Hydraulic and Hydrological Assessment: Cherry Orchard

Final Report

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This report describes work commissioned by Malone O'Regan Consulting Engineers, on behalf of the National Development Finance Agency, by an instruction dated Tuesday 11 June 2024. The Client's representative for the contract was Frank Moran of Malone O'Regan Consulting Engineers. Jemima Kivikoski, Laura Foster, Kate de Smeth and Tom Sampson of JBA Consulting carried out this work.

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1 Introduction

1.1 Background to the project

JBA Consulting have been engaged by Malone O'Regan Consulting Engineers to provide an hydraulic assessment to inform a Stage 3 Flood Risk Assessment of the proposed daylighting design for a culverted watercourse in the proposed Cherry Orchard Avenue social housing development. This development is overseen by the National Development Finance Agency (NDFA) and Dublin City Council (DCC). A hydromorphological assessment of the proposed design was carried out by JBA Consulting in September 2024 which informed the current design assessed in this report.

1.2 Study aims

The aim of this study is to evaluate the hydraulic characteristics of the current proposed channel design under a range of present day rainfall return periods: 1-in-2-year (50% Annual Exceedance Probability (AEP)), 1-in-10-year (10% AEP), 1-in-100-year (1% AEP) and 1-in-1000-year (0.1% AEP), and climate change scenarios representing the Medium-Range Future Scenario (MRFS) and High-End Future Scenario (HEFS). Maximum water levels and depth are the primary outputs used in evaluating flooding characteristics.

1.3 Proposed Development

The proposed development is a mixed-use development of 171 residential units and neighbourhood centre on a site of c.3.64 ha bound by Cherry Orchard Meadow, Blackditch Road, Croftwood Crescent and Cherry Orchard Avenue, Cherry Orchard, Dublin 10, which will consist of the following:

- Construction of new neighbourhood centre block at the corner of Blackditch Road and Cherry Orchard Avenue, ranging in height from 4 to 5 storeys comprising; 4 no. retail units (599 sqm retail space) and 3 no. community, arts and cultural space units (total 615 sqm) at ground floor level; and 107 'Older Persons' apartment units (94 no. 1-bed and 13 no. 2 bed) on all floors; and stores, bin and bicycle storage and a plant room;
- Construction of 64 no. 2 storey semi-detached/terraced houses (13 no. 2-bed and 51 no. 3-bed) arranged in 3 clusters facing adjoining roads and new internal streets;
- Provision of a new central park extending from Cherry Orchard Meadow to Cherry Orchard Avenue comprising 0.76 ha public open space which includes a restored watercourse feature from the daylighting of the Blackditch Stream (currently culverted underground), play area, civic plaza, pedestrian bridge and walkways along the stream;
- A total of 638 sqm communal open space;

- Provision of 91 no. car parking spaces, 4 no. motorcycle spaces and 1 no. loading bay;
- Provision of 228 no. bicycle parking spaces;
- Three vehicular accesses are proposed from Blackditch Road, Cherry Orchard Meadow and Cherry Orchard Avenue;
- Road works and public realm improvements along Cherry Orchard Avenue between Blackditch Road, Croftwood Crescent comprising; demolition of roundabout at junction of Cherry Orchard Avenue and Cherry Orchard Grove and replacement with a T-junction; new pedestrian crossings and relocation of 2 no. existing bus stops (Stop ID:2909 and Stop ID: 7507);
- Site clearance works including removal of existing fencing surrounding the site;
- Boundary treatments, public lighting, drainage works, traffic calming measures, new pedestrian crossings, internal roads and footpaths, landscaping and ESB substation; and
- All ancillary site services and development works above and below ground.

Figure 1-1 Proposed development layout (SHA, 2025)

2 Model Design

2.1 Input Parameters

A 1D-2D linked TuFLOW model was developed using data provided by Malone O'Regan Consulting Engineers and publicly available LiDAR data from the GSI Open Topographic Viewer.

The most up-to-date landscape drawings[1] provided to JBA were used to design a ground model representing the proposed channel, berms, and riparian zones. Drawings included zone boundaries, spot heights, wall heights and channel cross sections. Elevations provided were used to alter the existing LiDAR ground data to represent the post-development site topography. As the watercourse is currently culverted, pre-development scenarios were not modelled. The baseline flood extents discussed in the Section 3 represent flooding characteristics of the open channel, post-development.

The watercourse generally flows from northwest to southeast. The inflow and outflow points are represented as 1350mm circular culverts. The upstream invert level is given as 48.00 mAOD and the downstream invert is 47.58 mAOD, giving a gradient of approximately 1:2.42.

The proposed bridge was not included in the model parameters for reasons discussed in Section 3.2.

2.1.1 Hydrology

Flow inputs into the model were extracted from the Phase 2 Greater Dublin Strategic Drainage Study (GDSDS) Infoworks ICM model at the upstream node SO08334201.1, see Figure 2-1. Modelled flows are associated with 60-minute peak storm events for a range of rainfall return periods.

A double peak, see Figure 2-2, is apparent in the hydrograph of the 20-year return period and greater which suggests that upstream of the site surcharging of the drainage system or attenuation during large events may be occurring.

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[1] SHB5-CRD-DR-MAL-L-P1-0001 6.pdf, SHB5-CRD-DR-MAL-L-P1-0001-Sections.pdf (SHA, 2025)

