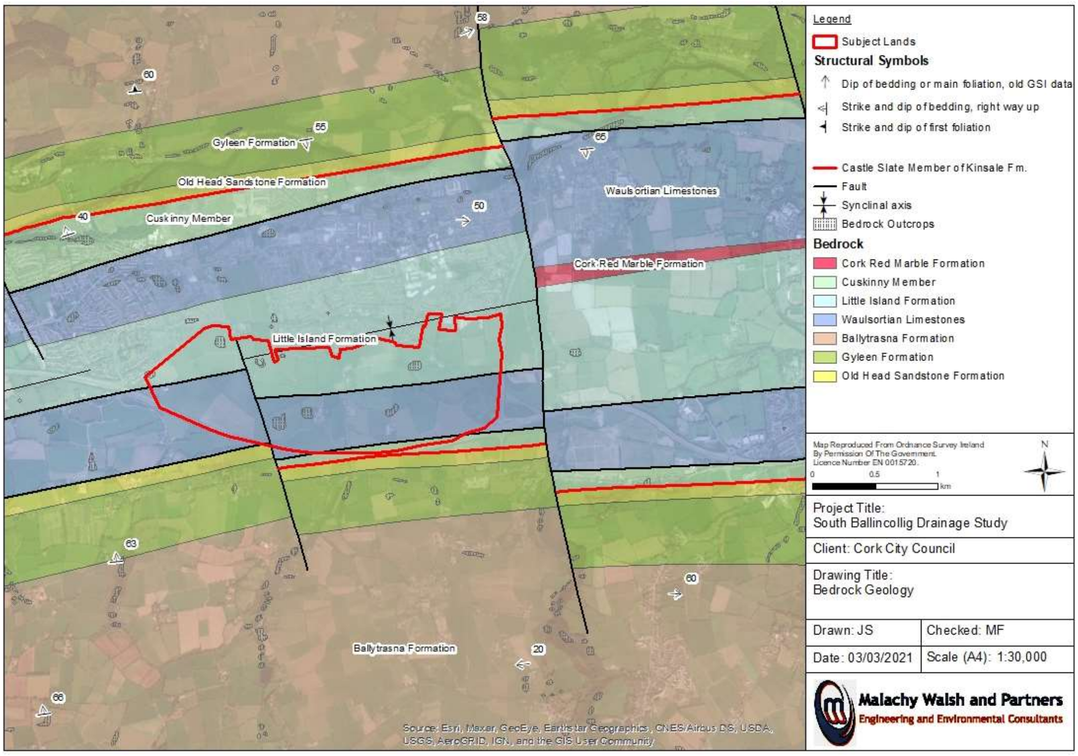


Figure 2.9: Bedrock Geology of Subject Area

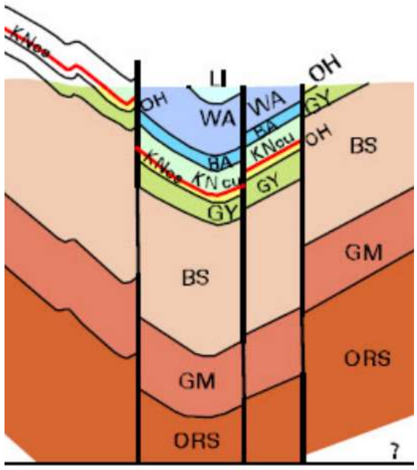


Figure 2.10: Geological Cross-section of the Cork Syncline (GSI Sheet 25)

2.4.2 Quaternary Sediments and Glacial Geomorphology

The subject lands are primarily underlain by till derived from Devonian sandstones with some areas exhibiting bedrock outcrop or subcrop in the central areas of the site. The northern boundaries are bordered by urban/made ground. The southern boundaries are bordered by gravels derived from Devonian sandstone in the form of a glaciofluvial terrace. A terrace is a step-up feature formed from glacial meltwater depositing sediment.

The presence of till is further supported by previous ground investigations that have been performed in close proximity to the study area.

Tills are often described as unsorted to poorly sorted diamicton which contains clays, silts, sands, and gravels. Glaciofluvial materials are often well sorted and contain larger clasts, in this case gravels and possible sands.

Meltwater channels have been noted to flow towards the subject lands from the north and the south.

2.4.3 Soils

The site is primarily composed of “AminDW - Deep well drained mineral (Mainly acidic)” sourced from Devonian Sandstone till.

Several areas are classed as “BminSW - Shallow well drained mineral (Mainly basic)” soils, sourced from Carboniferous Limestone parent materials and subcrop of the limestones beneath.

Several areas within the centre and east boundaries of the site are recorded as “AminPD - Mineral poorly drained (Mainly acidic)”, sourced from Devonian Sandstone till and contain groundwater gleysol. Gleysols are hydric soils in which they are saturated by groundwater for long periods of time. It appears these AminPD soils are associated with the Grange stream as they are on the banks of the river itself.

Some smaller sections of the northern boundary are classed made/urban ground.

Further site investigation will be required to determine both soil and bedrock characteristics. This may require carrying out infiltration tests to determine the infiltration potential of this site. Infiltration tests measures the rate at which water soaks away from the test pit and gives an infiltration rate. These are carried out in trial pits or boreholes if depth or access restraints are encountered.

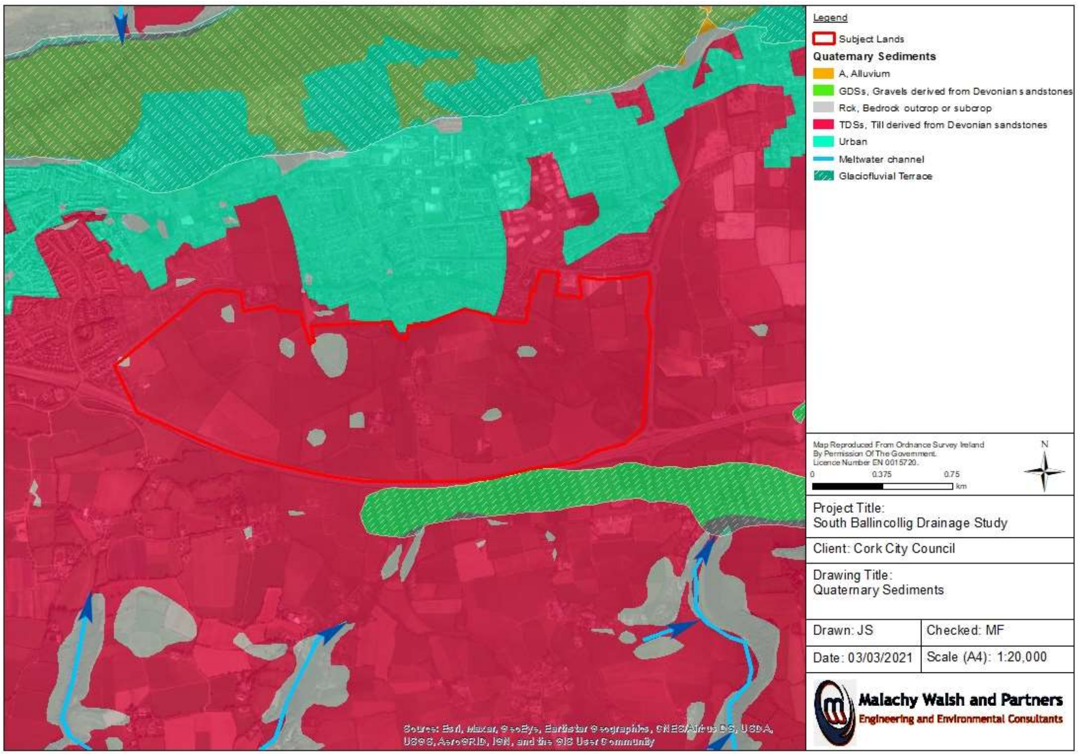


Figure 2.11: Quaternary Sediments of Subject Area

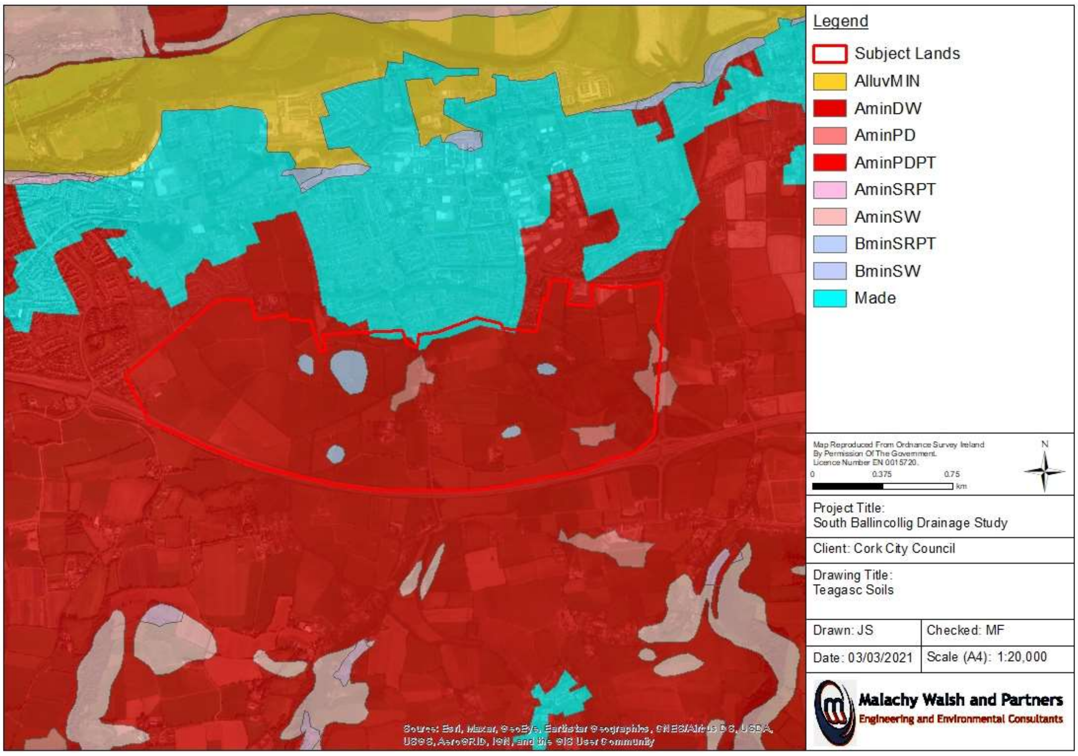


Figure 2.12: Soils of Subject Area

2.5 Hydrogeology & Hydrology

2.5.1 Hydrogeology

The vast majority of the site is situated within a "Rkd - Regionally Important Aquifer - Karstified (diffuse)", and "Ll - Locally Important Aquifer - Bedrock which is Moderately Productive only in Local Zones" within a small portion of the southern boundary. "Rg- Regionally Important Gravel Aquifer" has been noted north, northeast, and east of the site.

There are two karst features in close proximity to the site which have been noted, namely Coleman Cave (GSI reference 1405NEK001) which can be found approximately 2.5km west of the western boundary, and a Curraheen Spring (GSI reference 1405NEK002) 1.5km east of the eastern boundary.

There has been anecdotal evidence of a possible spring located within the subject lands known as the "Balleen" spring in the western portion of the site, however this is yet to be identified.

This possible karstification within the site is due to acidic rainfall percolating through the fractured rock and chemically weathering it. Overtime, this has produced complex groundwater flow paths through open cavities and fractures in the bedrock, which can penetrate through the bedrock for tens of meters. Karstification within the Carboniferous limestones has occurred since the Jurassic period, and many of the cavities have since been filled with Pleistocene glacial outwash sands and gravels (Meere et al., 2013).

Historic maps show evidence of flooded limestone quarries, lime kilns and pits which may be connected to the bedrock aquifers and may be hydrogeologically connected to karst features.

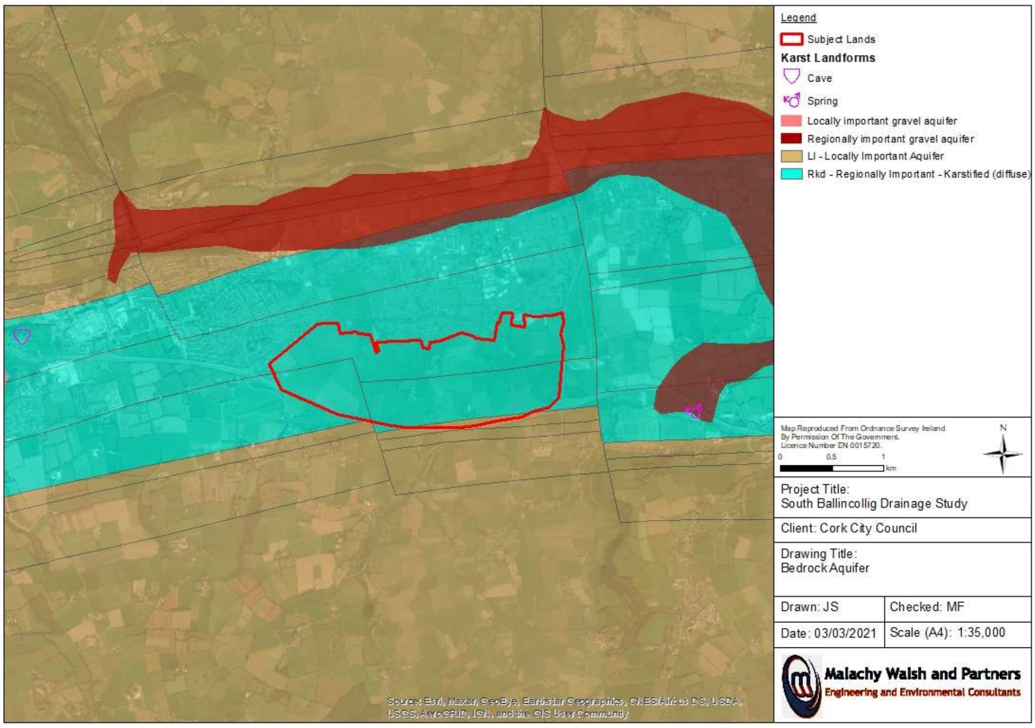


Figure 2.13: Aquifer Map of Subject Area

2.5.2 Groundwater Vulnerability

The GSI groundwater vulnerability map can be seen in Figure 2.14. The western and central portions of the subject land have been categorised as “high” and “extreme” groundwater vulnerability, while the eastern portion has been categorised as “moderate” groundwater vulnerability. Areas that exhibit rock outcrop or subcrop have been categorised and “rock at or near surface or karst”.

2.5.3 Groundwater Wells & Springs

As indicated on Figure 2.15, seven Groundwater Wells & Springs were identified within and nineteen in the vicinity of the subject lands’ boundary. Many of these wells were drilled by Geotech Drilling during the construction of the N22 bypass. Their logs lack hydrological data such as water strike and yield.

Groundwater well logs that do contain data were identified on the southern boundary of the site. Groundwater strike ranged between 1.2 – 1.9m bgl.

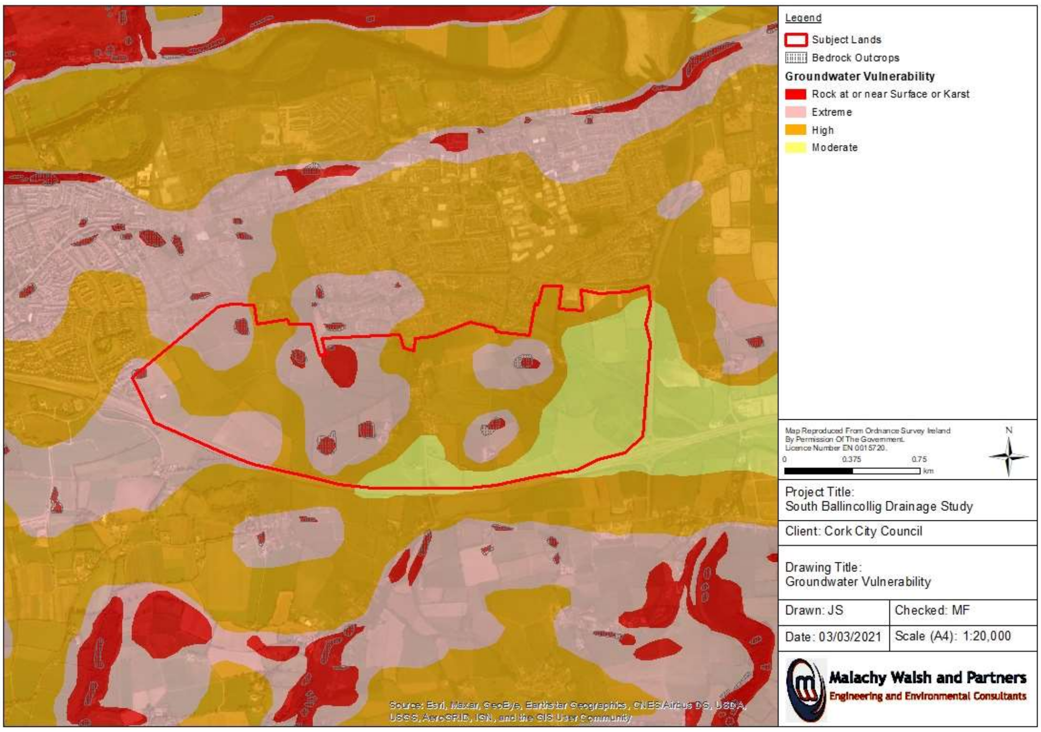


Figure 2.14: Groundwater Vulnerability of Subject Area

2.5.4 Subsoil Permeability

The GSI groundwater subsoil permeability map classifies how water can infiltrate downwards into the surface at a given point with relative ease (Figure 2.16). There are large areas within the subject lands that have not been mapped for groundwater subsoil permeability. This is likely due to its relation to the Groundwater Vulnerability map where rock outcrop and subcrop have been identified. However, the majority of the site is mapped as “Moderate”, and it can therefore be assumed that unmapped areas will also have moderate subsoil permeability due to similar subsoils and topsoil constituents. There is an area of “high” subsoil permeability immediately south of the southern boundary, which are associated with the Quaternary “Gravels derived from Devonian sandstone” that extend to the east.

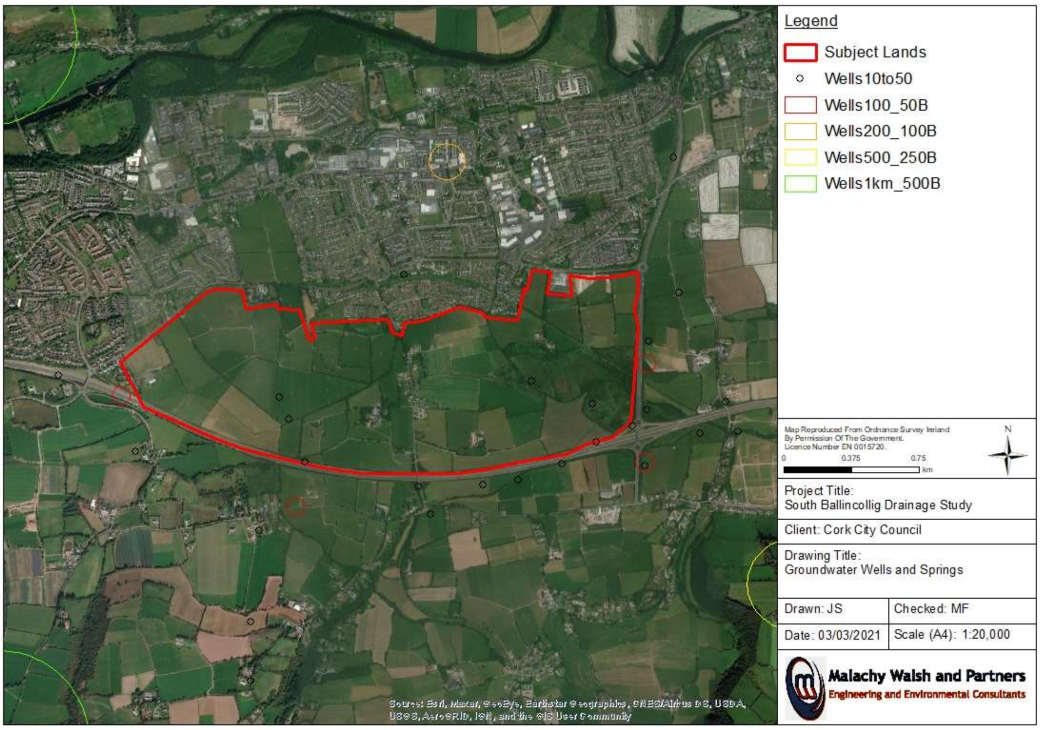


Figure 2.15: Groundwater Wells & Springs

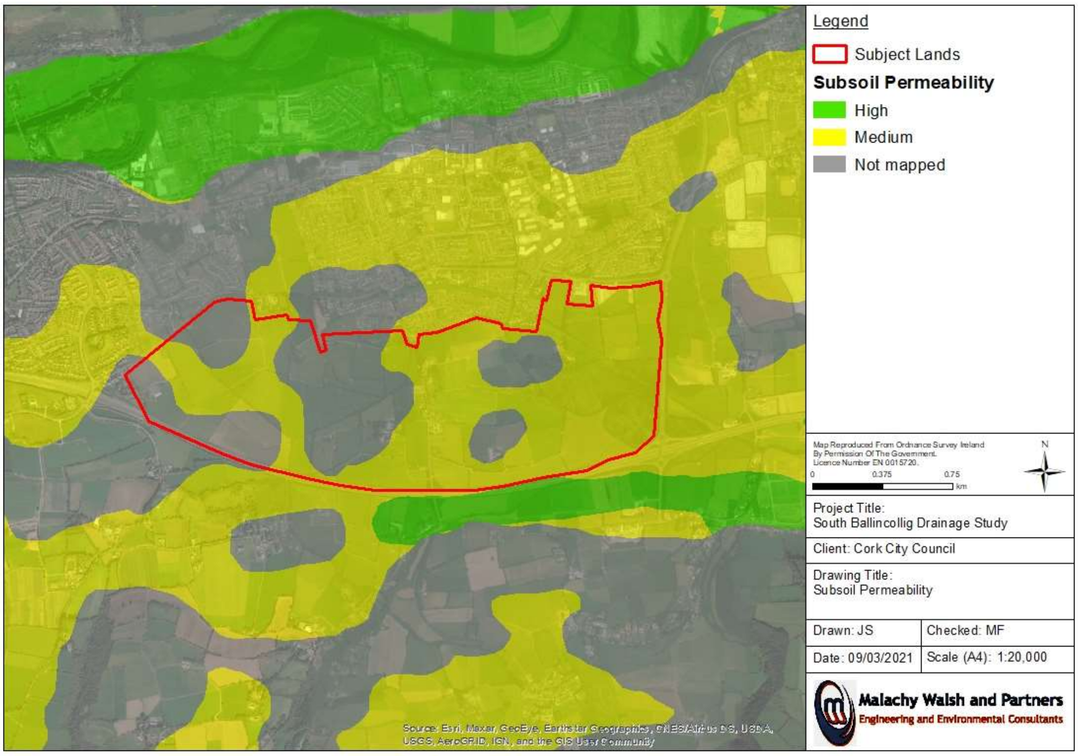


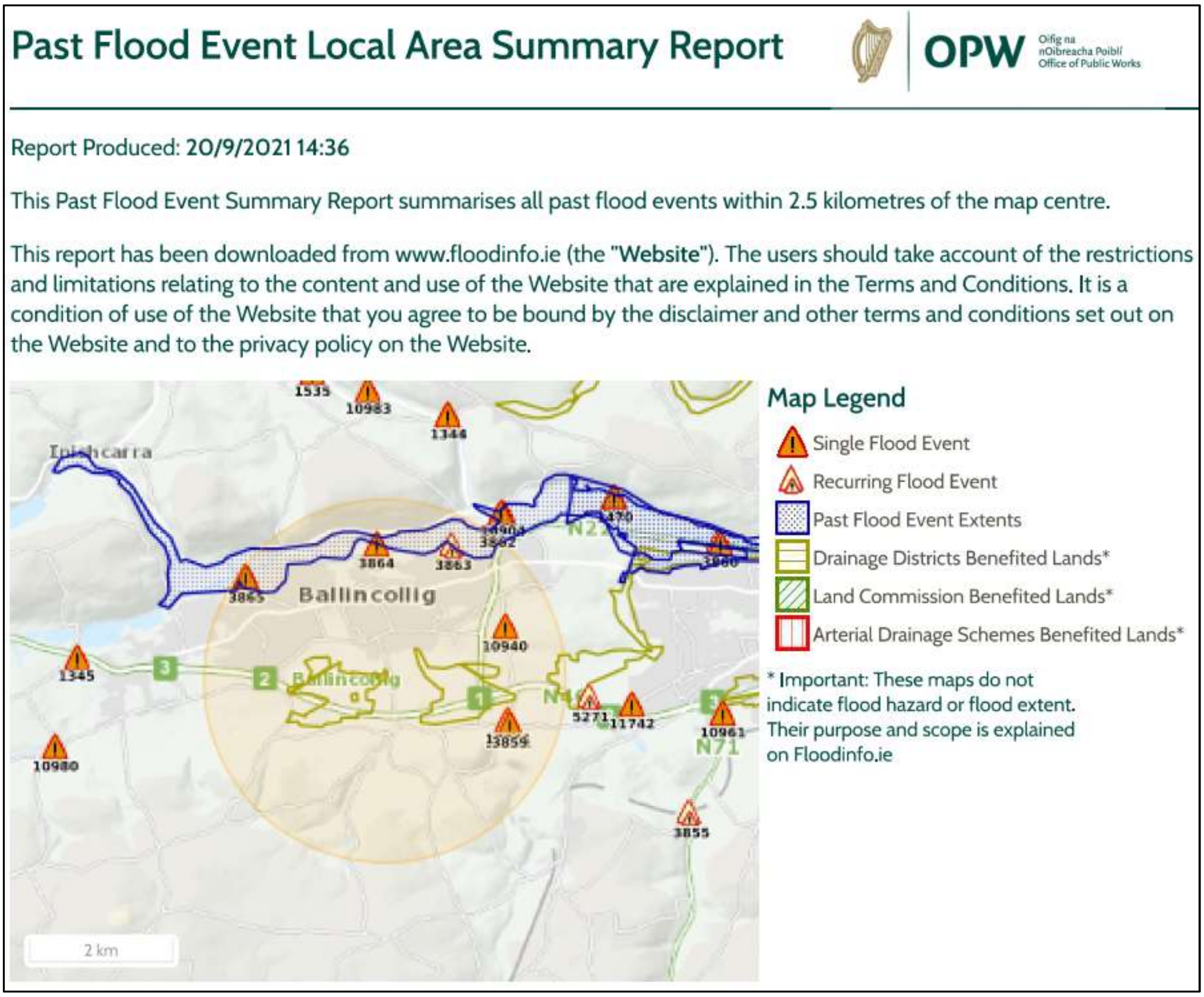
Figure 2.16: Subsoil Permeability of Subject Area

3. Existing Flood Risk Information & Historic Flooding

3.1 Flood History – OPW Local Area Reports

The Past Flood Event Local Area Summary Report which was obtained from the Office of Public Works (OPW) floodinfo.ie website is included on Figure 3.1 below. This report summarises all recorded past flood events within 2.5km of the site centre. There are no recorded flood events within the Plan Area however there are a number of incidences of flooding on the Curraheen River downstream of the Plan Area and also on Clash Road which is located to the east of the Plan Area. The following flood events are notable:

- Curraheen River Clash Road Recurring (Flood ID-3859)
- Clash Road Ballincollig, Co. Cork 19th November 2009 (Flood ID-10940)
- Curraheen, Ballincollig, Co. Cork Flooding 19th Nov. 2009 (Flood ID-10941)
- Curraheen, Ballincollig, Co. Cork Flooding Dec. 2009 and Jan. 2010 (Flood ID-11422)
- Curraheen, Co. Cork 28th June 2012 (Flood ID-11671).



Past Flood Event Local Area Summary Report

Report Produced: **20/9/2021 14:36**

This Past Flood Event Summary Report summarises all past flood events within 2.5 kilometres of the map centre.

This report has been downloaded from www.floodinfo.ie (the "**Website**"). The users should take account of the restrictions and limitations relating to the content and use of the Website that are explained in the Terms and Conditions. It is a condition of use of the Website that you agree to be bound by the disclaimer and other terms and conditions set out on the Website and to the privacy policy on the Website.

Figure 3.1: Local Area Summary Report

3.2 Lee CFRAM Study Flood Maps

The OPW in conjunction with Cork County Council and Cork City Council have carried out an extensive catchment based flood risk assessment and management study for the Lee Catchment (Lee CFRAM Study). The OPW has subsequently published Flood Maps for the River Lee Catchment and some of its tributaries. The predicted 10%, 1% and 0.1% AEP flood extents for the Maglin River are shown on Figure 3.2 below. This indicates that relatively large areas on the upstream (western) side of the site and the downstream (eastern) side of the site would be in Flood Zones A and B.

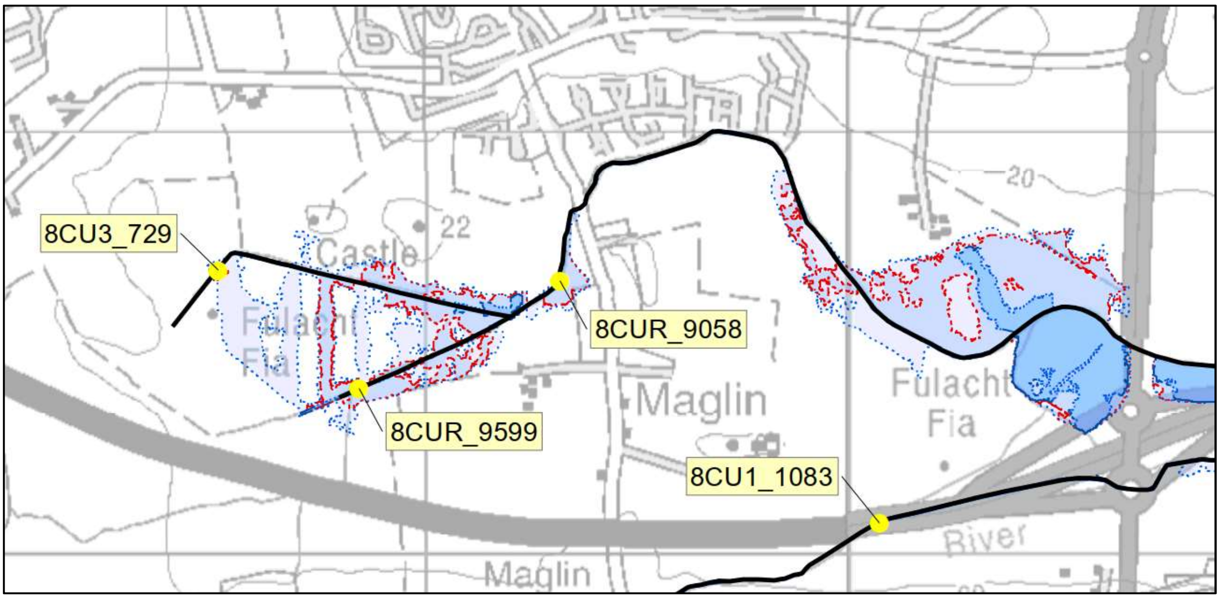


Figure 3.2: Lee CFRAM Study Flood Extent Map (Current Scenario)

3.3 Historic Mapping

Historic mapping has been reviewed and is discussed in Section 2.3. There are some flood risk indicators on these maps which suggest there is potential for flooding at these lands, such as:

- At the western side of the site, the historic maps indicates the “Greenfield Well” near the upstream end of the artificial drain;
- A small area in the central portion of the lands is included on Historic Flood Plains maps from the 1830’s;
- Lough Boy appears on the map adjacent to Maglin House.

3.4 Recent Flood Events

Following a period of prolonged rainfall starting from 17th February 2021, flooding at the subject site and the surrounding areas was observed on 19th February 2021. The source of flooding was the Maglin River. This confirms that there are flooding issues in the Plan Area which require further assessment. Specific information in relation to the February 2021 flood event is provided in Section 7.

4. Data Acquisition & Review

4.1 MWP Site Walkovers

A number of walkover surveys were carried out of the site and the surrounding lands which helped to identify and understand the local drainage features and flow paths as well as the river characteristics. Site walkovers were also used to assist in reviewing the adequacy of existing information and to identify any information gaps.

4.2 Lee CFRAM Study Survey Data

The Lee CFRAM Study survey data was provided by Cork City Council. This includes cross sections of the Maglin and Curraheen Rivers. This information was reviewed and the following key points were noted:

1. The data provided appears suitable for the purpose of the SFRA however some information gaps exist which require supplementary surveys;
2. Based on the observations during the site walkover, there are a number of minor culverts and bridges along the reach which were not captured in the survey data and which would be preferable to obtain to assist with the SFRA;
3. The hydrographic channel survey does not extend upstream of the locations indicated on Figure 4.1. If the river is not modelled upstream of this it is possible that the analysis would not be sufficient to identify areas at risk of flooding.

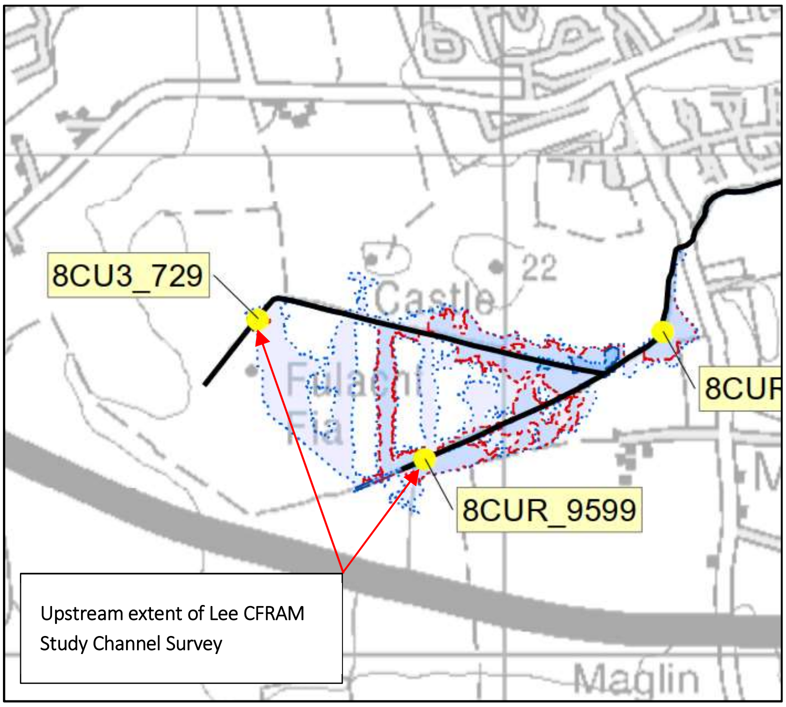


Figure 4.1: Upstream Extent of Lee CFRAM Study Channel Survey

4.3 Supplementary Hydrographic & Topographic Data

The information gaps identified in the Lee CFRAM Study survey data were supplemented by additional channel and structure surveys. The survey was carried out by HD Surveys in February 2021. This included twenty one additional channel/drain cross sections and nine culvert/bridge structures. The scope and extent of the survey was agreed with Cork City Council and it is deemed sufficient to for the purpose of the SFRA. Notwithstanding this, a refined analysis may be possible with the benefit of additional survey data, particularly in Maglin North downstream of Maglin Bridge.

4.4 LIDAR Data

2.0m LIDAR data was obtained from National Map Services in March 2021 which covers the full Plan Area and some surrounding lands.

4.5 Rainfall Data

Hourly rainfall data was obtained from Met Éireann for Cork Airport synoptic station covering the period from January to March 2021. Daily rainfall data was also provided for Ballinacurra, County Cork which was used to assist with the analysis of the Balledmond flow data.

Depth Duration Frequency (DDF) tables were also obtained for the Plan Area from Met Éireann to assist with the analysis of the recorded data.

Daily rainfall data from a number of gauges on the Lee Catchment was obtained from Cork City Council. This includes the Iniscarra gauge.

An extensive review and analysis of the rainfall data pertaining to the February 2021 event has been carried out as part of this report, as outlined in Section 7.

4.6 Soil Moisture Deficit

In order to assist with the hydrological analysis and the assessment of the antecedent conditions, Soil Moisture Deficit (SMD) values at Cork Airport for February 2021 were obtained from Met Éireann.

4.7 Hydrometric Data

There is no reliable flow data available for the Maglin River however it was possible to obtain intermittent stage records from the ESBi for Station 19036 on the Maglin River and 19049 on the Curraheen River which was used to inform the hydrological analysis.

Flow data was obtained from the EPA for the Owennacurra at Ballyedmond (19020) covering February and March 2021.

4.8 Storm & Foul Network Drawings

Existing Storm and Foul network drawings for Ballincollig as well as design drawings for the Ballincollig Sewerage Scheme were provided by Cork City Council. These were used to assist in determining the urban catchment area and to identify the locations of outfalls to the river.

5. Identification of Potential Flood Risk Sources

The existing flood risk information has been reviewed in conjunction with observations from site walkovers and desktop studies in order to identify all possible sources of flooding which might affect the Plan Area. Potential sources of flooding and their relevance to the flood risk in the Plan Area are summarised on Table 5.1 below. Based on this, the following flooding sources require further assessment as part of this SFRA:

1. Fluvial Flooding from the Maglin River and other artificial drains
2. Pluvial Flooding due to accumulation of surface water in topographical depressions
3. Groundwater Flooding

Type of Flooding/Flooding Source	Comments
Fluvial Flooding	The Maglin River and a number of artificial drain traverse the site. The Lee CFRAMS has identified that areas of these lands are at risk of fluvial flooding and there are previous flood events which are known to have affected the area. Consequently, fluvial flooding from the Maglin River requires further assessment in this SFRA. In order to inform this, a Hydrological Analysis has been carried out as summarised in Sections 6 and 7and fluvial hydraulic modelling was undertaken which is outlined in Section 8.
Coastal/Tidal or Estuarial Flooding	Coastal/tidal and estuarial flooding are not relevant to this Plan Area and does not require further consideration.
Pluvial Flooding	There is no recorded history of pluvial flooding at this site however the desktop review and site walkovers indicate that some topographical depressions exist which may be result in surface water runoff ponding. Pluvial flood has been assessed further, as outlined in Section 9.
Groundwater Flooding	There is no identified flood history associated with groundwater flooding however the site characterisation and site walkovers indicate that this cannot be ruled out in some areas of the lands without further assessment. A qualitative assessment of groundwater flood risk is provided in Section 10.
Flooding from Overland Flow	This risk is associated with flooding caused by surface water runoff when rainfall intensity exceeds the infiltration capacity of the ground. Overland flow is most likely to occur following periods of sustained and intense rainfall when the ground surface becomes saturated. There is no specific catchment to contribute to significant overland flows entering the lands from outside of the site boundary. Runoff from some areas to the north of the site might be conveyed through the site if not already intercepted by artificial urban drainage systems however the runoff rates/volumes would be relatively small and any such risk will not affect strategic planning and would be best addressed at a local scale for individual development sites.
Infrastructure Failure	There is no specific infrastructure which is relied upon to reduce flood risk at this site therefore this does not require further consideration in this SFRA. The risk of failure of any flood risk infrastructure incorporated into the future development should be addressed in site specific flood risk assessments.
Flooding from Urban Drainage Systems	There are no significant existing urban drainage systems that could affect the Plan Area. The risk of flooding from urban drainage systems is more relevant to the post-development scenario which should be managed by implementing an appropriate Surface Water Management Plan for each development site.

Table 5.1: Identification of Potential Flood Sources

6. Hydrological Analysis

6.1 Purpose

The purpose of the hydrological analysis is to estimate the flood flows throughout the site and the surrounding areas for various Annual Exceedance Probabilities (AEP's). The results of this analysis will then form a key input into the subsequent hydraulic modelling of the Study Area which will enable the flood levels and extents to be determined.

6.2 Catchment & River Reach Description

The Maglin River consists of two main tributaries. The northern tributary flows through the study area where it joins the southern tributary approximately 440m downstream of the site boundary. Due to the construction of the N22 road system, local changes have been made to the river alignment and catchment area of both the north and south tributaries. In particular, the confluence of the two tributaries was previously within the subject site and, whilst this has shifted further downstream, a link hydraulic connection exists between the two rivers. This takes the form of a twin culvert system under the N22 road which allows flows to pass between the tributaries via the floodplain a short distance upstream of HEP MN02 (discussed below). For this reason it is necessary to include Maglin South River in the hydrological analysis and hydraulic modelling.

The Maglin River discharges into the Curraheen River approximately 600m downstream of the study area boundary. In order to ensure that the downstream boundary conditions are suitably accounted for in the hydraulic analysis, the model has been extended a distance downstream of the confluence with the Curraheen River. Therefore, it is necessary to include the Curraheen River in the hydrological analysis.

The sub-catchment delineation of the key catchments included in this study is shown on Figure 6.1 below.

Maglin North has a catchment area of approximately 5.6km^2 and 9.8km^2 at the inflow point and outflow point of the study area respectively. The catchment elevation varies from circa 180mOD at the southwest divide to 10mOD at the downstream end of the study area. The upstream catchment is characterised by relatively steep reaches, whilst the gradient reduces significantly on the northern downstream portion of the catchment.

The soil classification for the catchment is Soil Class 2 according to the Flood Studies Report 1975 (FSR) soil maps. This is consistent with MWP's local knowledge of the area and the EPA soil mapping. The principal land uses on the catchment are plotted on Figure 6.2 based on Corine 2018.

The catchment of Maglin North River is also influenced by urban development in Ballincollig and the N22 road, primarily via inflows within or adjacent to the study area boundary. Estimation of the urban area contributing to the catchments was influenced by the available storm and foul network drawings which are available for the town.

Specific catchment characteristics are discussed in the following sub-sections for all relevant reaches.

On the lower reaches, parts of the Maglin and Curraheen River catchments are located within a drainage district. These lands were drained by the Commissioners of Public Works under various acts between 1842 the 1930's. The purpose of the schemes were to improve land for agriculture by lowering water levels. Typical works included channel and lake widening / deepening, embankment construction, weir removal and bridge replacement / modification. The extent of the benefited lands associated with this study are indicated on Figure 6.3. For Section 50 applications the OPW typically requires that design flows in drained catchments be increased by a factor of 1.6 to account for increased rate of runoff, and therefore peak flows, from the artificially drained areas. It is entirely possible that Local Authorities have not maintained the drainage districts in the recent past there any such increased flows may not be realised. In any case, this has been accounted for in the FSU methodology with the ARTDRAIN2 PCD.

The Standard Average Annual Rainfall (SAAR) for the Maglin catchment is approximately 1080mm according to the FSU database. It is slightly higher at circa 1100mm for the Curraheen catchment.

