# Poynton Reservoir Flood Study

Cheshire East Council

## Poynton Flood Study Report

D01 | V01

November 2019

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# Poynton Reservoir Flood Study

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## Contents

# Executive Summary


Following the recommendations of the 2016 Section 10 inspection of Poynton Lake Reservoir, a reservoir flood study has been carried out by Jacobs under a commission from Cheshire East Council issued in September 2019. Poynton Lake Reservoir is an ornamental lake which is located within the grounds of Poynton Park, Cheshire. The reservoir embankment carries the A523 highway, which sits on a berm on the embankment downstream face.

A computational hydraulic model of the reservoir has been constructed using new survey data. The hydrological and hydraulic analysis provides peak reservoir outflow and maximum water level for Poynton Lake Reservoir during a 0.01% AEP (10,000-year return period) safety check flood event. This has been accompanied by a sensitivity analysis on the results of the 0.01% AEP flood event assessment and the simulation of reservoir performance for the 0.1% AEP flood event (1,000-year return period design flood event).

Both storm events were simulated using the winter storm profile, as is standard for a predominantly rural catchment. The model simulations show the capacity of the overflow is exceeded and the embankment overtops early in both the 0.01% AEP and the 0.1% AEP flood events.

The modelling has shown that the overflow is inlet-controlled, operates as an orifice and has a capacity limited to approximately 0.85m$^3$/s, whilst the storm inflow to the reservoir peaks at approximately 7 and 11m$^3$/s for the 0.1% AEP and the 0.01% AEP flood events respectively.

Further analysis indicates that the capacity of the overflow will be exceeded, and that the dam will overtop, for modelled flood events in excess of approximately 2% AEP (50-year return period).

The level of the embankment clay core is unknown. It is recommended that this should be established along with other geotechnical properties of the embankment, in order to quantify the risk of seepage through the dam.

# 1. Introduction

## 1.1 Background

A Section 10 Inspection carried out in August 2016 (Mott MacDonald, 2016) for Poynton Lake Reservoir, recommended that an Emergency Drawdown Plan is prepared and that an updated Flood Study assessment is carried out for the reservoir.

## 1.2 Scope

Jacobs UK Ltd was commissioned by Cheshire East Council in September 2019 to undertake a flood study on Poynton Lake Reservoir, with the following agreed scope:

- Review available topographic data, assess suitability for use in the hydraulic modelling and if necessary, commission additional topographic survey work as required;
- Carry out a combined hydrology / hydraulic modelling analysis in accordance with the recommendations of the ICE Floods and Reservoirs Safety 4$^{th}$ Edition guidance;
- Determination of the maximum reservoir level and peak spillway flow for the 0.1% and 0.01% AEP (1,000-year and 10,000-year return period respectively) winter events;
- Wind wave overtopping calculations relative to the crest of the embankment.

## 1.3 Purpose of Report

The purpose of this report is to provide the design flood (0.1% AEP event) and safety check flood event (0.01% AEP) stillwater level and wave overtopping rate for Poynton Lake Reservoir. These are to satisfy recommendations made “in the interests of safety” in a report under Section 10 of the Reservoirs Act

## 1.4 Study Site

Poynton Lake Reservoir is an ornamental lake which is located within the grounds of Poynton Park, Poynton, Cheshire. The reservoir is impounded by an approximately 900m long embankment. The A523 road occupies a berm on the downstream face of the embankment. The location and the general arrangement of Poynton Lake Reservoir are shown in Figure 1-1 and Figure 1-2 respectively.

The reservoir overflow (Figure 1-4) features a weir which discharges into a box and then a pipe. The pipe runs through the embankment and discharges downstream of the embankment into Poynton Brook.

There are two catchments (one direct and one indirect) which drain to the reservoir (Figure 2-2). The direct catchment is located to the east of Poynton Lake, and is estimated to have an area of 1.96km$^2$, the land use of the direct catchment is predominantly agricultural with areas of woodland and sub-urban residential development.