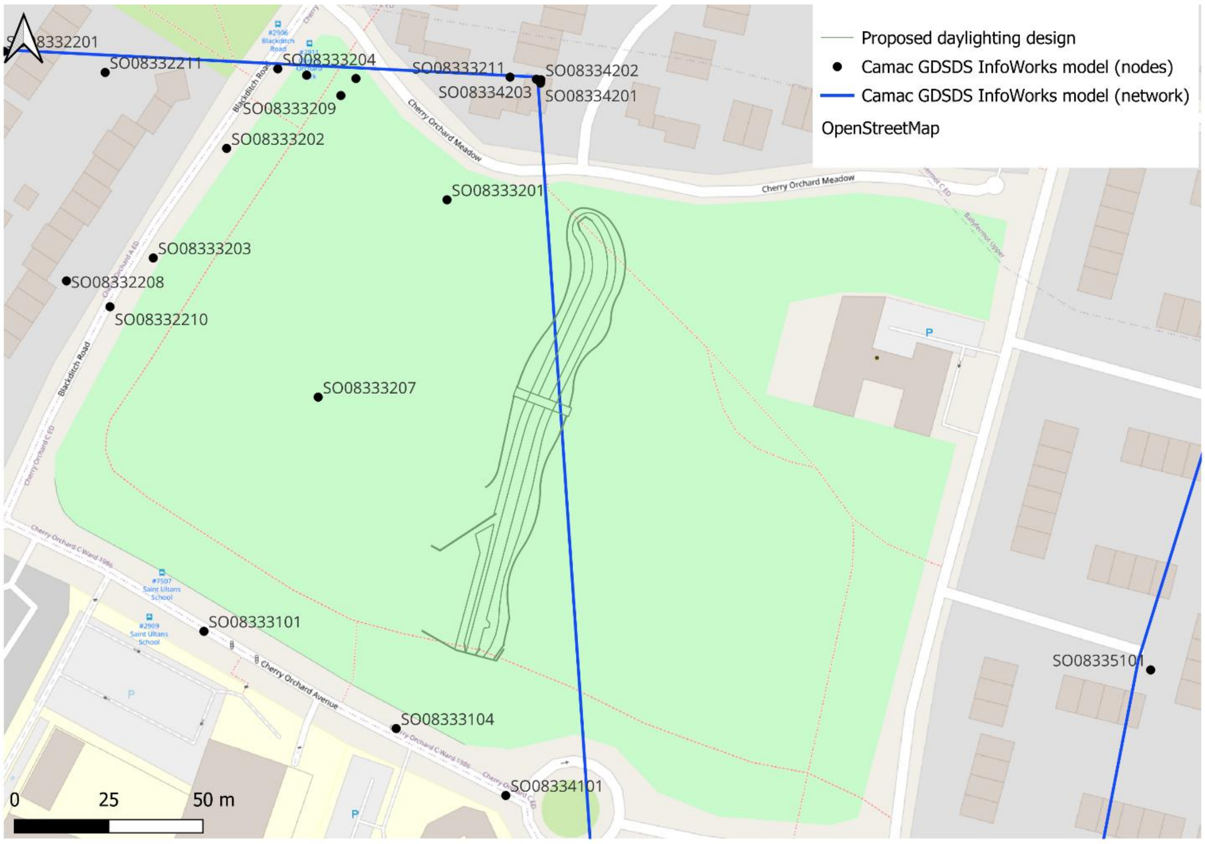

Figure 2-1 Camac GDSDS model with hydrological estimation nodes

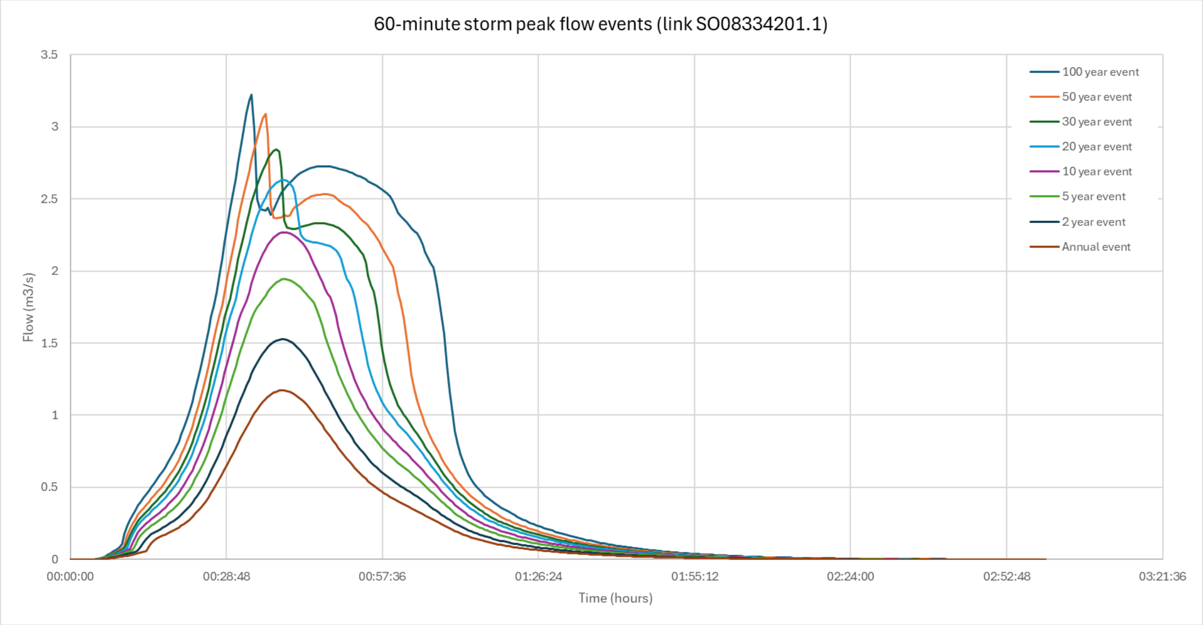

Figure 2-2 60-minute storm peak flow events at model link SO08334201.1 at the upstream end of the proposed daylighted culvert

The 1000-year event was estimated using the Rational Method and pipe flow equation (overpage) to derive a ratio between the 100-year event and the 1000-year event from which the 1000-year hydrograph was estimated. Rainfall intensities required for the Rational Method were taken from the updated 2023 Depth-Duration-Frequency (DDF) curves which provide rainfall data up to the 100-year event. Since the 2023 dataset does not include 1000-year return period values, a scaling ratio was derived from the older DDF curves to estimate the 1000-year rainfall intensity.

$$t_p = \frac{L}{V}$$

where,

L = pipe length (m)

V = average pipe velocity (m/s)

As two hydrograph shapes were derived from available catchment data, two 1000-year hydrographs were prepared, see Figure 2-3. One based on the 10-year event (M1000-60(a)) and one on the 100-year GDSDS hydrograph (M1000-60(b)).