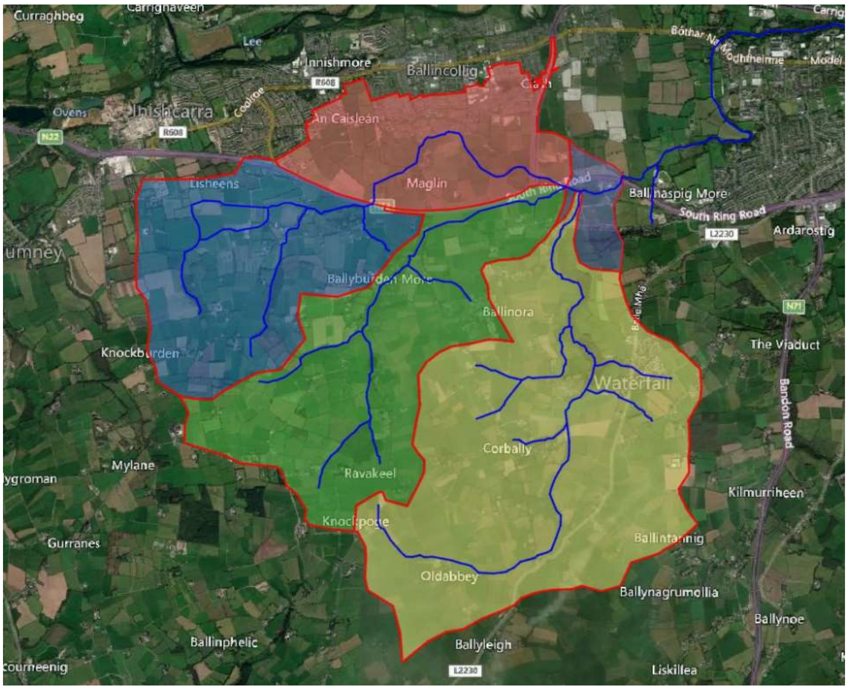


Figure 6.1: Sub-catchment Delineation

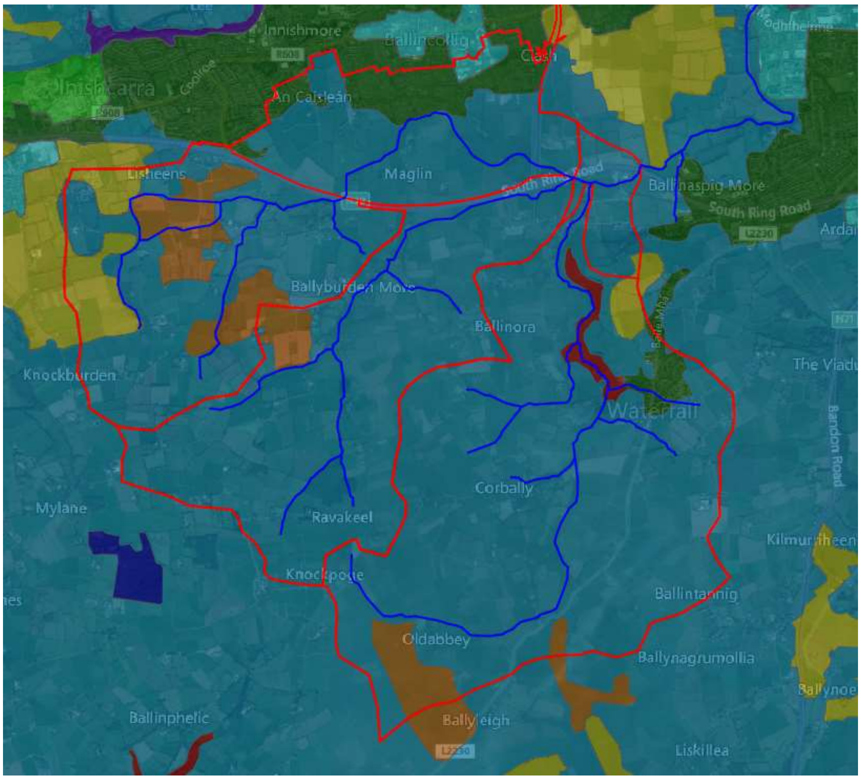


Figure 6.2: Corine Land Use 2018

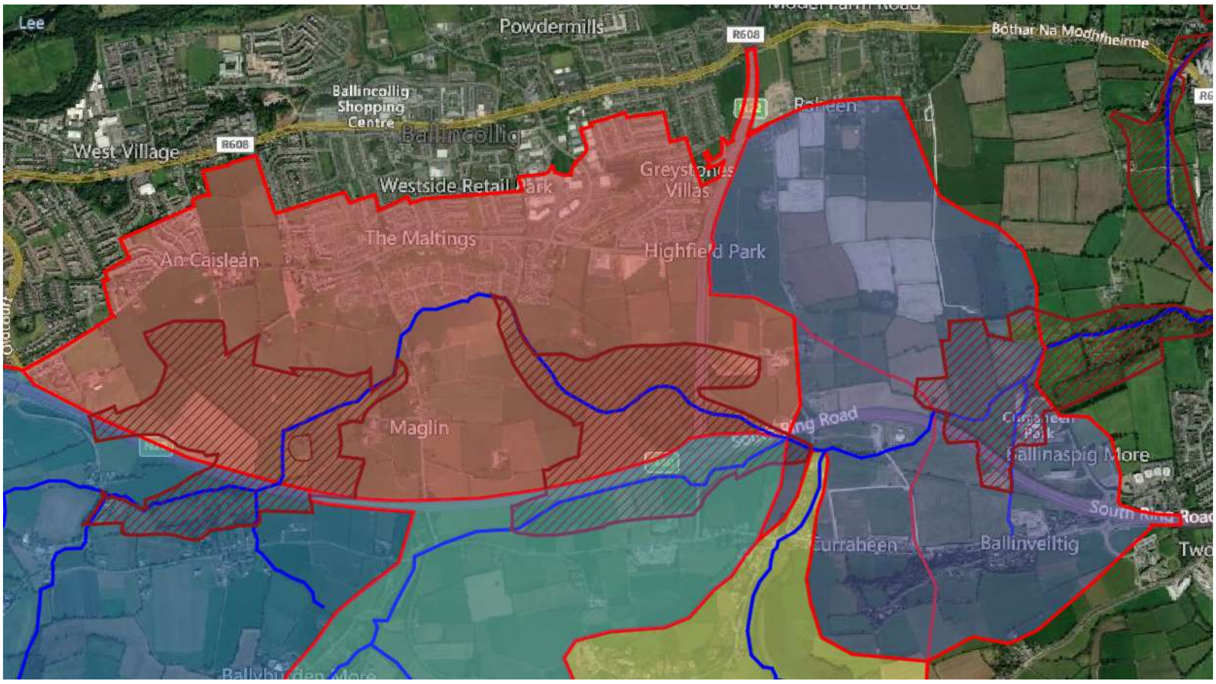


Figure 6.3: Benefiting Lands

6.3 Hydrological Estimation Points

In order to gain a suitable understanding of the flood sources and mechanisms and to build a representative hydraulic model of the system, it is necessary to estimate the relevant flows at suitable locations along the reach. Hydrological Estimation Points (HEP) were selected at relevant locations along the length of the rivers, typically at the location of significant changes to the catchment area or flow regime. The location of all HEP's selected for this study are summarised on Table 6.1 and Figure 6.4 below.

Flows at MN02 are not presented in the following sections because the initial flood estimation indicated that the difference in peak flow between upstream and downstream HEP is not significant.

HEP	River	Description
CU01	Curraheen	Near downstream end of model
CU02	Curraheen	N40 crossing
CU03	Curraheen	N40 crossing
MA01	Maglin Lower	Downstream of Confluence
MS01	Maglin South	Upstream of Confluence
MN01	Maglin North	Upstream of Confluence
MN02	Maglin North	Discharge Point from Study Area
MN03	Maglin North	Open Channel, downstream of urban inflows
MN04	Maglin North	Downstream of Artificial Drain confluence
MN05	Maglin North	Upstream of Artificial Drain confluence
MN06	Maglin North	Entry point to site/Downstream of N22 Culvert
AD01	Artificial Drain	Open Channel
AD02	Artificial Drain	Upstream end of model

Table 6.1: Hydrological Estimation Points

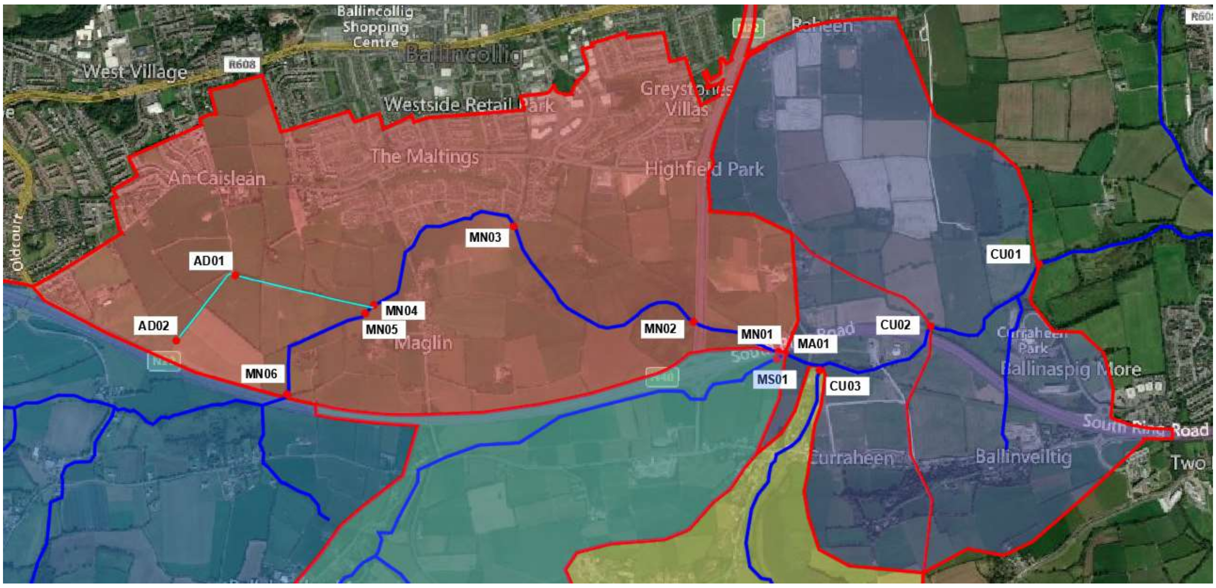


Figure 6.4: Hydrological Estimation Points

6.4 Flow Estimation

6.4.1 Overview of Methodology

The Flood Studies Update (FSU) programme was undertaken by the OPW in order to provide improved extreme rainfall and flood estimation methods for Ireland. It is the most recent major study of its kind to be carried out in Ireland and is broadly recognised as the best practice method for estimating peak flood flows.

One of the key outputs from the FSU was the 7 variable regression equation for estimating the Index Flood (i.e. Q_{MED}) based on Physical Catchment Descriptors (PCD's). The Index Flood is the flow that can statistically be expected to be equalled or exceeded once in a 2 year period. Ideally the application of this equation would be limited to catchments greater than 25km^2, although it has been shown to perform reasonably well for smaller catchments, albeit with a seemingly higher Factorial Standard Error (FSE) of 1.879 rather than 1.37 (Gebre et al., 2012). An alternative regression equation which was developed as part of FSU WP 4.2 for small and urbanised catchments was shown to perform slightly better than the 7 variable equation with a FSE of 1.674. However, it was concluded that the dataset on which the small catchment equation was based is very small and the FSU 7 variable equation is still the preferred method for catchments greater than 5km^2.

It is for these reasons, and others outlined below, that the flows estimated using the FSU 7 variable regression equation have been compared to the flows calculated using the following methods;

1. FSU WP4.2 Small Catchment Equation, modified by Gebre et. al. 2012;
2. Flood Studies Report (FSR) 1975 Rainfall Runoff (RR) Method;
3. Institute of Hydrology (IH) 124 3-variable Equation;
4. Poots & Cochrane, 1979

Since it is the flows associated with the northern tributary of the Maglin River that are of most interest to this study, design flows were initially estimated at MN01 and MN06 using the methods outlined above. The selected method was then applied to the other HEP's in the study area.

6.4.2 Index Flood Estimation

6.4.2.1 FSU 7 Variable Regression Equation

The FSU method for ungauged catchments uses Physical Catchment Descriptors (PCD's) to establish an initial estimate of the Index Flood (i.e. Q_{MED}) based on a seven variable regression equation. The initial PCD estimate can be improved by using data from a hydrologically and/or geographically similar gauged site, referred to as a Pivotal Site.

The PCD's which were initially obtained from the FSU Web Portal were checked for suitability using available mapping and land-use data. Various adjustments were made to the PCD's which would be deemed to provide a more reliable estimate of peak flow.

The general procedure for estimating the Index Flood at any HEP can be summarised as follows;

1. Review the Physical Catchment Descriptors at each HEP and identify suitable pivotal site(s);
2. Estimate the Index Flood at the potential pivotal site(s) using annual maxima data;
3. Estimate the Index Flood at the potential pivotal site(s) using Physical Catchment Descriptors and determine the appropriate adjustment factor (i.e. Q_{MED} Gauged / Q_{MED} PCD Rural);

4. Estimate the Index Flood at each HEP using Physical Catchment Descriptors;
5. Estimate the Design Index Flood flow at each HEP using the relevant gauging station as a pivotal site and adjust the rural estimate for urbanisation.

There are no reliable gauging stations on the Maglin and Curraheen Rivers. The ESB maintained gauging stations on the catchment for a relatively short period however the reliability of the records and the gaugings are not suitable for statistical analysis.

Potential pivotal sites were identified using the FSU Web Portal based on the hydrological similarity of the catchments. As a rule of thumb, a hydrological similarity less than 1 indicates high similarity, a value between 1 and 2 indicates medium similarity and a value greater than 2 indicates low similarity.

The Physical Catchment Descriptors (PCD's) for the Maglin River at MN01 and for the top three sites ranked in terms of hydrological similarity are tabulated on Table 6.2 below. The same ranking would apply to MN06 although the hydrological similarity across all sites is lower for this HEP.

Although all sites have a comparatively high hydrological similarity, it can be seen from the table that there is an appreciable difference in the pivotal site adjustment factors. Stations 30020 is a karstic catchment and was therefore rejected. Station 25040 would appear to be more acceptable however ARTDRAIN2 is zero and this is one of the important descriptors in predicting Q_{MED} in drained catchments. Station 25034 has a higher ARTDRAIN2 than the subject site which may be a contributory reason for having a higher adjustment factor. On balance Station 25034 has been selected as this would seem to best reflect the subject site PCD's when all parameters are considered and this is also indicated by the improved hydrological similarity.

Station	AREA	BFISOIL	SAAR	FARL	DRAIND	S1085	ARTDRAIN 2	URBEXT	Hydrologica l similarity	Adjustmen t Factor	Gauge Class
MN01	9.82	0.701	1080	1	1.192	5.430	0.317	0.131	-	-	-
25034	10.77	0.759	969	1	0.273	2.572	0.678	0	0.73	1.48 *	A2
30020	21.41	0.684	1191	1	1.846	2.891	0.678	0.010	0.84	0.74	B
25040	28.02	0.641	990	1	1.204	13.494	0	0.0618	1.05	0.61	A2

** FSU appears to use a different rating eqn than EPA but it cannot be obtained. The longer record from EPA clearly shows an increasing trend so this should not be disregarded. Therefore, the FSU Qmed has been scaled up by the ratio of the EPA increase between the two time series.*

Table 6.2: PCD's & Hydrological Similarity for HEP's and Potential Pivotal Sites

The Index Flow Q_{MED} is estimated using the following seven variable regression equation which was presented in FSU WP2.3:

$$Q_{MED} = 1.237x10^{-5} AREA^{0.937} BFIsoils^{-0.922} SAAR^{1.306} FARL^{2.217} DRAIND^{0.341} S1085^{0.185} (1 + ARTDRAIN2^{0.408})$$

The factorial standard error (FSE) of this equation was discussed above. The adjustment for urbanisation is made by applying the following equation:

$$Q_{MED\ URBAN} = Q_{MED}(1 + URBEXT)^{1.482}$$

When the above equations are applied to the Maglin River in conjunction with the pivotal site adjustment factor, the resulting estimate of $Q_{MED\ URBAN}$ at MN01 and MN06 is 3.86m^3/s and 2.34m^3/s respectively. It is notable that these flows are higher than the 68% confidence level flows that would be obtained using PCD's only.

6.4.2.2 FSU WP4.2 Small Catchment Equation

The FSU small catchment equation, modified by Gebre et al. (2012) uses the following simplified 5-variable equation:

$$Q_{MED} = 2.3848x10^{-5}AREA^{0.9245}SAAR^{1.2695}BFIsoils^{-0.9030}FARL^{2.3163}S1085^{-0.2513}$$

The factorial standard error (FSE) of this equation was discussed above and the urban adjustment factor has been applied using the same methodology as the 7-variable equation.

The equation was applied to the Maglin River at MN01 and MN06 in order to provide a comparative estimate of Q_{MED} which produced a $Q_{MED\ URBAN}$ flow of 3.54m^3/s and 2.43m^3/s respectively.

6.4.2.3 FSR Rainfall Runoff Method

The Flood Studies Report 1975 Rainfall Runoff Method (FSR RR) has also been applied to the catchment to MN01 and MN06 to provide a comparison with the FSU 7-variable equation flows. This method uses catchment characteristics to estimate the catchment response to rainfall using a synthetic unit hydrograph and losses model with a theoretical rainfall storm profile. Convolution of the design effective rainfall and the unit hydrograph is carried out to estimate the design flood hydrograph.

The method has the added advantage of explicitly receiving rainfall inputs which can be useful for assessing past flood events, as will be discussed in the following sections of the report.

The catchment characteristics adopted in the analysis correspond generally to those outlined above for the FSU techniques. Soil Type 2 was used across the entire catchment based on the FSR Winter Rain Acceptance Potential (WRAP) maps and the urban catchment area was measured from the 1:50,000 scale Discovery Series mapping.

The 75% winter profile has been adopted for the critical storm profile as recommended for predominantly rural catchments and the return period of the rainfall to produce each return period flood is in line with FSSR 5 and FSSR 16 recommendations.

The unit hydrograph time to peak (T_p) is a valued parameter in estimating hydrograph shape and peak flow using the FSR RR method. Various studies have demonstrated that an improvement can be made on flow estimates if site specific data is available to estimate T_p. It was possible to obtain recorded water levels for the inactive ESB gauging stations 19036 on Maglin South River and 19049 on the Curraheen River. Although the records are intermittent, there was sufficient data to make a reasonable estimate of catchment lag (LAG) and use this to derive the instantaneous unit hydrograph time to peak ($T_p(0)$) directly based on FSSR13, 16 and FEH1999 recommendations. This procedure suggests that it is appropriate to increase $T_p(0)$ on both the Maglin and Curraheen Rivers by a factor of 1.32.

The critical storm duration was estimated to be 13 hours and 9 hours at MN01 and MN06 respectively and the design rainfall depth was determined using the FSU DDF module. The respective Q_{MED} flows were then estimated to be 3.36 m^3/s and 2.16m^3/s.

It is not possible to assign a confidence level to the flows obtained using this technique. The method was tested on 36 Irish catchments and it was found that the 25 year return period flow was overestimated in 30 of the 36 catchments with 24 catchments being overestimated by over 150% (Bree et al., 1989). On the other hand, it was found in the same study that the method tends to underestimate percent runoff, particularly for upland catchments.

6.4.2.4 IH124 Equation

The Institute of Hydrology Report 124 method has been widely used in Ireland and the UK for flood estimation in small catchments. The equation uses three variables from the FSR to determine the mean annual flood flow Q_{bar}, namely SOIL, SAAR and AREA. The Mean Annual Flood Flow Q_{BAR} is estimated using the following three variable regression equation. This is the flow that can statistically be expected to be equalled or exceeded once in a 2.33 year period.

$$Q_{BAR} = 0.00108 AREA^{0.89} SAAR^{1.17} SOIL^{2.17}$$

A growth factor of 0.95 can be applied to Q_{bar} to estimate Index Flood, Q_{MED}. The application of this equation in conjunction with urban adjustment results in a Q_{MED} value of 2.54m^3/s and 1.33m^3/s at MN01 and MN06 respectively. The IH124 equation has a factorial standard error of 1.65.

6.4.2.5 Poots & Cochrane Equation

Poots & Cochrane method is published in the Institution of Civil Engineers TN229, Design Flood Estimation for Bridges, Culverts and Channel Improvement Works on Small Rural Catchments (1979).

The method, which is intended for rural catchments of area less than 20km^2, used the FSR catchment characteristics data from 42 catchments throughout the British Isles to derive a three-variable equation which is claimed to statistically provide better results that previous methods.

The Mean Annual Flood Flow Q_{BAR} is estimated using the following three variable regression equation:

$$Q_{BAR} = 0.0136 AREA^{0.866} RSMD^{1.413} SOIL^{1.521}$$

The equation has a standard error of 1.64 therefore the calculated value of Q_{bar} was increased by a factor of 1.64 in order to achieve a 68% confidence that the flow will not be exceeded within the specified return period (refer to previous sub-section for the basis of selecting this confidence level).

This publication was published prior to the revised SOIL parameters in FSSR 16 therefore the derivation of the equation relates to the original FSR 1975 values. However, for the purpose of this study the revised FSSR 16 coefficients have been adopted.

A growth factor of 0.95 can be applied to Q_{bar} to estimate Index Flood, Q_{MED}. The application of this equation in conjunction with urban adjustment results in a Q_{MED} value of 3.60m^3/s and 2.33m^3/s at MN01 and MN06 respectively.

6.4.3 Flood Frequency Analysis

In order to establish the design flows for various AEP's it is necessary to derive a suitable flood growth curve which is used to scale Q_{MED} for the return period of interest.

6.4.3.1 FSU Pooled Growth Curve

Based on the FSU guidance, an improved growth curve can generally be derived by pooling a number of station records. For this study a pooling group has been selected based on the most hydrologically similar gauged sites using the ranked list provided on the FSU Web Portal. For a target design event of 100 year return period, the 5T rule adopted by FEH 1999 and the FSU requires a minimum record length of 500 years.

The growth factors derived from the pooled analysis with the 3 parameter GLO distribution, the GEV distribution and the 2 parameter EV1 distributions are tabulated on Table 6.3 and plotted on Figure 6.5 for MN01 and MN06. The growth factors from the Lee CFRAM Study and the FSR Regional Growth Curve are also included on the figure for comparative purposes. The GEV growth curve could not be developed for MN06 because the parameter k is too close to zero.

The L-moment diagrams indicate that the GEV distribution provides a better fit than the GLO distribution because the L-skewness and L-Kurtosis distribution values for GEV pass more centrally through the L-moments of the pooling sites and the study average. This may be a contributory reason for MN06 GLO distribution being a clear outlier to the other growth curves plotted on Figure 6.5. The choice of GEV distribution is consistent with the Lee CFRAM Study findings however in this case it is apparent that the distribution provides for a concave downward shape which presents a risk of underestimation for rarer events. The growth curve derived for MN01 with EV1 distribution appears to be most consistent with the Lee CFRAM Study Growth Curve and the FSR Regional Growth Curve.

Notwithstanding the above comments, it is desirable for consistency to adopt a single growth curve for the entire study area and, on balance, it is considered that the growth curve derived at MN01 with GLO distribution provides the best overall representation of both the study growth curves, the Lee CFRAM Study and the FSR Regional Curve. The choice of GLO is consistent with FSU research which indicates that 3 parameter distributions are generally more suitable for ungauged catchments and furthermore it provides a more cautious estimate than the GEV and EV1 distributions for rarer events.

FSU Pooled Growth Factors						
Return Period, T	Probability	MN01 Pooled - GLO - X_T	MN06 Pooled - GLO - X_T	MN01 Pooled - GEV - X_T	MN01 Pooled - EV1 - X_T	MN06 Pooled - EV1 - X_T
2	0.50	1.00	1.00	1.00	1.00	1.00
5	0.20	1.26	1.30	1.29	1.30	1.33
10	0.10	1.44	1.51	1.47	1.49	1.55
20	0.05	1.62	1.73	1.64	1.68	1.76
30	0.033	1.74	1.87	1.74	1.79	1.88
50	0.02	1.88	2.06	1.85	1.92	2.03
100	0.01	2.10	2.33	2.00	2.11	2.24
200	0.005	2.33	2.64	2.14	2.29	2.44
500	0.002	2.67	3.10	2.32	2.53	2.71
1000	0.001	2.96	3.51	2.44	2.71	2.91

Table 6.3: FSU Pooled Growth Factors

6.4.3.2 FSR Rainfall Runoff Growth Curve

In order to estimate the Q_T/Q_{MED} growth curve for the FSR RR method, it is necessary to estimate the peak flow for each increment of return period and divide this by Q_{MED}. As noted above, the 75% summer rainstorm profile has been adopted. The resulting growth curve is plotted on Figure 6.5 below.

6.4.3.3 FSR Regional Growth Curve

The growth curve applied to the mean annual flow (Q_{bar}) estimated using the IH124 and Poots & Cochrane methods is the FSR regional growth curve for Ireland which is also shown on Figure 6.5 below.

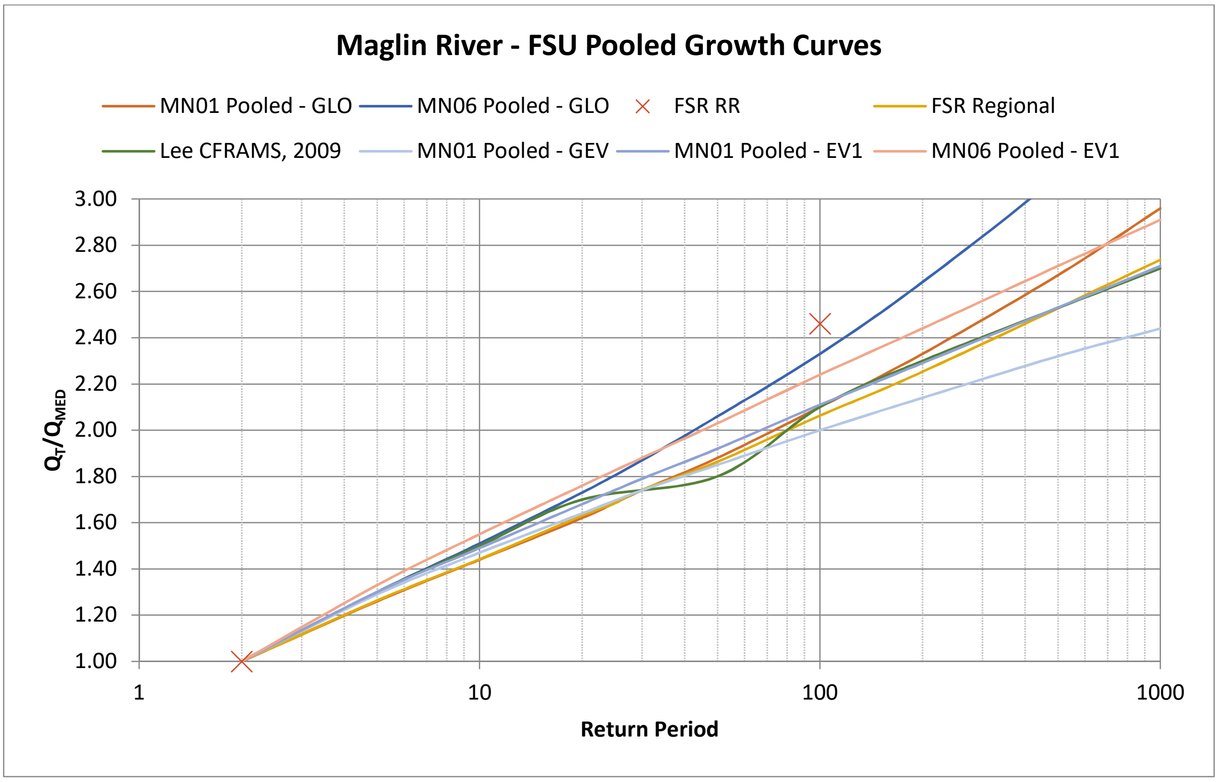


Figure 6.5: Flood Growth Curve Comparison (All Methods)

6.4.4 Summary of Flow Estimates for Various AEP's

The peak flows estimated at MN01 using the techniques outlined above are summarised on Table 6.4 and plotted on Figure 6.6 below for various AEP's. The peak flows estimated at MN06 are similarly presented on Table 6.5 and Figure 6.7. The IH124 is a clear outlier and is not considered further. It is encouraging that the estimated flows up to around 1% are broadly similar, with a maximum difference of around 10%. The difference increases to around 15% for the 0.1% AEP event. Poots & Cochrane method mainly differs for rarer events due to the difference in the adopted growth curves. When this method is eliminated, the maximum difference between the three remaining methods is less than 10% for all events.

The analysis carried out indicates that the FSU 7-variable equation with pivotal site adjustment is suitable for estimating Q_{MED} at all HEP's. The flows for various AEP's will be derived using the growth curves derived from the FSU pooling group.

The peak flows in the artificial drain (AD01 and AD02) are assumed to correspond to the difference between the peak flows of MN04 and MN05. This approach is considered reasonable based on a comparison with the flow estimated for the artificial drain using the IH124 method.

MN01 - Peak Flow Summary						
Return Period, T	AEP (%)	FSU 7-Variable	FSU Small Catchment Eqn.	FSR RR	IH124	Poots & Cochrane
2	0.50	3.86	3.54	3.36	2.54	3.60
5	0.20	4.86	4.46		3.21	4.55
10	0.10	5.56	5.09		3.66	5.19
20	0.05	6.25	5.73		4.14	5.87
30	0.033	6.71	6.16		4.42	6.27
50	0.02	7.25	6.65		4.73	6.71
100	0.01	8.10	7.43	8.27	5.24	7.43
200	0.005	8.99	8.24		5.72	8.11
1000	0.001	11.42	10.47		6.95	9.85

Table 6.4: MN01 - Summary of Peak Flows

MN06 - Peak Flow Summary						
Return Period, T	AEP (%)	FSU 7-Variable	FSU Small Catchment Eqn.	FSR RR	IH124	Poots & Cochrane
2	0.50	2.34	2.43	2.16	1.33	2.33
5	0.20	2.95	3.06		1.67	2.94
10	0.10	3.37	3.50		1.91	3.36
20	0.05	3.79	3.94		2.16	3.80
30	0.033	4.07	4.23		2.31	4.06
50	0.02	4.40	4.57		2.47	4.34
100	0.01	4.91	5.11	5.36	2.73	4.81
200	0.005	5.45	5.66		2.99	5.25
1000	0.001	6.92	7.20		3.63	6.38

Table 6.5: MN06 - Summary of Peak Flows

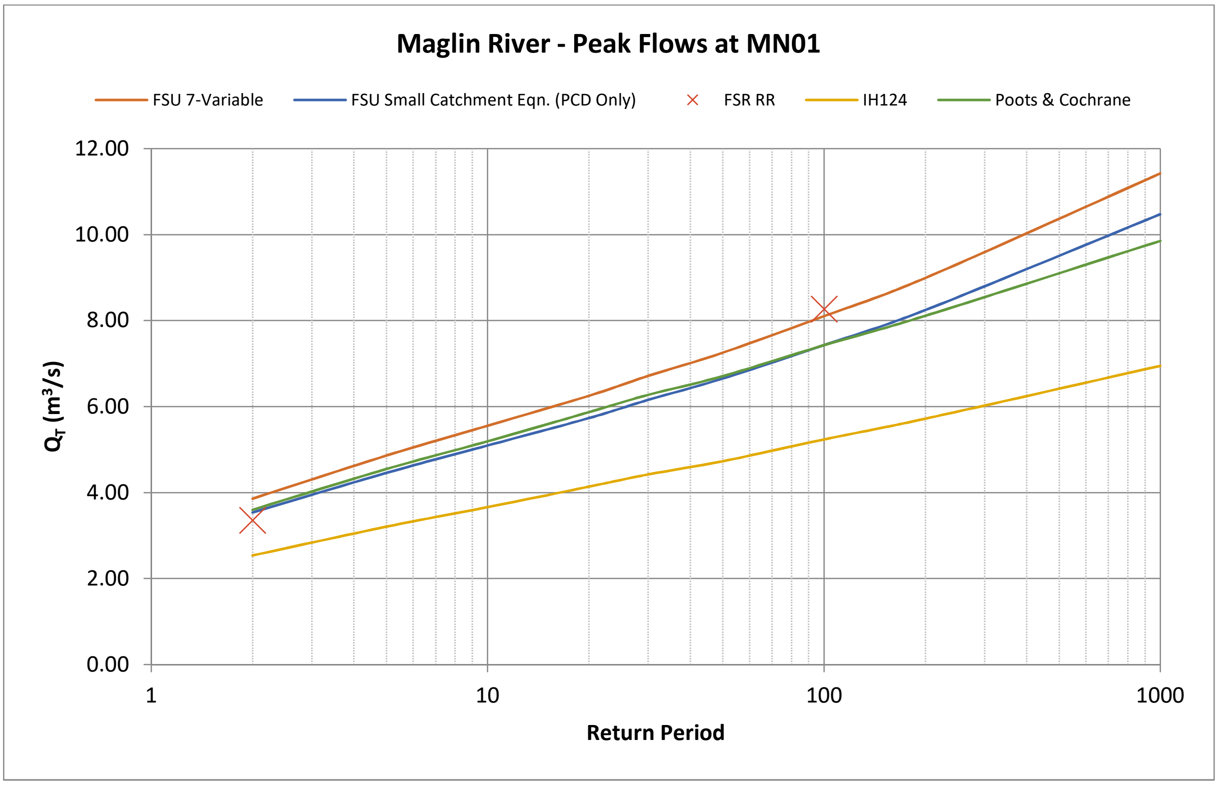


Figure 6.6: MN01 - Summary of Peak Flows

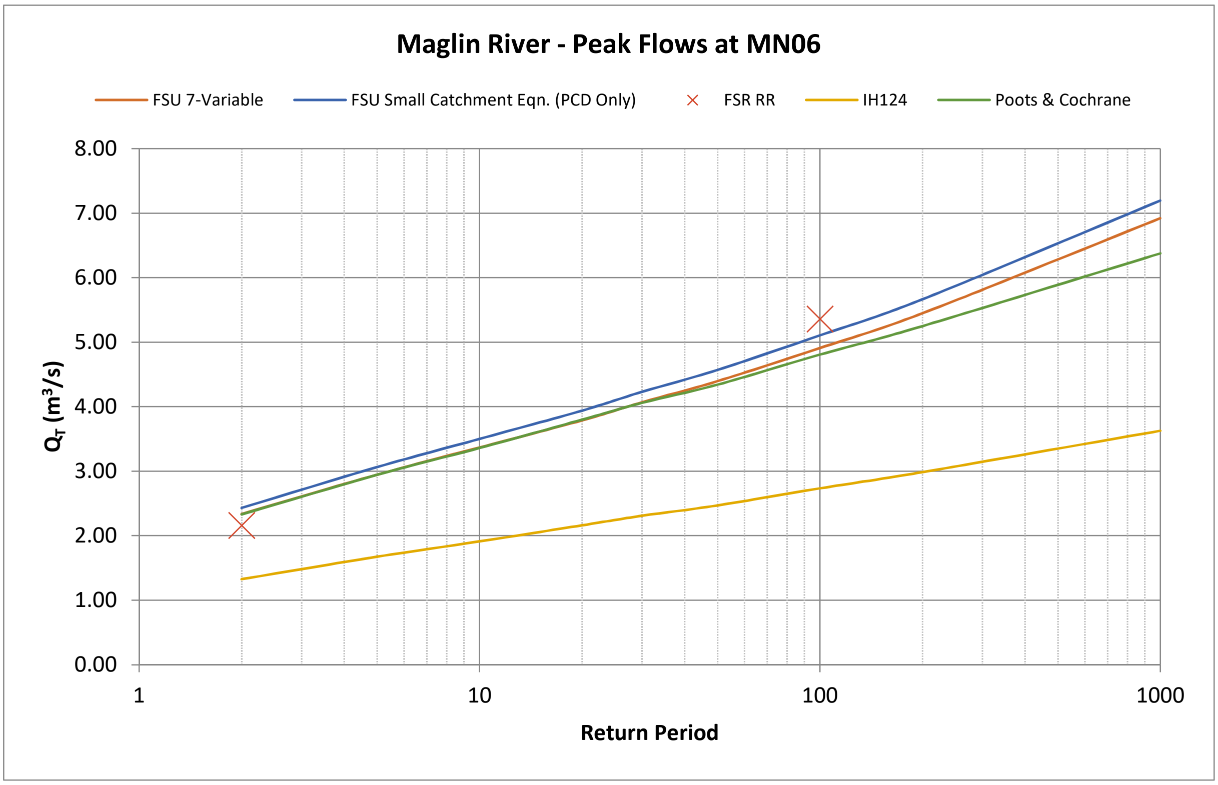


Figure 6.7: MN06 - Summary of Peak Flows

6.5 Hydrograph Derivation

The FSU Web Portal module allows the user derive a hydrograph for an ungauged site from a statistical analysis of the continuous flow records for gauged sites. However this method has no direct link to rainfall and the majority of catchments are much larger than the subject catchment which could result in significant distortion of the hydrograph shape.

Consequently the FSR hydrograph derived using site specific data for catchment lag is deemed more suitable and this is consistent with the approach used in the Lee CFRAMS for ungauged catchments. The derived FSR hydrographs were scaled to coincide with the design peak flows estimated using the FSU methods. As will be discussed below, this approach is also convenient as it facilitates flood event analysis.

The design hydrographs are plotted on Figure 6.8 below for all relevant HEP's. These hydrographs have been scaled using the FSU growth curves to establish the hydrograph shape and peak for each AEP event.

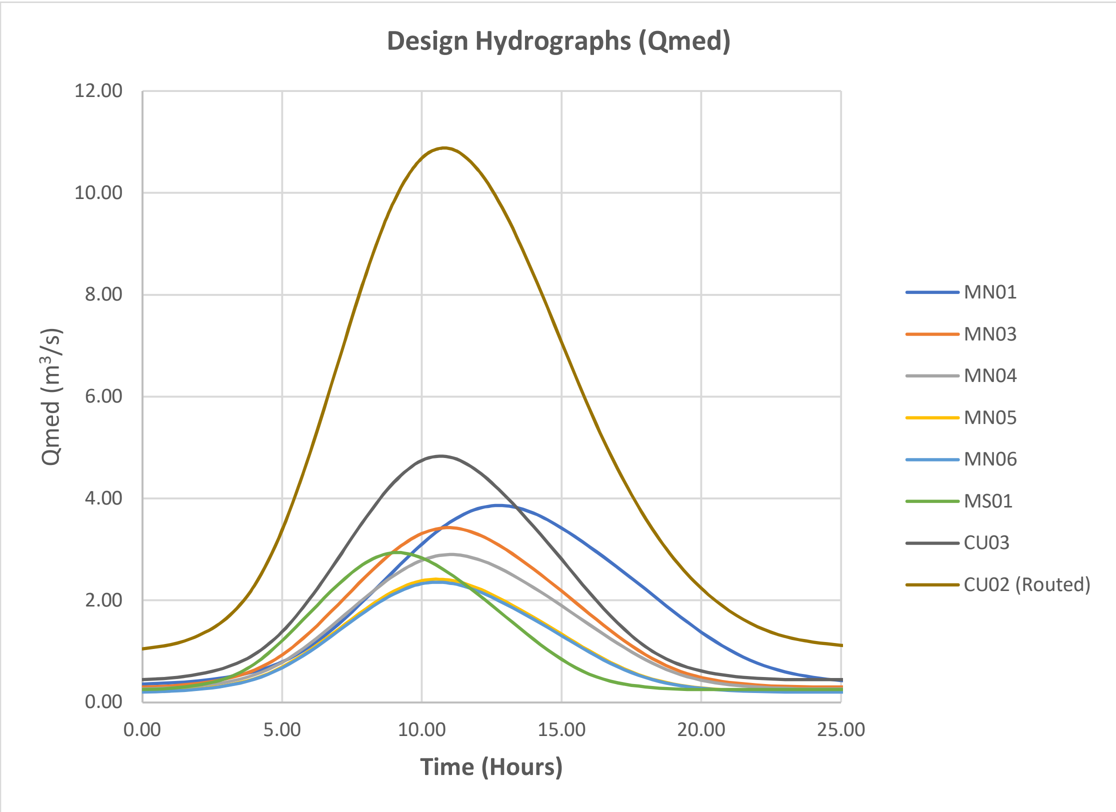


Figure 6.8: Design Flow Hydrographs

7. February 2021 Flood Event Analysis

7.1 Background

Following a period of prolonged rainfall starting from 17th February 2021, flooding at the subject site and the surrounding areas was observed on 19th February 2021. In order to further inform the hydrological and hydraulic analyses carried out as part of this SFRA, an assessment has been carried out to estimate the likely return period of the flood.

7.2 Observed Flood Extents

7.2.1 19th February 2021 Site Walkover

A walkover of the site and surrounding areas was carried out on 19th February 2021 when flooding was observed at various locations and photographic evidence of flood extents was collected. Photographic records of observed flooding at specific locations are included in Appendix A and a summary of the key comments is provided below.

Flooding of the western portion of the site was observed due to overtopping of the Maglin River. This occurred on the agricultural lands between the N22 and Maglin Road. Generally flooding was more extensive towards the upstream end of the reach, where the Maglin River flows northerly under the N22.

The artificial drainage ditch running through the site was not viewed during this walkover, therefore it is not possible to confirm if flooding occurred in this part of the lands.

Flooding of the lands downstream of Maglin Road to the N22 culvert appeared to be limited predominantly to the low lying lands at the southeast corner of the site. The culverts linking Maglin South and Maglin North tributaries were flowing full and were partially blocked. Downstream of the site, the culvert which carries the Maglin North under the N40 was almost submerged at the outlet.

Outside of the subject site, extensive flooding of the lands to the south of the N22 was observed due to overtopping of both the Maglin South and North tributaries.

The Curraheen River overtopped its banks at Curraheen Village resulting in a very small area of flooding on the public road. It is notable that more extensive flooding has been witnessed at this location on a number of occasions.

Extensive flooding from the Curraheen River was observed on the low lying lands to the west of the Greyhound Stadium. This area is known to flood frequently.

7.2.2 Surveyed Flood Levels

During the 19th February 2021 flood event, the flood level was measured at some discrete locations and a subsequent survey was carried out on in March 2021 to assist in estimating flood levels at other locations, based on photographic evidence available from the site walkover. A location of the surveyed points and the estimated flood levels are provided on Figure 8.3, Figure 8.4 and Table 8.1.

7.3 Rainfall Review & Analysis

A detailed review and analysis of the rainfall prior to the flood event has been carried out using data obtained from Met Éireann and Cork City Council. The purpose of this was to estimate the return period of the rainfall and to gain an appreciation of the antecedent conditions to inform the hydrological analysis.

Figure 7.1 below includes the daily and cumulative rainfall recorded at Cork Airport for a one month period prior to the flood event. It can be seen from this that the total rainfall amounts to 272mm which is predominantly due to two separate periods of prolonged rainfall. The first occurred from 26th January to 02nd February when and the second occurred from 11th February to 19th February. According to Met Eireann data, the 30 year average monthly rainfall at Cork Airport for January and February is 131mm and 98mm respectively. Therefore, the rainfall recorded in the month preceding the event was well over double the average values.

Data for Inniscarra rain gauge was provided by Cork City Council. This gauge is located to the northwest of the site and is slightly closer to the Maglin catchment centroid than Cork Airport however the gauge elevation is lower. The data from both gauges is likely to be relevant to the catchment response therefore the rainfall analysis has been carried out using the two datasets.

The hourly rainfall for the event is plotted on Figure 7.2 below. Rainfall on the evening of 18th February commenced at 21:00 and continued until 14:00 hours on 19th February. The storm duration was therefore 17 hours during which a rainfall depth of 42.7mm and 38.0mm was recorded at Cork Airport and Inniscarra respectively. The storm profile does not include any particular intense bursts, although the intensity was greater in the first eight hours or so.

Met Éireann DDF data was analysed and used to create a series of depth-duration curves for various return periods which is produced on Figure 7.3 and Figure 7.4 below for Cork Airport and Inniscarra respectively. The maximum recorded rainfall depths for durations of 3 to 17 hours were calculated using the rain gauge data and plotted onto the DDF curves in order to determine the return period of the storm. This analysis indicates that the maximum rainfall return period at Cork Airport was just over two years which occurred for the 15 and 16 hour storm durations. The analysis at Inniscarra suggests that the storm was less severe and the return period is likely to be less than one year.

In terms of the catchment rainfall, it appears likely that the storm return period would be somewhere between Cork Airport and Inniscarra. Therefore, it is concluded that the rainfall return period is likely to be one to two years. It is important to note that the assessment carried out in this section is in relation to the point rainfall return period only. It does not necessarily represent the catchment response to rainfall which requires an estimate of the flood return period, as described in the following section of the report.