The indirect catchment (catchment area 4.00km$^2$), is that of a tributary of the Poynton Brook. The tributary naturally bypasses the lake to the south and passes under the A523 through a culvert. About 170m upstream of the A523 crossing, there is an intake structure which diverts some of the flow to the Poynton Lake Reservoir. The bi-furcation structure is formed by a weir across the tributary and an intake structure/pipe. The catchwater structure is made of a culvert under Woodside Lane, followed by an open channel and a culvert under South Park Drive which discharges into Poynton Lake Reservoir.

The interception, and diversion out of the catchment of any run off, by drains or sewer systems (shown in Figure 1-3) is assumed to be negligible for the extreme flood events, and hence not considered for this study.

A summary of key reservoir details is provided in Table 1.1. More details about the reservoir characteristics are presented in Section 5.3.

Figure 1-1: Location of Poynton Lake Reservoir

Figure 1-2: Poynton Lake Reservoir General Arrangements

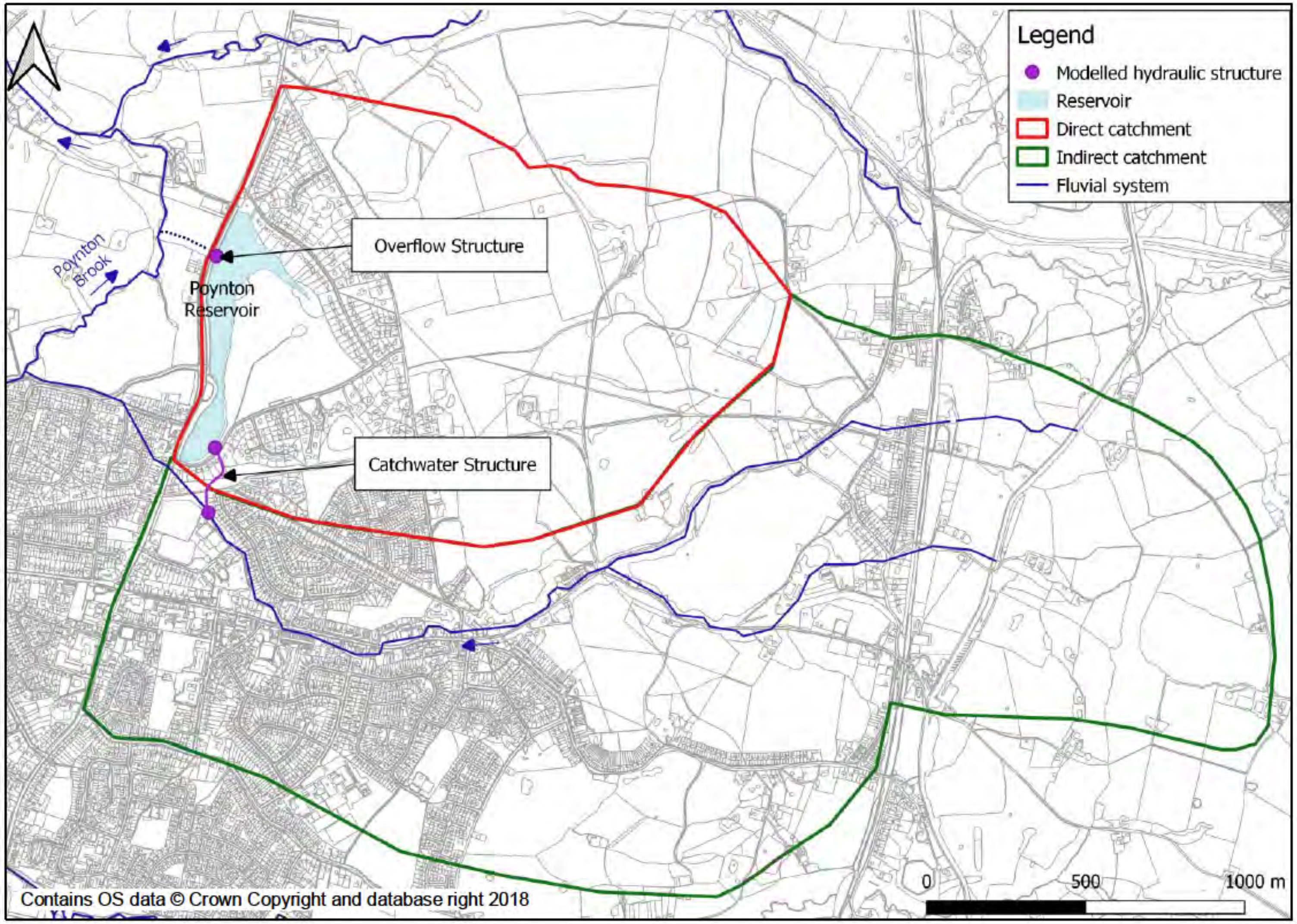


Figure 1-3: Surface Water Drainage Network at Poynton Lake Reservoir

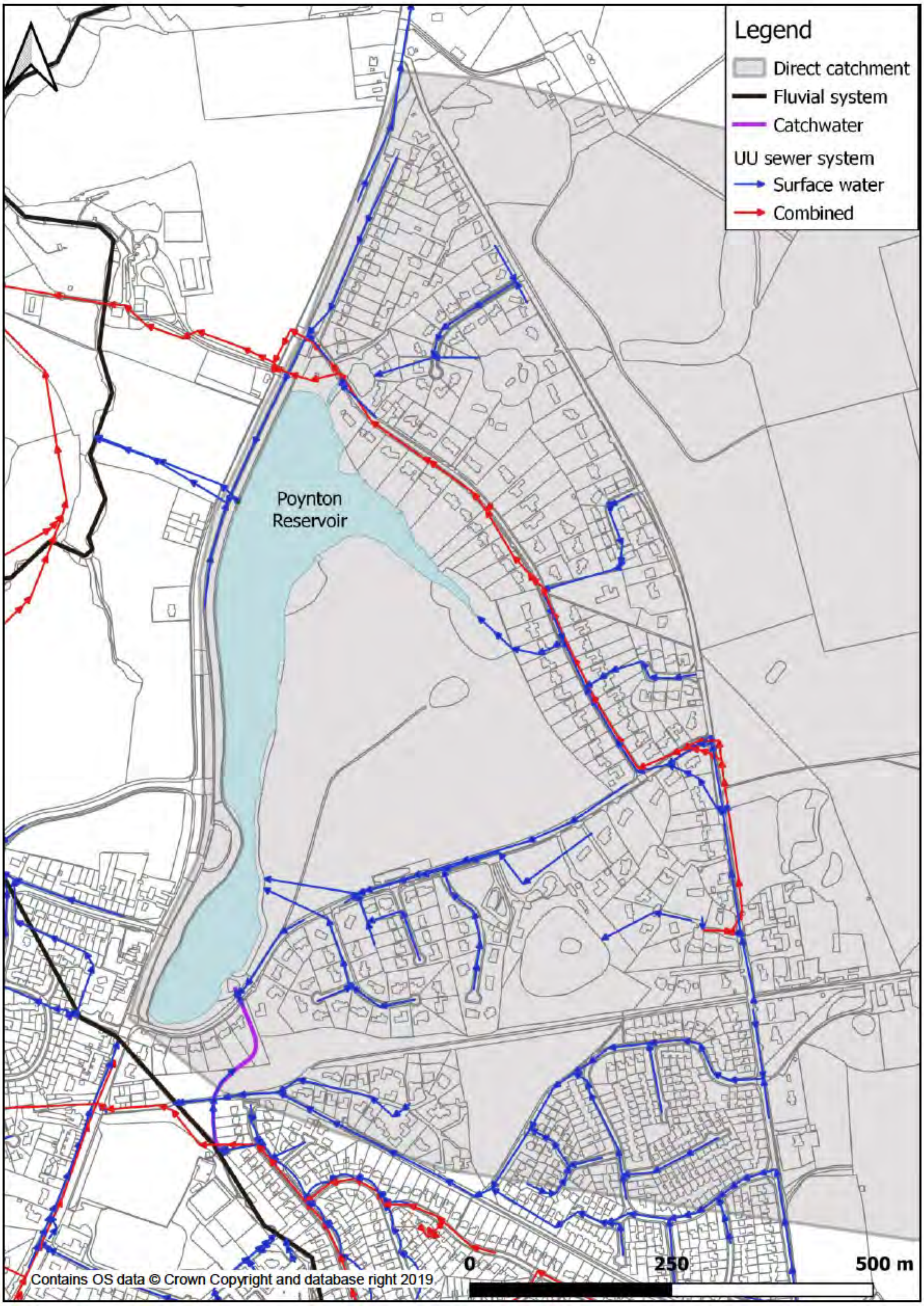


**Figure 1-4: Poynton Lake Reservoir Overflow Arrangement, photo inserts show (a) overflow box and (b) internal view of culvert inlet.**