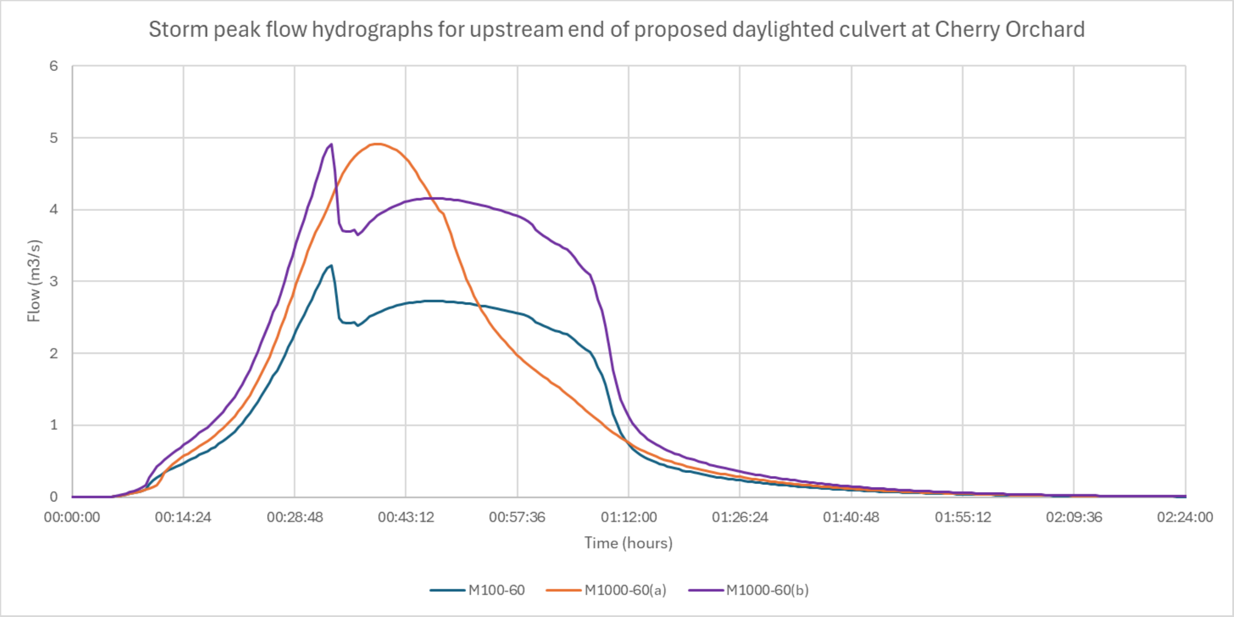

Figure 2-3 Estimated 1000-year storm peak flow hydrographs

2.1.2 Topography

The hydraulic model topography was based off landscape drawings and information provided by email. The model includes the channel, berms, streamside zone, middle zone and outer zone as delineated in the provided landscape drawings (SHA, 2025), which were digitised according to spot elevations and provided cross sections. Changes to ground elevations beyond the outer zone were deemed unnecessary, see Section 3.

3 Results and Discussion

3.1 Return Periods

The model was run using a series of return periods which represent a range of conditions, from normal storm conditions, e.g. Q2, ranging up to an extreme event e.g. Q1000.

As discussed in Section 2.1.1, two hydrographs were prepared for the Q1000 event. One shows the double peak associated with other return periods in the ICM model, the other has the smooth, single peak hydrograph shape. Despite these variations, the differences in the resulting extents and depths are negligible (see Table 3-1), and so we can conclude the model outputs are not sensitive to the selection of which Q1000 hydrograph shape to apply.

The Q1000 event in Figure 3-1 and Figure 3-2 refers to Q1000b (see Figure 2-3, refers to M1000-60(b) hydrograph).

Table 3-1 Maximum water level and depth for tested return periods

Return Period	Maximum Water Level (mAOD)	Maximum Depth (mAOD)
Q2	49.248	1.283
Q10	49.598	1.624
Q100	49.925	1.946
Q1000a	50.393	2.395
Q1000b	50.375	2.381

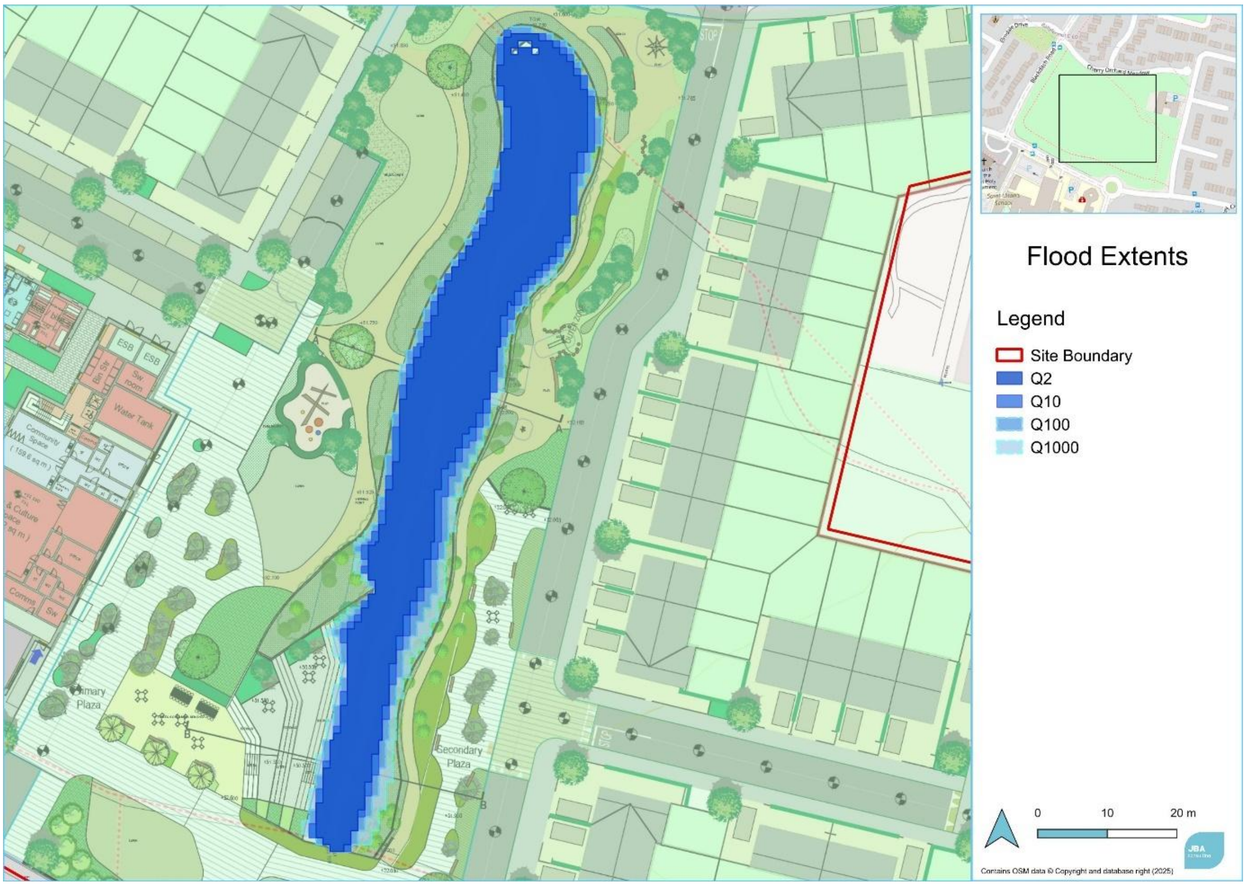

Figure 3-1 Flood extents

As the channel is set at a sufficiently low level and the retaining wall acts as a barrier in the riparian zone gradient, water is effectively contained within the riparian zones, see Figure 3-2. Further increases in water depth would be tolerated as there is adequate freeboard between the maximum Q1000 water level and the top of the retaining wall.