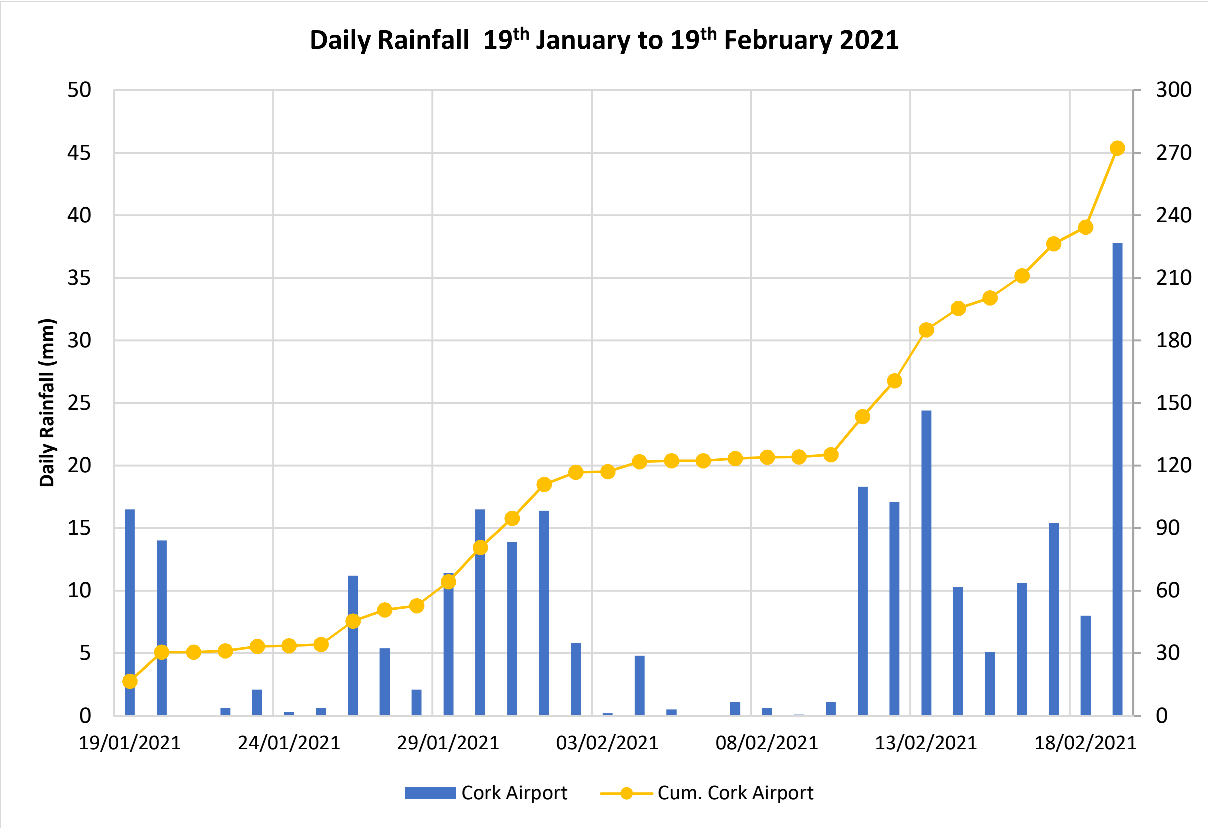


Figure 7.1: Cork Airport Daily & Cumulative Rainfall 19th January to 19th February 2021

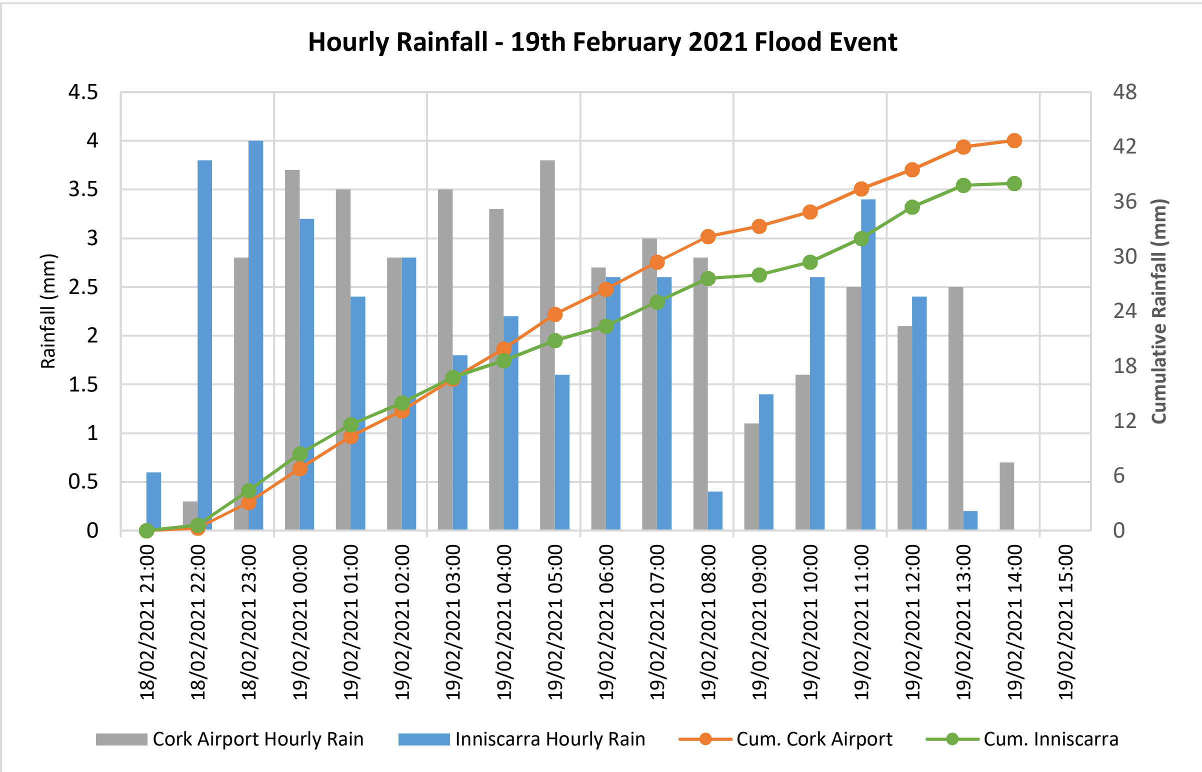


Figure 7.2: Hourly Rainfall Recorded at Cork Airport & Inniscarra for 19th February 2021 Flood Event

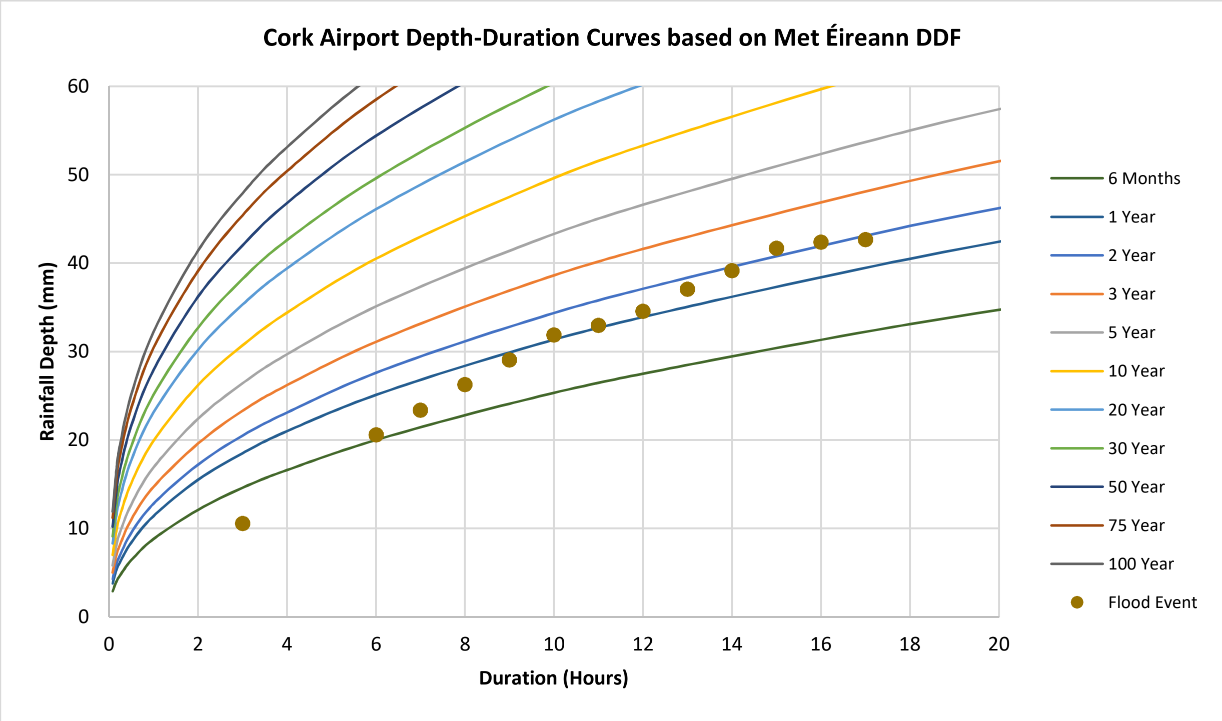


Figure 7.3: Cork Airport Depth Duration Curves Derived from Met Eireann Data with February 2021 Flood Depths Plotted

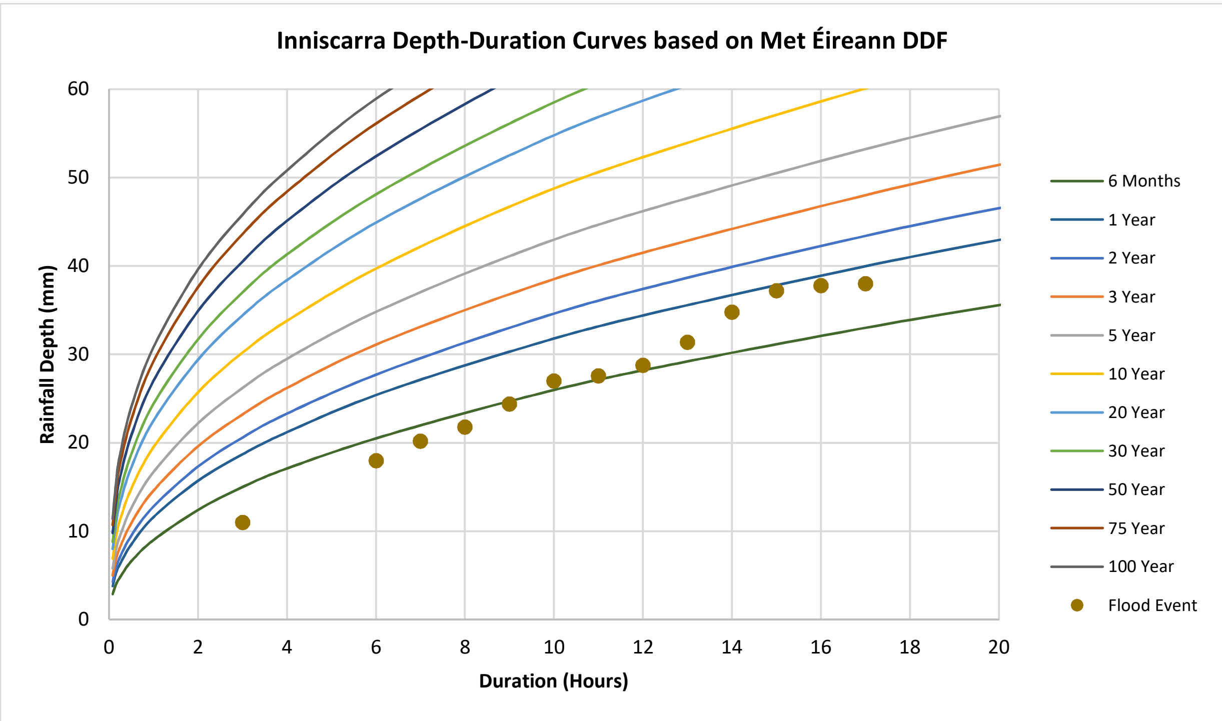


Figure 7.4: Inniscarra Depth Duration Curves Derived from Met Éireann Data with February 2021 Flood Depths Plotted

7.4 Hydrological Analysis of February 2021 Event

7.4.1 Overview

The analysis of the rainfall for the 19th February 2021 flood event was described above and the return period of the rainstorm would appear to be estimated with reasonable reliability. However, the storm return period would not necessarily correspond to the return period of the flood, for several reasons including variations in the catchment response and the rainfall duration and profile. Consequently it is necessary to analyse the observed rainfall inputs and convert these to a flow hydrograph for the event. Since gauged flow records are not available, the FSR RR method can be used to simulate the event, as described in FSSR 12 and FEH 1999. The simulated peak flow can then be compared to the flood growth curve derived in the previous section to obtain an estimate of the peak flow and return period.

7.4.2 Derivation of Inputs

Given the location of the two available hourly rain gauges relative to the Maglin catchment, there appears to be little benefit in developing possible weightings to establish the representative rainfall for the catchment. In this instance, it is deemed appropriate to adopt a catchment rainfall corresponding to the average of Cork Airport and Inniscarra hourly records. The average catchment rainfall established on this basis is plotted on Figure 7.5. It can be seen that the storm duration was 17 hours and the total rainfall for the event was 40.4mm.

There are no gauged records on this catchment. The use of gauged data from a number of adjacent catchments was considered however all nearby catchments were ruled out, mainly due to unreliable ratings or missing data. The closest suitable gauged site is 19020 at Ballyedmond. This site has a hydrological similarity to the Curraheen River of 0.85 however it increases to 1.71 on the Maglin at MN01. It was possible to use Ballyedmond gauge as a donor site to improve the estimates of the event percent runoff and baseflow. The transfer equations were in accordance with FEH1999.

A key aspect of the analysis is estimation of the percent runoff for the event which depends on the antecedent rainfall and catchment wetness. In order to estimate the catchment wetness index at the start of the event, and ultimately the percent runoff throughout the event, the antecedent rainfall and Soil Moisture Deficit (SMD) data was reviewed using the data supplied by Met Éireann for Cork Airport.

The SMD data indicates that the soils were saturated prior to the event with an SMD of 0mm, based on well drained soils.

The five day antecedent rainfall for the gauge at Cork Airport and Inniscarra is plotted on Figure 7.6 below. It is notable that, although the event rainfall at Cork Airport is higher than Inniscarra, the antecedent rainfall is lower and the respective five day antecedent precipitation index (API5) is 8.6 and 12.2mm.

Based on the above means of separating rainfall contributions and using the FSR techniques, the catchment wetness was estimated to be 132mm at the start of the event. A constant loss model was adopted and, using the same catchment characteristics that were applied earlier for peak flow estimation, the percent runoff for the event (PR_{RURAL}) was estimated to be 32.4% and 33.2% based on Cork Airport and Inniscarra gauges respectively.

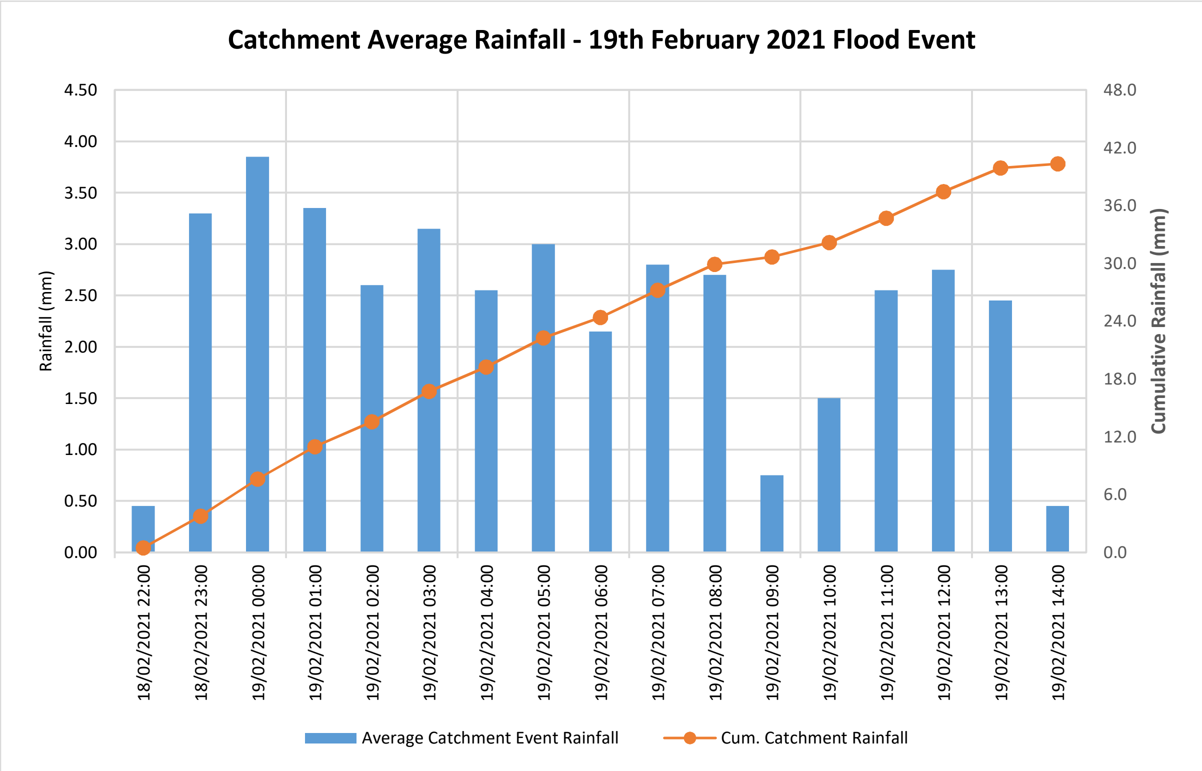


Figure 7.5: Catchment Average Rainfall

It was sought to improve this estimate using the gauged flow at Ballyedmond in conjunction with the Ballincurrig raingauge, the analysis of which indicated an adjustment factor of 1.16 would apply. Using this approach the percent runoff (PR_{RURAL}) for the Maglin catchment was estimated to be 38%. The cumulative net rainfall used in the convolution of the unit hydrograph will vary depending on the percent runoff (PR) which is adjusted at each HEP based on the urban area of the catchment.

The event baseflow at Ballyedmond was also reviewed and compared with the catchment descriptors baseflow from FSSR16. This indicated that a baseflow adjustment factor of 2.11 would be appropriate for the event.

Another consideration is the time to peak of the flood. It was estimated from the site walkover than the flood peak occurs sometime between 12pm and 2pm on 19th February. Based on the rainfall data available the catchment lag (LAG) would appear to be in the range of 6 to 8 hours with a corresponding IUH $T_p(0)$ of approximately 5 to 6.5 hours. $T_p(0)$ was estimated earlier at each HEP using stage records from 19036 and 19049 and it is notable that the value varies from 4.6 hours at MN06 to 5.7 hours at MN01. Therefore, it would appear that the shape of the unit hydrograph is broadly suited to this event.

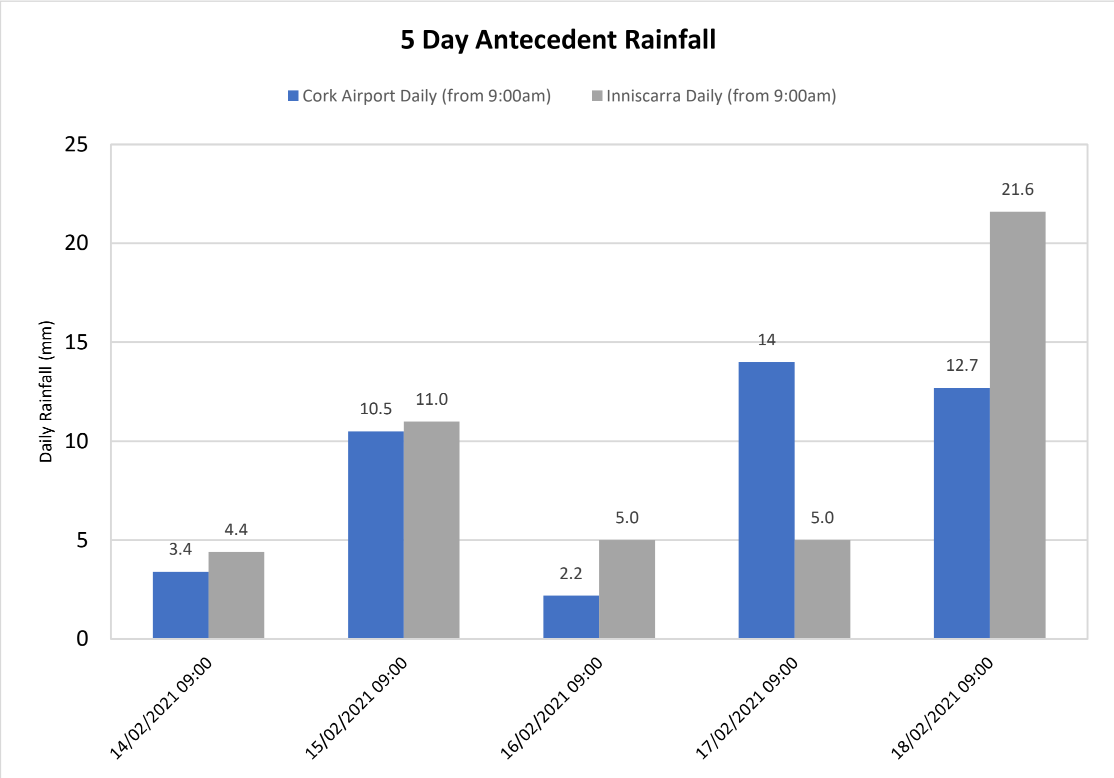


Figure 7.6: February 2021 Event – 5 Day Antecedent Rainfall

7.4.3 Hydrograph Simulation & Assessment of Return Period

The net rainfall was convoluted with the synthetic unit hydrograph for the catchment to produce the simulated flood hydrograph. Given the selected method for establishing design flows in the study area is with the FSU regression equation, the flows estimated for the flood event using the FSR approach were scaled to establish the final flow estimate for the event. The analysis indicated that the peak flow for the 19th February 2021 was slightly above Q_{MED} at all HEP's. Based on this, the scaling multipliers applied to Q_{MED} to establish the 'best estimate' peak flow at each HEP are provided on Table 7.1 below and the simulated hydrographs are shown on Figure 7.7 for HEP MN01 and MN06. It is therefore most likely that the return period of the flood was in the order to 2 to 3 years however given the potential spatial and temporal variations in rainfall and the limited hydrometric data available, there is some uncertainty associated with the estimate and the possibility of the flood return period being up to 5 years could not be ruled out.

HEP	River	Q_{MED} Multiplier	Best Estimate of Peak Flow (m^3/s)
CU01	Curraheen	1.05	13.46
CU02	Curraheen	1.05	12.64
CU03	Curraheen	1.05	5.05
MA01	Maglin Lower	1.00	5.82
MS01	Maglin South	1.00	2.94
MN01	Maglin North	1.07	4.13
MN03	Maglin North	1.07	3.67
MN04	Maglin North	1.02	2.96
MN05	Maglin North	1.02	2.45
MN06	Maglin North	1.02	2.38

Table 7.1: Peak Flows Estimated for February 2021 Event

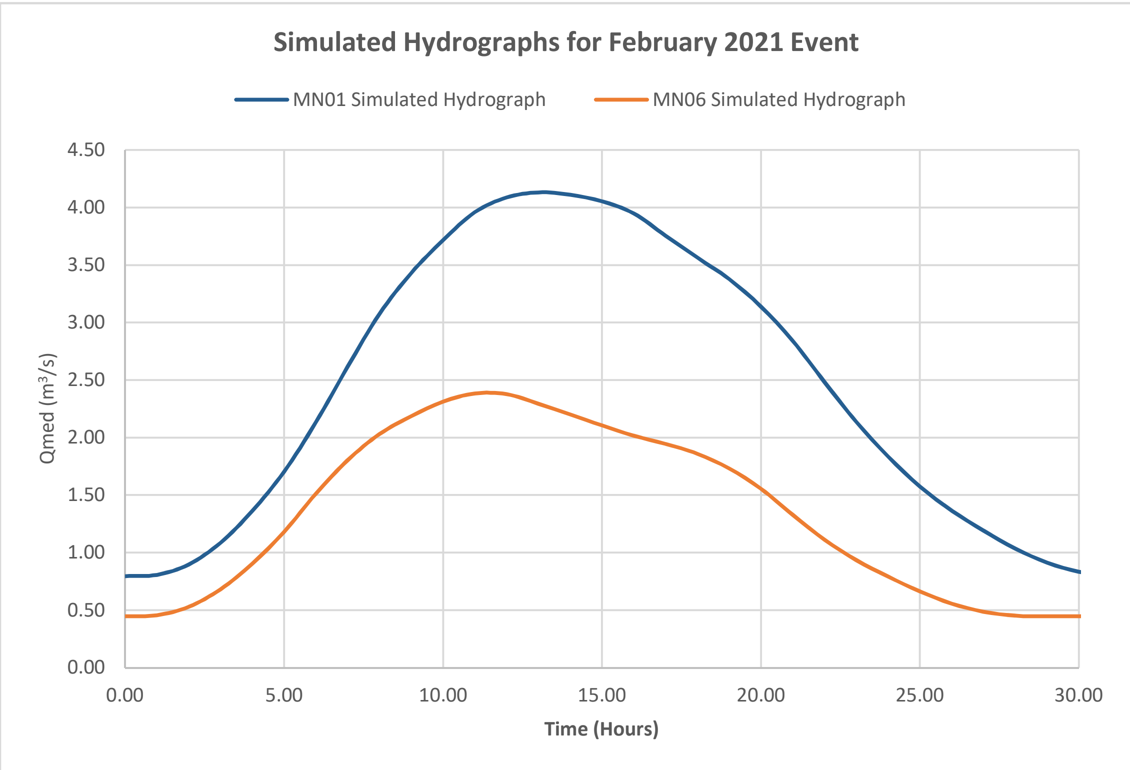


Figure 7.7: Simulated Hydrographs for February 2021 Flood Event

8. Fluvial Hydraulic Modelling

8.1 Hydraulic Modelling Approach & Overview

The hydraulic analysis was carried out using the Hydraulic Engineering Centre River Analysis System software which was developed by the US Army Corps of Engineers.

The initial model was developed as a combined 1D-2D hydraulic model using HEC-RAS Version 5.0.7. The model was updated following the release of HEC-RAS Version 6.1 in June 2021 and it was determined through various sensitivity checking that the results from a full 2D model were similar to the initial model and moreover the 2D model was found to provide better stability and robustness with less uncertainty associated with the efficiency of flow transfer between the 1D and 2D domains.

The final model includes approximately a 3.2km reach of the Maglin North River, 2.1km of Maglin South River and circa 1km of the Curraheen River as well as the artificial drainage channel running through the western portion of the lands.

The river channel was created using the 1-dimensional (1D) cross sections obtained from the OPW Lee CFRAMS survey data with a small number of supplementary cross sections surveyed by HD surveys for this SFRA. These 1D cross sections were used to create a digital terrain model DTM of the river channel to enable in-bank flows to be modelled using the hydrographic survey data since LIDAR data would not be suitable.

The remainder of the 2D model was developed using a DTM from LIDAR data. This has a total area of approximately 360 hectares and includes the lands within the study area, to the south of the N25 road as well as an extensive area downstream of the Maglin confluence with the Curraheen River. The DTM based on the 1D cross sections was then enforced onto the LIDAR DTM to provide a single terrain model for the entire area.

When selecting an appropriate computational mesh size for the model, cognisance was given to the approach HEC-RAS takes to represent the underlying terrain at each cell. Each cell can have between three and eight sides and each cell face consists effectively of a detailed cross section of the underlying terrain. This approach allows larger cells to be used without compromising accuracy, particularly when cells representing river channels are aligned in the direction of flow. Following various suitability checks, the 2D computational mesh adopted for the modelling generally incorporated a 10m x 10m cell size however this was refined to between 1 and 2m for the river and stream channels.

The model build focused on achieving maximum possible accuracy within the Plan Area. Although the model extends upstream and downstream of the Plan Area in order to adequately represent the boundary conditions, the flood extents and depths outside of the Plan Area have not been scrutinised and should not be used for strategic planning or flood risk assessment.

The model also includes 18no. existing bridge/culvert structures. The hydraulic model extents and schematic is included in Figure 8.1 below.

An unsteady flow analysis was performed using flow hydrographs which were derived during the hydrological analysis. The downstream boundary condition was set to match the water surface gradient at the downstream end of the reach.

Based on a walkover of the river reach and the model calibration discussed below, Manning's 'n' values were assigned based broadly on land use type and terrain. These are summarised on Figure 8.2 below.

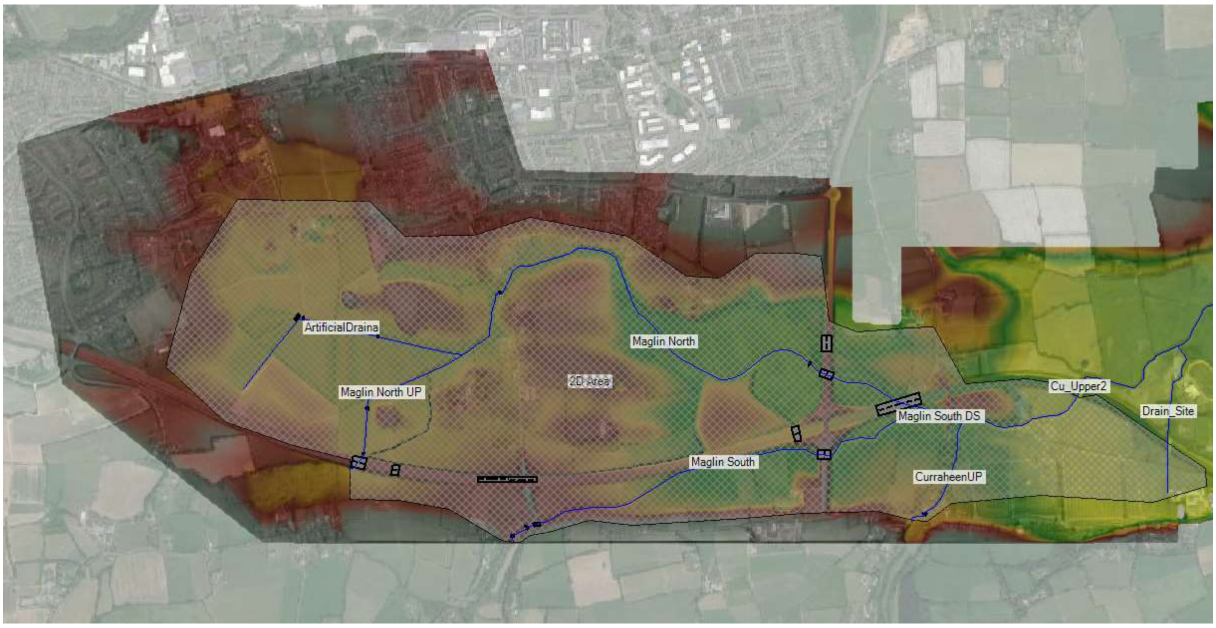


Figure 8.1: Hydraulic Model Schematic

Location	Manning's n
River Channel	0.055
Agricultural Lands	0.06
Urban Areas	0.15
Artificial Drain	0.07
Other Key Drainage Ditches	0.08

Figure 8.2: Manning's n Values

8.2 Model Calibration

The model was run for the February 2021 flood event using the scaled hydrographs which were derived during the hydrological analysis. Adjustments were made to the key input parameters, in particular channel roughness, until the performance was optimised insofar as possible.

The location of the control points where water levels were measured or estimated for this event are shown on Figure 8.3 and Figure 8.4 for the upper and lower reaches respectively. Table 8.1 includes a comparison of the observed and simulated flood levels at each control point. There are two categories of Control point based on the type of water level measurement which are as follows:

- Measurement Type A: Water level surveyed using a GPS at the time of the flood.
- Measurement Type B: Estimate obtained from surveying the observed extents or landmarks from photographs taken during the flood.

Overall the model appears to perform very well with 11 of the 17 control points being computed within 110mm of the observed flood levels. Any differences within this range may well be associated with inaccuracies in the estimated water levels for the event. The model overestimates the flood level by 140mm at Point No. 4 however this is offset by an underestimate at Point No. 5 which is located a short distance downstream. This indicates that there may be an anomaly with these recorded flood levels. The model performance is within 230mm at Control Points 16 and 17 which is acceptable given these points are outside of the site boundary and do not impact on the SFRA. It was found that the model under predicts the flood level at Control Point 9 by approximately 360mm. This is somewhat surprising given the model slightly over predicts upstream at Control Point 8 and downstream Control Point 11 (Maglin Road Bridge). Various possible causes of this anomaly were considered. Although there is some potential that it is due to a discrepancy in survey data or flow estimates, given the good overall performance of the model in this area, it appears more plausible that there was a partial blockage in the channel at some point upstream of Maglin Road Bridge during the flood event. The channel is lined with trees and other vegetation and could be susceptible to such blockages. Other potential causes include the possible contribution of pluvial flooding and, since this is a Type B estimate, possible inaccuracies in the estimated flood level at this location. Overall the model performance is considered to be acceptable and suitable for use in the SFRA.

Control Point ID	Measurement Type [1]	Observed Water Level (mOD)	Simulated Water Level (mOD)	Difference (m)	Comments
1	B	19.01	18.97	-0.04	Acceptable for SFRA
2	A	18.95	18.93	-0.02	Acceptable for SFRA
3	B	18.90	18.89	-0.01	Acceptable for SFRA.
4	B	18.57	18.71	0.14	Anomaly appears to exist between points 4 and 5. Acceptable for SFRA
5	B	18.71	18.60	-0.11	Anomaly appears to exist between points 4 and 5. Acceptable for SFRA
6	B	18.40	-	-	Area not flooded in simulated event.
7	B	18.50	18.56	0.06	Acceptable for SFRA.
8	B	18.40	18.51	0.11	Acceptable for SFRA.
9	B	18.28	17.92	-0.36	Under-estimate. See comments in main text.
10	B	17.95	-	-	Area not flooded in simulated event. Flood level in adjacent channel is 17.85m.
11	A	17.66	17.73	0.07	Acceptable for SFRA
12	A	17.55	17.66	0.11	Acceptable for SFRA
13	A	15.66	15.62	-0.04	Acceptable for SFRA
14	A	15.31	15.37	0.06	Acceptable for SFRA
15	A	15.29	15.36	0.07	Acceptable for SFRA
16	A	15.06	15.28	0.22	Point is outside of the site boundary and do not impact on SFRA
17	A	15.83	15.60	-0.23	Point is outside of the site boundary and do not impact on SFRA

[1] *Measurement Type A: Water level surveyed using a GPS at the time of the flood.*
Measurement Type B: Estimate obtained from surveying observed extents or landmarks from photographs taken during the flood.

Table 8.1: Model Calibration – Summary of Observed and Simulated Flood Levels for February 2021

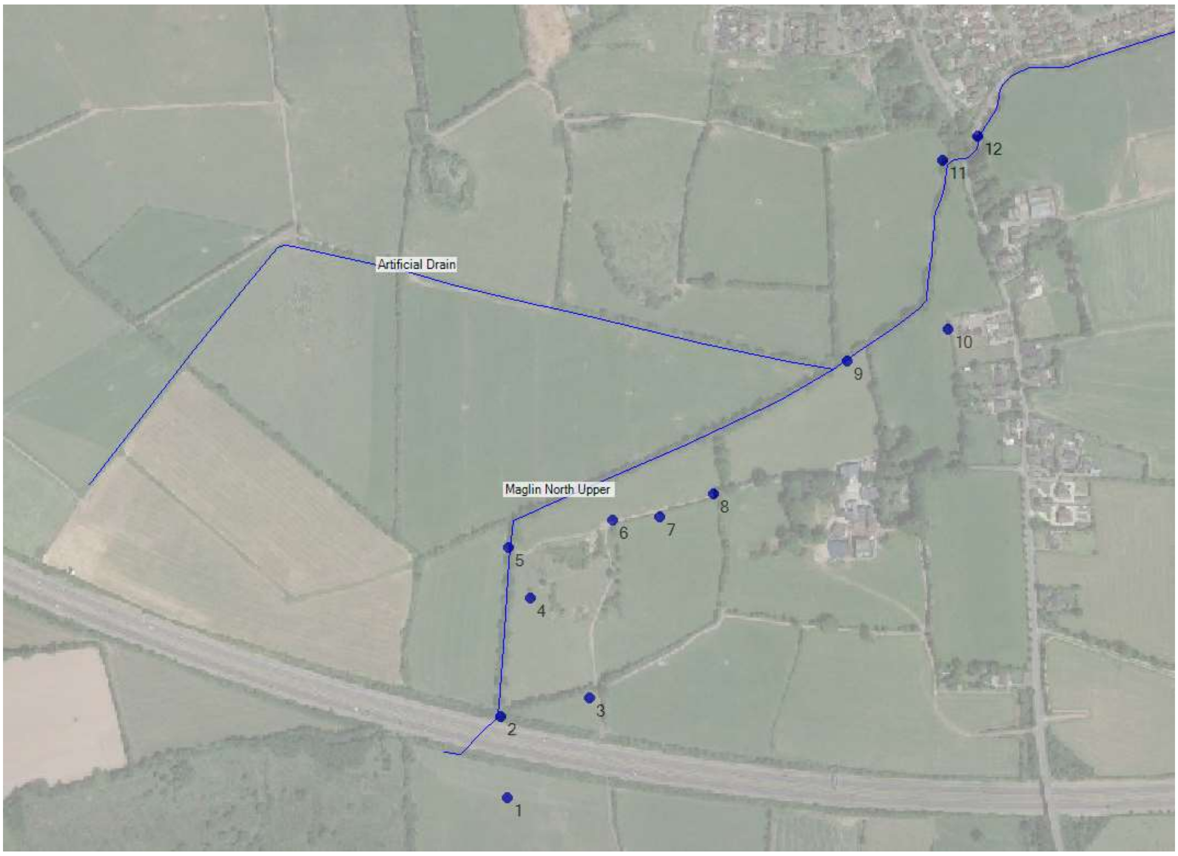


Figure 8.3: Location of Control Points for February 2021 Event (Upper Reaches)

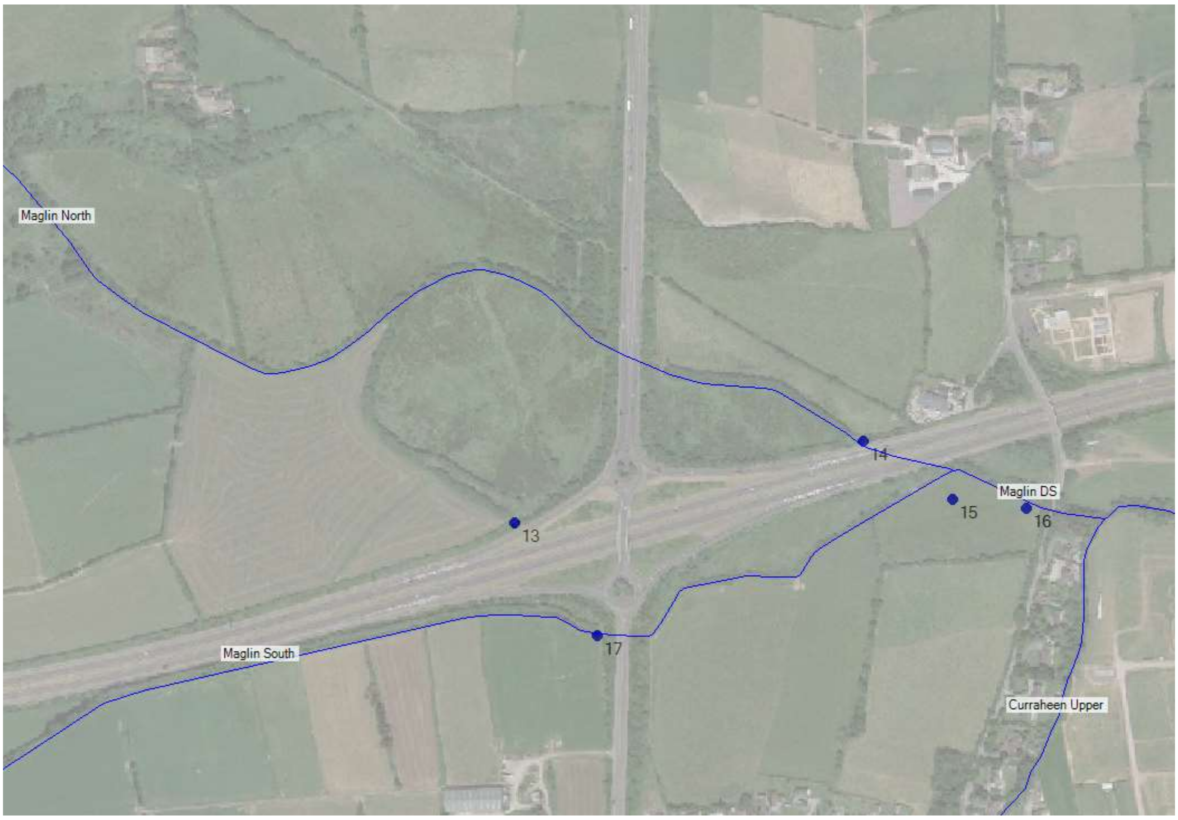


Figure 8.4: Location of Control Points for February 2021 Event (Lower Reaches)

8.3 Hydraulic Modelling Results

8.3.1 Baseline Situation

Following completion of calibration, the hydraulic model was run for the baseline situation (i.e. Pre-development) and the predicted flood extent maps are shown on

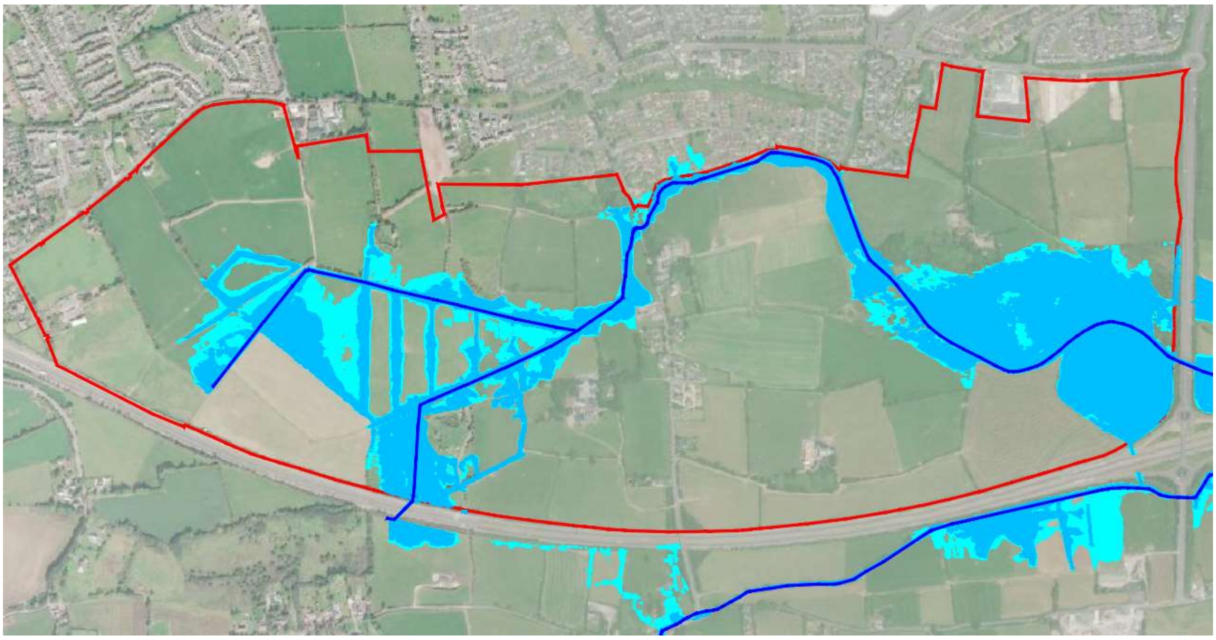


Figure 8.5 for 1% and 0.1% design events.

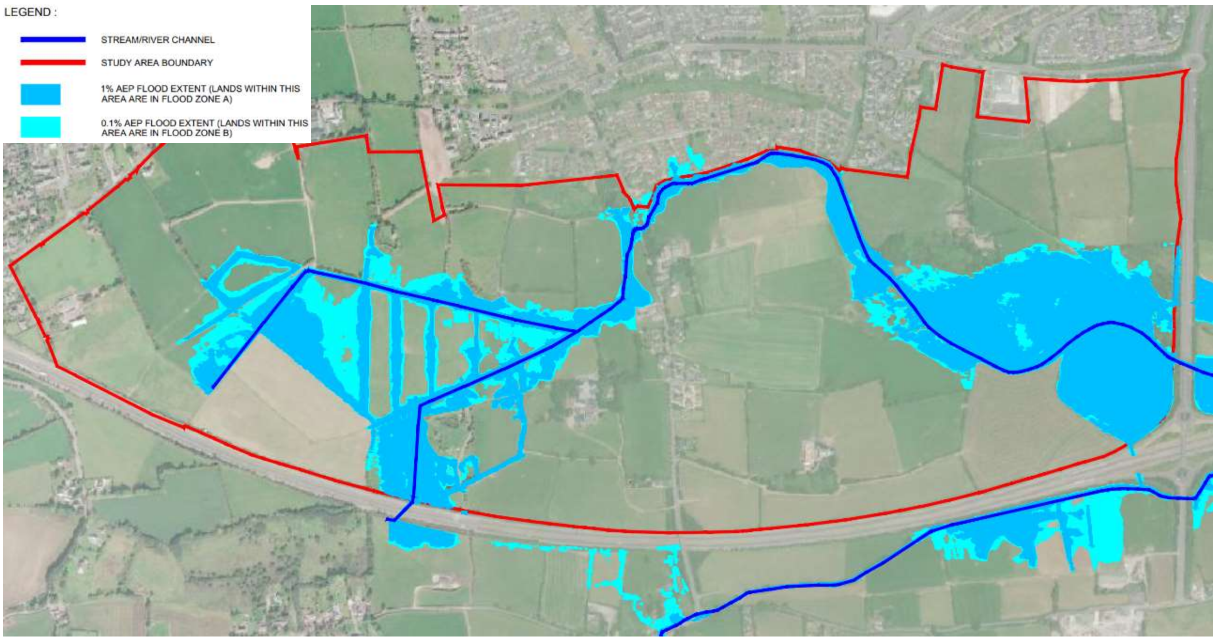


Figure 8.5: 1% AEP and 0.1% AEP Flood Extent Map (Pre-Development)

8.3.2 Comment on Lee CFRAM Study Predictions

The flood extents indicated on the Lee CFRAM Study Maps appear to be broadly consistent with those identified in this SFRA however at the upstream end of the Maglin North River and the Artificial Drain there are some specific differences. These are attributed mainly to a difference in the modelled extents in each hydraulic model. As indicated on Figure 8.6, it appears that the Lee CFRAM study hydraulic model did not include the upstream reaches of Maglin North or the Artificial Drain and this is expected to have a notable impact on modelled flood extents. The impact of this is highlighted by the fact that flooding in areas of the site was observed in February 2021 which were not identified as being at risk in the Lee CFRAMS.

The inclusion of 9no. additional bridge/culvert structures that were not included in the Lee CFRAMS model would also tend to increase flood levels due to hydraulic losses at each structure however the attenuating effect of the additional floodplains included in the SFRA model would have a tendency to reduce flood levels.

A comparison is made between the SFRA and the Lee CFRAM Study predictions on Table 8.2 for the 10% AEP, 1% AEP and 0.1% AEP flood events. It can be seen that the results from both models compare very well with predicted flood levels being generally within 50mm at all locations.

Lee CFRAM Study Node ID	10% AEP Flood Level - Lee CFRAM Study	10% AEP Flood Level - SFRA	1% AEP Flood Level - Lee CFRAM Study	1% AEP Flood Level - SFRA	0.1% AEP Flood Level - Lee CFRAM Study	0.1% AEP Flood Level - SFRA
8CUR_9599	18.49	18.53	18.69	18.70	18.78	18.77
8CUR9058	18.09	18.04	18.33	18.29	18.51	18.50
8CU3_729	18.24	18.34	18.46	18.49	18.58	18.65
8CU1_1083	16.76	16.79	16.90	16.97	17.03	17.11

Table 8.2: Comparison of SFRA and Lee CFRAM Study Predicted Flood Levels

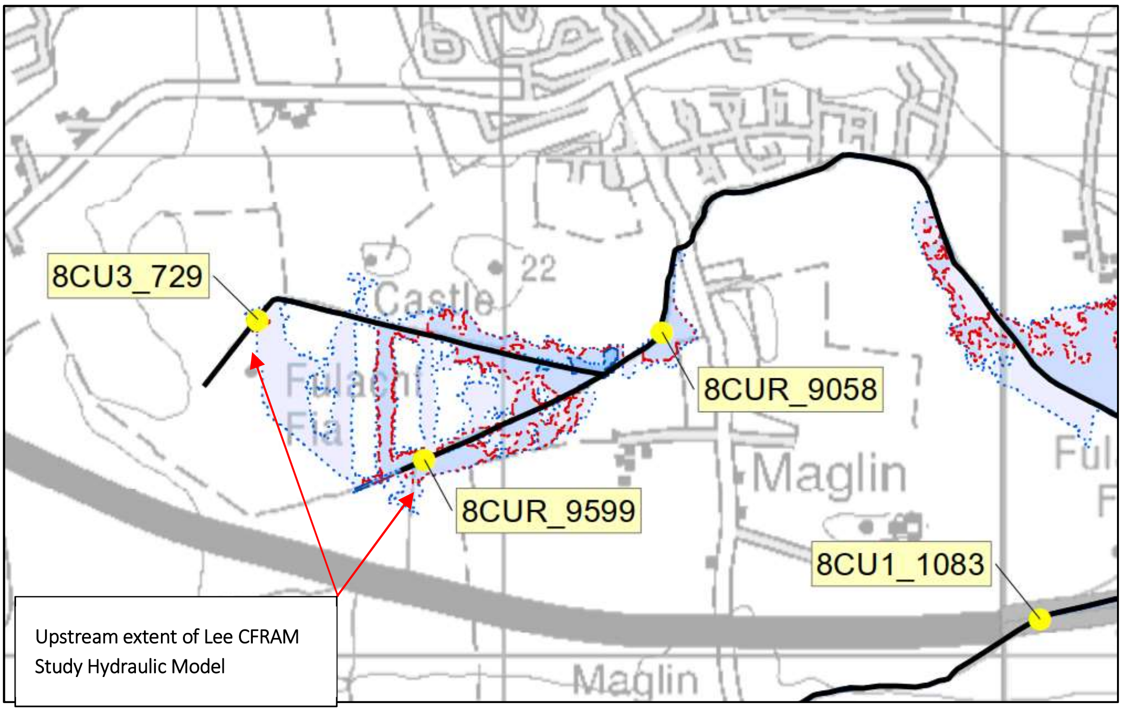


Figure 8.6: Lee CFRAM Study Hydraulic Model Extent

8.3.3 Post Development Situation

The post-development situation includes for the proposed new spine road which will cross the river and floodplain at a number of locations. It is assumed for the purpose of this SFRA that no other alterations will be made to the river or floodplains. The design of the spine road is yet to be completed therefore the analysis was carried out based on an indicative alignment which is expected to be broadly consistent with the final position. This approach is acceptable in the context of a SFRA for development plan zoning however at design stage, the possible implications of any deviations from the assumed alignment should be checked.