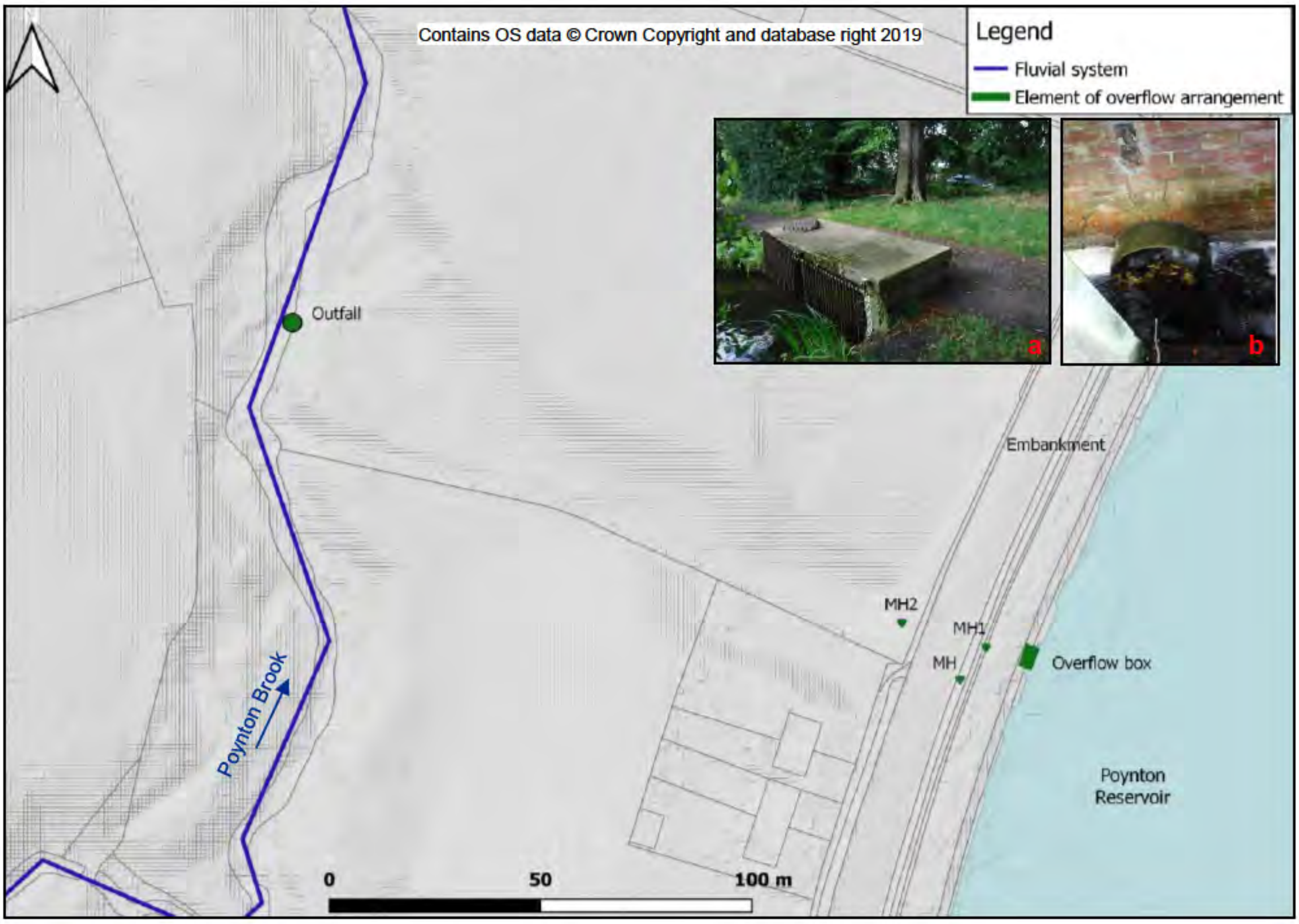



**Table 1.1: Reservoir Details**

| Poynton | | |
|---|---|---|
| **Parameter** | **Value** | **Source** |
| Surface area at overflow weir level ($m^2$) | 65,500 | OS Master Map data |
| Overflow weir crest level (mAOD) | 90.55 | 2019 Survey data |
| Overflow weir width (m) | 3.16 | 2019 Survey data |
| Minimum dam crest level (mAOD) | 90.88 | 2019 Survey data |
| Dam crest length (m) | 890 | MasterMap data and LiDAR data |

# 2. Methodology

## 2.1 General

The general approach for the determination of reservoir water levels and spillway discharges for the Poynton Lake Reservoir for the required flood events involved the development of an integrated hydrological and hydraulic model of the system using the Flood Modeller Pro software package, version 4.4 (Jacobs, 2018).

Key stages in the progression of the study were as follows:

1) Data collection from available historic reports and drawings and that obtained during the site visit.
2) Review of the (Jacobs in 2019) topographic survey data of the dam crest, the overflow facility and the catchwater intake and outfall.
3) Construction of the Flood Modeller Pro hydraulic routing model. The model representing the reservoir was built based upon; the topographic survey data and the LiDAR DTM data (Environment Agency, 2019).
4) Hydrological analysis for the derivation of reservoir model inflows for the 0.1% and 0.01% AEP (1,000-year and 10,000-year return period respectively) events using the methodology suggested in FEH Vol 4. The rainfall depths for T-year flood events were derived from the FEH2013 rainfall data set.