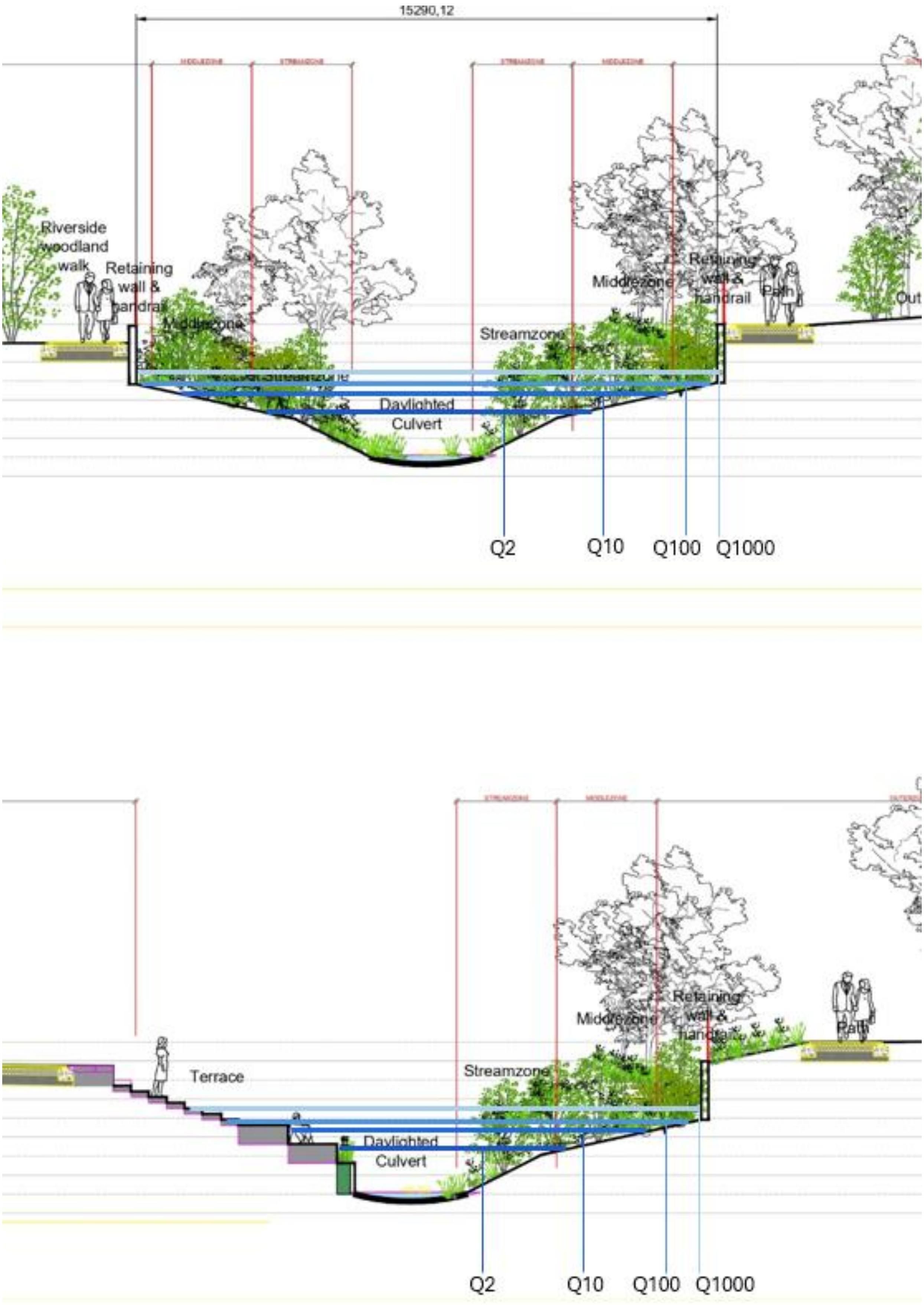

Figure 3-2 Cross sections and maximum water levels

3.2 Flow Monitoring

A comparison of hydrographs at selected points along the channel was undertaken to illustrate the influence of the daylighted channel on flows. The selected points were; the outlet pipe (most upstream, and representing the Inflow to the daylighted section), the inlet pipe (most downstream, and representing the Outflow from the daylighted section), and two intermediate points along the channel (labelled 'A' and 'B'), see Figure 3-3.

Figure 3-3 Flow monitoring locations

Given the watercourse is fully culverted, the baseline hydrographs derived in Section 2.1.1 and used as inputs for the model were taken to represent a pre-development 'baseline' scenario, denoted as 'Inflow' in Figure 3-3. Monitoring points A, B, and the point of Outflow from the channel were used to represent the post-development 'design' scenario, as at these points there has been an interaction between water and the daylighted channel.

To illustrate upstream flow conditions, the baseline and design hydrographs were combined on the same plot, shown in Figure 3-4. A small reduction in peak flow and slight shift to the right can be seen for the Q2 and Q10 return periods. This is likely due to the pooling observed behind the headwall which is discussed in Section 3.3. This effect lessens with the higher return periods.