An indicative road alignment and footprint was agreed with Cork City Council to enable the SFRA to be completed. The hydraulic model was then adjusted to include the new road alignment. This involved adjusting the DTM to include a 40m wide embankment along the line of the road and the provision of culverts at all locations where the road crosses the river channel. The dimensions of the culverts were adjusted iteratively until an acceptable solution was achieved, taking cognisance of anticipated OPW Section 50 requirements. The revised schematic of the model is indicated on Figure 8.7 below.

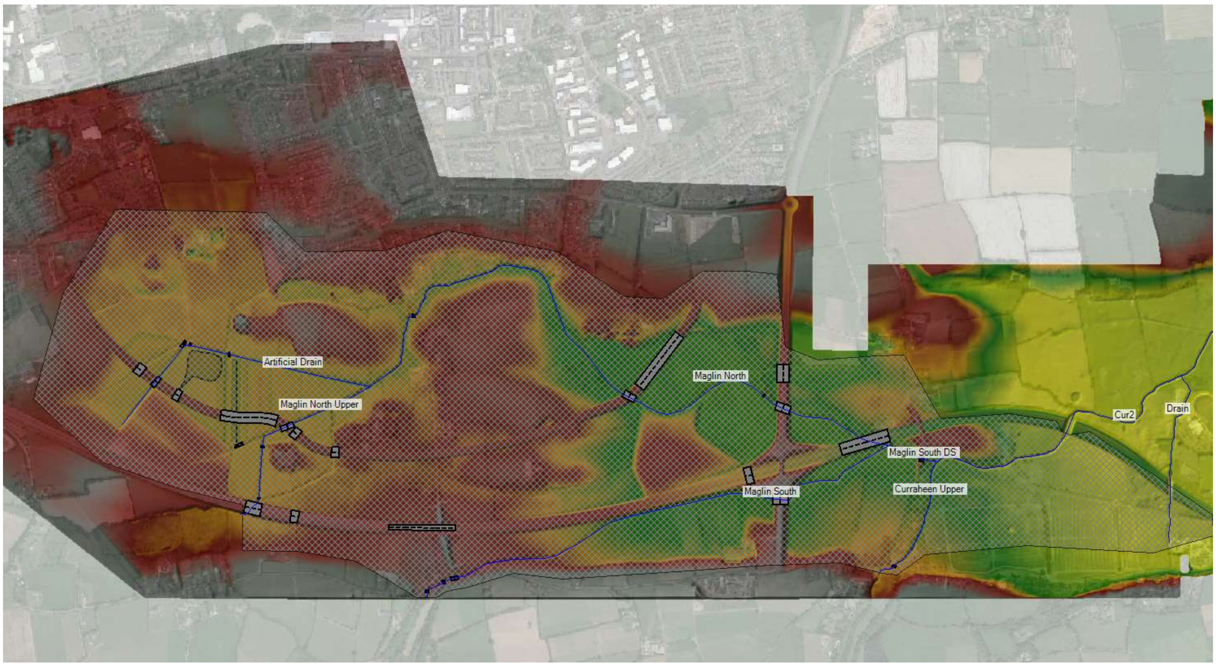


Figure 8.7: Hydraulic Model Schematic – with Proposed Spine Road

For the proposed situation with the spine road in place, the key criteria for an acceptable solution are:

1. Ensure the flood risk upstream and downstream of the site is not adversely impacted.
2. Ensure the flood risk within the site remains at an acceptable level.
3. Ensure the head loss caused by the introduction of new culverts/bridges is within acceptable limits.

The above criteria should be considered in the context of the Planning System and Flood Risk Management Guidelines which state that *"Flood risk to, and arising from, new development should be managed through location, layout and design incorporating Sustainable Drainage Systems and compensation for any loss of floodplain as a precautionary response to the potential incremental impacts in the catchment.*

For the purpose of the SFRA, the following possible solutions were examined.

8.3.3.1 Option 1 – No Mitigation

In order to test the potential impact of developing the spine road without any mitigation the model was first developed to include only the spine road embankment and three culverts at the crossing points of the Artificial Drain and Maglin North River. This initial model was run for the 0.1 % AEP event and the following key impacts were noted:

1. Flood levels upstream of the road embankment increase by over 300mm;
2. Flood levels upstream of the site increase slightly, in the order of 10mm;
3. Flood levels downstream of the site increase slightly, in the order of 20mm;
4. Flood levels along Maglin North River adjacent to Glincool and upstream to the confluence with the Artificial Drain increase by circa 20mm.

The above increases in flood levels are caused by a reduction in floodplain storage and changes to the flow regime through the site, such as blocking existing floodplain flow paths. Although the increases outside of the Plan Area are minimal, the Planning System and Flood Risk Management Guidelines require that a precautionary response to development on floodplains is taken in response to the potential incremental impacts in the catchment. This is considered relevant in the context of this area since there are existing developments downstream which are at risk of flooding and it is anticipated that further development of lands within this catchment may occur in the future. Therefore, this option is not considered appropriate in the context of the Guidelines.

8.3.3.2 Option 2 – Additional Culverts

The second option considered included the addition of numerous flood relief and balancing culverts through the proposed road embankment so that existing flow paths are maintained insofar as possible and to reduce hydraulic losses upstream of the road.

The results of the model for the 0.1% AEP event indicate that the flood level increase upstream of the road embankment would be much lower than Option 1 however it was found that the increased conveyance combined with the reduced floodplain storage tended to increase flood levels and peak flow further downstream. Although these increases are considered to be relatively small, the analysis of this option indicates that compensation storage is warranted in line with the Guidelines.

8.3.3.3 Option 3 – Flood Relief Culverts & Compensation Storage

The third option considered involved the provision of compensation storage in the Public Open Space on the western portion of the site together with a series of flood relief and balancing culverts through the embankment. A number of iterations of this option were examined. The final iteration included fewer flood relief culverts on the embankment crossing the eastern/downstream floodplain of Maglin North River so that additional storage is provided upstream of the embankment.

The concept layout of the proposed spine road, culvert system and compensation storage areas is shown on Figure 8.8 below.

The proposed additional storage areas proposed for this solution are predominantly indirect methods of compensation provision and accordingly require the full range of AEP events to be analysed to confirm the required spill levels and flow transfer methods. A range of events have been tested in this SFRA and the results were compared to the baseline situation. The analysis confirmed that this approach is the most effective combination of measures in terms of maintaining existing flow paths, minimising flood risk increases within the plan area and ensuring flood risk outside of the site boundary is not increased.

Notwithstanding this, the solution does result in an increase in flood levels and extents immediately upstream of the eastern crossing. This is a localised increase and it does impact on land or properties outside of the Plan Area. This localised increase within the Plan Area is justified because the solution would help to ensure the development does not have an adverse impact on flood risk downstream of the site. Figure 8.9 below shows the extent of lands affected by these increases for the 0.1% AEP Current event and Figure 8.10 shows the corresponding flood level increase.

With reference to the Output Node Locations shown on these figures,Figure 8.11 the water level difference between the existing and proposed situation is summarised as follows:

- Node 7: - 0.003m
- Node 8: + 0.02m
- Node 9: + 0.11m
- Node 10: + 0.02m
- Node 18: - 0.01m
- Node 20: + 0.16m

Importantly, these water level differences highlight that the increase is limited to a local area upstream of the embankment and that there would be a slight reduction in flood levels further upstream (i.e. at Node 7) and downstream (i.e. at Node 18).

The analysis carried out for this SFRA is sufficient to demonstrate that the road construction is acceptable in the context of flood risk and that suitable mitigation can be incorporated into the development design. It may however be possible to refine the solution or develop alternatives in conjunction with the road corridor design. In any case, the final details of the measures as well as the potential impact of any changes to the assumed road alignment should be confirmed in the design stage flood risk assessment.

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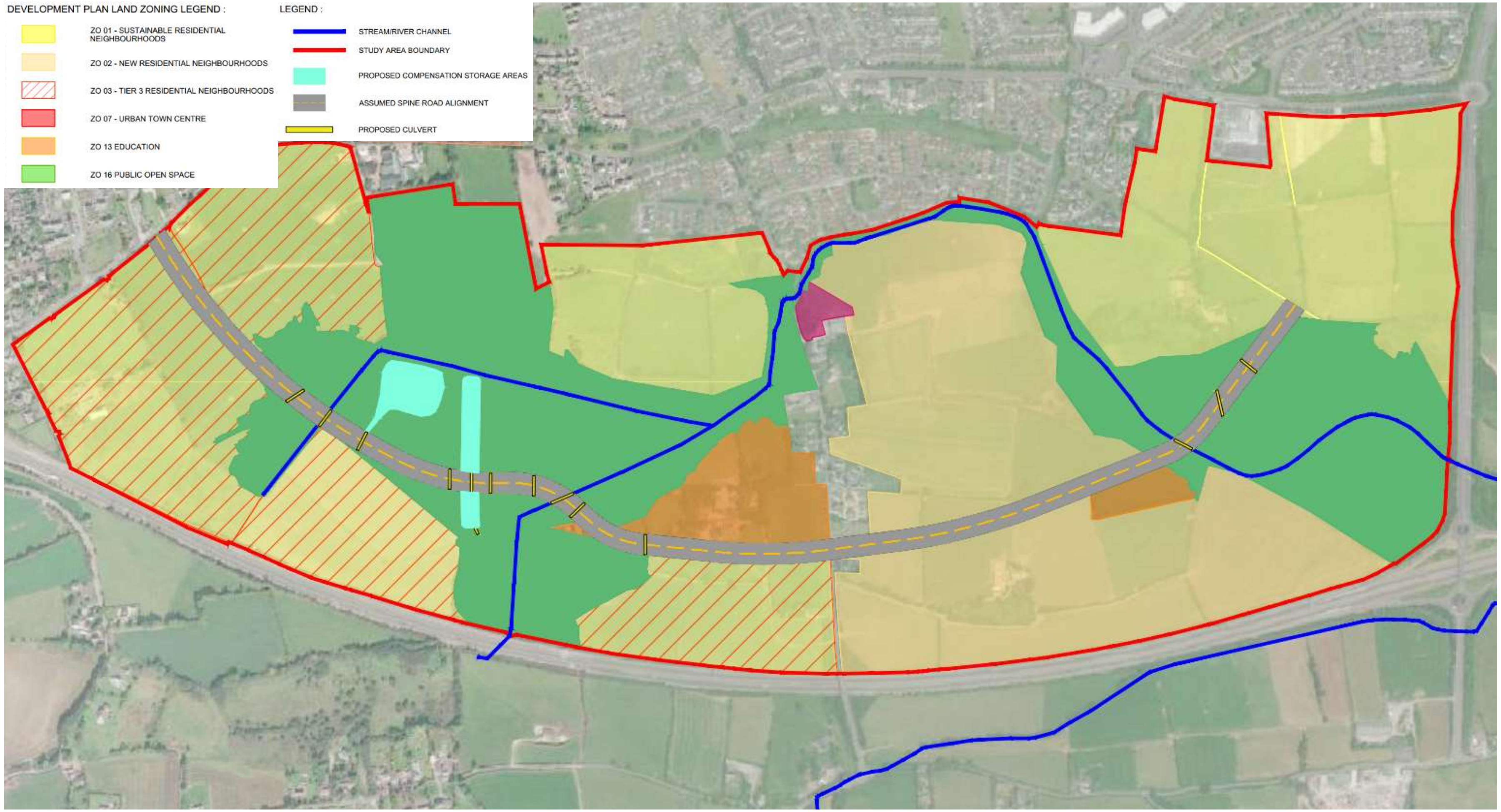

Figure 8.8: Proposed Spine Road Alignment, Culverts and Compensation Storage Areas

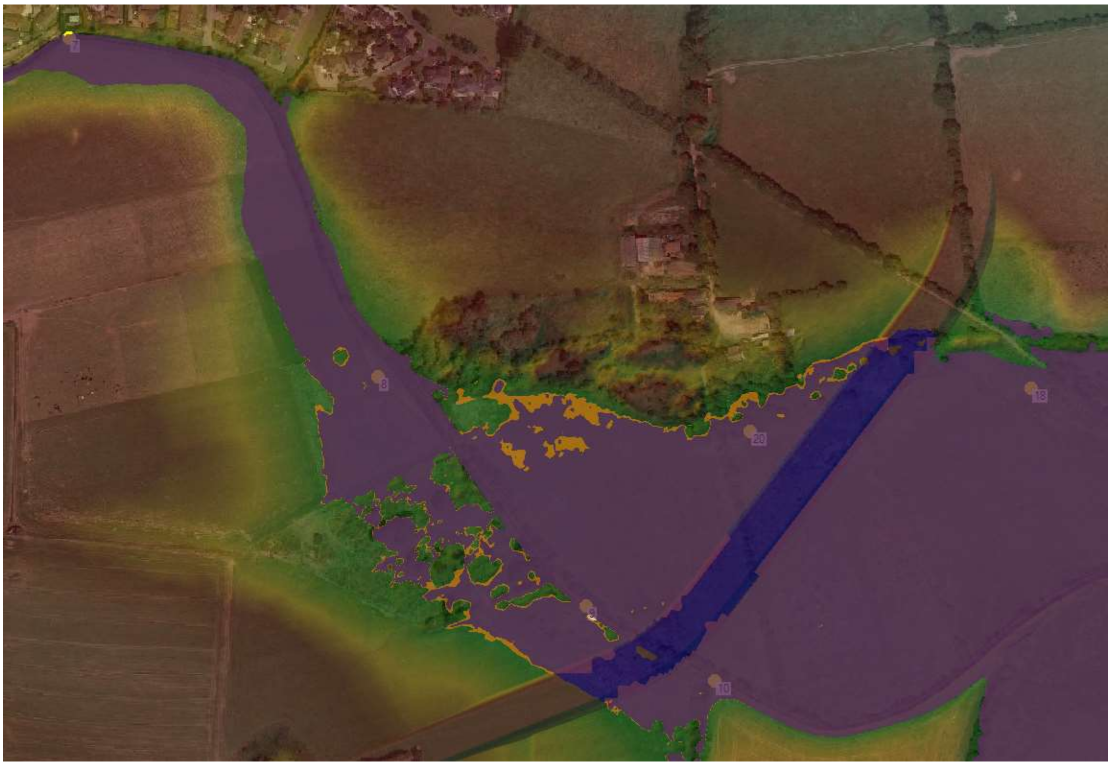

Figure 8.9: Flood Extent Increase Upstream of Eastern Spine Road Embankment – 0.1% AEP Current Event (Regions shown in orange denoted increased extents)

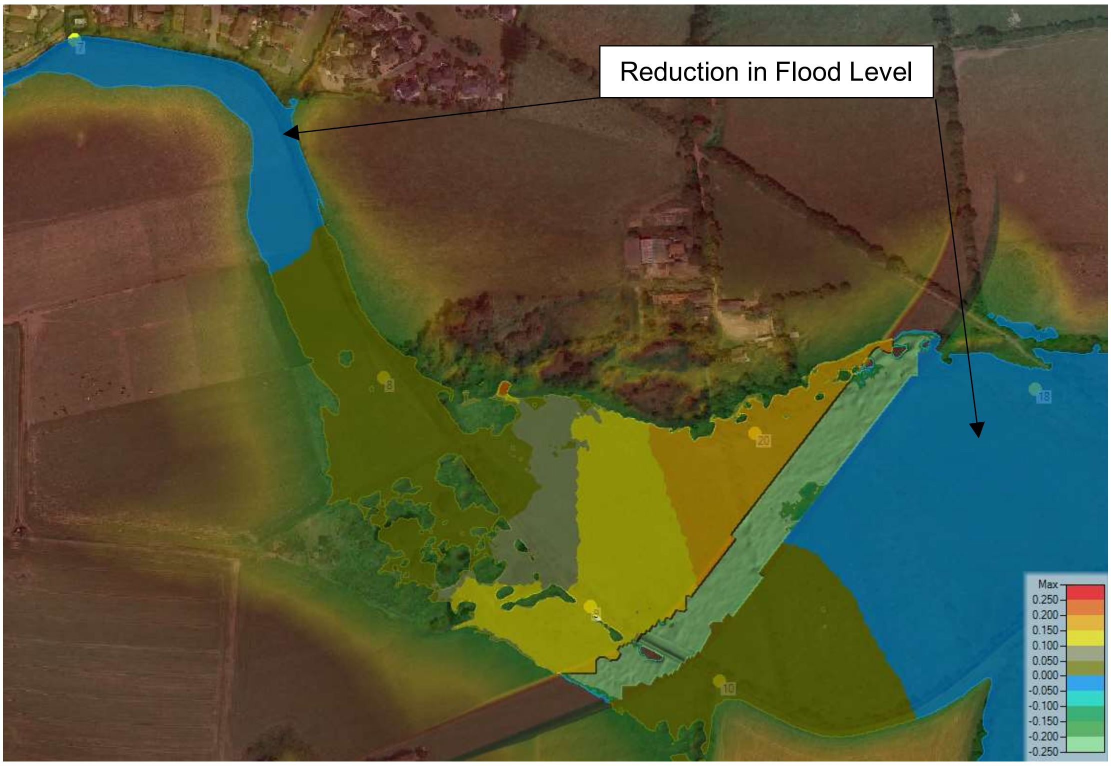

Figure 8.10: Flood Level Increase Upstream of Eastern Spine Road Embankment – 0.1% AEP Current Event

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8.3.4 Summary of Predicted Flood Levels

The predicted flood levels at specific output node locations in the Plan Area are tabulated on Table 8.3 for various events. The location of the output nodes is includes on Figure 8.11 and Figure 8.12.

Predicted Flood Levels at Node Points (Post-Development)							
Node ID	1% AEP Current	0.1% AEP Current	1% AEP MRFS	0.1% AEP MRFS	1% AEP HEFS	0.1% AEP HEFS	1% AEP MRFS (95% Confidence)
0	19.08	19.16	19.12	19.19	19.14	19.22	19.18
1	18.83	18.90	18.87	18.93	18.88	18.95	18.91
2	18.63	18.74	18.69	18.8	18.71	18.84	18.76
3	18.42	18.57	18.5	18.67	18.53	18.73	18.62
4	18.27	18.50	18.39	18.61	18.44	18.67	18.55
5	18.16	18.42	18.3	18.54	18.36	18.6	18.48
6	18	18.16	18.08	18.26	18.12	18.3	18.19
7	17.32	17.42	17.37	18.49	17.39	17.52	17.44
8	16.58	16.70	16.64	16.79	16.67	16.82	16.73
9	16.27	16.48	16.37	16.68	16.42	16.7	16.55
10	16.15	16.35	16.24	16.56	16.29	16.59	16.43
11	15.95	16.29	16.11	16.54	16.20	16.56	16.39
12	18.6	18.71	18.65	18.79	18.68	18.84	18.74
13	18.49	18.65	18.57	18.76	18.61	18.8	18.7
14	18.43	18.63	18.54	18.74	18.59	18.79	18.68
15	18.34	18.55	18.45	18.66	18.50	18.72	18.6
16	18.46	18.65	18.56	18.75	18.60	18.8	18.69
17	-	18.65	18.56	18.75	18.61	18.8	18.69
18	15.97	16.30	16.12	16.54	16.21	16.56	16.39
19	15.96	16.29	16.12	16.54	16.21	16.56	16.39
20	16.27	16.48	16.37	16.68	16.42	16.71	16.56

Table 8.3: Predicted Flood Levels

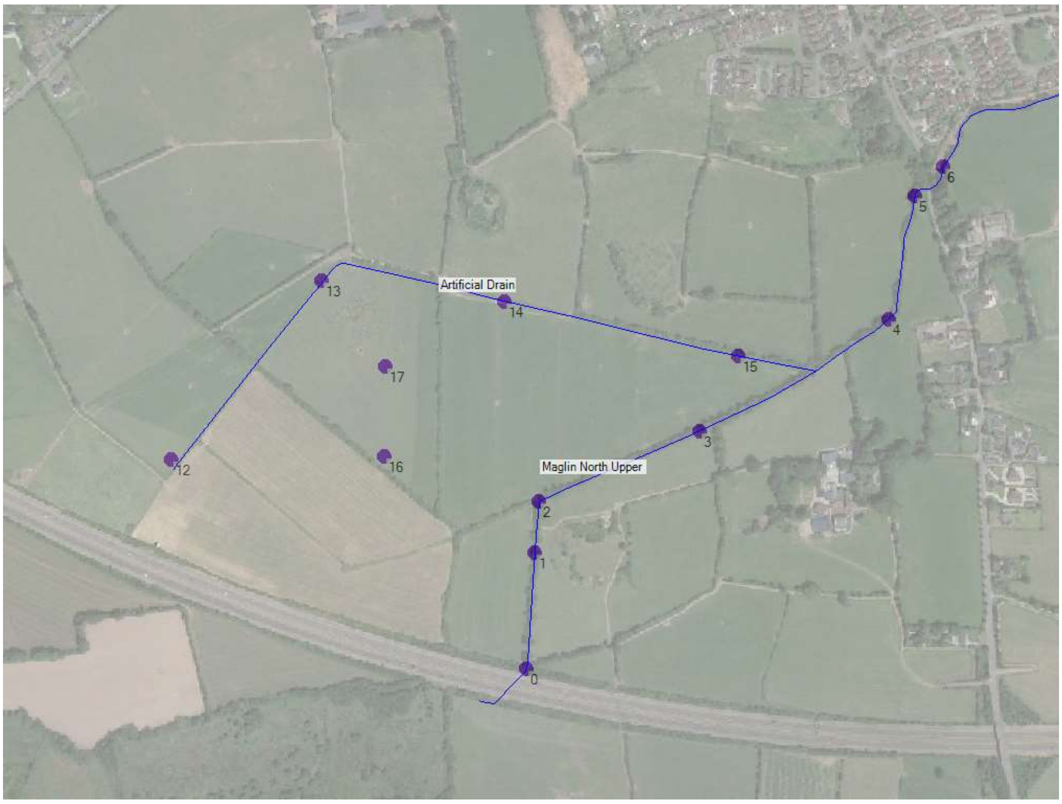


Figure 8.11: Output Node Locations (Upstream Reaches)

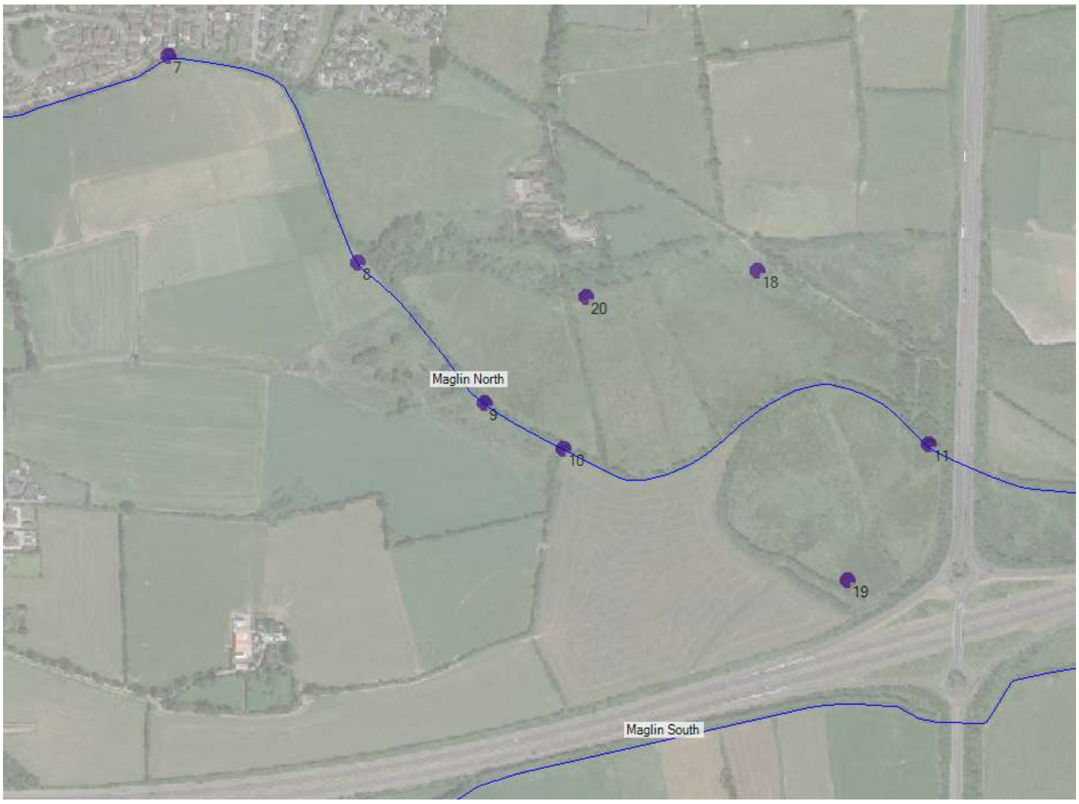


Figure 8.12: Output Node Locations (Downstream Reaches)

9. Pluvial Flood Risk

In order to identify locations within the Plan Area that may be at risk of pluvial flooding/surface water ponding, a 2D overland flow model was created using a DTM from LIDAR data. A distributed rainfall analysis was carried out for 1% AEP rainfall events with various storm durations. Rainfall depths were determined from the FSU rainfall DDF module and storm profiles were derived assuming a 75% winter storm based on the FSR RR model.

The results of the analysis for a 3 hour and 9 hour storm duration are shown on Figure 9.1 and Figure 9.2 respectively. It is important to note that the DTM is based predominantly on LIDAR data therefore local drainage features that could either impede or more likely assist with surface water drainage from depressions may not be adequately defined. Consequently the results of the analysis should be considered as indicative of areas that may be prone to flooding and further assessment of these areas should be undertaken at a site specific scale.

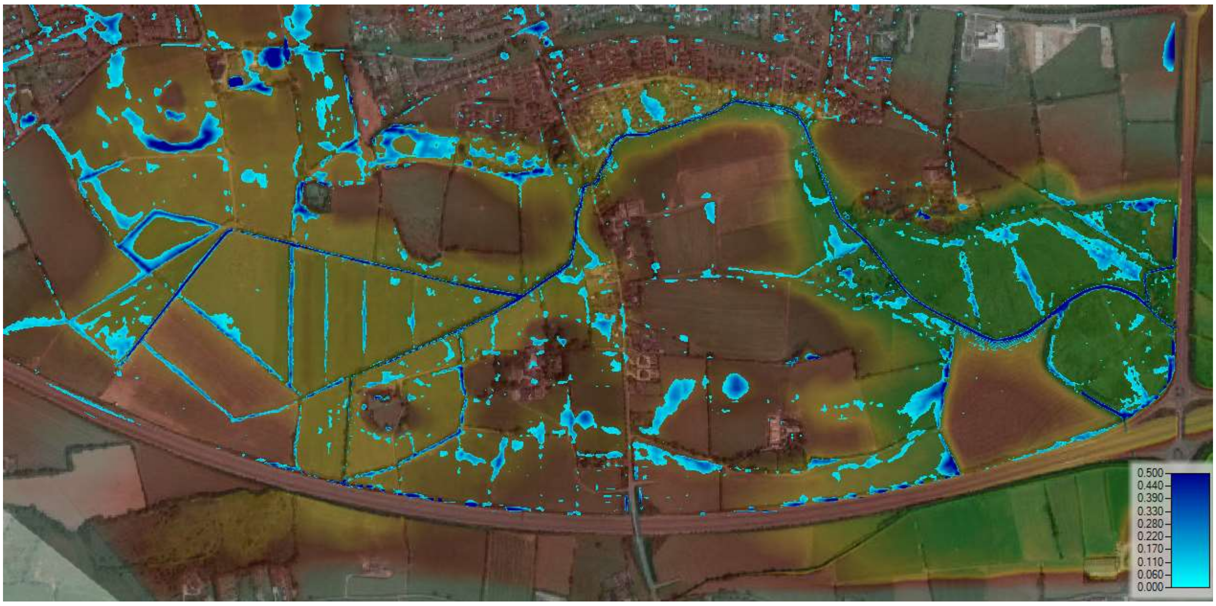


Figure 9.1: Indicative Pluvial Flood Risk Map for 1% AEP Event – 3 Hour Storm Duration

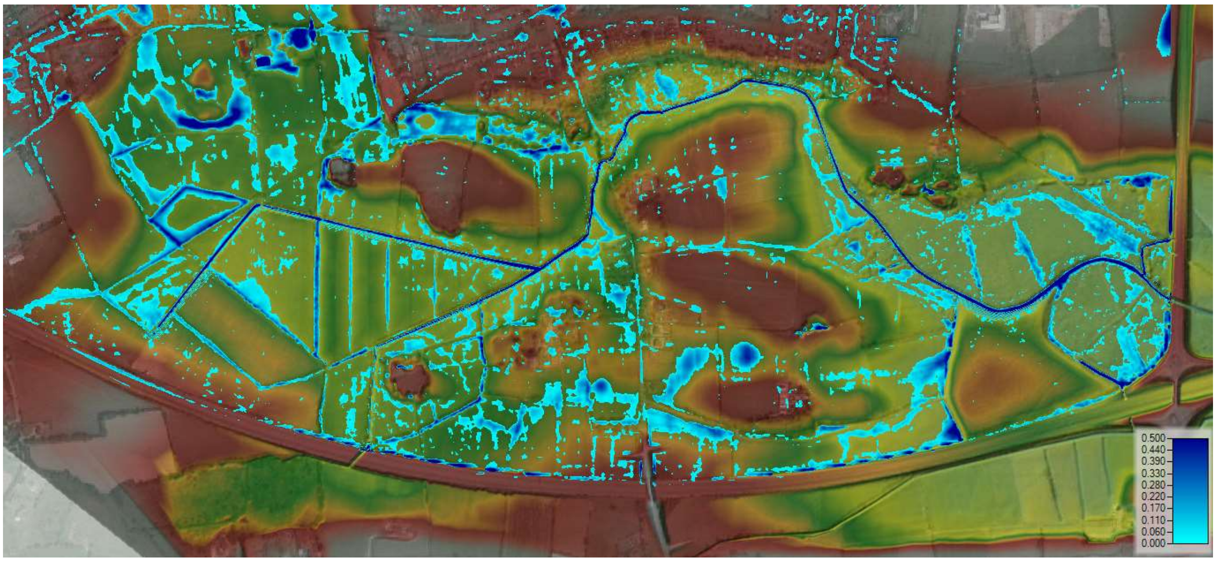


Figure 9.2: Indicative Pluvial Flood Risk Map for 1% AEP Event – 9 Hour Storm Duration

10. Groundwater Flood Risk

10.1 Introduction

In order to provide a qualitative assessment of groundwater flood risk based on existing information, a desk study has been carried out using available data.

The groundwater flood risk assessment was carried out as follows:

1. Complete a desk study to review historical mapping, topography, geology, hydrogeology and hydrology, as summarised in Section 2 of this report;
2. Assess the Hydrogeological Setting using previous ground investigation data and hydrogeological studies, as summarised in Section 10.2 below;
3. Prepare a Conceptual Site Model to understand potential groundwater flows and flooding mechanisms, as summarised in Section 10.3 below.

10.2 Hydrogeological Setting

10.2.1 Previous Ground Investigation Studies

Previous ground investigations within and in close proximity to the subject lands have produced detailed geological borehole and trial pit logs that provide information on the current bedrock, quaternary sediments and soils.

10.2.1.1 Ballincollig By-Pass Preliminary Site Investigation, 1996

The "Ballincollig By-Pass Preliminary Site Investigation", published by GEOTECH Specialists Ltd. in 1996, was produced for the N22 by-pass scheme that borders the south-eastern border of the site and several points within the subject lands. Nineteen shell & auger boreholes, sixteen rotary core borehole, and thirteen trial-pits were drilled/excavated. It should be noted that rotary core logs did not record coordinates and therefore do not feature on the CSM map below (Figure 10.2). Several geological cross-sections were provided which are summarised on Table 10.1.

10.2.1.2 Proposed Leisure Centre at Ballincollig, 2005

The "Proposed Leisure Centre at Ballincollig" GIR, published by Niall Fitzsimons & Co. Consulting Engineers in 2005, was produced for a site 400m north of the eastern extent of the subject lands. Two boreholes were drilled to a depth of 8.00m bgl and were composed of "MADE GROUND comprising of brown sandy gravelly CLAY with hard core and concrete throughout. Soft to Firm brown mottled purple slightly sandy slightly gravelly CLAY with cobbles, becoming stiff to very stiff at depth". Two sediment samples were taken at 2.00m and 5.00m depth at each borehole. Particle size distributions for these samples found that the material was very poorly sorted. Bedrock was not encountered.

Location	Chainage	BS5930 Description
Poulavane Link Road – Poulavane roundabout to Curraheen	0-1300m	Firm gravelly CLAY with some SAND, many cobbles and some boulders overlying LIMESTONE bedrock found at depths between 2 – 6.50m bgl (Core 1, 2, 16). Bedrock was described as moderately weak to moderately strong, slightly weathered to fresh, moderately cosed to very closed spaced, occasionally extremely closed clay-smeared to filled discontinuities. Trial pits TP2 found Soft to Firm gravelly sandy CLAY wth some cobbles overlying Stiff CLAY.
	1300-2100m	South, within the Maglin valley, found Soft sandy CLAY & SILT with some peat (BH10) and decayed roots (TP3). Beside the Maglin river found Firm to Stiff gravelly sandy CLAY. Trial pits found Firm to Stiff gravelly clayey SILT with boulders (BHR12). Further south, in TP3, Loose silty SAND & GRAVEL, increasing to Medium-dense with depth, unstable excavation.
	1400-1550m; 1900-1940m	Described as very soft/marshy ground.
Main Route – Bishopstown Roundabout to Curraheen	0-3500m	Rock was hit at 9.50m bgl in BH3 @ 260m chainage, underlaying till. The rock dips to 15 m and 15.50 bgl towards the east in BH core 5 and 6. Bedrock was described as LIMESTONE moderately strong, occasionally moderately weak, slightly weathered to fresh, close to very close, occasionally extremely close spaced clay filled and smeared discontinuities. Trial pits (TP4) found Firm boulder CLAY with boulders and a layer of very sandy GRAVEL at 4.40m bgl.
	1600-1900m; 3400m	Soft CLAY overlays Medium-dense to Dense silty sandy GRAVEL, and Firm to Stiff very gravelly SILT.
Main Northern Route – Curraheen to Barnagore (Ovens)	3500m	BH 13 and 14 found Soft to Firm silty/sandy CLAY overlaying Firm to Stiff boulder CLAY with some sands and gravels at 4-6m bgl depth. Trial pits 11 and 12 recorded Firm to Stiff boulder CLAY over large LIMESTONE boulder/ broken bedrock at 1.60m bgl. TP 13 and 14 found Medium-dense silty GRAVEL at 1.2 – 1.4m bgl under Firm CLAY. Bedrock was encountered at 8m bgl at Core 12 and dips to 20m bgl at Core 13. Bedrock was described as moderately weak to moderately strong, slightly weathered to fresh, moderate to very closely spaced discontinuities. At Core 13, there is possibility of irregular/undulating rock was found 17.50m bgl due to the cavitied nature of limestone and the proximity to an infilled cavity. BH16R found Dense GRAVEL at 2.24m bgl.
	3500-4000m	Groundwater was struck betweek 1.50 – 4.62m
	5600m	Strong groundwater flow noted in the gravels.
Main Southern Route – Curraheen to Greenfield Bridge	2950-6100m	Soft peaty CLAY over slightly clayey GRAVEL (BH1R), and Loose to Medium-dense very silty SAND, into Medium-dense/Dense sandy GRAVEL below 4.0-4.5m (BH3, BH7). BH8 & 11 close to the Maglin River found Firm CLAY and Dense GRAVEL with some sandy lenses. Bedrock was described as weak weathered to moderately strong LIMESTONE at 13.50m bgl at Core 15. It was very broken and fractured, weak to moderately weak SHALE, indicating the fault line. No bedrock recoded at 20m bgl @ 5460m chainage. Groundwater was struck between 1.20m and 1.90m from BH8R to BH6 & 7.

Table 10.1: Geological Description of GEOTECH Boreholes

10.2.1.3 Residential development, Heathfield, Carriganarra, Ballincollig, 2018

The "Residential development, Heathfield, Carriganarra, Ballincollig" Site investigation and interpretative report, published by Priority Geotechnical in 2018, was produced for the site which is located within the most north-eastern corner of the subject land boundary. Four rotary core boreholes and one cable percussion borehole were drilled in May 2018. The rotary cores reached depths between 11.8 – 16.4m bgl. The cable percussion borehole reached a depth of 3.6m bgl due to an obstruction. Three bulk sediment samples and rock core samples extracted from drilling were taken for further parameter testing.

Overall, ground conditions were described as Soft slightly gravelly SILT with low Cobble content present to depths up to 2.0m bgl. Below this was Firm slightly sandy slightly gravelly SILT with low Cobble content encountered up to 3.6m bgl. Rotary drilling encountered firm slightly sandy slightly gravelly SILT with low Cobble content to a depth of 4.1m bgl becoming stiff below this depth. These deposits are assumed present to depths above the LIMESTONE bedrock; encountered at 7.35m bgl to 13.3m bgl. The Limestone bedrock was described as "moderately strong to strong, moderately weathered, but non-intact in places especially in the parts owing to lower RQD% values. Three fracture sets were identified during the investigation, the first dipping 0-10° described as planar smooth with close to medium spacing, the second dipping 30-50° or 40-60° described as undulate smooth to rough, close to medium spacing, and the third dipping sub-vertical, wide spacing, undulated rough.

Ground water was assessed at a depth 9.0m bgl (seasonal variation may occur). Particular risks were identified as soft deposits and potentially karst within the Limestone bedrock.

10.2.2 Previous Hydrogeological Studies

10.2.2.1 Ballincollig Groundwater Body: Summary of Initial Characterisation, 2004

The following information was taken from the report which may aid in the research and conceptual site modelling of groundwater flooding in South Ballincollig:

- This GWB occupies the floor of an elongate east west trending valley extending west from Cork city. The body is generally flat to gently undulating (5-60 m OD), generally < 30 m OD in the east, 40-60 m OD west of Ovens. The River Lee joins the valley just west of Ballincollig and flows eastwards along the northern edge of the body, and its tributary the River Bride flows eastwards across the western half of the body.
- The GWB is bounded to the east by the coast and Lough Mahon. To the south the boundary is the contact with the low permeability sandstones and mudstones of the Ballinhassig GWB. Along the northern side of the body, the contact between the pure unbedded limestones and the underlying mudstones and sandstones (as shown on the 1:100,000 GSI Bedrock map) occurs 400-800 m into the valley floor, beneath alluvium and deep sand and gravel deposits. The northern boundary of the body extends to the edge of the valley floor, thus some areas of low permeability mudstones and sandstones along the northern margin of the body are also included within the body.
- The GWB is composed mainly of diffusely karstified, highly permeable pure limestones. To the north and south of the body are ridges of low permeability sandstones and mudstones. Overlying the bedrock in the GWB are glacial sand and gravel deposits. These vary in depth across the Lee and Bride Valley areas from several metres to approximately 40 m thick. The sand and gravel deposits provide a permeable pathway for recharge to the karstic aquifer and where saturated provide additional storage for the underlying bedrock aquifer. Along the northern boundary of the body these deposits overly the mapped contact between the pure unbedded limestones and the underlying mudstones and sandstones. The mudstones and sandstones that occur along the northern margin of the body are less productive than the overlying karstified limestones.
- The pure unbedded limestones in the synclinal valleys of South Munster are generally intensely fractured and have high frequency jointing and their permeability has been enhanced by subsequent karstification. Karst features such as caves, swallowholes and other collapse features occur in this GWB.
- Groundwater flows through faults and joints formed by deformation and subsequently enlarged by karstification. Most groundwater flow occurs in an upper shallow highly karstified weathered zone a few metres thick in which groundwater moves quickly in rapid response to recharge. Below this is a deeper zone where there are two components to groundwater flow. Groundwater flows through interconnected, solutionally enlarged conduits and cave systems that are controlled by structural deformation (influence of N-S jointing). In addition there is a more dispersed slow groundwater flow component in smaller fractures and joints outside the larger conduits. Generally this connected fractured zone extends to about 30 mbgl in pure limestones, however in the pure bedded limestones of the South Munster region, deep inflows from major zones of fissuring have been encountered to 40-50 mbgl.

- Groundwater in this body is unconfined. The water table is generally less than 10 metres below the surface. Groundwater gradients will be flat in the permeable limestones (0.001-0.002). The highly permeable aquifer can support regional scale flow systems. Groundwater flow paths can be up to several kilometres long, but may be significantly shorter in areas where the water table is very close to the surface. Overall groundwater flow away from the ridges to the north and south, towards the rivers draining the valley and ultimately to Lough Mahon in the east.
- Recharge to this GWB is both point and diffuse. The ridges to the north and south of this GWB (Ballinhassig GWB) provide runoff which recharges the limestone aquifer in the valley. Diffuse recharge will occur over the entire GWB. Swallowholes, collapse features provide the means for point recharge to the karstified aquifer. A relatively small volume of groundwater may cross as through-flow into this GWB from the adjacent low transmissivity GWBs.
- Most of the GWB is of High Vulnerability. Many areas of Extreme Vulnerability occur in the east of the body where rock outcrop and shallow rock are common. In this highly karstified aquifer the underlying limestone will have a very irregular surface. Subsoil depths in this GWB can therefore be highly variable within short distances.
- There is a high degree of interaction between surface water and groundwater in GWB underlain by karstified limestone.

10.2.2.2 Preliminary Flood Risk Assessments, 2010

"Preliminary Flood Risk Assessments" was published for the OPW in 2010 by Mott MacDonald as part of The Floods Directive (2007/60/EC) which came into force in November 2007. This report details and reviews the geological and hydrogeological aspects of the Republic of Ireland, outlines pathways in the source-pathway-receptor-consequence model, reviews groundwater flooding in Europe and mapping approaches used in the UK, discusses groundwater flooding in Rep. of Ireland using map models, and conclusions and recommendations of further works in the next phase of the Directive.

The following information was taken from the report which may aid in the research and conceptual site modelling of groundwater flooding in South Ballincollig:

- Groundwater flooding occurs when the natural underground drainage system cannot drain rainfall away quick enough, causing the water table to rise above the ground surface.
- The position of the water table relative to the ground surface is the principal indicator of whether or not groundwater flooding is likely for those aquifers whose characteristics are conducive to it. The position of the piezometric surface shows the level to which groundwater would rise in, for example, a well, were it not confined. Thus, for both confined and unconfined groundwater environments, the analysis of groundwater levels is a fundamental tool for the mapping of groundwater flooding.
- Springs occur where the water table surface intersects the ground surface. Therefore, springs are locations that may be susceptible to groundwater flooding.
- Groundwater flooding is described as `the type of flooding that can be identified as being caused by water originating from beneath the ground surface from permeable strata through a natural process'. Also, `groundwater flooding can also be differentiated from surface water flooding by its persistence, with a typical duration that is measured in weeks rather than hours and days and has a tendency to occur throughout the winter, often extending into spring and sometimes into the early summer'.
- Groundwater level rise in response to prolonged extreme rainfall in unconfined aquifers.

- Groundwater flooding mechanisms which may trigger an event in the bedrock aquifers are low/very low aquifer storage and high transmissivity.
- The available evidence supports the conclusion that the only groundwater floods of significance in the Republic of Ireland are those associated with regionally important karstified aquifers and that most occur on Pure Bedded Limestones (South Ballincollig is Pure Unbedded Limestones).
- The majority of groundwater flooding in Ireland occurs in turloughs, and in lowland karstic regions of Connacht and the west Mid-lands. Turloughs are groundwater floods that form temporary surface water features following sustained rainfall over a long duration of time that fluvial flooding. Unlike fluvial flooding, which may appear to be groundwater flooding, turloughs form at an already shallow water table rather than a rise in water table over time. Drainage of turloughs is correlated to the water table falling.

10.2.2.3 Flooding at Clash Road, Ballincollig, Co. Cork on 19th.November 2009

"Flooding at Clash Road, Ballincollig, Co.Cork on 19th.November 2009" was from a Flood Data Collection Form, published for the OPW by Cork County Council. The flooding occurred in the Lee catchment. The flood source was noted as a "high watertable" and cause as "inadequate surface water drainage". Flood level occurred at 20.4m AOD with a depth of 0.6m. Flooding was reported to occasionally occur at that locality in the past. The main impacts from this event were stated as "some pasture land was affected by the flooding event".

Figure 10.1 shows the extent of the groundwater flooding event and is approximately 200m from the most north eastern boundary of the subject lands. The GSI online mapping service Historic Groundwater Flooding noted that this event was groundwater flooding.