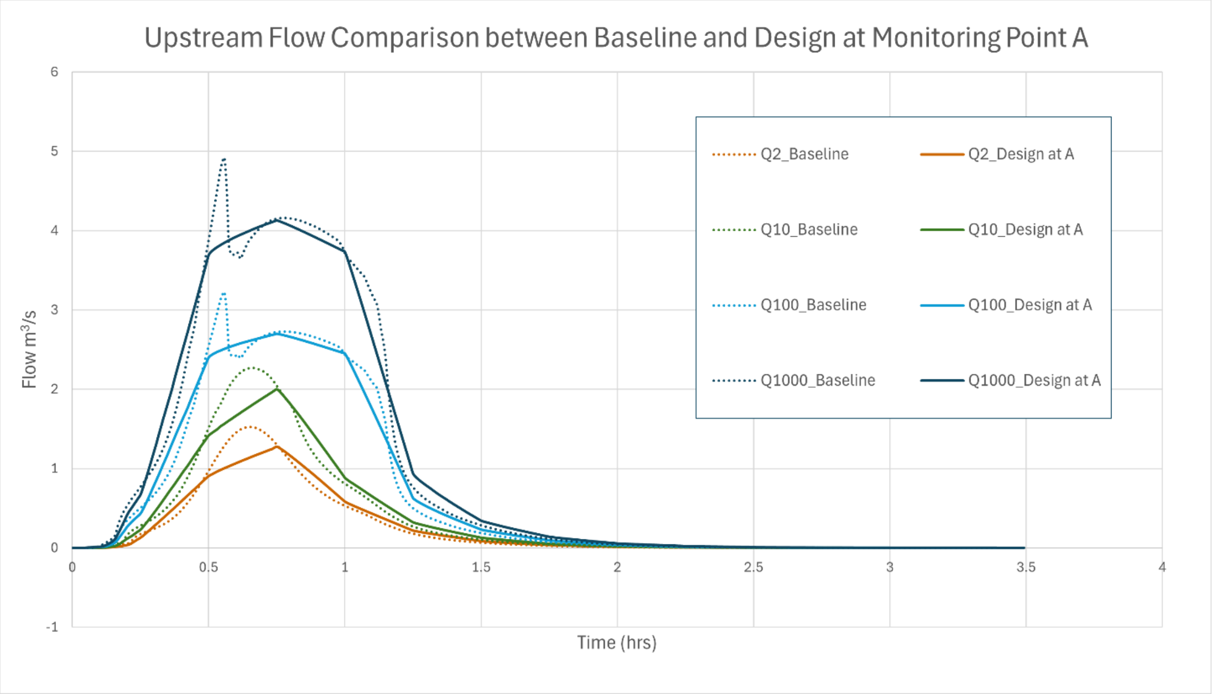

Figure 3-4 Modelled return periods at the point of inflow (baseline scenario) and at upstream point A (design scenario)

Figure 3-5 shows the outflow from the channel at the outflow point depicted in Figure 3-3. Some spiking of the flow, particularly under the Q10 return period, is likely due to some turbulence or recirculation as the outflow pipe inlet becomes partially submerged.

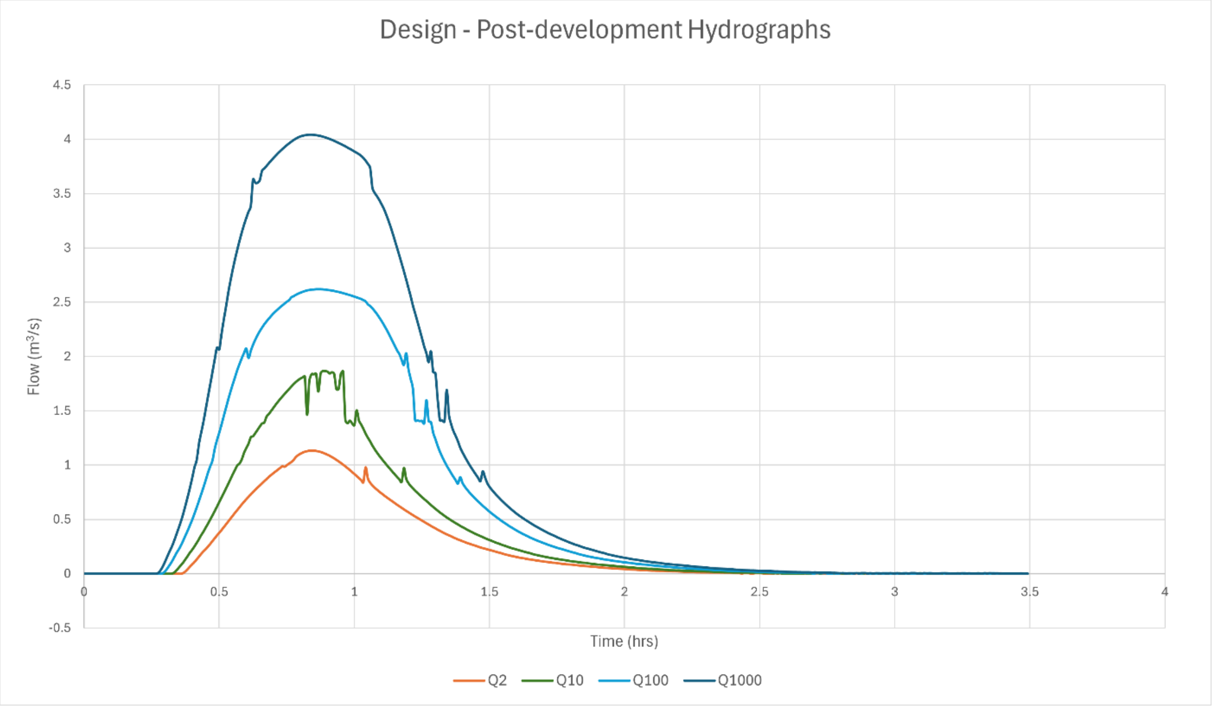

Figure 3-5 Hydrographs for modelled return period scenarios at the downstream outlet pipe entrance

A comparison between the baseline and design scenarios at the outflow point is shown in Figure 3-6, which demonstrates the shift in time to peak. The reductions in peak flow observed under each return period are summarised in Table 3-2 below.

Table 3-2 Differences observed between Baseline and Design peak flows at the outflow

Return Period	Q2	Q10	Q100	Q100
Baseline Peak (m3/s)	1.31	2.04	2.72	4.15
Design Peak (m3/s)	1.13	1.87	2.62	4.04
Difference (m3/s)	0.17	0.18	0.10	0.11

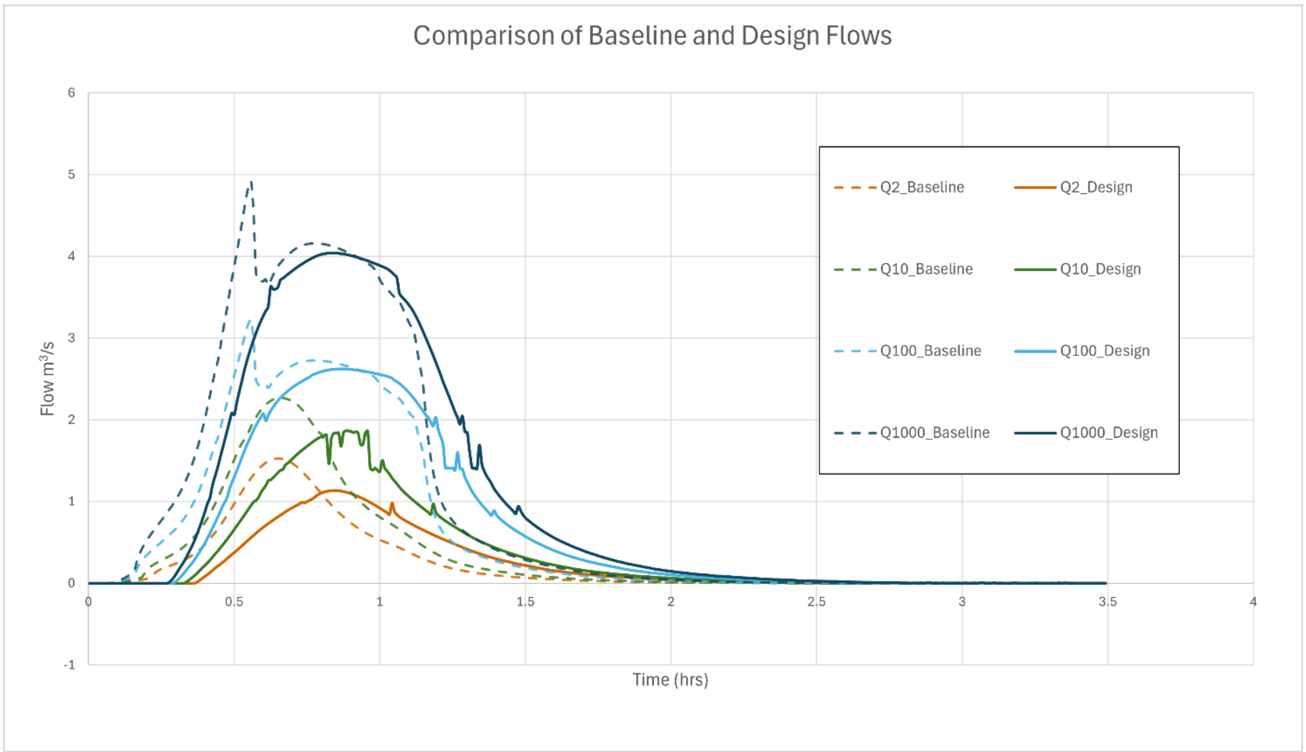

Figure 3-6 Modelled return periods at the point of inflow (baseline scenario) and point of outflow (design scenario)

Comparison of flow behaviour within the channel away from the inlet or outlet structures was assessed at the two in-channel locations, labelled 'A' and 'B', as shown in Figure 3-3. Spikes noted in the outlet pipe hydrographs are not present in the in-channel hydrographs suggesting there is good unidirectional flow (Figure 3-7). Hydrographs at these locations for each modelled return period closely match those of the inflow and outflow locations.

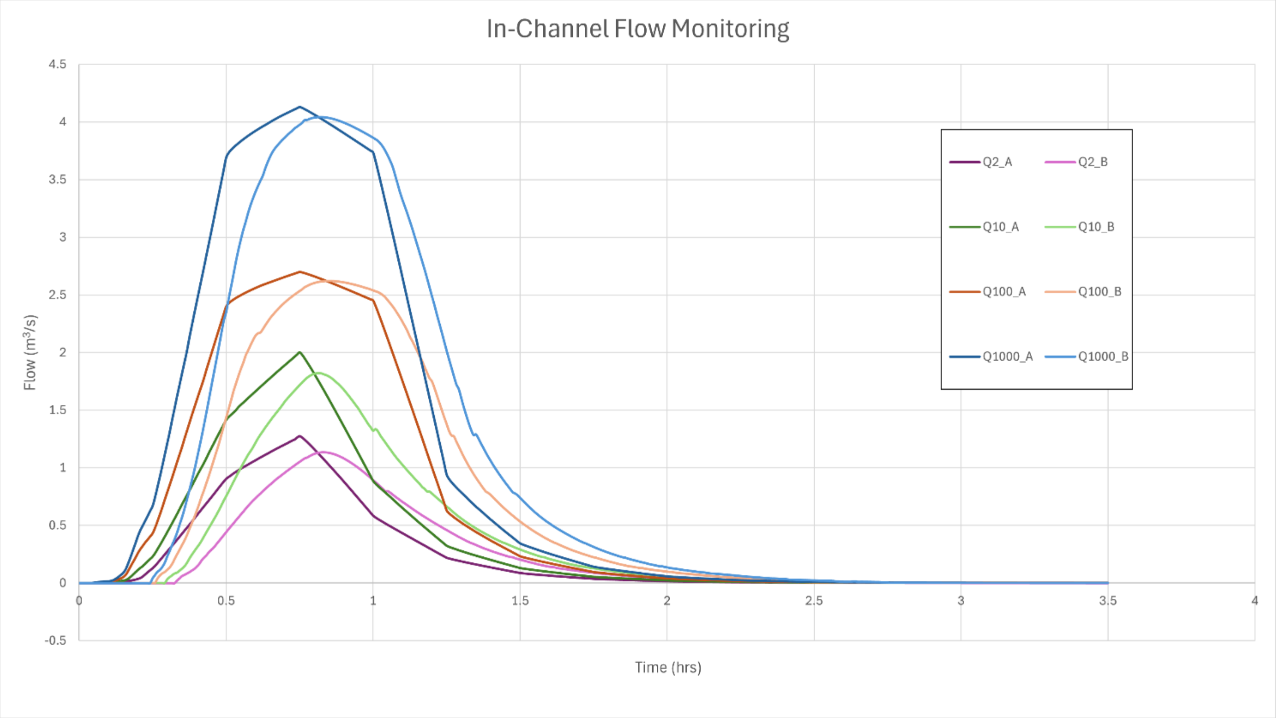

Figure 3-7 In-channel flow at upstream and downstream locations ('_A' refers to the upstream hydrograph, '_B' to the downstream hydrograph). Refer to Figure 3-3 for monitoring locations.

3.3 Hydraulic Behaviour

Water appears to accumulate towards the upstream end of the channel and behind the headwall under all modelled return periods. This is largely a product of the limited gradient of the channel which results in spilling into the narrow streamside and middle zones at the upstream end. A minor reduction in velocity, approximately 10m downstream of the upstream outlet, can be seen in Figure 3-10. This occurs immediately after the first inset berm and at the first bend in the channel. A slight loss in energy at the bend may also contribute to the upstream pooling behaviour. Along the channel, increases in velocity are noted around the inset berms.

Maximum depths along the length of the channel for Q2 (expected normal storm conditions) and Q1000 (an extreme scenario) can be seen in Figure 3-8 and Figure 3-9, respectively.

Under each return period, water is confined to the riparian streamside and middle zones by the retaining wall, see Figure 3-2. There is sufficient clearance between the Q1000 water level and the bridge deck that the bridge is not considered to have any hydraulic influence. For this reason, the bridge was not included in the model.