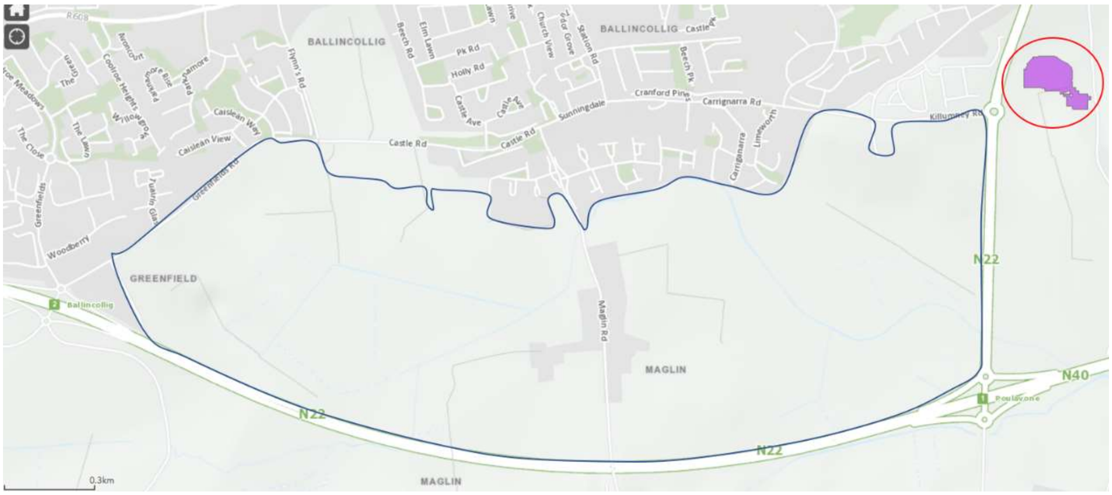


Figure 10.1: Location of the Flood Event Circled in Red (GSI online mapping)

10.3 Conceptual Site Model

Using water strike data that was collected from the Geotech Ballincollig Bypass data from the various borehole and trial pit logs, it was possible to estimate hydraulic head at each location with a water strike and elevation value. It is important to note that these data were collected during April-June 1995, which was before the N22 bypass was built which may have impacted the hydrogeological regime and the groundwater flow paths as a result, especially with the excavation of the road sub-base and piles for the fly-over. However, by assessing these data it may be possible to establish the groundwater flow direction of the time, which areas possess higher groundwater levels, and which areas may be at risk of flooding.

According to the EPA Groundwater Bodies map, there a divide between the Ballincollig and Ballincollig East groundwater bodies appears to be along the Pure Unbedded Limestone (green) and Dinantian Mudstone and Sandstones / Devonian Old Red Sandstone (purple)

Two sets of groundwater flow paths were identified (Figure 10.2), the first (in Navy) flows from the divide boundary north to south. The second (in White) flows from the divide boundary south to north, west to east in line with the fluvial systems, and north to south to converge towards the Maglin/Curraheen river.

Hydraulic head levels (in Yellow) show that two areas in particular have high hydraulic head readings, that being the western extent of the subject lands and the most south-eastern extent of the subject lands' boundary. These high head levels may trigger some groundwater flooding during a very high and/or several concurrent rainfall events. However, given the length of time between the data collected, the time of year these readings were taken, and the change in environment, further research is needed to confirm if these areas are at risk. It should be noted that these at-risk areas are in close proximity to surface water features, so the likelihood of flood events occurring will be fluvial.

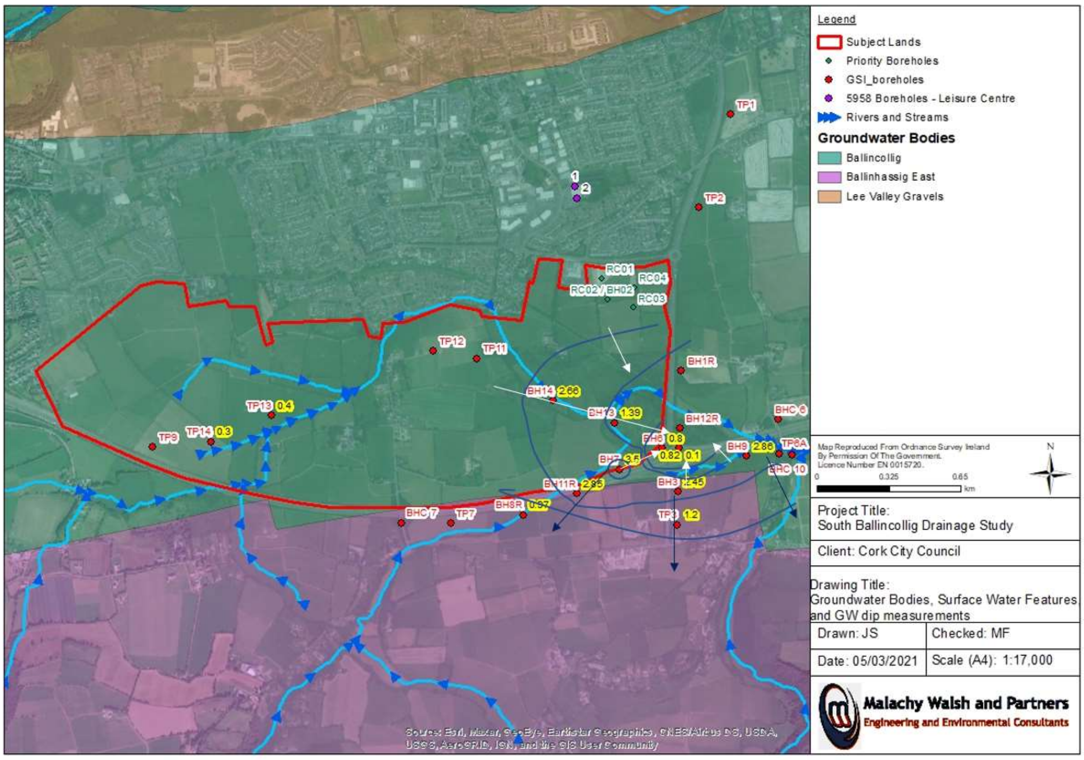


Figure 10.2: Conceptual Site Model of South Ballincollig

10.4 Groundwater Flood Risk Conclusion and Recommendations

Groundwater flooding is unlikely to be a significant risk within the Plan Area.

Although there are areas within the subject lands that exhibit high water strikes, it can be inferred through the CSM and groundwater flow paths that if flooding were to occur it would be due to fluvial flooding and a breach of the rivers' carrying capacities. The "Flooding at Clash Road, Ballincollig" report did not specifically mention that the flooding event in was groundwater flooding but did note the main source of the flooding was from the high water table and lack of surface water drainage in the area. It is yet to be established if the this event was due to groundwater flooding or surface water flooding.

Standing water was identified on site following extensive rainfall events, however these are not considered groundwater flooding as they would be able to drain across the surface or through the soils and subsoils and percolate to the water table

Fluvial flooding is a known risk in this site, and as such a flooding risk assessment should be drawn up to mitigate for it.

Groundwater monitoring is recommended for the western portion near the "Boiling Spring" as it is yet to be determined if this spring interacts with the groundwater table and if flooding is associated with that area. Groundwater monitoring is also recommended for the most south-eastern corner of the site as it is the converging point of two or more river channels, as well as exhibiting higher water strike readings.

Monitoring wells should be drilled within the site as there is a lack of recent groundwater data. These wells may also aid in the flooding risks associated with the River Lee to the north of the site. A full walkover and inspection of the risk areas stated above should also be carried out.

11. Strategic Flood Risk Management

11.1 Summary of Identified Flood Risk

It has been identified that fluvial flooding from the Maglin River is the main source of flooding within the Plan Area and parts of the lands are located in Flood Zones A and B. The flood risk is such that it will influence the decision making process in the Development Plan preparation and land zoning objectives.

There are areas of the site which may be subject to pluvial flood risk or ponding of surface water. An indicative flood risk map has been provided to highlight potential areas however it is considered that these flood risk issues are best dealt with on a site specific basis with the benefit of local data and that the management of such risks can generally be achieved by suitable surface water management and mitigation. Consequently, this potential source of flooding does not require further consideration in this SFRA.

There are no identified incidences of groundwater flooding within the Plan Area. Based on the information available for this SFRA, groundwater flooding is unlikely to be a significant risk however the risk should be considered on a site specific basis using local site investigation data. Notwithstanding this, it is envisaged that any possible risk associated with groundwater flooding would inherently be mitigated by incorporating a suitable surface water and fluvial flood risk management plan. Consequently, this potential source of flooding does not require further consideration in this SFRA.

11.2 Flood Zones & Development Plan Land Zoning

Cork City Council reviewed the flood zones identified during the SFRA preparation and applied the sequential approach when making decisions on land zoning in the Plan Area. Figure 11.1 shows the flood zones overlaid on the proposed Development Plan Zoning. Highly Vulnerable and Less Vulnerable development has been located in Flood Zone C and lands within Flood Zones A and B are restricted predominantly to Water-compatible Development. However, the following exceptions apply:

3. The proposed Spine Road will be located across Flood Zones A, B and C. The Spine Road is considered to be Less Vulnerable Development corresponding to Local Transport Infrastructure therefore a Development Plan Justification Test has been carried out, as summarised in Section 11.6.
4. The boundaries of some of the lands zoned for development other than water compatible development encroach slightly into Flood Zones A or B. The extent of lands encroaching into Flood Zones A or B other than those zoned as ZO 16 (Public Open Space) are highlighted in Figure 11.2. Development of these lands for High Vulnerability uses is not appropriate without a Justification Test and furthermore development for Less Vulnerable uses is not appropriate without Justification in Flood Zone A. It is recommended that one of the following approaches is taken to address these localised areas:
    a. The land zoning for those areas which encroach into Flood Zones A or B is revised so that only Water-Compatible development is in Flood Zone A and only Less Vulnerable development is in Flood Zone B.
    b. A Plan-making Justification Test is carried out to confirm that the development of these areas is acceptable in the context of the Planning System and Flood Risk Management Guidelines.
    c. An Objective is included in the Development Plan that precludes inappropriate development in Flood Zones A and B.

The Vulnerability Classification for all Zoning Objectives in the Plan Area is provided on Table 11.1 below together with the corresponding Flood Zones.

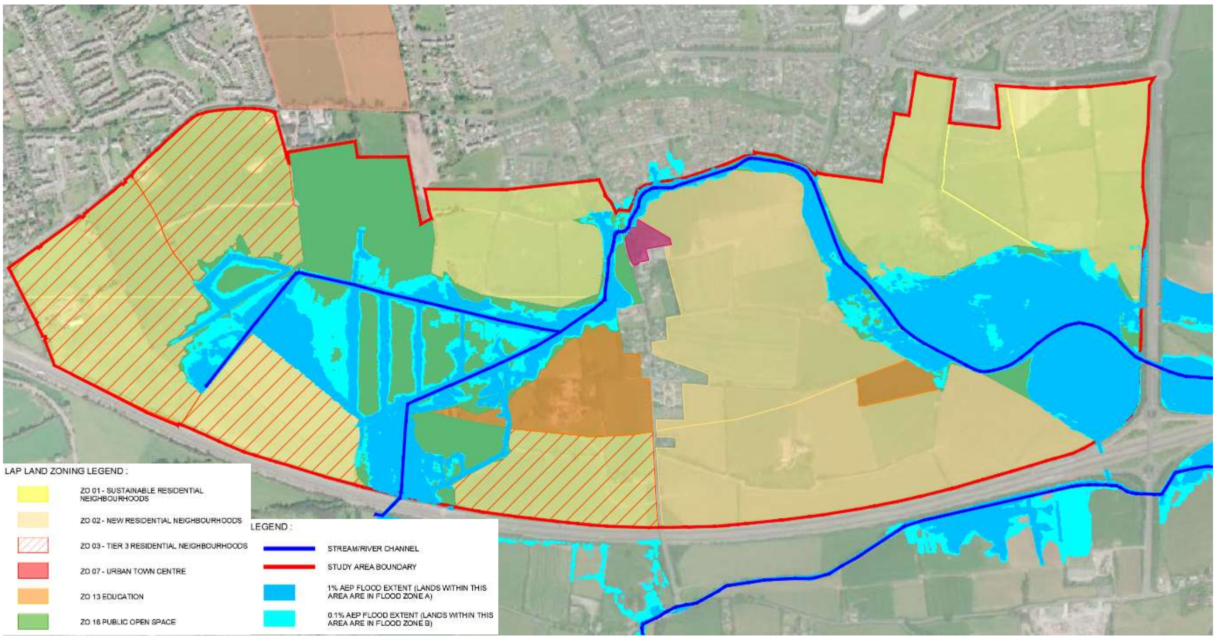


Figure 11.1: Flood Zones & Proposed Development Plan Zoning

Zoning Objective	Zoning Description	Vulnerability	Flood Zone *	Justification Test Requirements
ZO 01	Sustainable Residential Neighbourhoods	Highly Vulnerable Development	C	Not Required
ZO 02	New Residential Neighbourhoods	Highly Vulnerable Development	C	Not Required
ZO 03	Tier 3 Residential Neighbourhoods	Highly Vulnerable Development	C	Not Required
ZO 07	Urban Town Centre	Less Vulnerable Development	C	Not Required
ZO 13	Education	Highly Vulnerable Development	C	Not Required
ZO 16	Public Open Space	Water Compatible Development	A/B/C	Not Required
-	New Spine Road	Less Vulnerable Development	A/B/C	Required

** There may be some localised fringes of the Zoning located in Flood Zones A or B – Refer to Figure 11.2 and discussion in Section 11.2.*

Table 11.1: Land Use Zoning & Vulnerability Classification

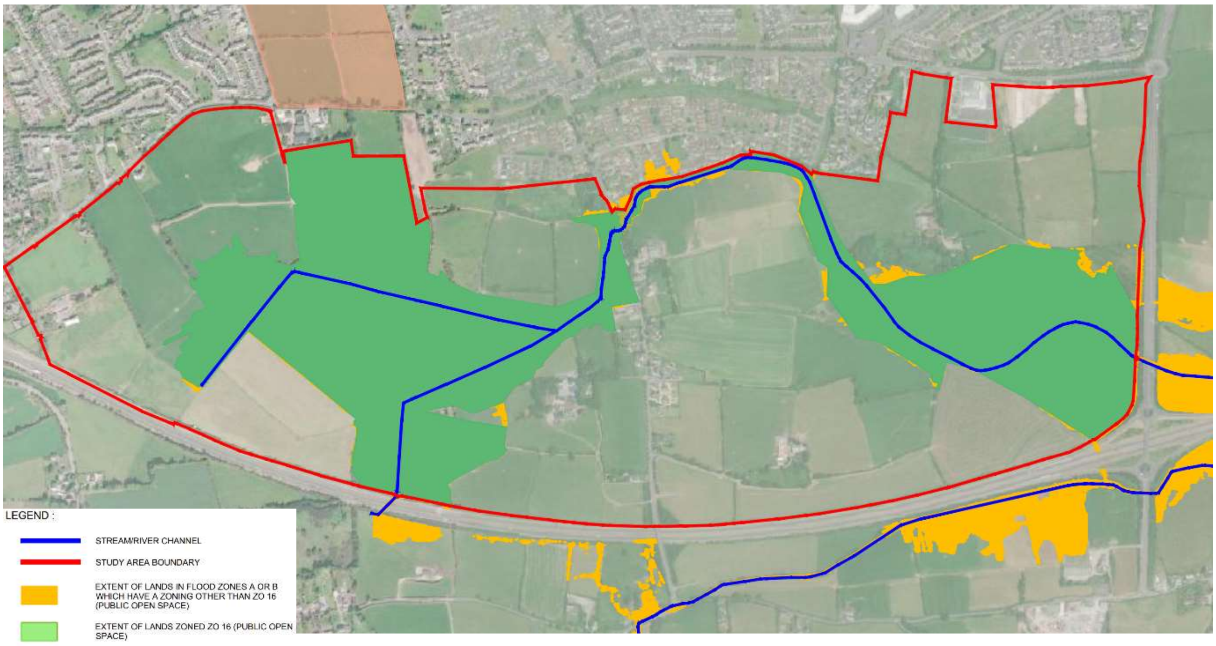


Figure 11.2: Extent of Lands in Flood Zones A or B which have a Zoning other than ZO16 (Public Open Space)

11.3 Hazard to People

As indicated on Figure 11.3 and Figure 11.4, flood depths in the Plan Area are relatively shallow. Flood depths for the 1% AEP MRFS away from the immediately vicinity of the main channels are generally less than 300mm however depths increase to approximately 1m on the downstream end of the Plan Area. Flow velocities in the floodplains are also low, and often close to zero where larger water depths accumulate.

Flood Hazard maps for the 1% AEP MRFS and 0.1% AEP MRFS are provided on Figure 11.5 and Figure 11.6 which were prepared using the guidance in DEFRA FD2320. Outside of the main channels, the flood hazard is generally less than 0.5 and is only greater than 0.75 in localised dips or drains. The hazard is therefore Low and is described as “Caution - Flood zone with shallow flowing water or deep standing water”.

Figure 11.3: Flood Depth – 1% AEP MRFS

Figure 11.4: Flood Depth – 0.1% AEP MRFS

Figure 11.5: Flood Hazard – 1% AEP MRFS

Figure 11.6: Flood Hazard – 0.1% AEP MRFS

11.4 Mitigation Measures

The mitigation measures outlined on Table 11.2 are recommended from a SFRA perspective to ensure that the flood risk to developments within the Plan Area is acceptable and to prevent an adverse impact upstream or downstream of the site.

SFRA Mitigation Measure Number	Measure	Mitigation
SFRA_MM01	Development on Floodplains	Cork City Council has adopted the sequential approach to land zoning in the Plan Area and development on floodplains has been avoided insofar as possible. The proposed spine road through the lands will be constructed through Flood Zones A and B and specific mitigation has been recommended for this, as summarised in SFRA_MM02 below. In line with the precautionary approach recommended in the Guidelines, compensation storage shall be provided for any development which results in a loss of floodplain up to the 0.1% AEP flood event. This shall be constructed prior to the development of the lands for which compensation is being provided.
SFRA_MM02	Proposed Spine Road Construction	A hydraulic analysis has been carried out to determine the impact of the road construction on flood risk. The analysis was based on an assumed road alignment. It was determined that the impact of the road construction could be mitigated by: 1. Incorporating suitably sized culverts at each point where the road crosses the river; 2. Providing additional flood relief/balancing culverts through the road embankment, positioned at strategic locations which allow the existing flow regime to be maintained; 3. Providing compensation storage in the upstream reaches of the site; 4. Restricting the flow through the eastern road crossing so that additional storage is provided upstream of the embankment at this location. The analysis carried out for this SFRA is sufficient to demonstrate that the road construction is acceptable in the context of flood risk and that suitable mitigation can be incorporated into the development design. It may however be possible to refine the solution or develop alternatives in conjunction with the road corridor design. In any case, the final details of the measures as well as the potential impact of any changes to the assumed road alignment should be confirmed in the design stage flood risk assessment.
SFRA_MM03	Maintaining Existing Drainage Regime & Flow Paths	The proposed land zoning minimises the extent of development on floodplains and provides suitable alternative flow paths where construction on floodplains is essential and justified. At a site specific scale, existing overland and channelized flow paths should be maintained and floodplain storage and conveyance areas should be protected. Where it is essential to modify flow paths or ground levels in floodplains, the impact of any such modifications should be quantified and mitigated as part of a site specific flood risk assessment.
SFRA_MM04	Finished Floor Levels	In order to ensure there is no unacceptable flood risk to people or property in the Plan Area, the finished floor level of all new developments should be constructed 500mm above the 1% AEP 95% confidence level MRFS flood level. These flood levels are summarised on Table 8.3. This recommendation is consistent with the requirements of Criterion 3 of the GDSDS although the 95% confidence level and freeboard requirement could be relaxed if further refined analysis is carried out at a site specific scale which demonstrates through sensitivity analyses and calibration that a lower freeboard still provides sufficient protection.
SFRA_MM05	Safe Access/Egress & Emergency Planning	The proposed spine road should be constructed at least 300mm above the 1% AEP 95% confidence level MRFS flood levels. This will provide for suitable and safe access/egress from individual development sites for both occupants and emergency services. An Emergency Plan should be developed for all developments interfacing with Flood Zones A and B as part of the development management. This should outline procedures to be following in the event of an extreme flood event and would typically include; the roles and responsibilities of management and stakeholders with contact details; flood sources and depths; flood awareness and flood warning sources; methods for disseminating flood warnings/alerts; evacuation procedures; stand-down, recovery and clean-up operations.

SFRA Mitigation Measure Number	Measure	Mitigation
SFRA_MM06	Surface Water Management	Appropriate surface water management is important for numerous reasons. In the context of flood risk, a key requirement is to ensure that development does not increase flood risk downstream of the site and it is therefore necessary to ensure that surface water runoff volumes and rates do not exceed the pre-development situation. This should be addressed on a site specific basis through the preparation of a Surface Water Management Plan for all developments which should build upon the Sustainable Drainage Strategy prepared for the Maglin / South Ballincollig Expansion Area. Pluvial flood risk in areas where surface water ponding could occur should also be addressed.
SFRA_MM07	Climate Change	The SFRA has considered climate change in the MRFS which includes for a 20% increase in fluvial flows according to current OPW guidance. Sensitivity checks were carried out for the HEFS and it was determined that the strategic mitigation measures provided in this SFRA are sufficiently robust to accommodate increases in flows without significantly increasing flood hazard to people or causing flooding of property, albeit with some residual risk that requires further consideration at site specific level. Similar requirements for considering climate change have been set out for surface water management in the Sustainable Drainage Strategy prepared for the Maglin / South Ballincollig Expansion Area. It is important to recognise that guidance in relation to climate change allowances may change during the lifetime of the Development Plan and the SFRA. Consequently, all future risk assessments and development designs should be based on the latest available guidance at the time of writing.
SFRA_MM08	Site Specific Flood Risk Assessment	A Site Specific Flood Risk Assessment should be carried out for all developments and should be completed in accordance with the Planning System and Flood Risk Management Guidelines (2009). It should build upon the strategic flood risk management measures outlined in this Strategic Flood Risk Assessment and also consider new information and site specific data to ensure that all potential flood risk issues are identified, mitigated and managed to an acceptable level. Residual Risks should be assessed at a site specific scale and appropriate measures should be implemented to manage all identified residual risk. This should include further consideration and development of solutions proposed to manage the residual risks identified in this SFRA.

Table 11.2: SFRA Mitigation Measures

11.5 Residual Risks

The main residual risks relevant to the SFRA are associated with:

- Blockages of Bridges or Culverts
- Exceedance Flows

11.5.1 Blockages in Spine Road Culverts

There may be some potential for blockages to occur in the new bridge/culverts in the Spine Road however the main structures are less likely to become significantly blocked due to the opening size that is expected in order to comply with current design practice. The consequences of any blockage would also be less severe than for the existing bridges in the area because there would be alternative flow paths available via the floodplain culverts and ample freeboard has been recommended to property floor level. Consequently this residual risk is considered to be acceptable and can be further mitigated by appropriate maintenance during the lifetime of the development.

11.5.2 Maglin South Bridge Blockage

It was identified during the model sensitivity checking that there is a potential overland flow path from Maglin South to the lands within this Plan Area whereby any water that enters the floodplain at the upstream end of Maglin South could flow in a northerly direction across the N22 and flow into the site. The analysis indicates that this would not occur for the 1% and 0.1% AEP Current events however there is potential for overtopping in the event of a blockage of the bridges or culverts on Maglin South River upstream of Maglin Road. This is considered to have a reasonably possibility of occurring due to the relatively small opening size of the structures and the extent of vegetation and trees on the upstream channel banks.

The flood map provided on Figure 11.7 indicates the extent of flooding that could occur in the present day (i.e. pre-development). For this example, which does not account for any drainage channels along the N22 Road, almost 2m^3/s could leave Maglin South and flow in a northerly direction towards the site. The flow would then split with some flowing in a westerly direction along the southern side of the N22 road embankment and the remainder potentially spilling over the N25 and flowing directly into the site.

There are existing drainage channels on either side of the N22 road which are likely to intercept some of this overland flow. These should be maintained and could enhanced where necessary to ensure that any such flows are conveyed through the site in a controlled manner that does not pose an excessive risk to property and people. The conveyance route should allow for flow to reach Maglin North River at Node 0 to the west and Node 10 to the east (Reference Figure 8.11 and Figure 8.12). The specific details of the measures required to manage this residual risk should be established in conjunction with the site specific flood risk assessments.

Figure 11.7: Flood Extent Map for Maglin South River with Blockage upstream of Maglin Road

11.5.3 Maglin Road Bridge Blockage

In order to test the effects of a blockage at Maglin Road Bridge, the bridge geometry was adjusted to include for a 50% blockage and the model was run for the 1% AEP MRFS flood event. The flood extent map for this scenario is provided on Figure 11.8 and the increase in flood level due to the blockage is provided on Figure 11.9. The following is noted:

1. The flood level upstream of the bridge increases by up to 300mm. This is considered acceptable given the freeboard to finished floors that is recommended in this SFRA.
2. The flood levels and extents downstream of the bridge generally reduce however the flood extents extends into one property to the north of the bridge which was not previously predicted to flood. This highlights the need to ensure appropriate development management and maintenance however the proposed development of this plan area does not increase the likelihood of a blockage or of flooding at this location. Therefore, this is considered acceptable.
3. Upstream of the bridge, there is a notable increase in flood extent northwest of the river. The flood depth at this location are generally no more than 300mm however there are some lower lying areas or local drainage channels where the depth could reach up to 1m. Again the incorporation of the mitigation measures recommended in this report would ensure that the residual risk to property within the Plan Area is acceptable however further consideration should be given at project level stage to the potential impact such an occurrence could have on downstream properties and provision should be made to manage this water within the site.

Figure 11.8: 1% AEP MRFS Flood Extents – Maglin Road Bridge Blockage

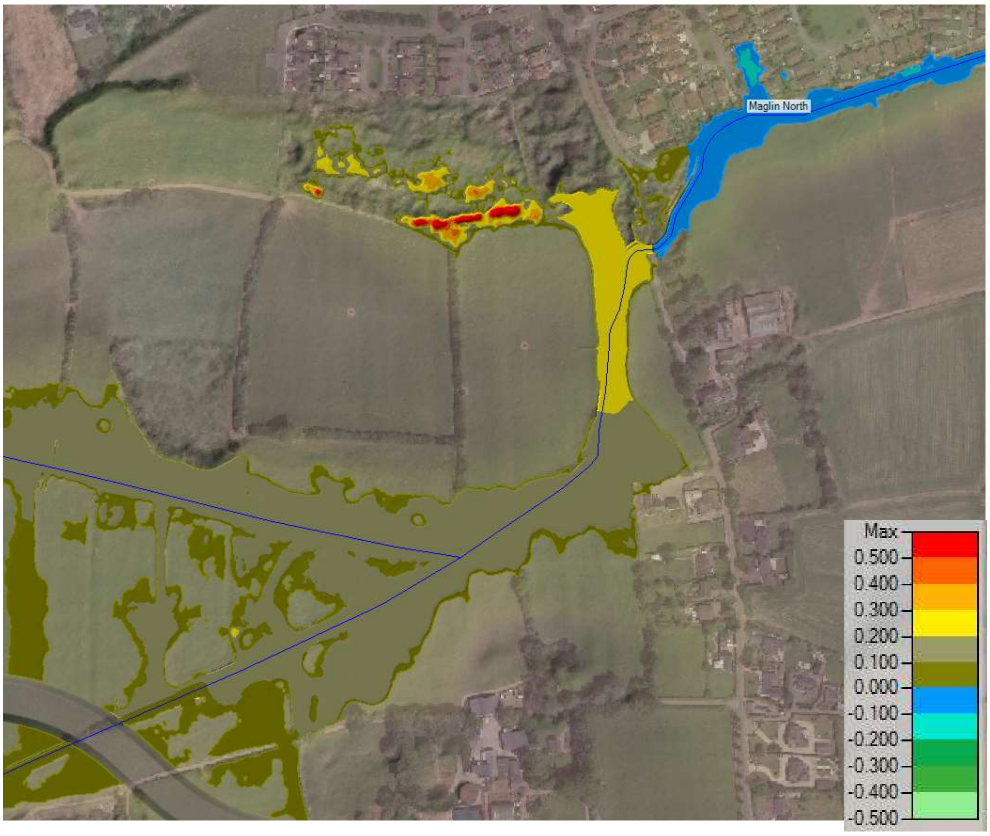


Figure 11.9: 1% AEP MRFS Flood Level Increase due to Maglin Road Bridge Blockage

11.5.4 Future Scenarios & Exceedance Flows

The 0.1% AEP HEFS flood extent map is provided on Figure 11.10 below. As would be expected, this event would result in an increase in flood level and extent throughout the Plan Area. At most locations the increase would not cause flooding to property and the extents would not differ significantly from the current scenario. However there are certain locations where an exceedance flow could have a more significant impact on flood risk. As can be seen on the map, these include:

1. The lands between the existing N22 road and the proposed spine road, on both sides of Maglin Road. This is due to overspill from Maglin South River entering the site. This can be appropriately managed by implementing the measures discussed in Section 11.5.2 - Maglin South Bridge Blockage
2. The flood extends into the lands on the northwest side of Maglin Road Bridge. The flood levels would not cause flooding of property within this Plan Area however provision should be made to manage this water within the site.

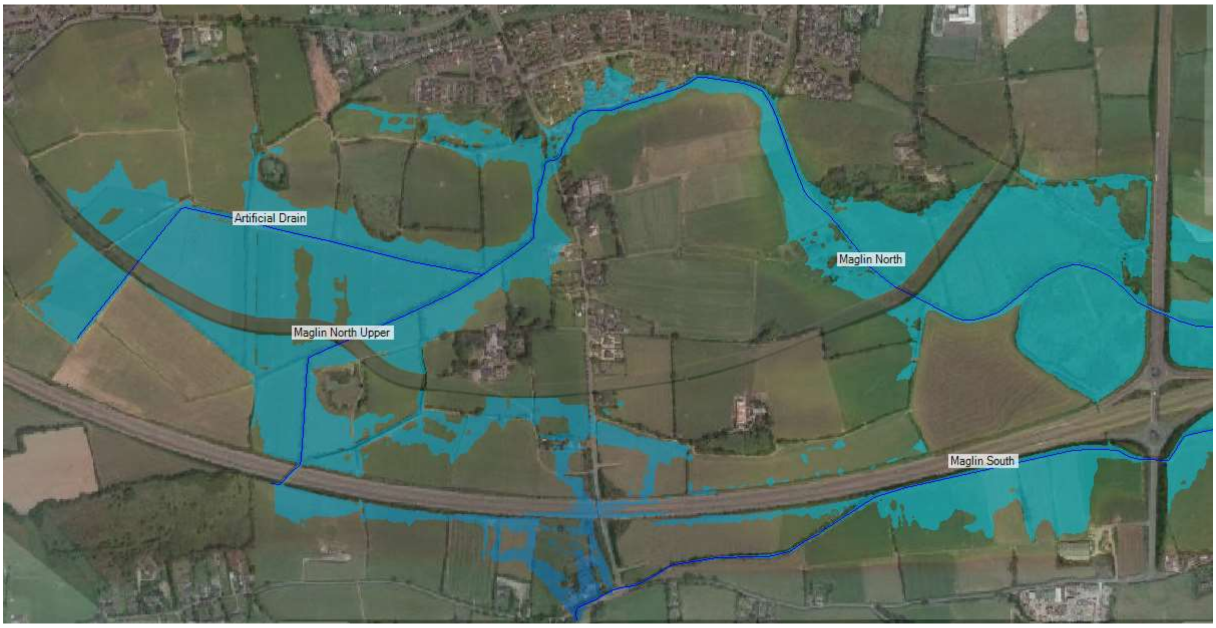


Figure 11.10: 0.1% AEP HEFS Flood Extent Map

11.6 Justification Test

11.6.1 Proposed Spine Road

As outlined in Section 11.2, the proposed spine road will be constructed in Flood Zones A and B therefore a Plan-making Justification Test was undertaken and is summarised on Table 11.3 below. It is concluded that the development of the spine road is acceptable in the context of the Planning System and Flood Risk Management Guidelines.

Justification Criteria	Justification
The urban settlement is targeted for growth in the National Spatial Strategy, Regional Planning Guidelines, or Statutory Plans defined under the provisions of the Planning and Development Act, 2000, as amended.	The South West Regional Planning Guidelines 2010 – 2022 state that additional growth will be directed to Ballincollig. The lands were identified under the Ballincollig-Carrigaline Municipal District Local Area Plan (Ballincollig-Carrigaline MD LAP, 2017) as the “Ballincollig (Maglin) Urban Expansion Area”. South Ballincollig (Maglin) is one of seven strategic consolidation and expansion areas listed in the Core Strategy of the Draft Cork City Development Plan 2022 – 2028. This is a key strategic site targeted for significant population growth. Objective 10.55 of the Draft Plan seeks “To support the compact growth and development of South Ballincollig Expansion Area as a strategic City consolidation and expansion area, as identified in the Core Strategy. All development shall be designed, planned and delivered in a co-ordinated and phased manner, using a layout and mix of uses that form part of an emerging neighbourhood integrated with the wider area”.
The zoning is required to achieve the proper planning and sustainable development of an urban settlement and is; o Essential to facilitate the regeneration and / or expansion of the centre of the urban settlement; o Comprises significant previously developed and / or under-utilised lands; o Is within or adjoining the core of an established or designated urban settlement; o Will be essential to achieving compact and sustainable urban growth; and There are no suitable alternative lands for the particular use in areas at lower risk of flooding within or adjoining the core of the urban settlement.	The construction of a spine road through the Plan Area is essential to facilitate the sustainable development of these lands and thereby provide for the targeted expansion of the town. These lands are not previously developed but are adjoining developed lands to the north and are enclosed on the south and east by the N40 road network. In the context of the strategic development objectives, such lands would be significantly under-utilised if not developed. The lands are adjoining the existing urban settlement and are essential to achieving the objectives of compact and sustainable growth in the Draft Cork City Development Plan 2022 – 2028. Whilst the zoning objectives have been developed by Cork City Council using the sequential approach, it is not possible to avoid the development of the proposed Spine Road in lower risk areas as it is required to traverse the site as part of the key infrastructure for the development of these lands and for the overall town.
A Flood Risk Assessment to the appropriate level of detail has been carried out as part of the SEA, which demonstrates that flood risk to the development can be adequately managed and that the development will not cause adverse impacts elsewhere.	A SFRA has been carried out which addresses the flood risk to and arising from the development of these lands. It is concluded that the flood risk can be adequately managed, the risk to people is low and the development of the site and the proposed spine road is not expected to cause adverse impacts elsewhere. At planning stage, a site specific flood risk assessment will be required in relation to the spine road and each development within the Plan Area. The assessment should be completed in accordance with the Planning System and Flood Risk Management Guidelines (2009) and the additional recommendations in this SFRA.

Table 11.3: New Spine Road – Justification Test

11.6.2 Zoned Lands with Minor Encroachment in Flood Zones A & B

As outlined in Section 11.2, prior to finalising the Development Plan, a Plan-making Justification Test may be required in relation to localised areas of lands which encroach into Flood Zones A or B.

12. Recommendations for Development Plan Objectives

It is recommended that the Development Plan Objectives address the key requirements for sustainable flood risk management of these lands. These are summarised on Table 12.1 below.

Recommendation Number	Recommendation
SFRA_R01	All new proposed development proposals within the Maglin / South Ballincollig Expansion Area shall be accompanied by individual site-specific flood risk assessments which are completed in accordance with the Planning System and Flood Risk Management Guidelines (2009) and which build upon the strategic flood risk management measures outlined in the Strategic Flood Risk Assessment prepared for the Maglin / South Ballincollig Expansion Area. Flood Risk Assessments should ensure that appropriate site specific mitigation measures are implemented and residual risks are managed to an acceptable level.
SFRA_R02	Inappropriate development in Flood Zones A and B is not permitted except for essential road and transport infrastructure which is necessary to facilitate suitable access to individual development sites and where such infrastructure cannot be located in Flood Zone C. With this exception, any portion of zoned lands within Flood Zone A and Flood Zone B shall be restricted to Water-compatible development and Less Vulnerable Development respectively.
SFRA_R03	It is recognised that minor modifications may need to be made to floodplains to provide suitably designed public open space and to facilitate safe access to individual development sites. Any such modifications shall ensure that existing overland and channelized flow paths are maintained and floodplain storage and conveyance areas are protected. Proposals shall be accompanied by a site specific flood risk assessment carried out to an appropriate level of detail to quantify the potential impacts of any proposal on flood risk.
SFRA_R04	Compensation storage shall be provided for any development which results in a loss of floodplain up to the 0.1% AEP flood event. This shall be constructed prior to the development of the lands for which compensation is being provided.
SFRA_R05	Incorporate Sustainable Urban Drainage Systems (SuDS) and other nature-based surface water drainage solutions as part of a site specific Surface Water Management Plan for all development proposals. Proposals shall build upon the Sustainable Drainage Strategy prepared for the Maglin / South Ballincollig Expansion Area. Proposals shall also address pluvial flood risk in areas where surface water ponding could occur and ensure that floor and street levels are designed to manage any potential risks or exceedances.
SFRA_R06	The design of amenity and public open space adjacent to the river and in areas of flood risk should incorporate Natural Flood Management Measures and seek to achieve Biodiversity Net Gain for the development.
SFRA_R07	Development along natural watercourses should comply with the Inland Fisheries Ireland Guidance 'Planning for Watercourses in the Urban Environment (2020), including in particular the maintenance of a minimum riparian zone of 35 metres for river channels greater that 10 metres in width, and 20 metres for rivers channels less than 10 metres in width.
SFRA_R08	Proposals to construct new or replacement culverts or bridges on watercourses in the Plan Area shall be subject to approval from the OPW, in accordance with Section 50 of the Arterial Drainage Act 1945.

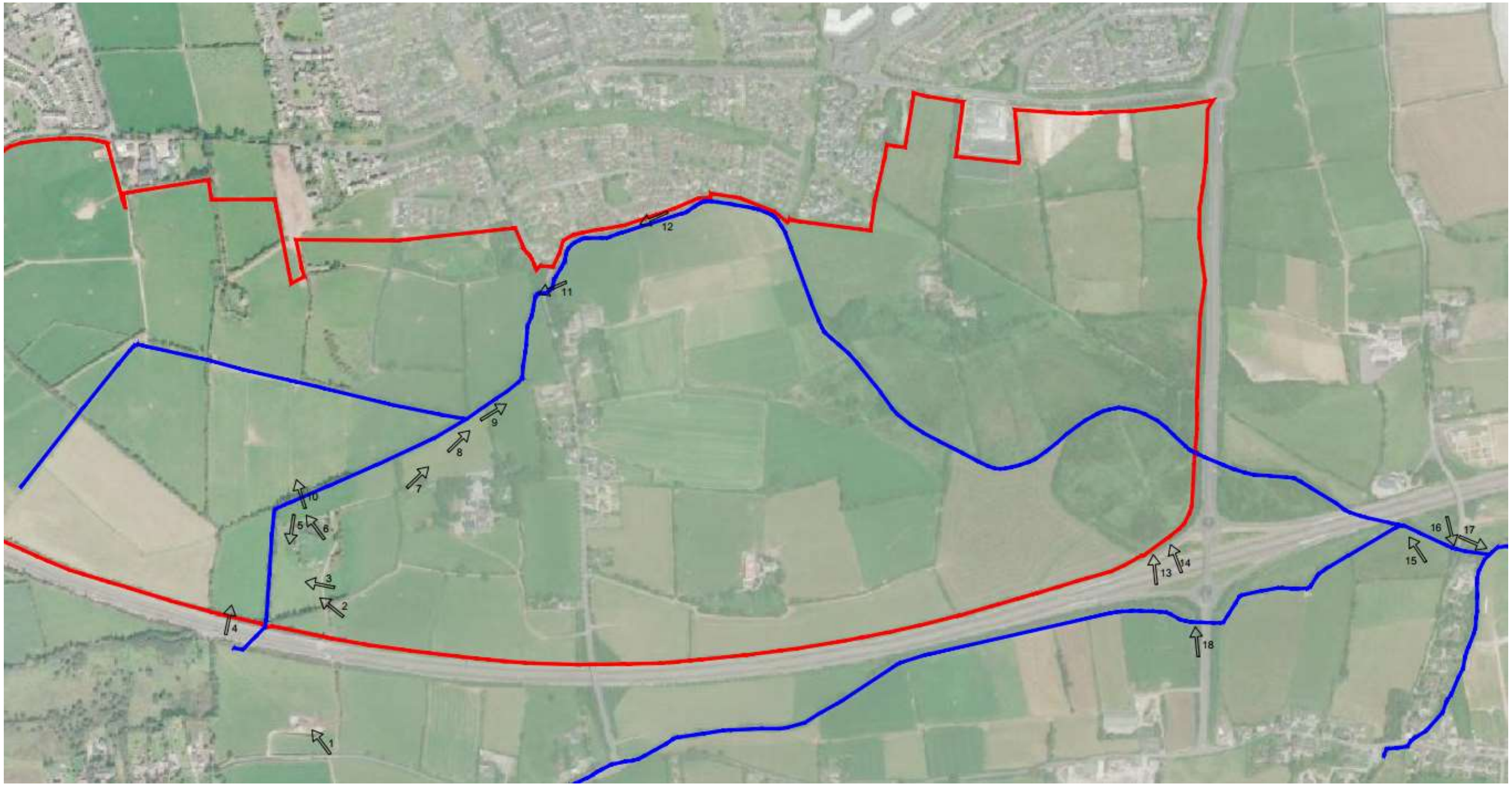
Table 12.1: Recommendations for Inclusion in Development Plan

Appendix A

19th February 2021 Flood Event Photographs

Cork City Council

MWP



Photograph Location Plan

Photo 1: Maglin North River, Upstream of Plan Area

Photo 2: East of Maglin North River, upstream of karst rock outcrop

Photo 3: East of Maglin North River, upstream of karst rock outcrop

Photo 4: West of Maglin North River, looking north from N22

Photo 5: East of Maglin North adjacent to karst rock outcrop, looking south

Photo 6: Downstream of karst rock outcrop, looking north

Photo 7: Maglin North, upstream of confluence with Artificial Drain

Photo 8: Maglin North, near confluence with Artificial Drain

Photo 9: Maglin North, upstream of Maglin Road Bridge

Photo 10: Maglin North, looking north across river towards Ballincollig Castle

Photo 11: Maglin North, looking upstream of Maglin Road Bridge

Photo 12: Maglin North, downstream of Maglin Road Bridge

Photo 13: Twin pipe culvert under N22 linking Maglin South with Maglin North

Photo 14: Southeast end of Plan Area

Photo 15: Maglin North Culvert under N22, just upstream of confluence with Maglin South

Photo 16: Clash Road Bridge, upstream

Photo 17: Clash Road Bridge, looking downstream

Photo 18: Maglin South at N22 link road

Appendix B

Drawings

- Existing Flood Extent Map – Current Scenario
- Development Plan Zoning & Flood Extent Map
- Proposed Zoning Map with Spine Road, Culverts & Compensation Storage – Concept Plan
- Extent of Lands in Flood Zones A or B which have a Zoning other than ZO 16 (Public Open Space)

MWP

SUSTAINABLE DRAINAGE STRATEGY

South Ballincollig Drainage Study

Cork City Council

December 2021

Contents

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Figures

Project No.	Doc. No.	Rev.	Date	Prepared By	Checked By	Approved By	Status
21721	6002	A	14/08/2021	C. Ahern	M. Fenton	M. Fenton	DRAFT
21721	6002	A	20/12/2021	C. Ahern	M. Fenton	M. Fenton	Final

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Glossary of Acronyms and Terms

AEP	Annual Exceedance Probability
CFRAMS	Catchment Flood Risk Assessment and Management Study
CIRIA	Construction Industry Research and Information Association
DEFRA	Department for Environment, Food and Rural Affairs
DTM	Digital Terrain Model
EPA	Environmental Protection Agency
FFL	Finished Floor Level
FRA	Flood Risk Assessment
GDSDS	Greater Dublin Strategic Drainage Study
HEFS	High End Future Scenario
LAP	Local Area Plan
mOD	Metres Above Ordnance Datum
MRFS	Mid Range Future Scenario
MWP	Malachy Walsh & Partners
OPW	Office of Public Works
PSFRM	The Planning System and Flood Risk Management Guidelines, November 2009
SAAR	Standard Average Annual Rainfall
SuDS	Sustainable Urban Drainage Systems
SDS	Sustainable Drainage Strategy

1 Introduction and Background

1.1 General

Malachy Walsh and Partners (MWP) Consulting Engineers have been commissioned by Cork City Council (CCC) to carry out a Strategic Flood Risk Assessment (SFRA) and Sustainable Drainage Strategy Study (SuDS) in relation to the development of lands in Maglin, Ballincollig, Cork.

This report relates the Sustainable Drainage Strategy for the proposed Ballincollig (Maglin) Urban Expansion Area.

1.2 Scope of this Report

The purpose of this report is to prepare a Sustainable Drainage Strategy (SDS) for the site. This includes:

- Provide a clear, creative, cost-effective and innovative SuDS which would be adopted for the main infrastructure design and by future designers on a site specific basis;
- Identify suitable locations for different types of SuDS;
- Identify key amenity features & potential habitats, such as wetlands or ponds;
- Set out all mitigation measures required for each area of the site, including confirmation of the minimum finished floor levels and ground levels throughout the study area & the maximum discharge rates from the developed sites;
- Identify the route, inverts & outfalls of the proposed main drainage system serving the developed area, including interfaces with existing or future foul trunk mains, rising mains or other utilities;
- Set out firm and clear requirements in relation to the incorporation of SuDS in both the development masterplan and the future development of the zoned lands.

1.3 Subject Site Description

1.3.1 Current Site Overview

The subject lands for this study are located in Maglin, Ballincollig, Cork. The lands were formerly under the jurisdiction of Cork County Council but transferred to the jurisdiction of Cork City Council as part of the city boundary extension in 2019. Cork City contains five local electoral areas, and the subject lands lie within the Cork City South West Local Electoral Area. The subject lands are currently undeveloped. The current land use of the subject land is largely agricultural pastureland as evident by the most recent Corine Landcover Use map dated 2018.

The Maglin River flows into the western portion of the site via a culvert under the N22 road and continues to flow through the site until it discharges near the southeast corner. A series of artificial drainage channels extend through the agricultural lands to the west of the Maglin River. The site has been identified as at risk of fluvial flooding from the Maglin River and karst features may exist within the site which will need to be assessed and managed. There may also be a risk of high groundwater levels within the site which will need to be assessed further and managed.

There are a number of existing features on the site including Fulacht Fia's, Lime Kilns, a standing stone, Maglin House, Ballincollig cave, Ballincollig Castle which may provide restrictions.