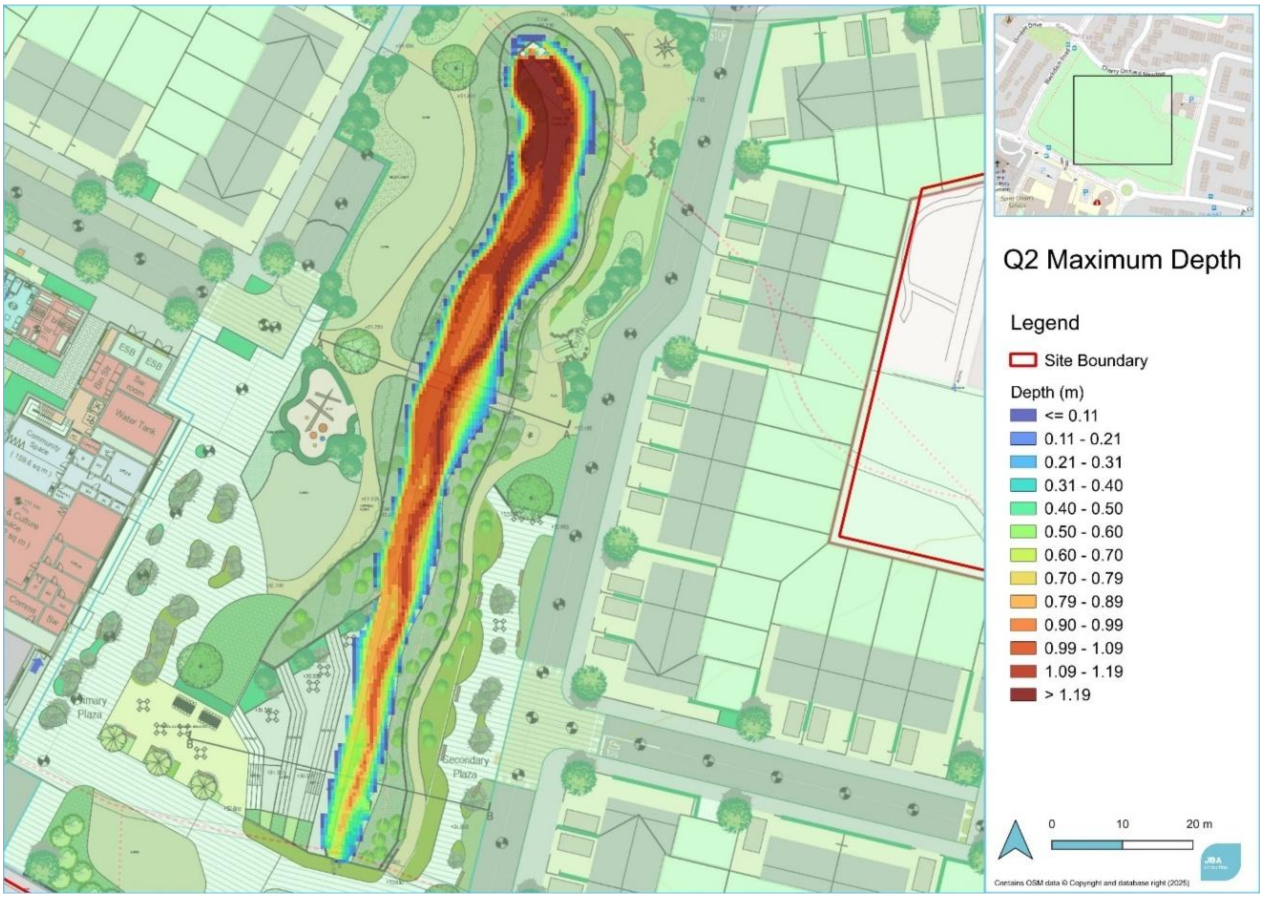

Figure 3-8 Maximum water depth during Q2 return period scenario showing upstream pooling