1.3.2 Proposed Site Overview

The subject lands were identified under the Ballincollig-Carrigaline Municipal District Local Area Plan (Ballincollig-Carrigaline MD LAP, 2017) as the "Ballincollig (Maglin) Urban Expansion Area". Ballincollig has experienced a high level of growth since 2000. The vision for Ballincollig is that it will continue to grow as a major centre for population and employment. The Cork City Draft Development Plan 2022 - 2028 identifies suitable locations for residential and employment growth in the town and co-ordinates this growth with the upgrading of infrastructure services. The land south of Ballincollig town, Maglin, represents a major strategic housing development opportunity for Cork City. Cork City Draft Development Plan 2022 - 2028 facilitates the delivery of these lands for development.

The subject lands cover approximately 220 ha and are zoned for mixed use including residential, educational, and recreational. The zoning of these lands is illustrated in Figure 1.1 below. There are a total of six different land zones within the subject lands. The majority of the land has been zoned as Sustainable Residential Neighbourhoods (ZO 01), New Residential neighbourhoods (ZO 02), Tier 3 Residential Neighbourhoods (ZO 03) and Public Open Space (ZO 16). There is also areas zoned for Education (ZO 13) and an area zoned as the Urban Town Centre (ZO 07). The public open space land use has been largely influenced by the strategic flood risk assessment which indicates these areas will be inundated during 0.1% and 1% AEP flood events. It is also proposed to construct a new spine road through the expansion area which would cross the Maglin River at a number of locations.

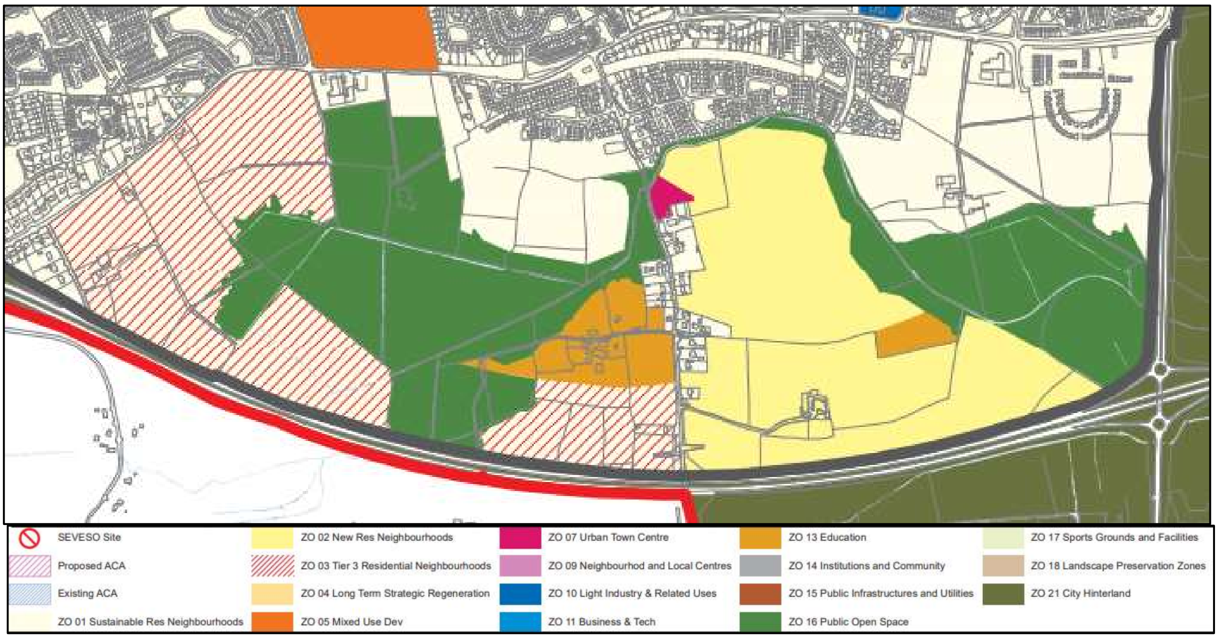


Figure 1.1: Ballincollig (Maglin) Urban Expansion Area Land Zoning Map (Cork City Development Plan 2022-2028)

2 Sustainable Drainage Systems (SuDS)

2.1 Introduction to SuDS

Surface water is a valuable resource, and this should be reflected in the way it is managed and used in the bult environment. The overarching objective of the SuDS approach is to ensure that development on each area of land within the subject area does not result in an increase in run-off to rivers and other discharge points when compared to a natural hydrological baseline pre-development. To maximise these benefits, sustainable drainage systems should be considered from the beginning of the development planning process and throughout the design. This may influence site layout and design, and the use and characteristics of urban spaces. As well as the benefits listed below, SuDS generally provide robust, low maintenance and usually cheaper methods of dealing with rainfall for new developments.

In order to enable a SuDS design to be successfully undertaken an understanding of the ground conditions on the site and an estimation of the storm water runoff from the proposed development is required. A suitable site investigation should be completed for all developments with the plan area to inform the detailed design of SuDS, in particular to determine the infiltration and groundwater characteristics.

2.2 Policy & Guidance

These policy documents give effect to the EU Floods Directive 2007 which requires the assessment of watercourses to determine flood risk and to take adequate and co-ordinated measures to reduce that risk where it exists. They also give effect to the EU Water Framework Directive adopted in 2000 which was transposed into Irish law in 2003 by the European Committees Regulations 2003 (S.I. No. 722 of 2003) where Member States must ensure that their waters achieve at least good status by 2015 and that their status does not deteriorate. Both of these EU Policy Documents have been transposed into Irish Law. These changes towards the SuDS approach is now supported by a number of infrastructural policy documents. The following policy and guidance documents are used in the design of SUDS systems for development:

- Cork City Draft Development Plan 2022 – 2028
- European Union Water Framework Directive (2000/60/EC)
- CIRIA SuDS Manual 2015 (Latest)
- Greater Dublin Strategic Drainage Study (2005)

2.3 Philosophy of SuDS

SuDS should not be thought of as an individual component but rather as an interconnected system designed to manage, treat, and make the best use of surface water, from where it falls as rain to the point at which it is discharged into the receiving environment beyond the boundaries of the site. A central design concept for SuDS is the SuDS Management Train. This will be discussed in more detail in Section 5, but this essentially describes the use of a sequence of components that collectively provide the necessary processes to control the frequency of runoff, the flow rates, the volume of runoff, reduce concentrations of contaminants to acceptable levels, and provide amenity and biodiversity benefits.

3 SuDS Objectives & Design Requirements

3.1 Design Basis & Criteria

The overarching requirement of a successful SuDS strategy is to ensure that drainage of newly developed areas is performed without increasing on pre-development runoff rates and volumes. This will be achieved principally by controlling flows through various SuDS systems.

Sustainable Drainage Systems are designed to replicate natural drainage systems. The philosophy of SuDS is about maximizing the benefits and minimizing the negative impacts of surface water run-off from developed areas. The types of benefits that can be achieved by SuDS can be broadly divided into four categories, also known as the four pillars of SuDS design and are the main drivers supporting SuDS philosophy. These four pillars are water quantity, water quality, amenity, and biodiversity. These drivers control the design criteria and objectives of each SuDS component.

The design criteria of SuDS controlled by the four pillars of SuDS can be summarized as follows as stated in the CIRIA SuDS Manual C753 2015.

Objective	Design Criteria
Water Quantity	1. Use surface water runoff as a resource. 2. Support the effective management of flood risk in the receiving catchment. 3. Protect morphology and ecology in receiving surface waters. 4. Preserve and protect natural hydrological systems on the site. 5. Drain the site effectively. 6. Manage on-site flood risk. 7. Design in system flexibility/adaptability to cope with future change.
Water Quality	1. Support the management of water quality in receiving surface waters and groundwaters. 2. Design system resilience to cope with future change.
Amenity	1. Maximise multi-functionality. 2. Enhance visual character. 3. Deliver safe surface water management systems. 4. Support development resilience/adaptability to future change. 5. Maximise legibility. 6. Support community environmental learning.
Biodiversity	1. Support and protect natural local habitat and species. 2. Contribute to the delivery of local biodiversity objectives. 3. Contribute to habitat connectivity. 4. Create diverse, self-sustaining, and resilient ecosystems.

Table 3.1: Design Criteria Summary

These design criteria should be given full consideration during the development design. The extent and way in which these criteria can be delivered will be controlled by the site characteristics and development objectives. It is likely that the water quantity and water quality design criteria will prove to be the main drivers in determining the design philosophy of this development site and these criteria are supported by standards which should be implemented to ensure the successful implementation of the SuDS system. Maximizing delivery of amenity and biodiversity criteria will likely ensure the delivery of other planning outcomes/objectives for this site.

To maximize the opportunities and associated benefits of the SuDS, these criteria should be considered at an early stage and fully integrated into the surface water management and urban design process. Other general design criteria which should be considered include constructability, maintainability, cost-effectiveness, and health and safety.

3.2 Water Quantity

The overarching water quantity design objective is to control the volume of runoff to support the management of flood risk and maintain and protect the natural water cycle. This is achieved by controlling how fast the runoff is discharged from the site and how much runoff is discharged from the site. Several techniques can be implemented to control the peak runoff rate and volume of runoff from a development. These techniques include infiltration, attenuation, conveyance, and water harvesting. Each technique presents different opportunities for stormwater control, flood risk management, water conservation and groundwater recharge. Various SuDS components can utilize any one or a number of these techniques to control the water quantity runoff from this development.

3.2.1 Climate Change Allowance

Rainfall depths used for the design of the drainage system and associated SuDS components should include for the effects of climate change. Current industry standard is to accommodate the Mid-Range Future Scenario (MRFS) which corresponds to a 20% uplift on extreme rainfall depths. However, the High-End Future Scenario (HEFS) corresponding to a 30% increase should be considered on a case by case basis for certain development such as critical infrastructure or where the consequences of exceedance are high. The implications of any flooding associated with a HEFS event should be examined and understood for all drainage proposals.

3.2.2 Urban Creep

In order to allow for future increases in impermeable area within development sites, for example due to small extensions, patios etc., an increased in paved surface area of 10% should be included in all drainage designs, unless it can be justified and agreed with Cork City Council that this would not apply.

3.2.3 Water Quantity Design Standards

1. Controlling runoff volume

The use of infiltration and rainwater harvesting are important mechanisms for delivering volume control: the greater the volume of runoff that is infiltrated or used on site, the lower the volume of runoff discharged. The acceptable runoff volumes for various rainfall events is described below.

a) Frequent Rainfall Events: The prevention of runoff from the site for the majority of frequent rainfall events (or for the initial depth of rainfall for larger events) is called interception. Interception of at least 5mm of runoff (preferably 10mm should be achieved) from the site should be achieved for frequent rainfall events (i.e., events with <1 year return period). This will ensure run off from the site does not occur for the majority of small rainfall events.

b) Extreme Rainfall Events: The SuDS should be designed so that the volume of runoff discharged from the site during extreme rainfall events (i.e., 1 in 100 year event plus climate change) is controlled. The volume of runoff from the proposed development should not exceed the volume of runoff from the same area pre-development. Where controlling greenfield volumes is considered unachievable, then the runoff volume should be reduced as much as possible and any additional volume should be stored and released at a rate of no more than 2 litres/s/ha or Q_{BAR}, whichever is greater.

2. Control of peak rate of runoff

Attenuation is an important mechanism in controlling peak runoff rates from the development. Peak runoff rates should also be controlled during both frequent and extreme rainfall events.

a) Frequent Rainfall Events: The SuDS should be designed so that peak runoff rates from the site for frequent rainfall events (<1 year return period) that are likely to have an impact on morphology, ecology, or capacity of receiving surface waters are constrained to the greenfield rates of runoff for the same return period or 2l/s/ha, whichever is greater. These events are likely to produce bankfull events for streams and rivers so replicating greenfield runoff rates will protect the morphology and ecology of the receiving area.

b) Extreme Rainfall Events: The SuDS should be designed so that the peak runoff rates for extreme rainfall events are constrained to the greenfield rates of runoff for the same event up to an including the 100 year event with climate change allowance. Site critical duration storm to be used to assess attenuation storage volume.

3. Control of on-site flood risk arising from the surface water management system.

a) The SuDS should be designed so that runoff is completely contained within the designated drainage system and no flooding on site should occur for 1 in 30 year rainfall events unless specifically designated to flood. However, since peak runoff rates will usually require control up to the 1 in 100 year rainfall events, SuDS components may be designed to manage events up to this size.

b) The risks associated with events that exceed the capacity of the drainage system should be evaluated, and the design of the site and the drainage system should be integrated so that flooding is appropriately managed. Properties should be fully protected against flooding from the site drainage system for the 1 in 100 year event including climate change with floor levels at least 500mm above maximum river level and any onsite storage retention. The South Ballincollig Drainage Study Strategic Flood Risk Assessment provides recommendations for minimum finished floor levels of properties to mitigate the risk of fluvial flooding.

4. Exceedance Design

All drainage and SuDS components should be designed to mitigate the risk to people and property in the event of the drainage system becoming overloaded. This risk can be managed by providing suitable alternative flow paths which direct flows away from buildings and other risk areas via roads, carparks, amenity areas and the like. Such flows should generally be routed to the outfall position of each site so that the intended drainage regime is maintained and increased flood risk does not occur outside of the site.

3.2.4 Allowable Discharge Rate

Where infiltration is not feasible, the allowable discharge rate from developed sites can be determined using the IH124 small catchment equation to estimate the existing greenfield runoff rate. The Mean Annual Flood Flow Q_{BAR} is estimated using the following three variable regression equation:

$$Q_{BAR} = 0.00108AREA^{0.89}SAAR^{1.17}SOIL^{2.17}$$

The mean annual flood flow is the flow that can statistically be expected to be equalled or exceeded once in a 2.33 year period. The peak flow for other return periods can be estimated using the FSR 1975 growth curve for Ireland.

Since it is anticipated that all sub-catchments within the plan area will be less than 50 ha, the formula should be applied for 50 ha and the result factored based on the ratio of the actual site area and the applied area (i.e. 50 ha).

The Standard Average Annual Rainfall (SAAR) at this site is approximately 1080mm. The Soil Type is 2 accordance to the FSR WRAP maps therefore the corresponding soil coefficient is 0.3.

The allowable discharge rate from developed sites is summarised on Table 3.2 below. It should be noted that these rates only apply where post-development discharge volume is also limited to greenfield runoff volumes. Where volume is not controlled, the discharge rate should be limited to Q_{BAR}.

Return Period (Years)	Allowable Discharge Rate (l/s/ha)
1 in 1 Year	2.57
1 in 2.33 Years (Q_{BAR})	3.03
1 in 30 Years	4.99
1 in 100 Years	5.93
1 in 100 Years plus 20% (MRFS)	7.12
1 in 100 Years plus 30% (HEFS)	7.71

Table 3.2: Allowable Discharge Rates

3.3 Water Quality

The Water Framework Directive Monitoring Programme became operational in 2006. The introduction of Water Framework Directive has been a key driver in the implementation of SuDS. Groundwater and receiving surface waters are now required to be protected from factors that may compromise its water quality status such as diffuse urban pollution. Water quality therefore is a driving factor for the design criteria of SuDS and the increased use of SuDS is an important means of reducing urban runoff and improving the water quality of that runoff.

SuDS can treat and clean surface water runoff from urban areas so that the receiving environment is protected, while at the same time conveying, storing, and infiltrating surface water to protect flood risk, river morphology and water resources, and delivering amenity and biodiversity value for the development. The level of treatment provided is dependent on the design of the SUDS system proposed.

This improvement in quality of run-off can be achieved by cleaning the flow through a SuDS Management Train or Treatment Train approach. This quality control should be designed to work simultaneously with water quantity design criteria.

There are a number of water quality treatment mechanisms which will be utilized to ensure water quality criteria are achieved in this development. These vary from each SuDS components and often components will be combined in a treatment train type scenario to fully maximise the benefits of each SuDS component. These mechanisms include sedimentation, filtration, absorption, biodegradation, volatilisation, precipitation, plant uptake and nitrification.

3.3.1 Water Quality Design Standards

1. Prevent runoff from the site to receiving surface waters for the majority of frequent rainfall events: Runoff should not be discharged from the site to receiving surface waters or drainage systems for the majority of frequent rainfall events. This is achieved through interception which provides both water quality and water quantity benefits. Interception of at least 5mm, and preferably 10mm should be achieved for frequent rainfall events with a return period of <1 year.
2. Treat runoff to prevent negative impacts on receiving water quality: Runoff should be adequately treated to protect the receiving water body from both short term acute pollution and long term chronic pollution. The treatment processes provided by different SuDS components will have varying capabilities to remove different types of contaminants. Most of the proposed land uses for this development will be relatively low risk and this risk can be mitigated by implementing SuDS components in close proximity to the source of runoff and in sequence if necessary to form a management train type SuDS. SuDS components usually offer a range of treatment processes and, in sequence, deliver gradual improvements in water quality, as well as providing an environmental buffer for accidental spills or unexpected high pollutant loadings from the site.

3.3.2 Water Quality Requirements

The proposed land use is largely residential, educational, and public open space and therefore is considered to be low risk in terms of water quality requirements. Analysis of the effectiveness of the chosen SuDS components to achieve water quality criteria follows the simple index approach described in the CIRIA C753 SuDS Manual chapter 26. As per simple index approach, each SuDS component is assigned a ‘mitigation index’ relative to the three primary sources of pollution listed below: TSS, metals and hydrocarbons. Mitigation indices are added together, and water quality criteria are met if the mitigation index is greater than the risk index.

Where the mitigation index of an individual component is insufficient, two components (or more) in series will be required to ensure mitigation indices are met. Mitigation indices should be reduced by 50% to account for the reduced performance of secondary or tertiary components associated with already reduced inflow concentrations.

Land Use	Pollution Hazard Level	Total Suspended Solids (TSS)	Metals	Hydrocarbons
Residential Roofs	Very Low	0.2	0.2	0.05
Other Roofs	Low	0.3	0.2	0.05
Residential car parks, school car parks, low traffic roads [1]	Low	0.5	0.4	0.4
[1] *For higher volume roads, refer also to "Drainage Design for National Road Schemes – Sustainable Drainage Options", NRA*				

Table 3.3: Land Use Risk Indices

SuDS Component	TSS	Metals	Hydrocarbons
Filter Strip	0.4	0.4	0.5
Filter Drain	0.4	0.4	0.4
Swale	0.5	0.6	0.6
Bioretention System	0.8	0.8	0.8
Permeable Pavement	0.7	0.6	0.7
Detention Basin	0.5	0.5	0.6
Pond	0.7	0.7	0.5
Wetland	0.8	0.8	0.8

Table 3.4: SuDS Component Mitigation Indices

3.4 Amenity

The amenity design criteria presented above should be applied to maximise the amenity value from a SuDS scheme for this development. Amenity may be defined as "a useful or pleasant facility or service". This definition is particularly relevant for describing the multi-functional opportunities associated with SuDS designs. There are many amenity benefits that are intrinsic to SuDS. These benefits often provides improved SuDS performance as amenity benefits can also deliver water quantity, water quality and biodiversity benefits. There are a wide variety of benefits to amenity that can be delivered by SuDS when the afore mentioned design criteria are implemented. These benefits include:

- Air quality improvements.
- Air and building temperature regulation.
- Biodiversity and ecology.
- Carbon emission reduction and sequestration.
- Community cohesion and crime reduction.
- Economic growth and inward investment.
- Education.
- Health and well-being.
- Noise reduction.
- Security of water supply.

- Recreation.

These benefits should meet the afore mentioned design criteria which will ensure a successful SuDS is implemented. The scale in which these benefits will be implemented depends on local requirements, site-specific characteristics, and the proposed land use of the development. The proposed land use includes residential, educational, and public open space land use which should allow ample opportunity to implement a wide variety of amenity benefits.

3.5 Biodiversity

The design criteria for biodiversity have been presented previously. These criteria should be applied to maximise the benefits in which biodiversity can contribute to this SuDS scheme, the proposed development, and the wider environment. The extent to which each biodiversity design criterion can be addressed by the designer will depend on local requirements, site-specific characteristics, and proposed land uses. Both amenity and biodiversity design criteria should be considered together and at an early stage and fully integrated into the design process in order to maximise the opportunities that can be achieved by the scheme at no or minimal extra cost.

Design of the new environment will effectively see a greater human population and reduction of arable lands. Human wellbeing and ecological sensitivities will therefore need careful consideration. The human aspects relate to places and spaces to relax and exercise safely and reside in an environment with a low risk of flooding, taking account of climate change. Ecological sensitivities relate to the flora and fauna (biota) that occur in the proposed development lands and potential future biota. Key principles required for proper rollout of an ecologically friendly environment are:

Biodiversity Net Gain (BNG): BNG is development that leaves biodiversity in a better state than before.

Use of Nature Based Solutions: The International Union for the Conservation of Nature (IUCN) defines Nature Based Solutions (NbS) as “actions to protect, sustainably manage and restore natural or modified ecosystems, which address societal challenges effectively and adaptively, while simultaneously providing human well-being and biodiversity benefits”.

These principles can be achieved by utilizing riparian areas and buffer strips and the “River Continuum Concept”. Buffer strips stabilise banks along watercourse and prevent/reduce nutrient/sediment entry whereas a bare strip of earth/concrete will do little in preventing sediment from entering a watercourse. Additionally, a good buffer assists bank protection and limits bank slippage, thereby helping to keep the channel clear. The provision of such buffer strips would:

- Provide bank stability and prevent excessive erosion.
- Preserve water quality by filtering sediment from runoff before it enters the river.
- Provide an undeveloped flood plain to accommodate flood waters during extreme flooding events.
- Provide food and habitat for fish and wildlife.
- Partially shade the channel. This provides a camouflage effect for fish and helps to reduce high summer temperatures.
- Provide decaying vegetation in the channel which is a food source for certain macroinvertebrates.

A complex unbroken buffer strips function best so high species diversity and continuity is desired in the plan. Runoff from roads and hard surfaces should be directed to attenuation areas (settlement ponds) which then feed dedicated integrated constructed wetlands (ICWs) next to the watercourse that flows through the site. There should be a connection between these ICWs and the watercourse in times of flood.

The "River Continuum Concept" by Vannote et al. (1980) describes the ecological function of rivers as linear ecosystems and the effects of interruptions of their connectivity. Any river enhancement proposals should be considered in cognisance of the river continuum concept and would need to be informed by surveys at locations upstream and downstream of the site. The following guidance should be consulted in relation to stream enhancement:

- Channels and Challenges - the enhancement of salmonid rivers (O'Grady, 2006)
- River Enhancement Programmes in Ireland (O'Grady et al, 2017).

The continuity of the semi-natural landscape to the south should permeate into the proposed development site by adopting the following measures:
Using only native species, preferably sourced from the locality when planting

- Using species suitable for the soil conditions e.g. willow species and hazel in low-lying poorly drained areas
- Connectivity of hedgerow and treeline habitats along walkways and watercourses i.e. the fewer dead ends the better, so loops are desired
- Limited and ecologically sensitive lighting
- Soft engineering practices should be used for watercourses, wetlands and swales. Use of concrete in the environment should be minimised insofar as possible
- Habitat creation and repair should follow recognised and proven literature and practices.

4 Site Characterisation

4.1 Site Characteristics

The various site characteristics which influence SuDS techniques are outlined below.

4.1.1 Area Draining to SuDS Components

The subject lands comprise of approximately 220ha in total with varying proposed land uses such as residential, educational, and recreational land uses. The catchment area draining to the Maglin river is relatively small compared to the Plan Area. Consequently, changes to the pre-development runoff regime have higher potential to adversely impact downstream flood risk, morphology and ecology.

4.1.2 Topography & Slope

The topography of South Ballincollig is relatively flat flanked on the north and south by small, east-west trending hills. The subject lands are situated south of the River Lee, which is prone to flooding. Elevation within the site was found to be between 20 – 30m AOD, with an average of 23m AOD. The site has a slight dip from west to east as ground level approaches sea-level within Cork City. Several rock outcrops have been identified within the site (*Figure 4.3*) which illustrates these outcrops have an average elevation of 22 – 25m AOD.

Figure 4.1: Topography of Plan Area

4.1.3 Existing Drainage Paths & Flow Routes

The existing drainage paths and basic flow routes within the Plan Area have been established and are illustrated on Figure 4.2 below. There are a number of existing natural and artificial flow paths which should be replicated insofar as possible in the detailed design of the overall plan area and on a site specific basis. Effective management of the existing drainage regime should be achieved through the use of an appropriate SuDS design.

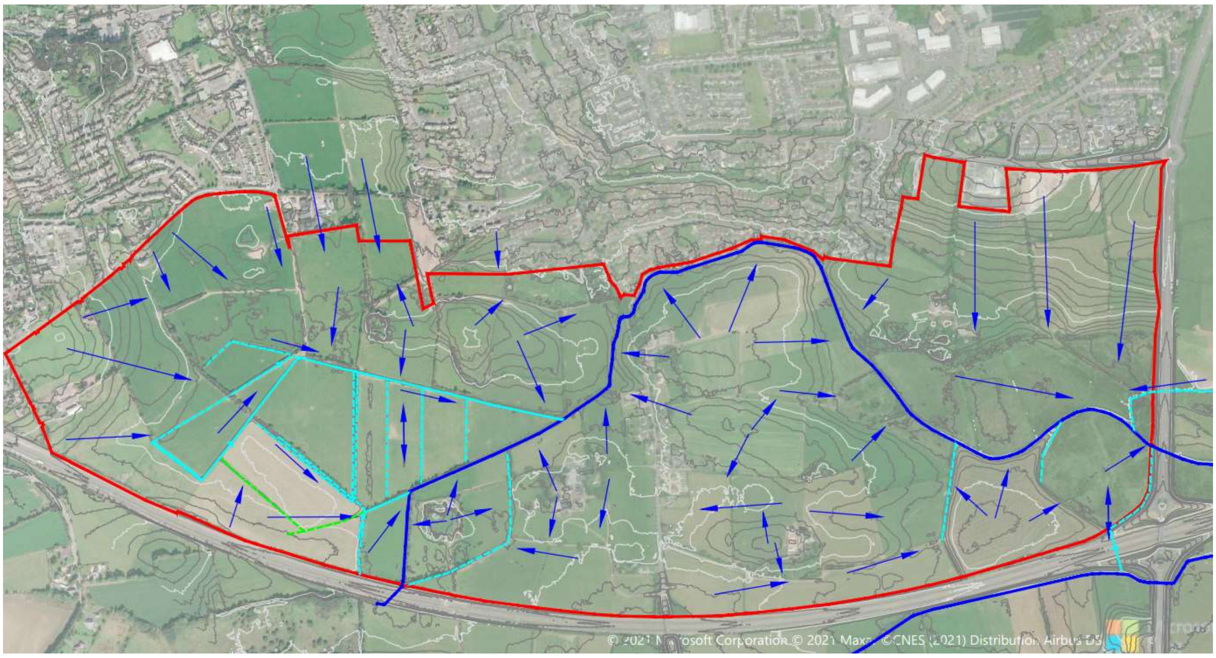


Figure 4.2: Existing Drainage Paths & Flow Routes

4.1.4 Solid & Structural Geology

The bedrock geology of the site is primarily composed of Carboniferous Limestones. Located within the Cork Syncline of the Munster Basin, the site lies within a limestone valley flanked by the north and south by sandstone ridges. This area of the Munster Basin has been heavily folded and subsequently eroded which has produced a series of east-west trending anticlines and synclines that have influenced the local topography. Sandstones are found at higher-elevation anticlines while limestones are found at lower-elevations synclines due to differential erosion as the limestone was eroded at a quicker rate than the sandstone.

Bedding on either side of the Cork Syncline is steeply dipping at an average of 55° to the south on the northern limb, and 60° to the north on the southern limb. The Cork Syncline axis runs directly through the northern boundary of the site.

There are two sets of cross-cutting faults that have been identified by GSI mapping: the first strikes east to west which appears to follow strike of bedding, the second strikes north to south.

The bedrock found within and immediately adjacent to the site are described from literature below with the symbol for each formation given in brackets for cross-reference purposes with the GSI 1:100,000 scale bedrock geology map.

Little Island Formation (CDLITT). Described as Carboniferous massive calcilutite limestones (mudbank facies) and crinoidal calcilutites. The formation is a uniform succession of crinoidal biomicrite limestones (wackestones) and massive unbedded calcilutite limestones of mudbank facies. The top of the formation is highly crinoidal and passes up to poorly bedded wackestones. The formation is estimated to be up to 500m thick.

Waulsortian Limestones (CDWAUL). Described as Carboniferous massive, unbedded limes mudstones. Sometimes informally called "reef" limestones, although inaccurate. Dominantly pale grey, crudely bedded or massive limestone. Known to be moderately to intensely karstified. Typically, 300 - 500 m thick.

Cuskinny Member (CDKINS2). Described as Carboniferous flaser-bedded sandstone & mudstone. The member is sand dominant and characterised by alternations of flaser-bedded sandstones, lenticular-bedded (linsen) mudstones, massive sandstones, and nodular mudstones. Thin quartz-pebbly sandstones and conglomerates also occur throughout the member. Thickness can vary between 199-235m.

Castle Slate Member of Kinsale Formation (KNcs). Described as Carboniferous uniform well-cleaved dark-grey slaty mudstones. The member consists of uniform dark grey, well cleaved massive mudstones. Comminuted crinoidal debris is common in some beds as are phosphatic nodules and disseminated pyrite. Rhythmic upward grading from sediment of medium silt size to fine silt and mud. Thicknesses of 5 – 62m have been reported.

Old Head Sandstone Formation (DUOHSF). Described as Devonian grey flaser-bedded sandstones, fine grained sandstones and minor mudstones and lenticular bedded mudstones. The formation is dominated by lithologies belonging to the Heterolithic Facies (mainly flaser-bedded sandstones), wavy bedded fine-grained sandstones and minor mudstones. Thickness has been reported as between 899-1098m.

The bedrock geology within the subject area is dominated by Carboniferous limestones in both the northern and southern half of the subject area with carboniferous sandstones and mudstones located at the very southern edge of the subject area. There are also a number of bedrock outcrops located within the subject area.

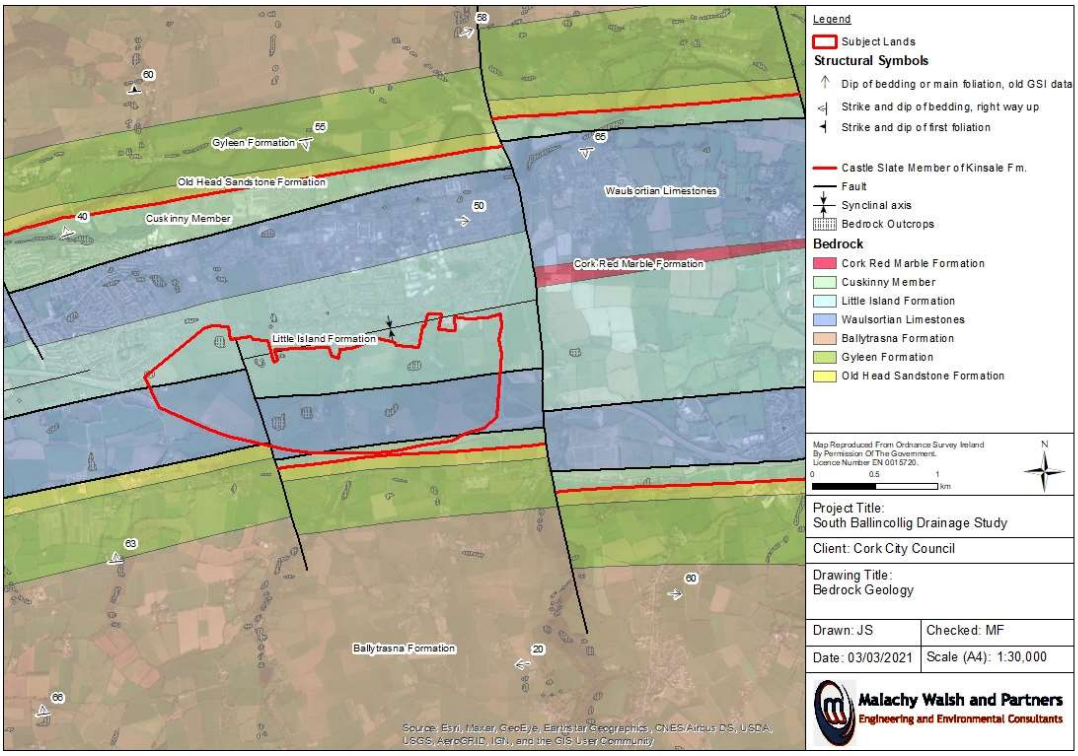


Figure 4.3: Bedrock Geology of Subject Area

4.1.5 Quaternary Sediments and Glacial Geomorphology

The subject lands are primarily underlain by till derived from Devonian sandstones with some areas exhibiting bedrock outcrop or subcrop in the central areas of the site. The northern boundaries are bordered by urban/made ground. The southern boundaries are bordered by gravels derived from Devonian sandstone in the form of a glaciofluvial terrace. A terrace is a step-up feature formed from glacial meltwater depositing sediment.

The presence of till is further supported by previous ground investigations that have been performed in close proximity to the study area.

Tills are often described as unsorted to poorly sorted diamicton which contains clays, silts, sands, and gravels. Glaciofluvial materials are often well sorted and contain larger clasts, in this case gravels and possible sands.

Meltwater channels have been noted to flow towards the subject lands from the north and the south.

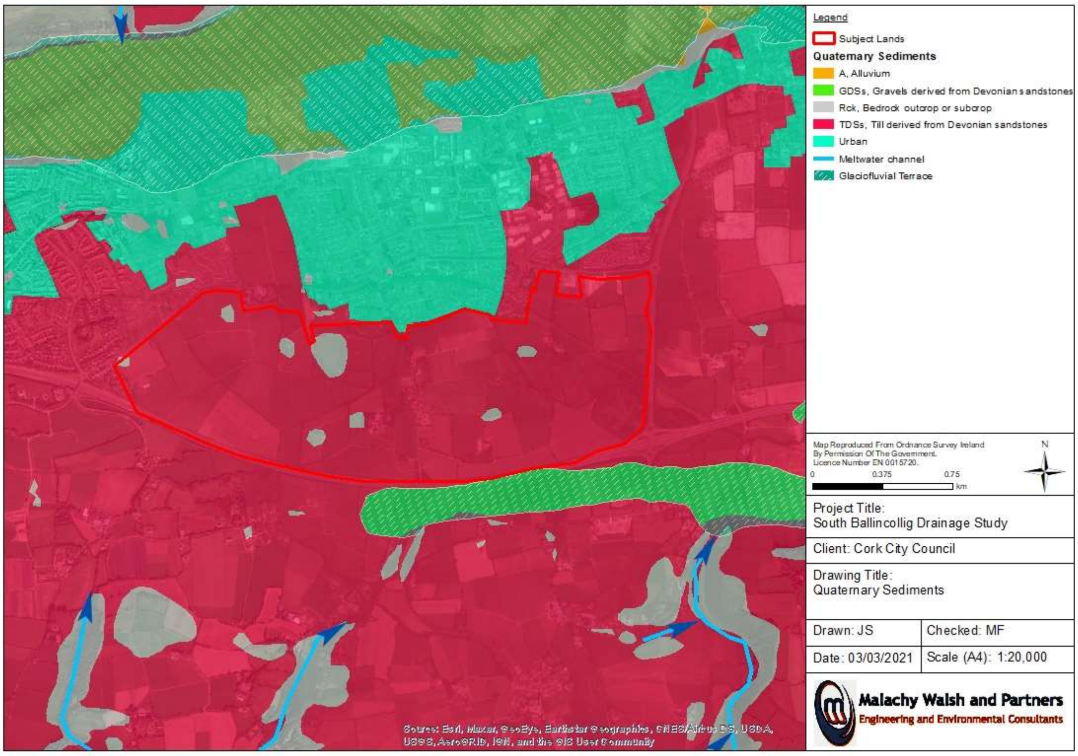


Figure 4.4: Quaternary Sediments of Subject Area

4.1.6 Soils

The site is primarily composed of "AminDW - Deep well drained mineral (Mainly acidic)" sourced from Devonian Sandstone till.

Several areas are classed as "BminSW - Shallow well drained mineral (Mainly basic)" soils, sourced from Carboniferous Limestone parent materials and subcrop of the limestones beneath.

Several areas within the centre and east boundaries of the site are recorded as "AminPD - Mineral poorly drained (Mainly acidic)", sourced from Devonian Sandstone till and contain groundwater gleysol. Gleysols are hydric soils in which they are saturated by groundwater for long periods of time. It appears these AminPD soils are associated with the Grange stream as they are on the banks of the river itself.

Some smaller sections of the northern boundary are classed made/urban ground.

Further site investigation will be required to determine both soil and bedrock characteristics. This may require carrying out infiltration tests to determine the infiltration potential of this site. Infiltration tests measures the rate at which water soaks away from the test pit and gives an infiltration rate. These are carried out in trial pits or boreholes if depth or access restraints are encountered.

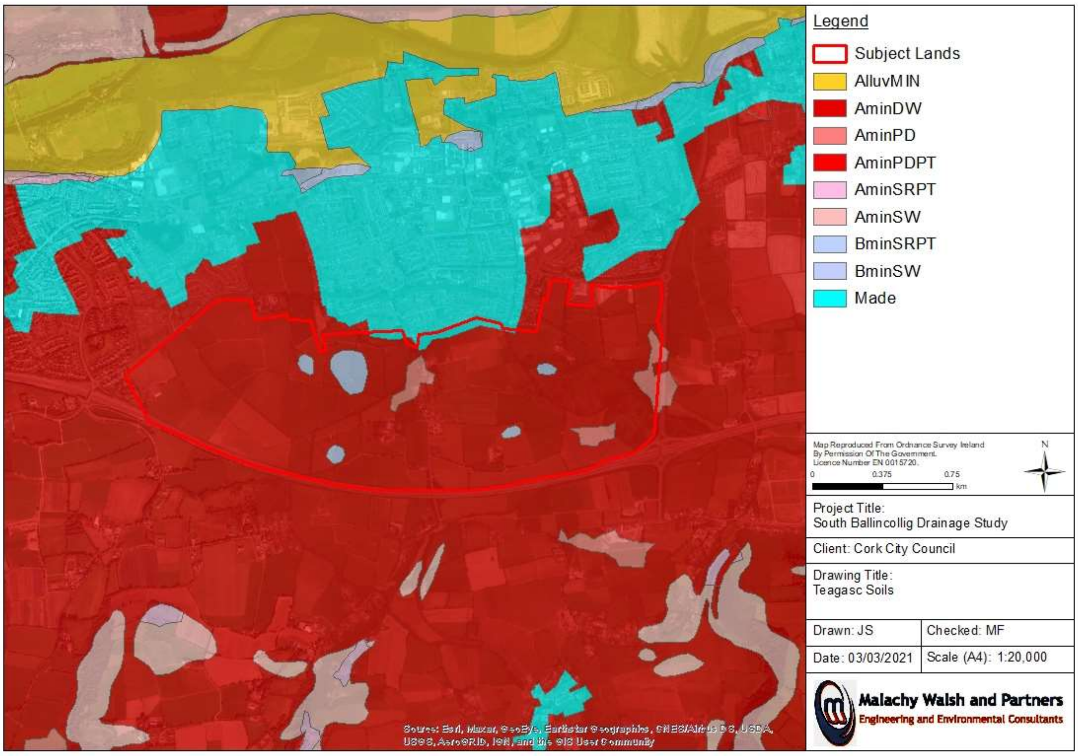


Figure 4.5: Soils of Subject Area

4.1.7 Available Space

Given the extent of undeveloped land within the plan area, there would be ample space available to incorporate SuDS features as part of all future developments.

4.2 Catchment Characteristics

4.2.1 Hydrogeology

The vast majority of the site is situated within a "Rkd - Regionally Important Aquifer - Karstified (diffuse)", and "Ll - Locally Important Aquifer - Bedrock which is Moderately Productive only in Local Zones" within a small portion of the southern boundary. "Rg- Regionally Important Gravel Aquifer" has been noted north, northeast, and east of the site.

There are two karst features in close proximity to the site which have been noted, namely Coleman Cave (GSI reference 1405NEK001) which can be found approximately 2.5km west of the western boundary, and a Curraheen Spring (GSI reference 1405NEK002) 1.5km east of the eastern boundary.

There has been anecdotal evidence of a possible spring located within the subject lands known as the "Balleen" spring in the western portion of the site, however this is yet to be identified.

This possible karstification within the site is due to acidic rainfall percolating through the fractured rock and chemically weathering it. Overtime, this has produced complex groundwater flow paths through open cavities and fractures in the bedrock, which can penetrate through the bedrock for tens of meters. Karstification within the Carboniferous limestones has occurred since the Jurassic period, and many of the cavities have since been filled with Pleistocene glacial outwash sands and gravels (Meere et al., 2013).

Historic maps show evidence of flooded limestone quarries, lime kilns and pits which may be connected to the bedrock aquifers and may be hydrogeologically connected to karst features.

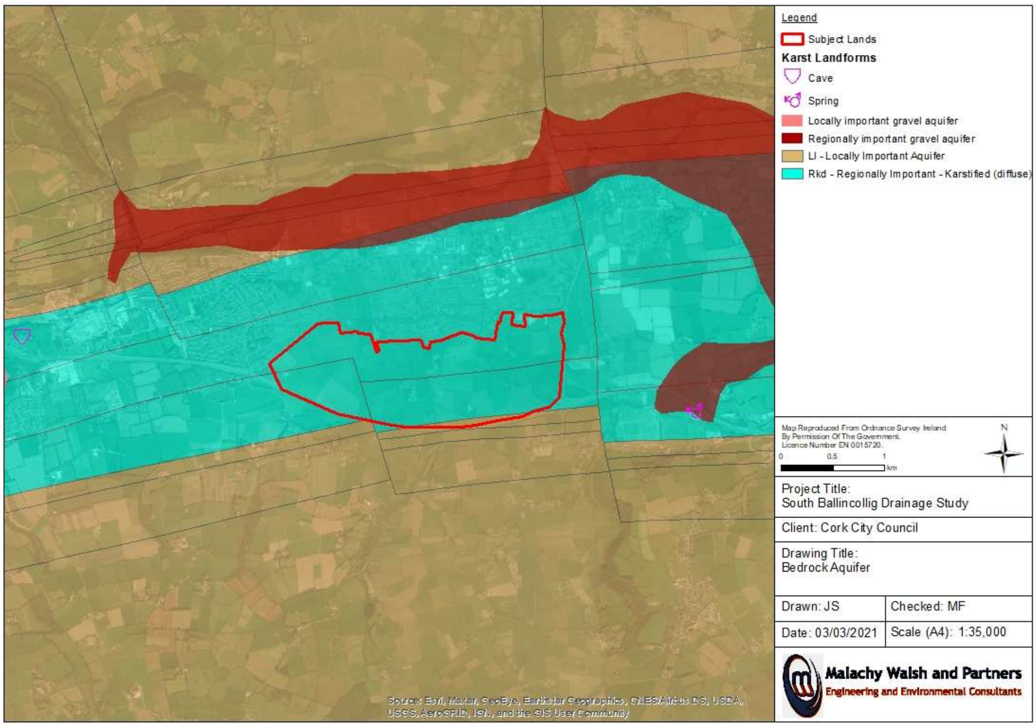


Figure 4.6: Aquifer Map of Subject Area

4.2.2 Groundwater Vulnerability

The GSI groundwater vulnerability map can be seen in *Figure 4.7*. The western and central portions of the subject land have been categorised as "high" and "extreme" groundwater vulnerability, while the eastern portion has been categorised as "moderate" groundwater vulnerability. Areas that exhibit rock outcrop or subcrop have been categorised and "rock at or near surface or karst".

Fig

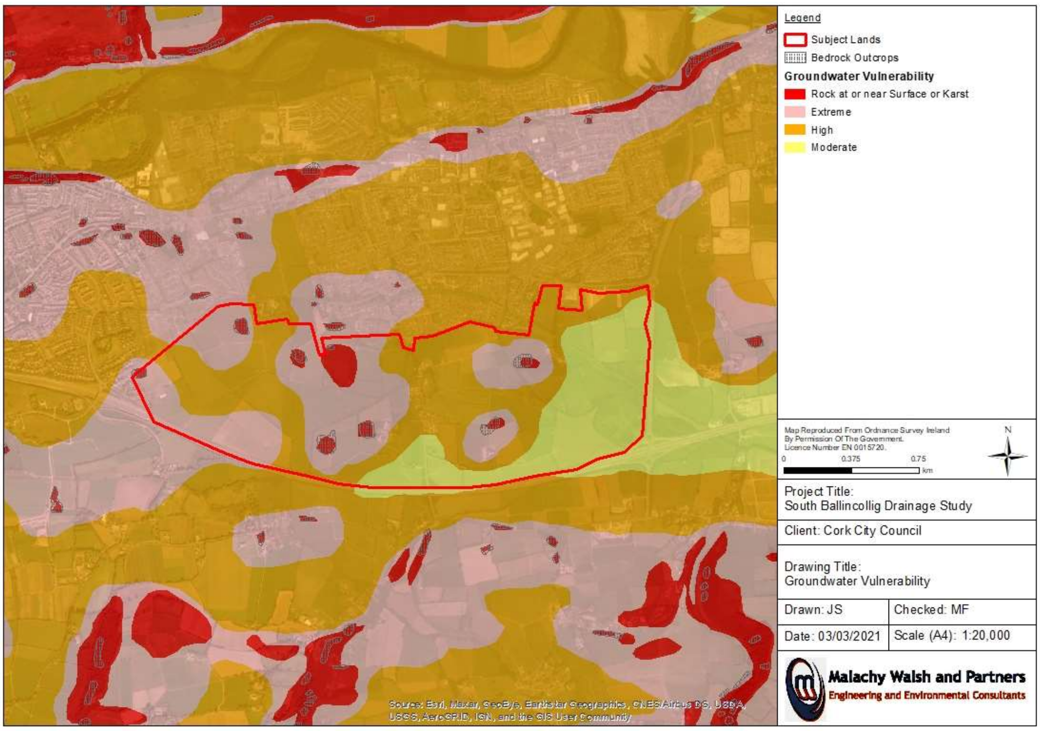


Figure 4.7: Groundwater Vulnerability of Subject Area

4.2.3 Subsoil Permeability

The GSI groundwater subsoil permeability map classifies how water can infiltrate downwards into the surface at a given point with relative ease (*Figure 4.8*). There are large areas within the subject lands that have not been mapped for groundwater subsoil permeability. This is likely due to its relation to the Groundwater Vulnerability map where rock outcrop and subcrop have been identified. However, the majority of the site is mapped as "Moderate", and it can therefore be assumed that unmapped areas will also have moderate subsoil permeability due to similar subsoils and topsoil constituents. There is an area of "high" subsoil permeability immediately south of the southern boundary, which are associated with the Quaternary "Gravels derived from Devonian sandstone" that extend to the east.

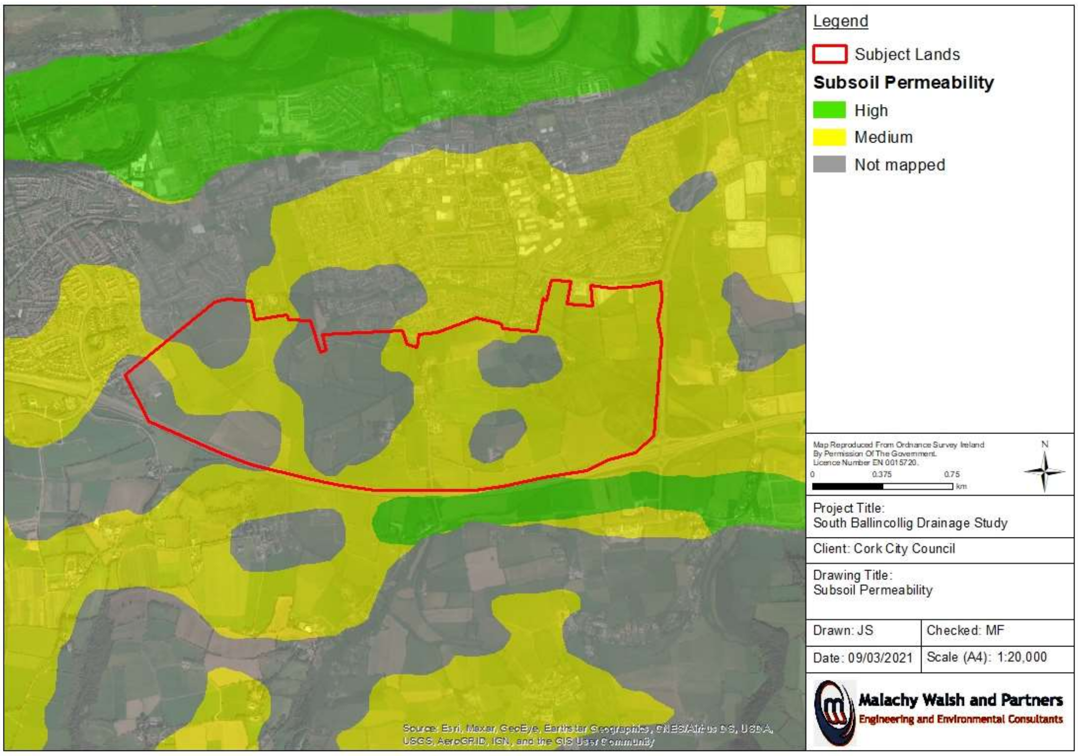


Figure 4.8: Subsoil Permeability of Subject Area

4.2.4 Strategic Flood Risk Assessment

Figure 4.9 illustrates the predicted pre-development flood extents within the Plan Area for both the 1% AEP and the 0.1% AEP current scenarios. This prediction has largely influenced the zoning of public open space as this type of space is more suitable for areas which may be subjected to flooding when compared to that of residential zones. The development designs should take full account of the potential impact fluvial flood levels could have on the selection, siting, and design of suitable SuDS.

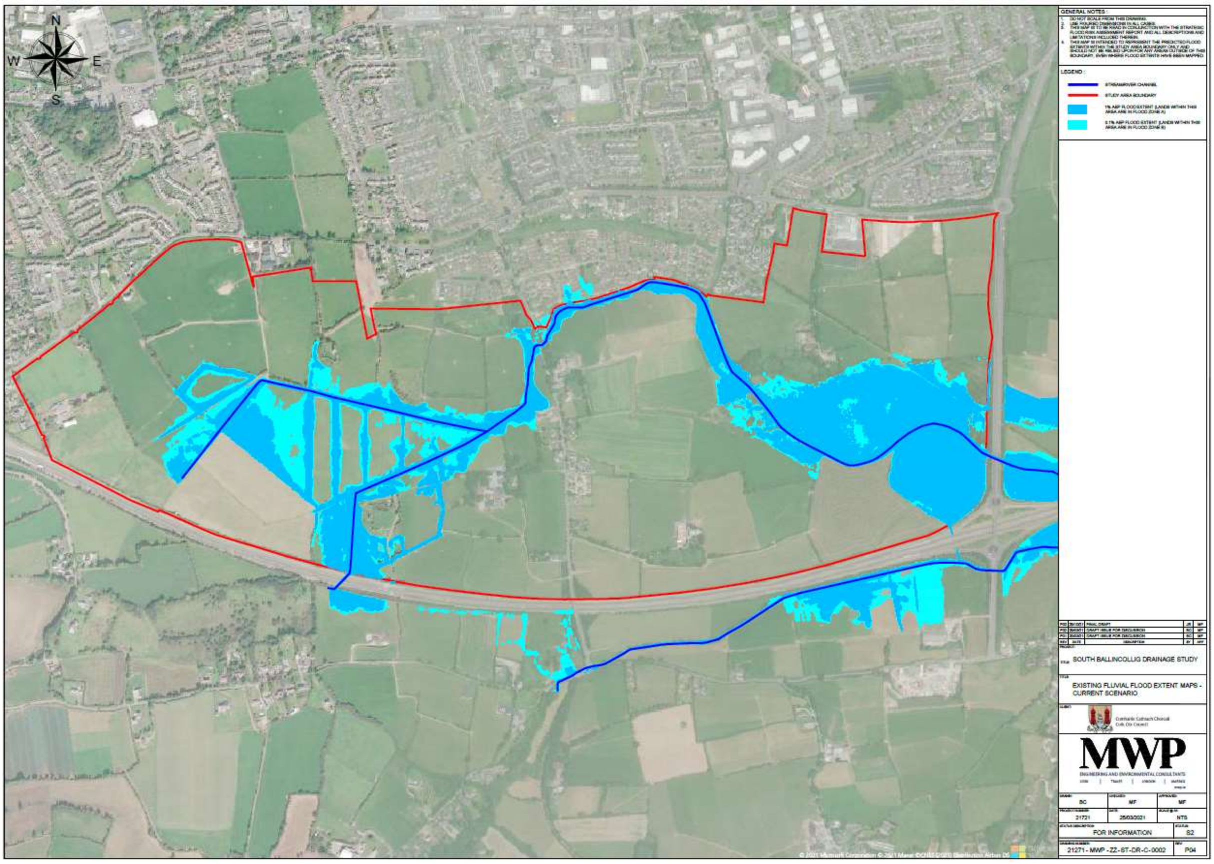


Figure 4-9: Flood Risk Assessment of Subject Area

4.3 Key Challenges Posed by Subject Lands

4.3.1 Potential High Groundwater Levels

When designing a surface water management system for a site that overlies high groundwater levels (i.e., maximum likely groundwater levels are within 1m of the base of the SuDS component) the following challenges should be considered within the design process:

1. The use of infiltration may not be suitable due to reduced hydraulic and treatment capacity.
2. If SuDS are constructed below the maximum likely groundwater level, then groundwater can potentially enter the SuDS component and reduce the storage capacity.
3. Flotation and structural design risks to storage structures or impermeable liners can occur because of the extra loads imposed by the groundwater and the buoyancy of the tanks or liner.

Two areas in particular have recorded high hydraulic head readings, these areas are located in the western extent of the subject lands and the most south-eastern extent of the subject lands' boundary. These high head levels may trigger some groundwater flooding during a very high and/or several concurrent rainfall events. However, given the length of time between the data collected, the time of year these readings were taken, and the change in environment since these recordings were made, further site investigation is needed to confirm if these areas are at risk. It should be noted that these at-risk areas are in close proximity to surface water features, so the likelihood of flood events occurring will be fluvial. Further site investigation is needed to assess the risk of a high groundwater level as these conclusions have been developed from water strike data collected in 1995 before the construction of the Ballincollig bypass.

The preferred and most sustainable mechanism of infiltration may not be suitable if the groundwater levels within a particular area are found to be within 1m of the infiltration surface. Groundwater conditions such as this compromises the pollution protection that is normally provided by soil of sufficient depth to the groundwater level. A high groundwater level can potentially directly pollute groundwater if the groundwater is hydraulically linked to water within the SuDS. This issue may be avoided through careful use of land raising. However, this will require advice from a ground engineering specialist, to ensure that the infiltration capacity and risk of settlement or instability is acceptable. Depending on the depth of groundwater below the site it may be possible to use shallow infiltration basins or permeable pavements. It is recommended that if groundwater levels are assessed to be high in a particular area that alternative methods to infiltration be considered that utilize shallow or surface SuDS components.

It is important to keep storage and conveyance systems above maximum likely groundwater levels, wherever possible. This will avoid difficulties during construction caused by water flows into excavations and will ensure that the hydraulic and treatment capacity of the SuDS component is retained at all times. It is very difficult to completely seal a geomembrane around a geocellular tank or permeable pavement, and any water ingress can impact on the available internal hydraulic storage capacity. The use of tanks with membranes below groundwater is not recommended. It should also be noted that surface linear channel systems cannot be assumed to be completely sealed and, if they extend below the groundwater table, then water will leak into them.

It is also important to avoid locating storage tanks or lined sub-base systems below the maximum likely groundwater level, if at all possible. There are two reasons for this:

The lateral loads on the side of tanks increase significantly if groundwater is applying pressure to the side of the tank and will therefore impact on the structural design of the system.

Buoyancy of the tank or lined sub-base can cause uplift failure of the system. Flotation should be prevented by having sufficient counter force, which will be derived from the self-weight of the tank construction and the weight of permanent backfill over the top of the tank. Shallow 150 mm deep attenuation tanks have been successfully used below concrete slabs where groundwater was very close to the surface. In some cases where it has not been possible to resist uplift forces by dead weight alone, extra resistance has been provided using anchorage systems.

If a high groundwater table has been determined then SuDS selection will focus on surface and shallow features that avoid infiltration. Some SuDS components (e.g., permeable pavement, swales etc) that usually allow infiltration may possibly still be suitable if used in conjunction with an impermeable liner (such as a waterproof membrane or compacted native clay) to prevent infiltration. The minimum clearance between the base of infiltration SuDS and peak seasonal groundwater may be reduced if a risk assessment shows it is acceptable to do so.

4.3.2 Flat Topography

The topography of the majority of the land in the subject area is relatively flat. This can provide challenges when designing a SuDS System. These challenges relate to issues achieving sufficient gradients to drain runoff effectively and difficulty in meeting outlet levels for existing piped systems or watercourses.

On flat sites differences in elevations between piped drainage systems and SuDS components can lead to a number of issues. Systems which do not have sufficient falls can lead to a reduction in self cleansing velocities which can affect system performance, shallow or high capacity linear SuDS components may provide a solution to this issue. If even shallow SuDS components end up below the minimum allowable outfall levels then this may require the use of pumping. However, pumping should always be considered a last resort and only permitted when guaranteed maintenance of the pumping systems can be ensured.

On flat sites, or indeed any site where a storage tank or layer has a flat base, ponding of water may occur in the base of the storage. If this occurs in systems that are not lined (where the soils are impermeable and do not therefore allow infiltration), the water could be in contact with the underlying soils for a significant length of time and could ultimately reduce the strength of the soil. If possible, water ponding should be avoided. If this cannot be provided, then the reduction in strength due to waterlogging should be considered in the structural design of any SuDS components.

In general, to overcome the challenge of flat topography it is recommended that surface water runoff should be managed using shallow or surface SuDS components that are utilized as close to its source as possible (e.g., swales, pervious pavements). Good SuDS design should manage the site into small sub-catchments and provide source control.

4.3.3 Unstable Subsurface/Potential Karstification

This site potentially has areas which are prone karstification as the subject area is underlain by carboniferous limestone bedrock. This may lead to instability issues within these areas if water is encouraged to infiltrate these areas. Further site investigation is recommended to assess the risk of unstable/karstified subsurface. A number of issues can arise when implementing a SuDS scheme on a site that may be prone to subsurface soil or rock instability. These issues arise mainly due to the effects of SuDS components that encourage water infiltration. Water infiltration into the ground can cause instability in poorly consolidated soils because the water can wash out the soil or cause it to compact. Also, rocks such as the limestone and sandstones found in this area may slowly dissolve over time. Water from infiltration systems can also cause slope or retaining wall failure, because water pressure may increase in the soil behind the slope or wall.

SuDS for sites where there are soils or rocks prone to instability should not use infiltration as a runoff destination, and it may be necessary to line systems to prevent any water infiltrating the ground. This will depend on the risks associated with any potential instability and the likely volume of water discharging to the infiltration device when compared to natural exposure to rainfall on the same area. A small amount of infiltration to provide interception may not be a problem. An appropriate risk assessment may need to be carried out on a case by case basis. The use of SuDS in such areas with rock/soil instability risks will require advice from a geotechnical specialist.

4.3.4 SuDS on Floodplains

The land that has been zoned for public open space is mainly situated on the floodplain of the Maglin river or has been predicted to flood during a 1% or 0.1% AEP flood event. The role of a floodplain is primarily to mitigate flood risk from rivers or tides, and during extreme events these areas will naturally experience flooding, making them ineffective for use in storing surface water runoff. All storage volume should normally be provided within the development footprint, excluding volumes stored on the floodplain. The presence of a floodplain, however, should not restrict the site from including SuDS, as they could still be effective in managing routine rainfall, and runoff may need to be discharged safely across the floodplain. Water quality treatment may also be provided. SuDS in the floodplain may also be acceptable in terms of providing treatment for frequent events. The design of those parts of SuDS in a floodplain should not reduce floodplain storage or conveyance.

SuDS should be selected and designed taking into consideration the likely high groundwater table and susceptibility to erosion during periods of high flows/water levels. Surface discharges from SuDS should be dispersed (allowed to shed off as sheet flow), and point discharges minimised or eliminated.

4.3.5 Designing for Roads/Highways

Ballincollig-Carrigaline LAP includes a new spine road through the expansion area which crosses the Maglin River at a number of locations. There may also be a number of low trafficked roads constructed in conjunction with proposed residential and education zones. The drainage of roads using SuDS is a design application scenario that has specific challenges owing to the need to protect the road pavement from damage and ensure that extra safety risks are not introduced by the design of the drainage system. Road design requirements will also be a key influence, and there needs to be co-operation between road and SuDS designers to achieve the most effective schemes.

If SuDS components provide multiple uses and drain both the road and adjacent parts of the development, it will be more efficient and it is more likely that the drainage system will be approved and adopted by the drainage approving body. Discussions should be held early in the design process to ensure that SuDS for roads are designed to the standards which achieve these requirements.

Roads can also be designed or modified to maximise their exceedance flow conveyance or storage capacity, but great care is needed to ensure that the flows do not cause problems further downstream and do not represent a hazard to vehicles and pedestrians.

There are several SuDS components that are particularly suitable for draining roads. These include:

Pervious pavements: Often used for low trafficked roads, particularly in residential areas. These can include permeable paving, porous asphalt, pervious concrete, or reinforced grass systems.

Swales: Swales are an extremely useful method for draining long stretches of road where the road is close to existing ground level and there are few buried services alongside or crossing the road.

Bioretention systems: Bioretention systems can be fitted within road build-outs as traffic calming features and within dead space in car parks or turning areas, providing amenity and biodiversity benefits within urban areas.

Detention Basins/Ponds/Wetlands: These components may be suitable for roundabouts or junctions and are also used extensively on the motorway and trunk road network, where there is space in open countryside.

4.3.6 Designing for Urban Environment

There is a small area zoned as urban town centre in this proposed development. For successful integration of SuDS into urban design, early consideration of surface water as part of an integrated design process is required. This involves a collaborative, interdisciplinary design team that brings together developers, engineers, planners, landscape architects, architects, ecologists, and the local community. Sites within urban areas are often confined and restricted.

Planning and design constraints are often tighter than at other sites, and land is often more valuable. Introducing SuDS can appear challenging when faced with competing development objectives, but SuDS can be integrated into a development without negatively impacting upon the primary function of the urban space. Opportunities for the creation of SuDS can be found in even the smallest spaces, and a perceived lack of space is not a justifiable reason for not using SuDS.

4.3.7 Site Constraints Summary

Table 4.1 summarises the main constraints of the site which will inform the development of the SuDS design.

Attribute	Description	Influence on SuDS Design
Fluvial Flooding	The SFRA assesses the flood risk within the site. Parts of the site have been identified as being at risk of flooding. The areas at risk follow the path of the Maglin River through the site and are illustrated on the SFRA map.	The siting and design of SuDS should take cognisance of the predicted flood extents and levels.
Karstification	The site has a number of physical constraints in the form of a karst limestone underground cavers system associated with an extensive central area at risk of flooding which form tributaries to Maglin Steam, running west to east.	Areas that experience karstification may limit the feasibility of infiltration solutions. Further ground investigation is required to confirm this constraint.
Topography	Elevation within the site was found to be between 20 – 30m AOD, with an average of 23m AOD. The site has a slight dip from west to east as ground level approaches sea-level within Cork City.	Flat topography influences existing and proposed flow routes and should be considered at design stage of SuDS components.
Groundwater Flooding	Groundwater flooding was not identified as a significant risk in the plan area.	This is based off relatively old data and further monitoring is warranted.
Archaeological Heritage & Architectural Heritage	There are 8 archaeological features recorded on the site as shown on the map. There are 4 Fulacht Fia's, 2 Lime Kilns, a standing stone, and a country house Maglin House which are all protected. Ballincollig cave pNHA 001249 is the only proposed Natural Heritage Area in the site. Most notable of the features is the Ballincollig Castle, which is a protected structure (RPS 467). It is important that any proposed development does not diminish the importance of the castle in the landscape and that some of its original purpose as a look out over the surrounding landscape is maintained.	Ballincollig Castle requires a 100m buffer around the castle. SuDS features should not impact on protected structures/recorded monuments.
Land Use	The site is largely undeveloped agricultural land that has been zoned for residential, educational, public open space and also contains a proposed spine road.	Proposed land use is relatively low risk to water quality requirements with the exception of the proposed spine road.
Groundwater Levels, Soil Permeability & Infiltration Potential	Groundwater levels are not confirmed however it is anticipated that certain areas of the site could experience high groundwater. GSI mapping indicates soils would have medium to high permeability.	Solutions relying on infiltration may be unsuitable where high GW levels occur. Where high GW is not present, soil permeability should be sufficient to accommodate infiltration. Further investigation required.

Table 4.1: Site Constraints

5 SuDS Components & System Guidance

5.1 The Management Train

The SuDS philosophy, and effective stormwater management in general, requires a series of SuDS features, linked together, to form a stormwater management system to treat and attenuate surface water runoff as close to the source of runoff as possible, before being conveyed downstream for further treatment and storage. The individual components described below do not constitute SuDS, if applied in isolation. As indicated on Figure 5.1, the management train essentially comprises of four stages: Prevention, Source Control, Site Control and Regional Control.

Except where otherwise agreed with Cork City Council, developers are required to address all necessary stages within individual development sites. It is not currently proposed to use shared regional controls for development sites within areas zoned as Public Open Space. Regional Controls provided for the overall development are intended to cater for the main link road and where possible to provide additional treatment over and above SuDS compliant designs for individual sites.

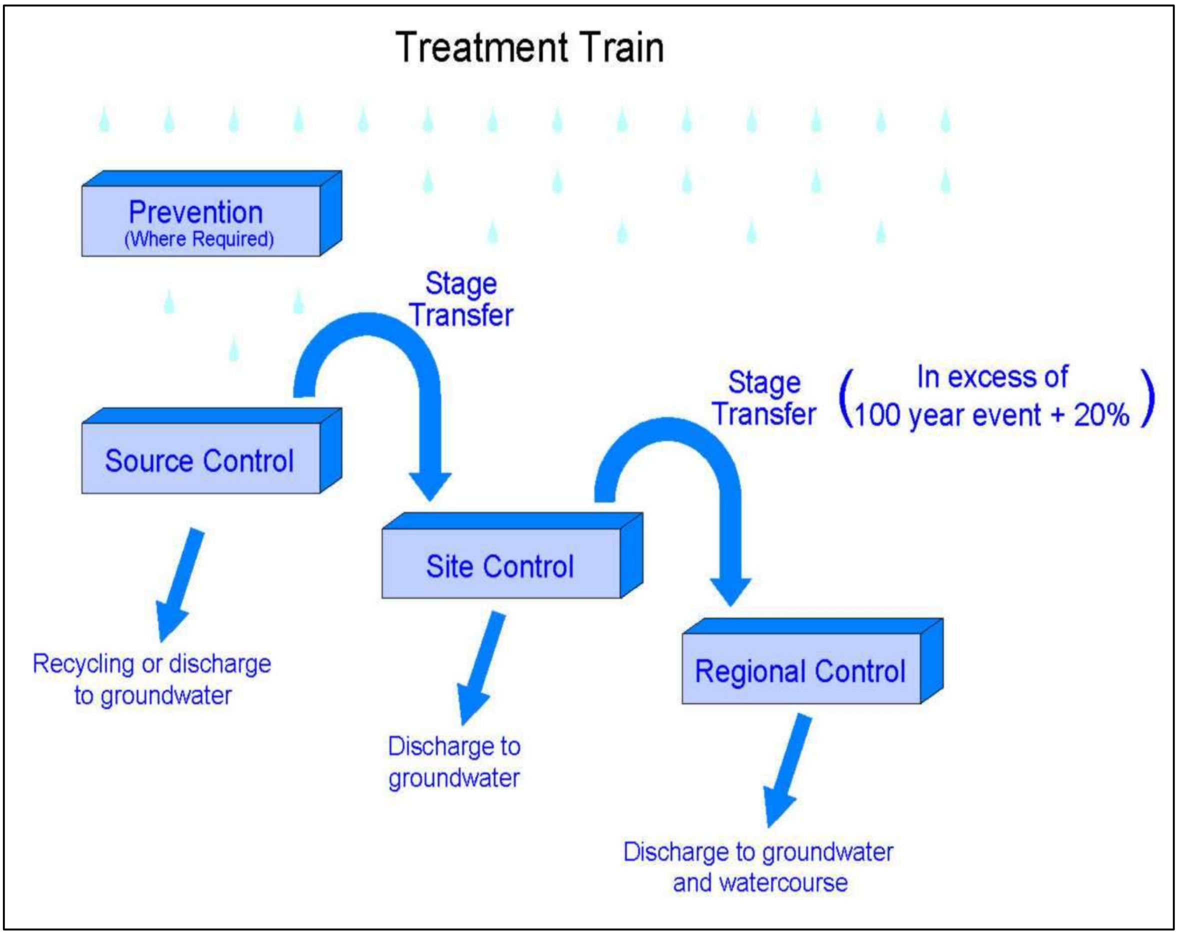


Figure 5.1: SuDS Management Train

Prevention: Prevention measures are required to guard against peak runoff and immediate threat of pollution and water contamination by chemicals and other pollutants. This can be achieved using a combination of methods including rainwater harvesting, permeable paving, and soakaways, diverting small roof areas to grassed areas.

Source Control: The prevention measures if required, should be closely linked to source control which provides interception stages to manage run-off as close as possible to where it falls as rain. Roof water runoff should be captured and treated within the curtilage of each individual site. This could be typically achieved by using SuDS measures such as green roofs, rain gardens, filter trenches or bioretention systems. All other areas such as residential driveways, or car parking areas and access roads in commercial sites will require two stages of SuDS treatment. This could be typically achieved using SuDS measures such as improved swales, permeable paving and bioretention systems. Where two stages of SuDS treatment cannot be achieved at source, site control SuDS should be considered.

Site Control: Where volumes of runoff exceed the capability of source control measures, site control measures are provided such as detention basins, ponds, permeable paving areas, vegetated low flow channels, planted areas and swales which will provide additional cleaning of run-off and ensure that surface water is managed within the site.

Regional Control: Regional control shall be provided for the link road which will be designed for storage and treatment and may provide some additional storage for residual run-off from individual developments in excess of the 100 year event (+20% for MRFS).

The principle of the Management Train is that wherever possible, surface water should be managed locally in small, sub-catchments as close to source as possible rather than being conveyed to and managed in large systems further down the catchment. Different SuDS components may be able to perform at multiple source/site/regional control levels. Drainage designs for developments should follow the sustainability hierarchy summarised on Table 5-1 below.

Sustainability Level	SuDS Component	Water Quantity	Water Quality	Amenity & Biodiversity
Most Sustainable	Rainwater harvesting, Green/Blue Roofs	✓	✓	✓
	Trees	✓	✓	✓
	Infiltration Systems (Infiltration trenches & basins etc.)	✓	✓	✓
	Filter Strips & Swales	✓	✓	✓
	Detention Basins & Wetlands & Ponds	✓	✓	✓
	Pervious Paving & Filter Drains	✓	✓	
Least Sustainable	Attenuation Tanks & Oversize Pipes	✓		

Table 5.1: Sustainability Hierarchy

5.2 Surface Water Discharge Hierarchy

The following discharge hierarchy will apply to all drainage designs in the plan area:

1. At source reductions and re-use;
2. Infiltration to ground;
3. Discharge to surface water body;
4. Discharge to public surface water sewer (only applicable where discharge to link road sewer is deemed beneficial with an associated strategy for regional treatment).

Where infiltration is suitable at the site, it should always be selected over discharge to surface water bodies or sewers.

5.3 Source Controls

5.3.1 Rainwater Harvesting (RWH)

General: Rainwater harvesting involves collection of rainwater from roofs and hard surfaces, rainwater butts may also be included as rainwater harvesting but generally perform on a much smaller scale. RWH utilizes runoff that has been collected from roofs and other impermeable areas. This run off is then stored, treated (where required) and then used as a supply of water for domestic, commercial, industrial and/or institutional properties. This runoff is generally used for non-potable purposes such as irrigation, flushing toilets and washing machines. RWH systems are rarely used to provide potable water for consumption or bathing, as this requires specialised treatment and monitoring to manage the contamination risks. RWH, whether designed for water conservation only or surface water management as well, provides benefits in delivering interception for all connected surfaces – where demand from the system is regular and consistent through the year.

Selection & Siting: Careful consideration of the likely contaminants present in the runoff due to the nature of the impermeable surface it comes in contact with should be applied. This will influence the suitability of runoff for various types of harvesting. For example, runoff from roofing materials containing copper or zinc, or treated with fungicides or herbicides, may not be suitable, depending on the purpose for which the water is to be used.

Although RWH selection and siting normally depends on the size, access requirements of the tank and physical constraints of the site RWH can generally be used in residential, commercial, or industrial development for new or retrofit projects for water conservation and surface water management. Storage tanks should be placed in a safe, secure location either underground, indoors, on roofs or adjacent to buildings. Tanks should not generally be placed on filled ground, and an adequate geotechnical investigation should be undertaken to ensure the suitability of the soils for the tank foundation. The storage tank is quite likely to be empty for periods of time and, where groundwater levels are close to the ground surface, the issue of flotation will need to be addressed.

RWH systems have a number of benefits when correctly utilized. They can meet some of the building's water demand, delivering sustainability and climate resilience benefits. They can help reduce the volume of runoff from a site and they can help reduce the volume of attenuation storage required on the site.

Water Quantity: RWH systems provide benefits in delivering interception for all connected surfaces when demand from the system is regular and consistent throughout the year. RWH systems can also provide benefits in controlling runoff volumes for both frequent and extreme rainfall events. However, for extreme rainfall events they will likely only capture the volume of the early stages of the extreme rainfall events until capacity has been reached. RWH systems cannot be assumed to contribute to a reduction in peak flow rate on a consistent basis, and therefore site conveyance design should not assume that any peak flow rate reduction is achieved.

Water Quality: By reducing the volume of runoff generated from the site, particularly for small events, RWH systems can directly reduce the pollutant load discharged to receiving waters. However, rainwater harvesting will only improve water quality if treatment measures such as pre-treatment, filtration, biological treatment, and disinfection are performed. Storing the water below ground can reduce the need for water quality treatment by keeping the water cool.

Amenity & Biodiversity: RWH systems provide indirect amenity value by supporting the resilience of developments and their landscape to changes in climate and water resource availability. However, RWH systems do not have any direct amenity or biodiversity value, but they can help to reduce flows on the downstream system, and this can help facilitate biodiversity delivery in those areas.

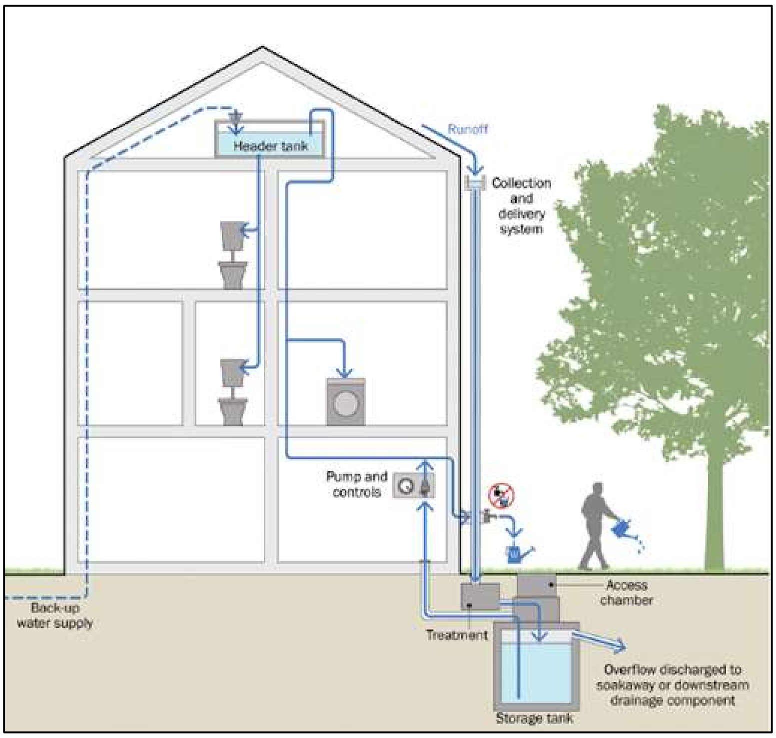


Figure 5.2 Rainwater Harvesting Schematic (CIRIA SuDS Manual C753 2015)

5.3.2 Pervious Pavements

General: Permeable pavements provide a pavement suitable for pedestrian and/or vehicular traffic, while allowing rainwater infiltrate through the surface and into the underlying layers where it is subsequently infiltrates to the ground and/or is collected and conveyed to the drainage network. Permeable pavements are most suitable for areas with light traffic loads and volume. The pavement generally caters for rainwater which lands directly on its surface but in certain cases, can accept runoff from other impermeable areas, such as Water Butts, Modified Planters or directly from rainwater goods and paved areas. Pervious pavements can be used as both source and site control components.

Selection & Siting: Pervious pavements are suitable for use on most sites and should be suitable for any of the proposed land zone uses if required. However, care must be taken to use pervious pavements in appropriate locations. They can often be combined with other solutions such as detention basins, ponds and wetlands allowing these subsequent attenuation and treatment features to be shallower and smaller. Pervious can be implemented on most types of ground conditions including flat terrain and areas which are not suitable for infiltration when combined with a liner. Unlined pavements should not be used in locations where infiltrating water may cause slope instability or foundation problems unless a full assessment of the risks has been carried out by a suitably qualified geotechnical engineer or engineering geologist. Pervious pavements are typically built as an alternative to impermeable surfaces and therefore require no extra development space for their construction.

Water Quantity: Pervious pavements offer benefits for all water quantity design criteria. Interception can be achieved with pervious pavements when infiltration can be utilized. Pervious pavements have also been demonstrated to control peak run off rates and runoff volumes for both frequent and extreme rainfall events.

Water Quality: Permeable pavement drainage has been shown to have decreased concentrations of a range of surface water pollutants when compared to impermeable surface drainage, including heavy metals, oil and grease, sediment, and some nutrients.

Amenity & Biodiversity: Pervious pavements provide amenity benefits by offering flexible use of space for a wide variety of activities. Pervious pavements do not provide any direct biodiversity benefits, although they are very useful for treating and controlling water to maximise the biodiversity in any downstream ponds or wetlands.

Figure 5.3 Pervious Pavement (www.susdrain.org)

5.3.3 Green Roofs/Blue Roofs

General: Green roofs are areas of living vegetation, installed on the top of buildings, for a range of reasons including visual benefit, ecological value, enhanced building performance and the reduction of surface water runoff. A green roof consists of a system in which several materials are layered to achieve the desired vegetative cover and drainage characteristics. These layers usually include vegetation, substrate, filter fabric, drainage layer, root barrier, waterproof membrane, and the roof deck. They come in two forms: extensive roofs and intensive roofs. Extensive roofs, have low substrate depths, simple planting, and low maintenance requirements. Intensive roofs otherwise known as roof gardens have deeper substrates that can support a wide variety of planting, but which tend to require more intensive maintenance; they are usually accessible.

A blue roof is a roof design that is explicitly intended to store water. This storage can be designed as attenuation storage or as storage for use such as irrigation, cooling water or non-potable use within the building, and/or for recreational opportunities.

Both systems are designed to intercept and retain precipitation which reduces the volume and rate of surface water runoff. Both systems can be integrated on a variety of roof types and sizes, although larger roof areas are typically more cost effective. They are particularly suited to flat/gently sloping roofs on commercial buildings, sports centres, schools, apartment blocks and other similar buildings.

Selection & Siting: Green roofs can be used on a variety of roof types and on any property size. They can be applied to a range of rooftop slopes, but steeper pitches will normally mean that less storage capacity is available, and the water drains away faster, unless the underlying drainage layer is specifically designed to capture and control flows. The greater the volume of water stored, the greater the potential loadings on the building, which may be an important design consideration. Other considerations include environmental factors such as exposure to wind, orientation to the sun, shading from surrounding buildings, climate of the area and the microclimate of the roof. Green roofs can be easily retrofitted providing there is sufficient structural capacity in the roof to support them, and provided that suitable and robust waterproofing can be installed.

Water Quantity: All green/blue roofs can be assumed to meet interception requirements during the summer but may struggle during the winter when they are likely to be more saturated for longer periods of time. The hydraulic performance of green roofs once saturated tends to be fairly similar to standard roofs. Green roofs can control runoff volumes for smaller more frequent events, in particular during the summer months. However, for larger more extreme rainfall events volume control will be significantly reduced once the green roof is saturated. Green roofs can also provide benefits in terms of reducing peak flow rates to the site drainage system – principally for small and medium-sized events. The storage characteristics of blue roofs will only be a function of the hydraulic controls at the outlet of the system. Although they can be designed to attenuate flows, any volumetric reduction will be limited to evaporation. The performance of a blue roof is more predictable than a green roof, as it usually constitutes a standard arrangement of attenuation storage and throttled outlets. There is a growing body of evidence that green roofs considerably reduce the volume of runoff in warmer periods when the soil moisture deficit is high. Blue roofs can also provide a reduction in the volume of runoff due to evaporation, and this process will be enhanced due to the solar warming of the water because of its shallow depth and exposed location.

Water Quality: Through a variety of physical, biological, and chemical treatment processes, within the soil and root uptake zone, which filter airborne pollutants and pollutants entrained within rainwater, green and blue roofs can help to reduce the amount of pollution delivered to the local drainage system and, ultimately, to receiving waters.

Amenity & Biodiversity: Green roofs can be used to provide valuable amenity if the roof is intended to be accessible or is overlooked. They can improve the roofscape for the surrounding community of office occupiers as well as users of the green roof space itself, with the variety of planting and habitats creating a more colourful, aesthetically pleasing, and natural environment, particularly in dense urban areas. Blue roofs tend to be constructed below the open space areas on podium decks, so the amenity value will be defined by the aesthetics and use of this space.

Green roofs can be designed to provide high ecological value. They can help to conserve valuable habitat and biodiversity and provide an oasis of life in an otherwise sterile urban environment. They can also contribute to networks, clusters and corridors of green space that connect previously fragmented habitats.

Figure 5.4 Green Roof (www.bauder.co.uk)

5.3.4 Infiltration Systems

General: Infiltration systems include SuDS components such as infiltration trenches, soakaways, infiltration blankets and infiltration basins. Infiltration can contribute to reducing runoff rates and volumes while supporting baseflow and groundwater recharge processes. The rate at which water can be infiltrated depends on the infiltration capacity of the surrounding soils.

Soakaways are excavations that are filled with a void-forming material that allows the temporary storage of water before it soaks into the ground. Infiltration trenches are simply linear soakaways. The advantages of trenches over cuboid soakaways is that they can often be kept shallower and, in variable soils, can help distribute the infiltration area so that the impact of less permeable areas of soil is less pronounced. Infiltration basins are flat-bottomed, shallow landscape depressions that store runoff before infiltration into the subsurface soils. Infiltration blankets are large shallow systems that are typically constructed using permeable aggregate or geocellular units that act as extensive soakaway systems.

Selection & Siting: Infiltration systems facilitate the discharge of surface water runoff to the ground and ultimately into groundwater. It is therefore crucial that any runoff is suitably clean before entering the infiltration component so that the groundwater is not put at risk of contamination. The performance of infiltration systems is dependent on the infiltration capacity of the surrounding soils and the depth to groundwater. A minimum distance of 1m between the base of the infiltration system and the maximum likely groundwater level should always be adopted. This is to minimise the risk of groundwater rising into the infiltration component and reducing the available storage volume, to protect the functionality of the infiltration process by ensuring a sufficient depth of unsaturated material and to protect the groundwater from any contamination in the runoff. A number of issues should be assessed before implementing any infiltration system for surface water management. Instability or subsidence in the subsurface/slope due to infiltration should be considered. The risk of pollution to both groundwater and surface water should be assessed and the risk of groundwater flooding should also be assessed. Soakaways are best suited to the infiltration of runoff from small areas such as roofs of residential housing. Infiltration components can often be retrofitted into existing developments, to drain small areas such as private driveways and roof drainage, providing there is sufficient offset from structures, slopes etc. Infiltration systems should not normally be used to drain landscaping or other areas with high risks of soil erosion and loadings, due to the risk of sediment blockage and clogging of the soils surrounding the component.

Water Quantity: Infiltration systems should be designed to manage storms based on a 1in 100 year event plus an allowance for climate change, the performance of infiltration systems under such conditions needs to be known. The infiltration component should discharge from full to half-full within a reasonable time so that the risk of it not being able to manage a subsequent rainfall event is minimised. Infiltration can play an important role in providing interception – the capture and retention of the first 5mm of any rainfall event, even on sites with low infiltration rates. Infiltration systems can help reduce the required attenuation storage volumes. Infiltration also reduces the volume of runoff for both frequent and extreme events.

Water Quality: Infiltration systems can provide excellent water quality benefits if design allows. The acceptability of infiltration design, from a groundwater protection perspective, will depend on the extent of the likely runoff contamination and site and ground characteristics. Sedimentation tends to occur within the temporary storage area and an allowance should always be made for this or, preferably, upstream SuDS components put in place to remove sediment before entering the component. The deeper an infiltration system is, the greater the risk of bypassing the protective upper soil layers and decreasing the distance to the water table. This can lead to an increased risk of groundwater pollution. In this respect shallow and dispersed systems are usually best. Geotextile layers can be used within infiltration components for additional trapping of surface water runoff particulates and hydrocarbons.

Amenity & Biodiversity: Infiltration systems do not usually have any inherent amenity value, but subsurface systems can promote the multi-functional use of space by allowing the overlying surface to be used for recreation or other amenity facilities. The ecological value of the system can be enhanced by diversifying the planting (e.g., including trees, woody shrubs, wildflower mixes) or by including bioretention areas within the design.

Figure 5.6: Infiltration Trench

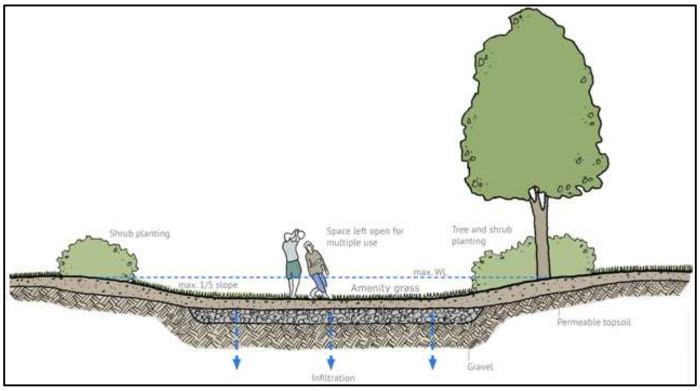


Figure 5.5: Infiltration Basin Schematic (www.susdrain.org)

5.3.5 Trees

General: Trees provide benefits to surface water management by providing transpiration, interception, increased infiltration phytoremediation. Trees can be planted within a range of infiltration SuDS components (e.g., bioretention systems, detention basins, swales) to improve their performance, or they can be used as standalone features within soil-filled tree pits, tree planters or structural soils. Tree pits and planters can be designed to collect and attenuate runoff by providing additional storage within the underlying structure. The soils around trees can also be used to filter out pollutants from runoff directly.

Selection & Siting: Trees can be used on a wide variety of development site types and ground conditions. Suitable trees should be chosen on a site-by-site basis, based on the constraints and opportunities afforded by a particular location, and to achieve optimum delivery of hydraulic, water quality, amenity, and biodiversity objectives. A landscape architect should advise on the most appropriate trees for a particular development scenario that are suitable.

Water Quantity: Interception provided by the tree canopy will vary with tree type and will increase over the life of a tree as it grows. For the first few years the Interception may be negligible. It is therefore best to ignore this aspect in the hydraulic design of SuDS, while recognising that it will have a long-term benefit and will reduce volumetric runoff loads to the surface water system in the future. Tree pits can help reduce flow rates from a site by facilitating infiltration and/or by providing attenuation storage.

Water Quality: Tree pits will filter out pollutants from runoff and, by reducing the volume of runoff, will also help to reduce pollutant loadings to receiving surface waters.

Amenity & Biodiversity: Trees should be selected and planted to maximise their potential delivery of surface water management objectives, but also so that they have a positive visual impact on the urban environment, with wide ranging seasonal interest. The implementation of trees for surface water runoff management should be integrated with the delivery of green street and green infrastructure strategies, transport (including walking, cycling and highways) strategies and with the overall urban design strategy. Trees can play a critical role in enhancing urban wildlife. The potential contribution of trees to local biodiversity strategies and habitat connectivity should be fully considered. Trees can act as bridges, maintaining connectivity for species through the urban landscape. Trees support wildlife in urban environments in many ways, providing food, shelter and habitat for birds, invertebrates, and other species. The selection and mix of tree species will influence the habitat diversity that is provided.

Figure 5.8: Tree Pits

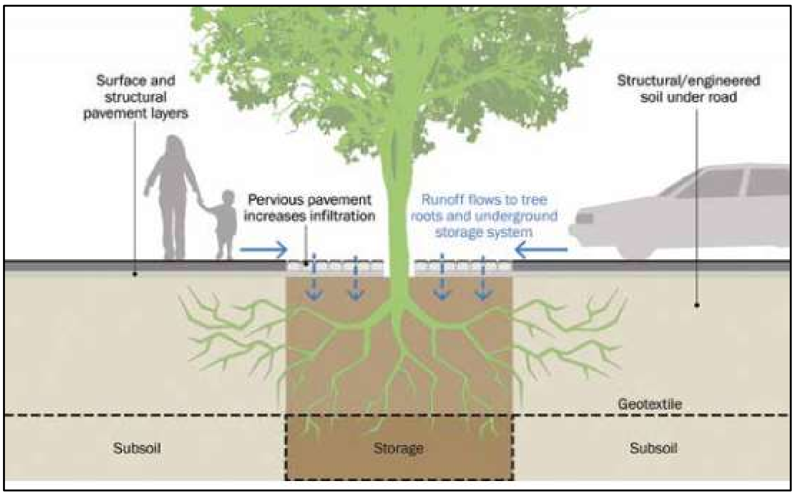


Figure 5.7: Tree Pit Schematic (CIRIA SuDS Manual C753 2015)

5.3.6 Filter Drains

General: Filter drains are shallow excavations backfilled with granular material that create temporary subsurface storage for either filtration or infiltration of stormwater runoff. Filter drains can contain a perforated pipe at the base to convey runoff to further SuDS components in the management train. Filter drains should ideally receive lateral inflow from an adjacent impermeable surface that is pre-treated using a vegetated filter strip or equivalent. They are not normally intended to function as sediment traps and should be implemented downstream of a pre-treatment system in order to prevent clogging and failure. Where there is no effective upstream removal of sediments and silts, a geotextile layer below the filter drain surface, at a shallow depth, is required that can be regularly removed and cleaned or replaced.

Selection & Siting: Filter drains are best located adjacent to impermeable surfaces such as car parks or roads/highways with upstream pre-treatment systems. Subject to appropriate ground conditions, filter drains are recommended for draining residential back gardens and other small, grassed areas where subsoil permeability is low. Filter drains can also be used to drain carriageways. The base of the filter drain should be a minimum 500mm above highest expected groundwater table level. The use of filter drains is typically restricted to sites without significant slopes unless they can be placed parallel to contours. Filter drains should not be sited on unstable ground, and ground stability should be verified by assessing site soil and groundwater conditions. They are designed for intermittent flow and should be allowed to drain and re-aerate between rainfall events. They should not, therefore, be used on sites with a continuous flow.

Water Quantity: Filter drains can only deliver a small contribution to interception where they do not allow infiltration. As well as determining the degree of filtration, the particle size of the medium also determines travel time in the filter and can therefore play a role in meeting peak flow discharge rate control requirements. They do not provide volume control for both frequent and extreme events.

Water Quality: Unless infiltration is allowed, filter drains will not provide a significant reduction in contaminant loads to surface waters via volumetric runoff control, as they can only provide limited interception.