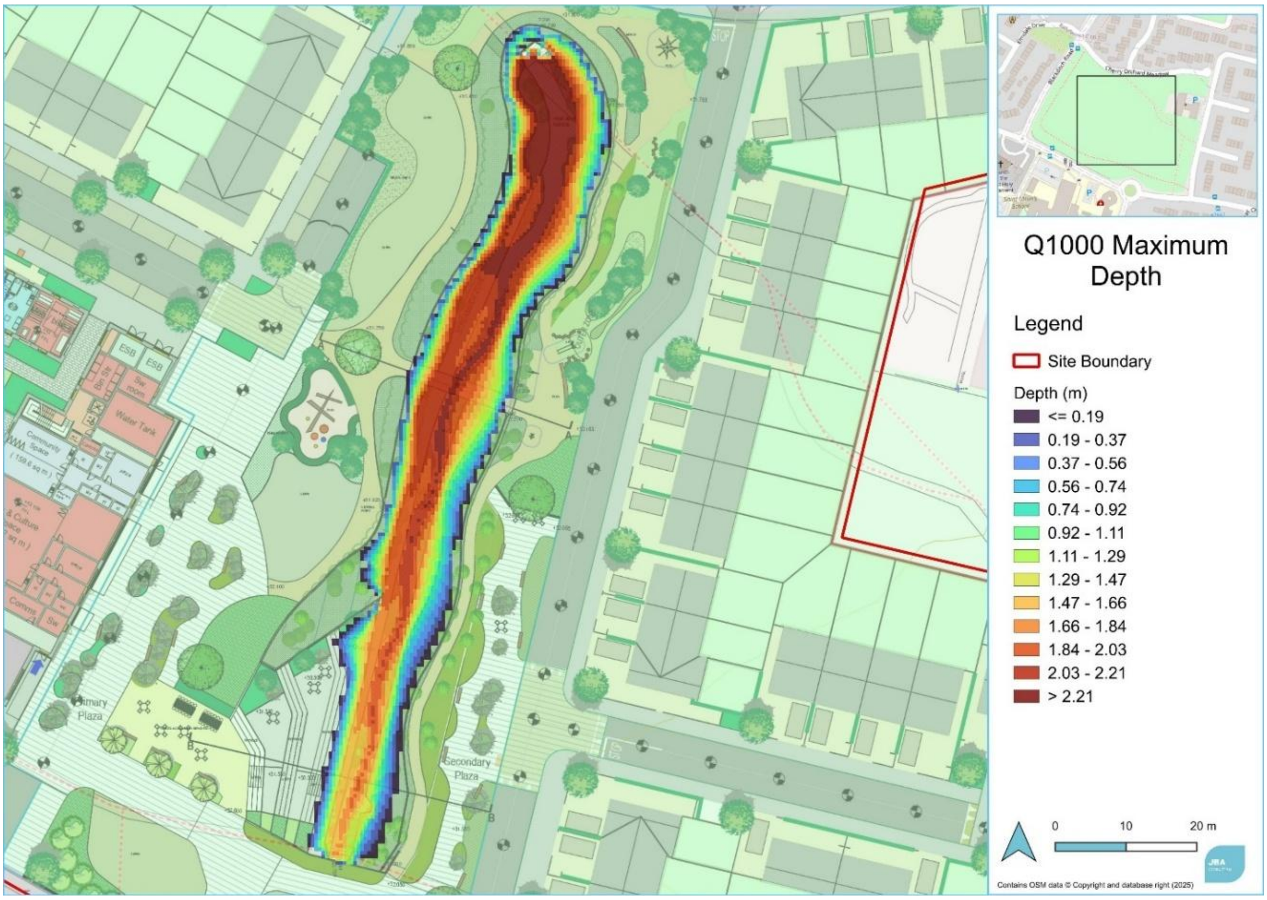

Figure 3-9 Maximum water depth during the Q1000 return period scenario

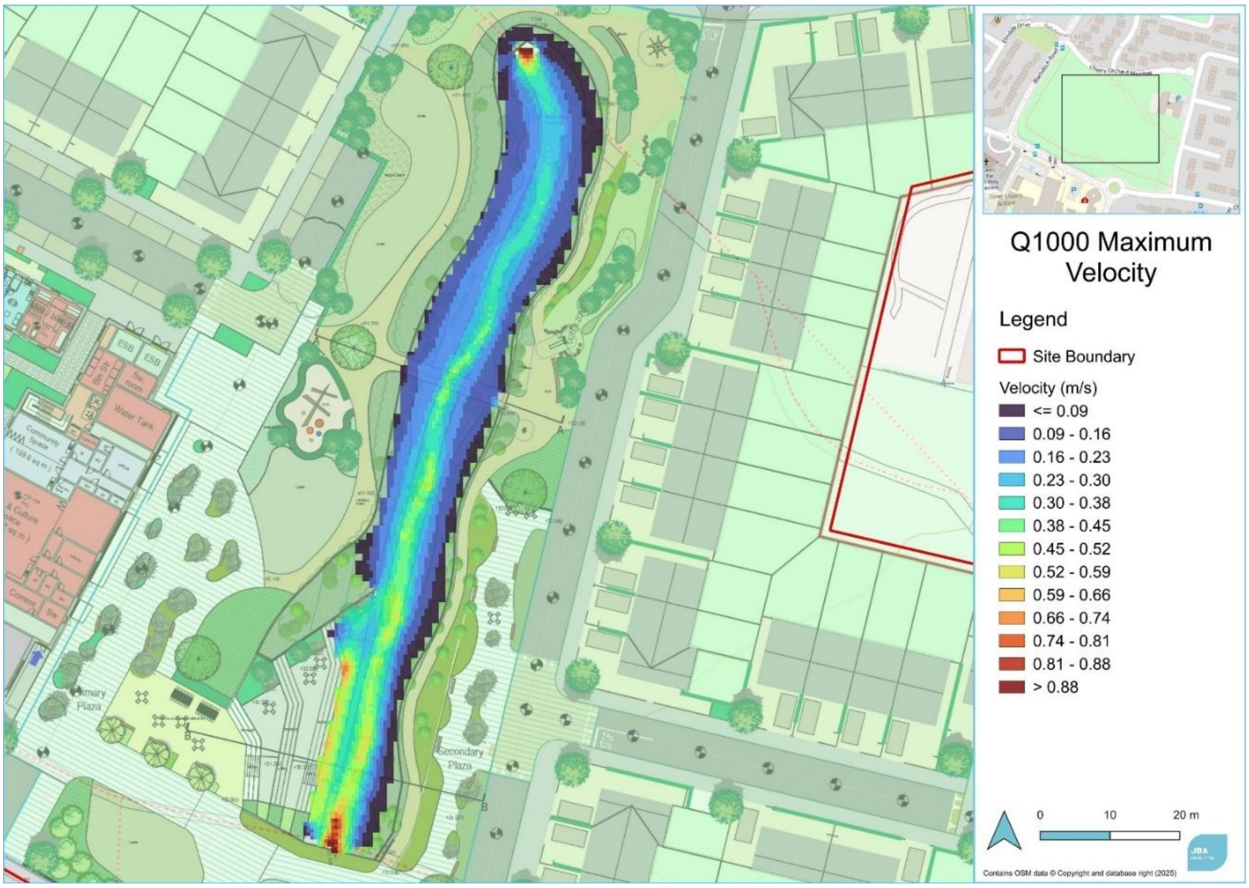

Figure 3-10 Maximum velocity within the channel and riparian zones under the Q1000 return period scenario

3.4 Climate Change Scenarios

To assess the behaviour of the channel under climate change, flows representing the Medium Range Future Scenario (MRFS) and High-End Future Scenario (HEFS) were also tested. These are derived from the Q100 return period with a 20% and 30% increase, respectively.

Table 3-3 Maximum water levels and depths for climate change scenarios

Scenario	Maximum Water Level (mAOD)	Maximum Depth (mAOD)
MRFS	50.11	2.13
HEFS	50.20	2.21

Resulting extents show little variation between the scenarios (Figure 3-11).

Figure 3-11 Flood extents under climate change scenarios

4 Conclusion

JBA Consulting has undertaken a hydraulic assessment of the current design of the culvert daylighting which forms part of the proposed social housing development at Cherry Orchard, Co. Dublin.

A hydraulic model was developed using data provided by Malone O'Regan Consulting Engineers and publicly available LiDAR data from the GSI Open Topographic Viewer. Hydrological inputs ranging from the annual return period up to the 100-year rainfall return period were extracted from the GDSDS Infoworks ICM model. The 1000-year return period was estimated using the highest flood extent and levels from two hydrograph shapes from the GDSDS.

The channel floods up to the base of the retaining wall which separates the middle riparian zone from the outer zone (as delineated in the provided landscape drawings). There is significant clearance between maximum water levels and the proposed bridge deck, illustrating that the channel is able to contain volumes generated by each return period. The culvert does not appear to be a cause of backlog and the pooling noted at the upstream end of the channel, with accumulation behind the headwall, is likely due to the shallow gradient of the channel. The dimensions of outlet culvert are sufficient in conveying the range of modelled return periods. Differences between different Q1000 results were considered to be negligible.

Increases in rainfall as a result of climate change representing the MRFS and HEFS were also tested. Results show that climate change scenarios have comparable results to the present day Q100 and Q1000 events.

This assessment is based on details of the development available at the time of preparation of this report. The assessment should be reviewed following any changes to the current design.

References

SHA (2025). Three (3) updated design drawings: Proposed Site Layout Landscape (dated 01/04/2025), SUDS Layout (dated 06/03/2025), Sections Through Open Culvert (dated 06/03/2025). Seán Harrington Architects. Dublin, Ireland.