Amenity & Biodiversity: Filter drains can be designed creatively to provide attractive boundary lines or edging. Filter drains may be protected with geotextile and covered with topsoil and planted with grass, in a landscaped area. However, this increases the risk that maintenance responsibilities will be overlooked, which could cause performance failure of the system and should therefore be implemented with caution. Gravel media can host microorganisms and provide breeding grounds for insects and amphibians. Adjacent biodiverse planting or overlying grass can also deliver additional opportunities for biodiversity.

Figure 5.9: Roadside Filter Drain

5.3.7 Filter Strips

General: Filter strips are uniformly graded and gently sloping strips of grass or other dense vegetation that are designed to treat runoff from adjacent impermeable areas by promoting sedimentation, filtration, and infiltration. The runoff is designed to flow as a sheet across the filter strip at sufficiently low velocities that treatment processes can take place effectively. They are often used as either a pre-treatment component before swales, bioretention systems and trenches (to extend the life of these components by capturing sediment) or as a treatment component.

Selection & Siting: Filter strips can be used in a variety of situations but are particularly well suited for managing runoff from roads because they are a linear feature and easily incorporated into roadside space. They are also suitable for managing runoff from car parks and other impermeable and permeable areas. Filter strips should be effectively incorporated into landscaping and public open spaces, so that their function is not compromised by activity in the area.

Unlined filter strips should not be used to treat runoff from areas with high contaminant loads if the risk of groundwater pollution due to infiltration is unacceptably high. Where a liner is used to prevent infiltration, the seasonally high groundwater level should be below the level of the liner. If infiltration is allowed, the maximum likely groundwater level should be at least 1 m below the base of the system. Filter strips should not be located in areas where trees or structures will cause shade conditions that limit grass growth.

Water Quantity: Where topsoil is suitably permeable, and underlying soils have some capacity to store and/or infiltrate runoff, then filter strips with very shallow slopes can be designed to deliver interception. Filter strips do not provide control for volumes from extreme events or help control peak run off volumes.

Water Quality: Filter strips can help retain runoff from small events on site helping to reduce the contaminant load discharged to surface waters via volumetric control. They can also treat the residual runoff by facilitating sedimentation and filtration. The acceptability of allowing infiltration from the filter strip will depend on the extent of the likely runoff contamination and site characteristics which would improve its treatment potential.

Amenity & Biodiversity: Filter strips deliver green, vegetated open space adjacent to impermeable areas and should be integrated with the overall site design and surrounding landscaping. Landscaping and layout of the filter strip and its adjacent area should be such that pedestrian traffic (and cycling) is kept to a minimum. The location of filter strips should be well defined on a site, as their function and value to the surface water management system is often not obvious to those using the site. A grass strip within the overall site landscaping will support biodiversity by providing feeding and foraging areas for various animals, and steeping stone habitats in urban areas. More diverse planting, possibly including areas of wildflowers, will encourage wider biodiversity.

Figure 5.10: Filter Strip (www.susdrain.org)

5.4 Site Controls

5.4.1 Swales

General: Swales are broad, shallow, vegetated drainage channels which can be used to convey or store surface water. Swales are generally suited for small catchments with impermeable areas. They are typically provided along roads in grass verges. Swales can be designed for infiltration to subsoil or detention and conveyance to another stage in the management train. Conveyance can be in the open channel or in a perforated pipe within a filter bed below the base of the channel. Swales are recommended to cater for runoff from access roads, providing water treatment and reduction in peak flow. Depending on local subsoil conditions, dry swales are recommended which provide infiltration and further reduce runoff volume. Where vehicle and pedestrian access is required across a swale, a causeway can be provided. The levels at the outer swale banks will be higher than at the centre of the crossing point. This drop in level acts as an exceedance route for runoff from the swale during extreme rainfall events.

Selection & Siting: Swales can be used in a wide variety of situations. They are well suited for managing runoff from roads because they are a linear feature and easily incorporated into the roadside space. They are also suitable for managing runoff from car parks and other impermeable and permeable areas. Swales should be incorporated into landscape and public open spaces, as they tend to demand significant land-take due to their shallow side slopes. They are also much easier to maintain on sites with high sediment loads than any other type of component. Unlined swales should not be used to treat runoff from areas with high contaminant loadings if the risk of groundwater pollution due to infiltration is unacceptably high.

Water Quantity: Swales normally provide interception because there is usually no runoff from them for the majority of small rainfall events. The water soaks into the surface vegetated soil layers and into the underlying soils or other media and is removed by evapotranspiration and infiltration. Swales can help reduce peak flow rates from a site by facilitating infiltration and/or by providing attenuation storage. Swales are not normally assumed to provide any reduction in volume of runoff for more extreme events.

Amenity & Biodiversity: Swales can be designed to fit into many different landscape types in an aesthetically pleasing manner, often delivering attractive vegetated corridors into streetscapes and road/parking corridors. Swales are generally shallow surface features that do not present significant risk or danger to the health and safety of the general public. Any residual risks can be mitigated through the design of shallow side slopes and shallow flow depths. Swales can include a variety of planting that will help provide a positive biodiversity value by providing habitat and food for insects, invertebrates, and birds.

Figure 5.12: Roadside Swale

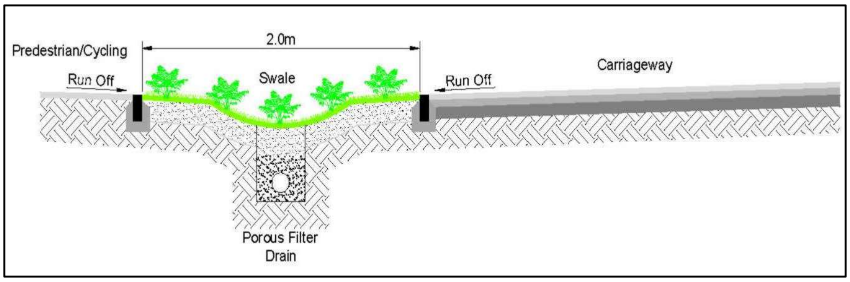


Figure 5.11: Swale Schematic

5.4.2 Bioretention Areas/Rain Gardens

General: Bioretention areas or "rain gardens" are stormwater controls that collect and treat stormwater runoff. The runoff is treated using soils and vegetation in shallow landscaped basins to remove pollutants. Treated runoff can be collected and conveyed further downstream and/or allowed infiltrate into the subsoil. Part of the runoff volume will be removed by evaporation and plant transpiration. Bioretention areas are recommended to cater for runoff from residential neighbourhoods and car parks.

Siting & Selection: Bioretention systems are applicable to most types of development and can be used in both residential and non-residential areas to mitigate polluted runoff from roads. They can be implemented in private curtilage for managing runoff from single properties, in small shared public areas, on car park islands, roundabouts, footpaths, traffic calming and pedestrian zones.

Water Quantity: Bioretention systems help deliver interception because there is no runoff from them for the majority of small rainfall events. The water soaks into the filter medium and is removed by evapotranspiration and infiltration where allowed. Bioretention systems can help reduce flow rates from a site by providing some attenuation storage and can reduce storage volume requirements where infiltration occurs.

Water Quality: Bioretention systems can provide very effective treatment functionality. the removal of sediments and associated pollutants by filtration through surface vegetation and groundcover. the removal of fine particulates and associated contaminants by infiltration through the underlying filter and some biological uptake by vegetation and subsoil biota.

Amenity & Biodiversity: Bioretention systems are very flexible in terms of size and appearance. They can deliver significant aesthetic benefits by incorporating vegetation into streetscape and general landscape features. They can be easily integrated into a variety of urban space including car parks, pedestrian areas, streetscapes, plazas, and forecourts. Bioretention systems can be used to treat runoff before use for non-potable requirements, and they can deliver water-efficient landscaping, particularly in dry areas, by storing runoff as a saturated layer beneath the system, creating a reservoir that can be accessed by overlying plants.

Bioretention systems can provide quality habitat conditions for wildlife, contributing positively to biodiversity enhancement in urban areas. The systems are relatively flexible in terms of the planting that can be included. Therefore, they can be designed to support local biodiversity requirements where appropriate.

Figure 5.13: Bioretention Area (www.greenblue.com)

5.4.3 Retention and Detention Basins

General: Detention Basins are dry basins that attenuate stormwater runoff by providing temporary storage with flow control of the attenuated runoff. Detention basins are generally applicable to most types of developments. In residential areas they are normally dry and often function as a recreational facility, e.g., sports fields or playgrounds. They may be constructed such that surface runoff is routed through them during storm events with an outflow restriction, or such that runoff typically bypasses the detention basin until a design storm event occurs when runoff is received by a flow diverter or overflow and temporarily stored until the inflow recedes below a design level.

Selection & Siting: Detention basins are generally applicable to most types of development and can be used in both residential and non-residential areas. They can often be designed as multi-functional spaces, creating an open area within a development, part of which can be made available for recreational purposes. Roundabouts can also often provide suitable space that is redundant within the existing or proposed development landscape. Groundwater levels should be checked to ensure that during periods of high groundwater, the storage capacity of the detention basin is retained and that hydraulic connectivity between the surface water runoff and the groundwater is acceptable from a water quality perspective.

Water Quantity: Vegetated detention basins can help deliver some interception because there tends to be no runoff from them for the majority of small rainfall events. The water soaks into the basin topsoil layer and is removed by evapotranspiration and even very small amounts of infiltration. Detention basins can help reduce flow rates from a site by controlling the discharge rate and allowing the basin storage to fill during storm events. Detention basins do not normally contribute to volumetric control of runoff, but can be used as Long-Term Storage areas, or to deliver further attenuation where Long-Term Storage is not practical. An exceedance flow route will be required for rainfall events that exceed the design capacity of the detention basin.

Water Quality: Vegetated detention basins can help retain runoff from small events on site, helping reduce the contaminant load via volumetric control. They can also treat the residual runoff, primarily via the gravitational settling of particulate pollutants, although some filtration will occur through the vegetation on the basin base and underlying soils together with biodegradation and photolytic breakdown of hydrocarbons during the drying processes between runoff events.

Amenity & Biodiversity: Detention basins can be important parts of the landscape design for public open space – defining the topography for green or hard landscaped areas. Basin design can take many forms, from naturalistic and irregular to formal and geometric. This will depend on the planned future use of the space and the landscape, amenity, and biodiversity objectives for the site. The value of basins can often be enhanced with footpaths or cycle paths, and enhanced with structural and diverse plants, wetland planted areas and wildflower mixes to enhance their beauty and amenity contribution. Detention basins may be constructed to serve more than one purpose, and can be used as car parks, playgrounds, or sports fields. When constructed for multiple purposes, the detention basin should be usable for the function other than surface water detention for most of the time. Where multi-functional use is intended, the recreational area should normally have a relatively low flooding frequency depending on its use.

Vegetated detention basins can include a variety of structurally diverse planting that will help make a positive contribution to urban biodiversity – providing habitat and food for invertebrates and birds. Some plants and animals specifically require ephemeral water bodies as part of their life cycle, and suitable wildflower mixes can provide important nectar sources for insects. Detention basins should provide a high biodiversity value to the area.

Figure 5.14: Detention Basin (www.susdrain.org)

5.5 Regional Controls

Ponds and wetlands are features with a permanent pool of water that provide both attenuation and treatment of surface water runoff. They can support emergent and submerged aquatic vegetation along their shoreline and in shallow, marshy (wetland) zones, which helps enhance treatment processes and has both amenity and biodiversity benefits.

5.5.1 Ponds & Wetlands

General: Ponds are basins which have a permanent depth of water. They can be constructed in an existing depression, by excavating a new depression or by constructing embankments. Runoff which enters the pond is detained and treated by settlement and often biological uptake before outfalling. Through the effective use of upstream source control measures, SuDS ponds can usually be designed as small features that blend unobtrusively into the landscape. Constructed Wetlands comprise of shallow ponds and marshy areas which are designed primarily for stormwater treatment but can also provide some attenuation above the permanent water level. Well designed and maintained wetlands can offer significant aesthetic, amenity, and biodiversity opportunities. Constructed wetlands require a continuous baseflow to support a plant-rich community.

Ponds are recommended at the end of proposed surface water drainage networks following previous SuDS techniques in the Management Train. Outflow from any proposed ponds may be restricted at times due to high tide levels and as such may require additional attenuation volume. Inclusion of several independent cells is encouraged which will enhance biodiversity, improve water quality levels, and provide a more environmentally effective management programme.

Selection & Siting: Ponds or wetlands are generally suitable for most types of new development and redevelopment and can be used in both residential and non-residential areas. Ponds are also appropriate for use in retrofit situations where land is available at a suitable point near the outlet of the drainage system. It may be difficult to site a pond on steeply sloping sites, and ponds should not be sited on unstable ground. Ground stability should be verified by assessing site soil and groundwater conditions. Ponds and wetlands should be placed in developments, so they are overlooked by housing and not hidden in an unseen corner. Alternatively, they can be located in larger areas of open space. This ensures that the water features are a valued part of a development. Where the groundwater table is close to the base of the pond, hydraulic connectivity between the water in the pond and the groundwater should be prevented, unless the water quality requirements for infiltration have been met.

Water Quantity: Ponds and wetlands can be designed to provide some interception if designed for infiltration. The extent of any volumetric reduction achieved in this way is likely to be very limited and requires careful assessment and design. They can help reduce peak flow rates from a site by controlling the discharge rate and allowing the temporary storage volume to fill during storm events. Ponds and wetlands normally do not contribute to volumetric control of runoff for both frequent and extreme events.

Water Quality: Ponds and wetlands treat incoming runoff through settling and biological uptake. Silts and suspended sediments are treated through settling. Uptake of pollutants, particularly nutrients, also occurs to some degree through the biological activity of the pond/wetland. Where ponds/wetlands are proposed as final polishing components to downstream components such as permeable pavements, swales or bioretention systems within the management train, then smaller sizes may be acceptable, depending on the effectiveness of the upstream SuDS components. Ponds should not accept contaminated runoff directly, as this will cause failure in delivering amenity and biodiversity performance and will increase maintenance risks and costs.

Amenity & Biodiversity: Pond and wetlands design should add value to the amenity and biodiversity of the local communities and be of an appropriate scale and form to suit the surrounding landscape character. The extent to which the pond/wetland is designed as an amenity feature will depend on the likely level of contamination of the inflows. Where it is being used as a final polishing component, amenity potential will be high. Ponds/wetlands can provide an educational resource for local schools and community environment groups and be the focal point of a recreational area. Biodiversity value is also likely to be very high where contamination is kept to minimal levels.

5.6 SuDS Potential Summary

A summary of possible SuDS components is included on Table 5-1 below. Since there is currently limited site specific information with respect to groundwater levels and infiltration potential, and also because the site layouts have not yet been established, very few components have been eliminated. This provides developers flexibility to develop suitable systems in accordance with the sustainability and discharge hierarchies provided above.

Further technical detail on how these components can be designed for various ground conditions can be examined in "*Part D: Technical Detail of the CIRIA SuDS Manual C753*".

SuDS components that have been disregarded include attenuation tanks and oversized pipes. These components should only be used as a last resort and it is envisaged that more sustainable measures will be feasible for developments in this plan area.

SuDS Component	Control Type	Design Criteria						If Infiltration Suitable	If Infiltration Unsuitable	Comments
		Peak Runoff	Frequent Runoff Volumes	Extreme Runoff Volumes	Water Quality	Amenity	Bio-diversity			
RWH	Source		✓	✓		✓		✓	✓	Suitable for residential and education zones.
Green/Blue Roofs	Source		✓		✓	✓	✓	✓	✓	Suitable for residential and education zones.
Infiltration Systems	Source/Site	✓	✓	✓	✓	✓	✓	✓		Only suitable if groundwater level and subsurface conditions allow.
Trees	Source/Site	✓	✓		✓	✓	✓	✓	✓	Suitable for wide variety of land use types and site conditions.
Filter Strips	Source/Site		✓		✓			✓	✓*	May be used with a liner if infiltration not suitable.
Filter Drains	Source/Site	✓			✓			✓		Not suitable in unstable ground conditions or high groundwater levels.
Swales	Source/Site	✓	✓	✓	✓	✓	✓	✓	✓*	May be used with impermeable lining in areas where infiltration unsuitable.
Pervious Pavements	Source/Site	✓	✓	✓	✓			✓	✓*	May be used with impermeable lining in areas where infiltration unsuitable.
Bioretention Systems	Site	✓	✓	✓	✓	✓	✓	✓	✓*	Can be designed for areas where infiltration not suitable.
Detention Basins	Site/Regional	✓	✓		✓	✓	✓	✓	✓*	Ensure storage capacity not reduced by high groundwater levels.
Ponds & Wetlands	Regional	✓			✓	✓	✓	✓	✓*	Can be used with liner if infiltration undesirable.

Table 5.2: SuDS Potential for the Plan Area

✓* = Alteration to design may be required.

6 Strategic Drainage Concept & Protocols for Developments

6.1 Strategic Drainage Concept

The strategic drainage concept for the plan area is illustrated on Figure 6.1 below.

The developable lands have been divided into sixteen sub-catchments with individual outfalls to the River or to drainage ditches. The sub-catchment delineation has been influenced largely by the existing paths and flow routes and also by the proposed land uses. The outfall positions mimics the existing drainage regime.

Source and Site controls within individual sub-catchments upstream of outfalls have not been provided as the location and design of the SuDS components will depend largely on the development layout and the outcome of site specific investigations. SuDS systems on a site specific scale should be incorporated into future development designs by building upon this strategic drainage concept, although it is accepted that adjustments to the sub-catchment delineation or outfalls may be justified at development design stage.

Open space and amenity areas have been provided within the floodplains. These areas are linked with a public walkway that follows the river alignments and also leads to key archaeological & architectural heritage features.

Natural Flood Management (NFM) measures are provided on the floodplains at the eastern side of the plan area. Such measures could include floodplain and riparian woodlands, offline storage and constructed wetlands. Features such as leaky barriers could also be considered in the main river channel. The incorporation of these measures in lands that are otherwise unsuitable for development provide opportunities to harness the natural features of the site and maximise amenity and biodiversity benefits. It is intended that any NFM measures provided in the plan area serve as an enhancement to the sustainability of the developed lands and should not be used as a means to reducing SuDS features elsewhere in the plan area.

The proposed link road has been provided with a further three sub-catchments which includes provision for storage and treatment as a feature in amenity areas. Existing flow routes have been maintained through the proposed link road by incorporating culverts at key locations along the road. The suitability of this approach has been confirmed in the Strategic Flood Risk Assessment. Outside of the river and main drainage channels, the culverts would only convey flow during flood events and outside of these times they can serve to enhance amenity by providing a pedestrian link between different public spaces that are separated by the link road.

There may be merit in providing shared regional controls for development sites within areas zoned as Public Open Space however this has not been provided for in the concept presented. Any such proposals should be discussed and agreed with Cork City Council.

It is intended that this concept will be developed further on a site specific scale using the principals applied here and taking into account the overall objectives of the SDS.

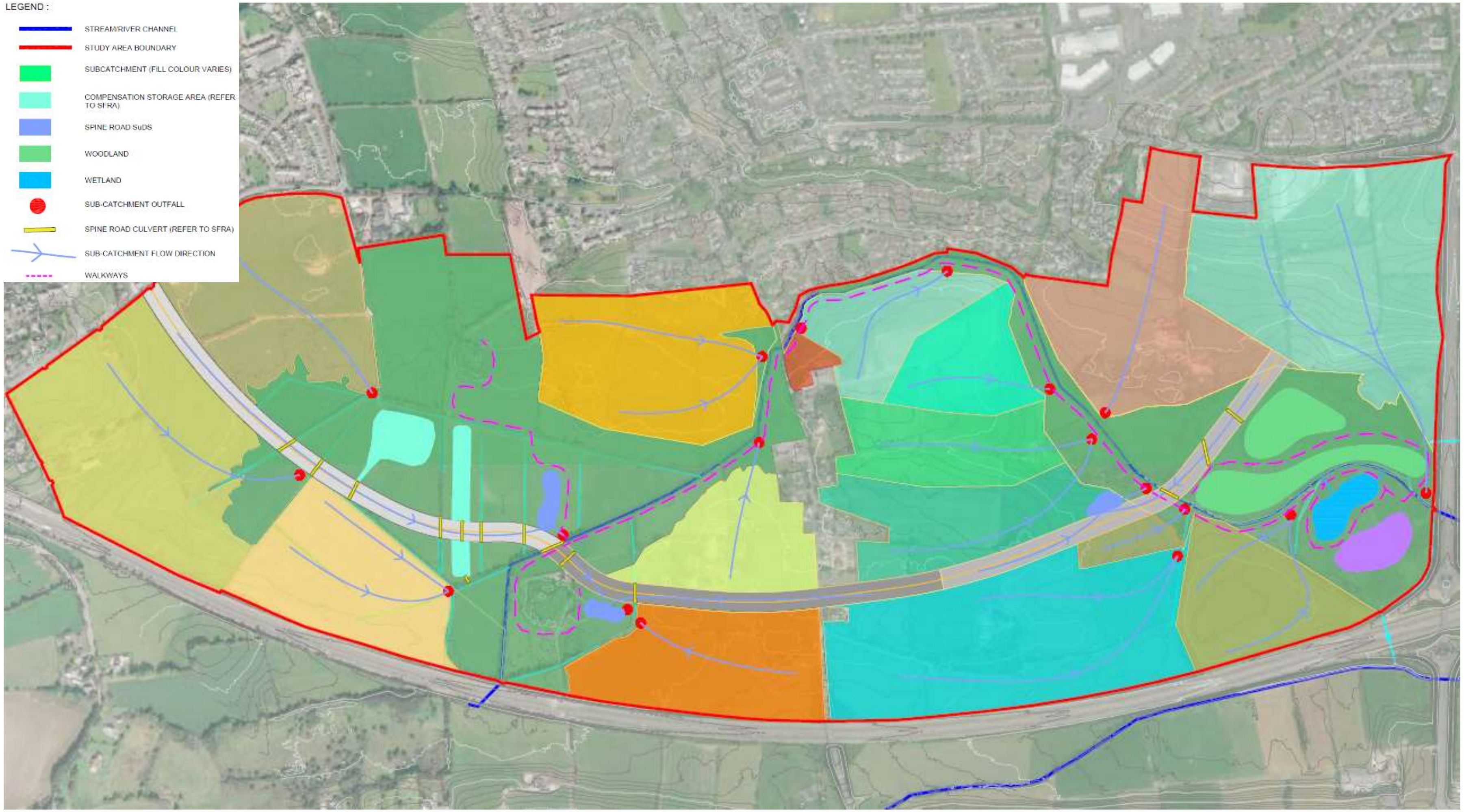


Figure 6.1: Strategic Drainage Concept

6.2 SuDS Protocol for New Development

The following protocol should be adopted for developments within the Ballincollig (Maglin) Urban Expansion Plan.

This protocol will provide guidance for assessing the resilience of SuDS to climate change during periods of drought, flash flooding, temperature extremes and periods of persistent rainfall and to propose appropriate resilient SuDS strategies to manage stormwater runoff arising from severe rainfall events now and into the future.

1. Identify most suitable method of discharge and suitable outfalls. This may require geotechnical or hydrogeological site investigations to determine subsurface conditions.
2. Establish allowable discharge rates and water quality discharge limits based on the requirements included in this SDS
3. Consideration of local environmental factors to determine appropriate SuDS components.
4. Development of a suitable SuDS Management Train for the proposed development.

6.1.3 Drainage Proposal for Residential Developments

It is required that for areas zoned for residential use that:

- That all runoff within the curtilage of the property boundary pass through at least one suitable SuDS component prior to discharging to downstream SuDS components.
- Storage up to the 1 in 100 year event including climate change allowance shall be provided within the curtilage of the property boundary, with maximum discharge rates/volumes in accordance with Section 3.
- Runoff from public areas (such as roads, parking areas, hard and soft landscaped areas, and footpaths) shall pass through at least one SuDS component prior to discharging to the final downstream detention/retention SuDS components.
- Runoff from roofs can be managed using water butts, rainwater harvesting, rain gardens, infiltration trenches, permeable paving, and soakaways.
- Runoff from hardstanding areas can be managed using permeable paving, filter strips, swales, and bioretention systems.
- The final SuDS components located within the development shall comprise basins/ponds/wetlands, prior to discharge to the Maglin River or local surface water sewer.

6.2.4 Drainage Proposal for Education

It is required that for areas zoned for education and residential areas which will develop apartments that:

- Runoff from roofs shall pass through at least one SuDS feature prior to discharge to on-site surface water retention features.
- Appropriate SuDS components shall be provided to store the 1 in 100 year event with an allowance for climate change, with maximum discharge rates/volumes in accordance with Section 3.
- Runoff from roads and parking areas shall past through an appropriate number of SuDS features prior to discharge to the final on-site surface water retention features.

- Runoff from roofs can be managed using rainwater harvesting, rain gardens, infiltration trenches, permeable paving, and soakaways.
- Runoff from hardstanding areas can be managed using permeable paving, filter strips, swales, and bioretention systems.
- Runoff should pass through a surface water retention feature such as a basin/pond/wetland, prior to discharging to the river or drainage channels.

6.2.5 Drainage Proposal for Public Open Space

The majority of lands zoned as Public Open Space are located on the floodplain of the Maglin River. This makes the public open space ineffective for use in storing surface water runoff during flood events. For this reason, storage capacities of the public open space should not be allowed to meet the developments total surface water attenuation requirement.

The presence of this floodplain should not preclude the public open space from including SuDS components as it can still be effective in managing routine rainfall, and runoff may need to be discharged safely across the floodplain. SuDS in the floodplain may also be acceptable in terms of providing treatment for frequent events. The design of any SuDS in the public open space should not reduce floodplain storage or conveyance for more extreme rainfall events.

SuDS components used within the floodplain should include systems that enhance amenity and biodiversity and provide opportunities for new habitat creation.

6.2.6 Drainage Proposal for Proposed Link Road

It is recommended that filter strips, swales, and detention basins (as a minimum) be utilised to cater for runoff from the proposed link road, providing water quality treatment and reduction in peak flow. Where space allows ponds/wetlands may be beneficial. Bioretention areas can be fitted as traffic calming features and within dead space in car parks, providing amenity and biodiversity benefits.

6.2.7 Drainage Proposal for Urban Town Centre

Considering SuDS early in the design process is key to ensuring that spatial requirements of SuDS components for the urban town centre are planned appropriately. SuDS components that utilize space and do not require extra space other than that proposed for the development are therefore recommended for the urban town centre where possible.

There are a range of SuDS components which can be easily designed into tight urban settings. Space efficient SuDS components include green roofs, bioretention systems, permeable paving, rills, rainwater harvesting, micro-wetlands, and tree pits.

7 Conclusions & Recommendations

7.1 Conclusions

- As part of the proposed Ballincollig (Maglin) Urban Expansion Area, SuDS will be required to manage surface water within the development.
- SuDS measures will be required as part of this new development to ensure water quantity, water quality, amenity and biodiversity design criteria are met, and the downstream water bodies must be enhanced and protected by SuDS components.
- A number of challenges may arise within this development such as issues surrounding high groundwater levels, unstable subsurface/karstification and flat topography. These issues can be overcome by implementing suitable SuDS components.
- Further site investigation is recommended to assess ground conditions such as infiltration capabilities and subsurface stability. Hydrogeological and geotechnical site investigations will be required.
- Developers within the Ballincollig (Maglin) Urban Expansion Area should implement SuDS measures in line with the recommendations of this report to suit individual site layouts and local ground conditions.
- Design of SuDS should be in accordance with the CIRIA C753 SuDS Manual.
- A management train should be incorporated during the design stage whereby surface water should be managed locally in small sub-catchments rather than being conveyed to and managed in large systems further down the catchment. This is consistent with the concept strategy provided and it is expected that this concept will be developed further on a site specific scale.
- It is recommended that areas zoned for residential utilize SuDS components such as water butts, rainwater harvesting, infiltration trenches, soakaways, permeable paving, filter strips, swales, bioretention systems and green/blue roofs where applicable to manage runoff from roofs and hard standing areas.
- It is recommended that areas zoned for education utilize SuDS components such as rainwater harvesting, rain gardens, infiltration trenches, permeable paving, soakaways, filter strips, swales, and bioretention systems.
- It is recommended that filter strips, swales, and detention basins (as a minimum) be utilised to cater for runoff from the proposed link road.
- The urban town centre should utilize space efficient SuDS components such as green roofs, bioretention systems, permeable paving, rills, rainwater harvesting, micro-wetlands, and tree pits.
- All drainage designs should be consistent with the mitigation measures and recommendations included in the South Ballincollig Drainage Study Strategic Flood Risk Assessment.

7.2 Recommendations for Development Plan Objectives

It is recommended that the Development Plan Objectives address the key requirements for sustainable flood risk management of these lands. These are summarised on Table 7.1 below.

Recommendation Number	Recommendation
SDS_R01	All new proposed development proposals within the Maglin / South Ballincollig Expansion Area shall be accompanied by individual site-specific flood risk assessments which are completed in accordance with the Planning System and Flood Risk Management Guidelines (2009) and which build upon the strategic flood risk management measures outlined in the Strategic Flood Risk Assessment prepared for the Maglin / South Ballincollig Expansion Area. Flood Risk Assessments should ensure that appropriate site-specific mitigation measures are implemented and residual risks are managed to an acceptable level.
SDS_R02	Incorporate Sustainable Urban Drainage Systems (SuDS) and other nature-based surface water drainage solutions as part of a site-specific Surface Water Management Plan for all development proposals. Proposals shall build upon the Sustainable Drainage Strategy prepared for the Maglin / South Ballincollig Expansion Area. Proposals shall also address pluvial flood risk in areas where surface water ponding could occur and ensure that floor and street levels are designed to manage any potential risks or exceedances.
SDS_R03	Maintain existing overland and channelized flow paths and protect floodplain storage and conveyance areas. Where it is essential to modify flow paths or ground levels in floodplains, the impact of any such modifications should be quantified and mitigated as part of a site-specific flood risk assessment.
SDS_R04	The design of amenity and public open space adjacent to the river and in areas of flood risk should incorporate Natural Flood Management Measures and seek to achieve Biodiversity Net Gain for the development.
SDS_R05	Development along natural watercourses should comply with the Inland Fisheries Ireland Guidance 'Planning for Watercourses in the Urban Environment (2020), including in particular the maintenance of a minimum riparian zone of 35 metres for river channels greater that 10 metres in width, and 20 metres for rivers channels less than 10 metres in width.
SDS_R06	Proposals to construct new or replacement culverts or bridges on watercourses in the Plan Area shall be subject to approval from the OPW, in accordance with Section 50 of the Arterial Drainage Act 1945.

Table 7.1: Recommendations for Inclusion in Development Plan

8 References

Construction Industry Research and Information Association (CIRIA) C753 - The SuDS Manual (2015)

Cork City Development Plan 2015 – 2021

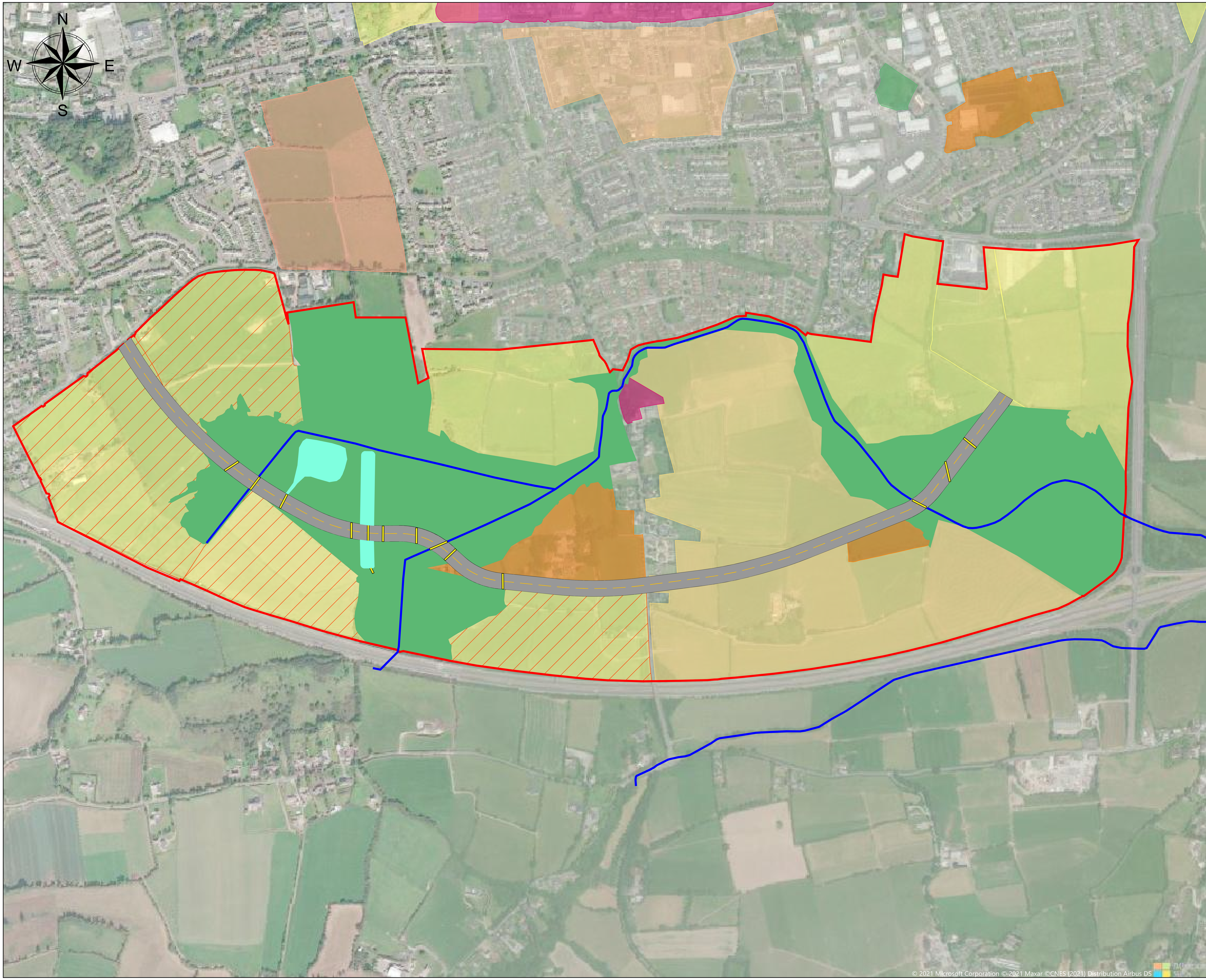
Environmental Protection Agency Maps: www.epa.ie

European Union Water Framework Directive (2000/60/EC)

Geohive Online Maps: www.geohive.ie

Geological Survey Ireland (GSI): www.gsi.ie

Greater Dublin Strategic Drainage Study (GDSDS), 2005



GENERAL NOTES :

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4. THIS MAP IS INTENDED TO REPRESENT THE PREDICTED FLOOD EXTENTS WITHIN THE STUDY AREA BOUNDARY ONLY AND SHOULD NOT BE RELIED UPON FOR ANY AREAS OUTSIDE OF THIS BOUNDARY, EVEN WHERE FLOOD EXTENTS HAVE BEEN MAPPED.

LEGEND :

STREAM/RIVER CHANNEL
STUDY AREA BOUNDARY
PROPOSED COMPENSATION STORAGE AREAS
ASSUMED SPINE ROAD ALIGNMENT
PROPOSED CULVERT

DEVELOPMENT PLAN LAND ZONING LEGEND :

ZO 01 - SUSTAINABLE RESIDENTIAL NEIGHBOURHOODS
ZO 02 - NEW RESIDENTIAL NEIGHBOURHOODS
ZO 03 - TIER 3 RESIDENTIAL NEIGHBOURHOODS
ZO 07 - URBAN TOWN CENTRE
ZO 13 EDUCATION
ZO 16 PUBLIC OPEN SPACE

REV	DATE	DESCRIPTION	BY	APP
P01	29/10/21	DRAFT ISSUE	JR	MF

PROJECT:

SOUTH BALLINCOLLIG DRAINAGE STUDY

TITLE:

PROPOSED ZONING MAP WITH SPINE ROAD, CULVERTS & COMPENSATION STORAGE - CONCEPT PLAN

CLIENT:

Comhairle Cathrach Chorcaí
Cork City Council

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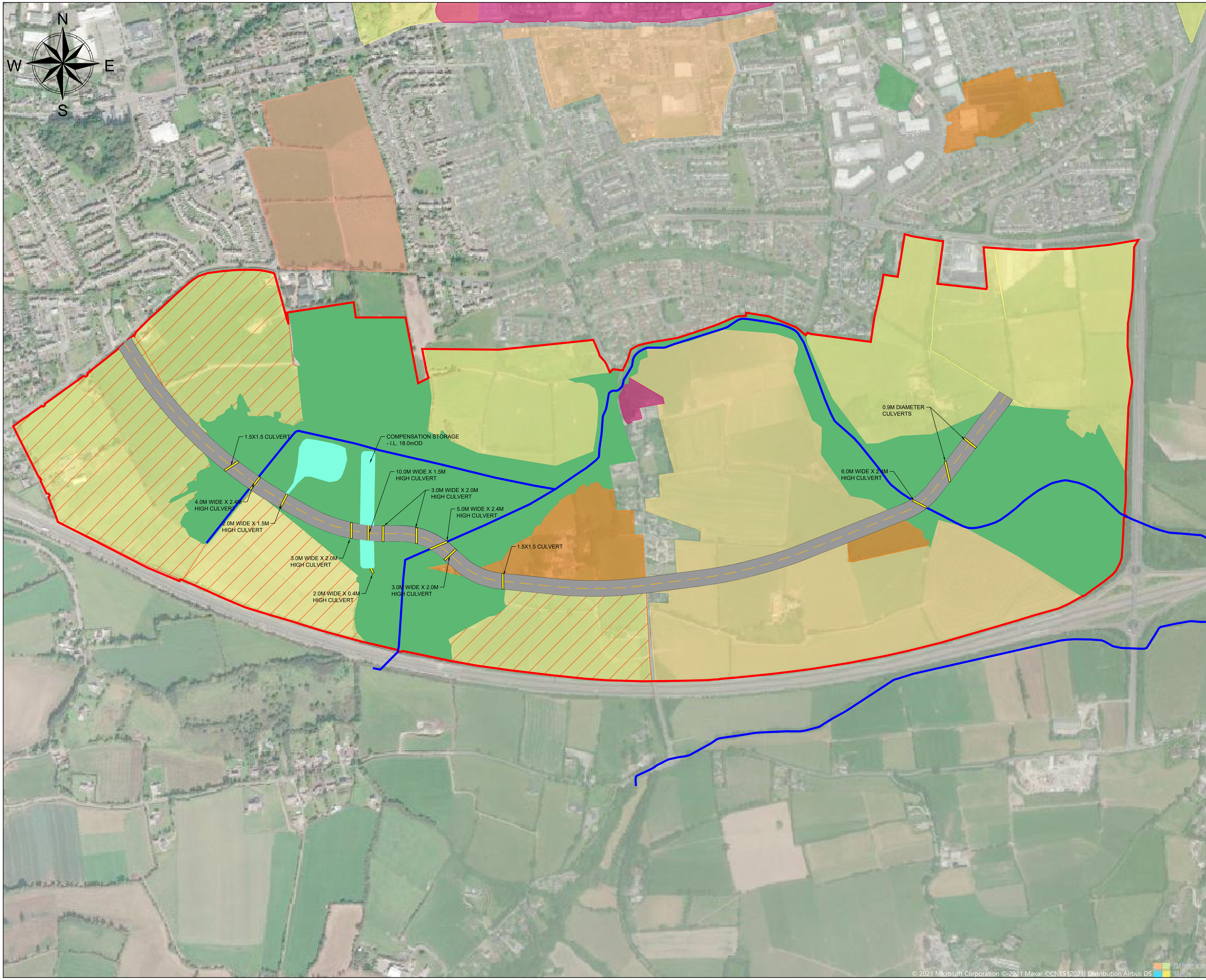
JR	MF	MF
PROJECT NUMBER: 21721	DATE: 29/10/2021	SCALE @ A1: NTS

STATUS DESCRIPTION: FOR INFORMATION
STATUS: S2

DRAWING NUMBER: 21271 - MWP -ZZ-ST-DR-C- 0003
REV: P01



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LEGEND :

STREAM/RIVER CHANNEL

STUDY AREA BOUNDARY

PROPOSED COMPENSATION STORAGE AREAS

ASSUMED SPINE ROAD ALIGNMENT

PROPOSED CULVERT

DEVELOPMENT PLAN LAND ZONING LEGEND :

ZO 01 - SUSTAINABLE RESIDENTIAL NEIGHBOURHOODS

ZO 02 - NEW RESIDENTIAL NEIGHBOURHOODS

ZO 03 - TIER 3 RESIDENTIAL NEIGHBOURHOODS

ZO 07 - URBAN TOWN CENTRE

ZO 13 EDUCATION

ZO 16 PUBLIC OPEN SPACE

NOTE ON CULVERT SIZING & COMPENSATION STORAGE AREAS:
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P02	23/12/21	INDICATIVE CULVERT SIZING ADDED	JR	MF
P01	29/10/21	DRAFT ISSUE	JR	MF

PROJECT:

SOUTH BALLINCOLLIG DRAINAGE STUDY

TITLE:

PROPOSED ZONING MAP WITH SPINE ROAD, CULVERTS & COMPENSATION STORAGE - CONCEPT PLAN

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FOR INFORMATION	S2

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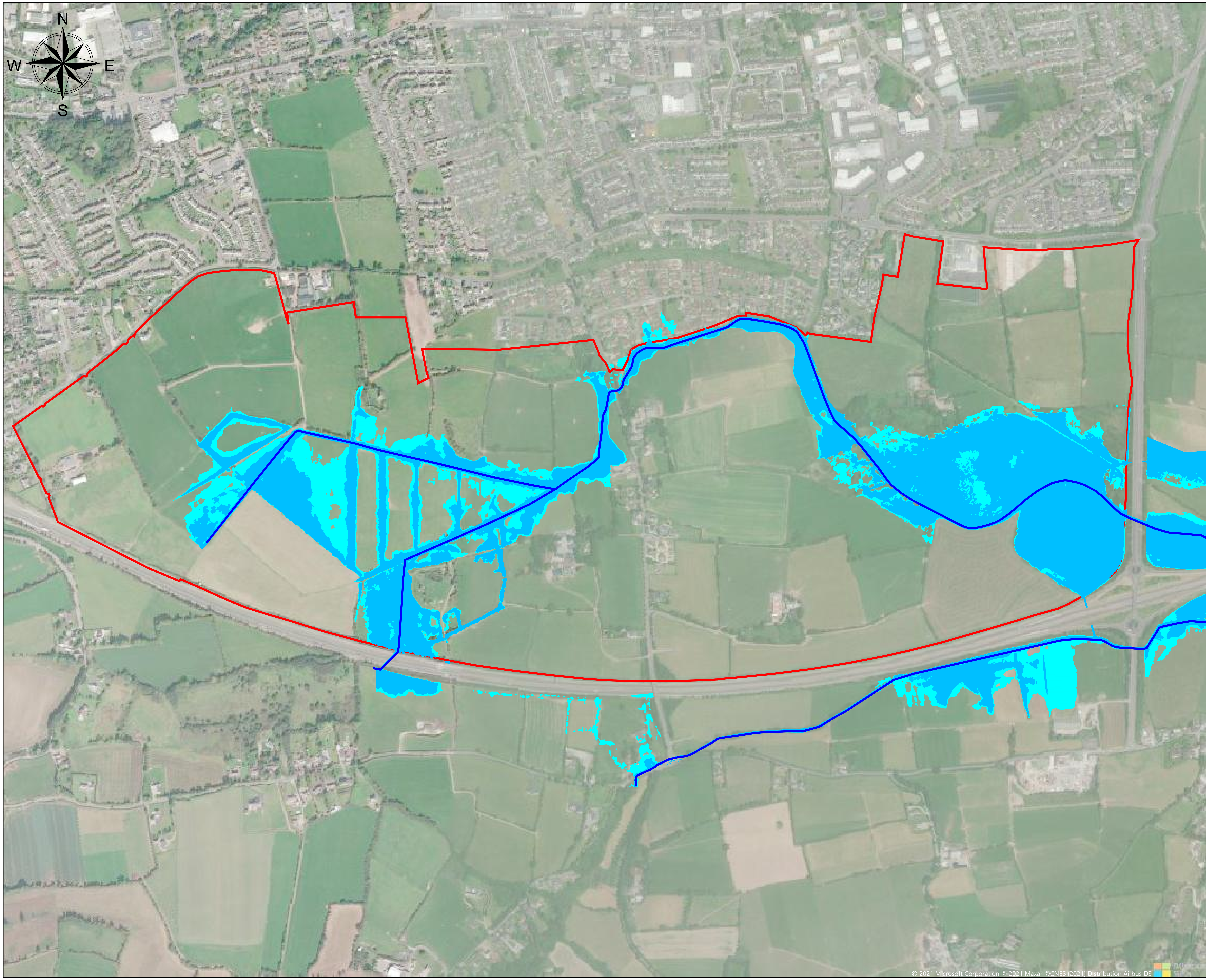
21271 - MWP -ZZ-ST-DR-C- 0003	P02



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LEGEND :

STREAM/RIVER CHANNEL

STUDY AREA BOUNDARY

1% AEP FLOOD EXTENT (LANDS WITHIN THIS AREA ARE IN FLOOD ZONE A)

0.1% AEP FLOOD EXTENT (LANDS WITHIN THIS AREA ARE IN FLOOD ZONE B)

REV	DATE	DESCRIPTION	BY	APP
P03	29/10/21	FINAL DRAFT	JR	MF
P02	29/03/21	DRAFT ISSUE FOR DISCUSSION	BC	MF
P01	25/03/21	DRAFT ISSUE FOR DISCUSSION	BC	MF

PROJECT:

TITLE: SOUTH BALLINCOLLIG DRAINAGE STUDY

TITLE:

EXISTING FLUVIAL FLOOD EXTENT MAPS - CURRENT SCENARIO

CLIENT:

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BC	MF	MF
PROJECT NUMBER: 21721	DATE: 25/03/2021	SCALE @ A1: NTS

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