5) Model simulations were undertaken to test for the specified range of design flood events, the 0.1% AEP and 0.01% AEP (1,000-year and 10,000-year return periods respectively). The model was further tested with the 0.01% AEP (10,000-year return period) fluvial flood inflows uplifted by 30% following the North-West England 2080s scenario central (50%tile) allowance (EA, 2016)[1] in order to take climate change into consideration.
6) Additional selected return period events were simulated were carried out to estimate the return period capacity of the reservoir overflow culvert.
7) Determination of wave overtopping using Floods and Reservoir Safety (4th Edition).
8) Model output - Flood levels in the reservoir and discharge in the overflow arrangement for Poynton Lake reservoir were output from the model.

[1] Flood Risk Assessment: Climate Change Allowance (Environment Agency, 2016)

# 3. Input Data

## 3.1 Topographic Survey

Topographic survey data was carried out by Jacobs in October 2019. The survey consisted of:

- Levels on the overflow weir, levels on the approach of the overflow, dimensions of the overflow box, overflow pipe invert level and diameter, arrangement and dimensions of pipes in the manholes;
- Dam crest long section, 7 embankment cross-sections, toe of the wall long section on pavement side;
- Catchwater intake and outfall structure dimensions, 2 cross-sections of the weir across the tributary (top and bottom) and 2 cross-sections upstream and downstream of the weir.

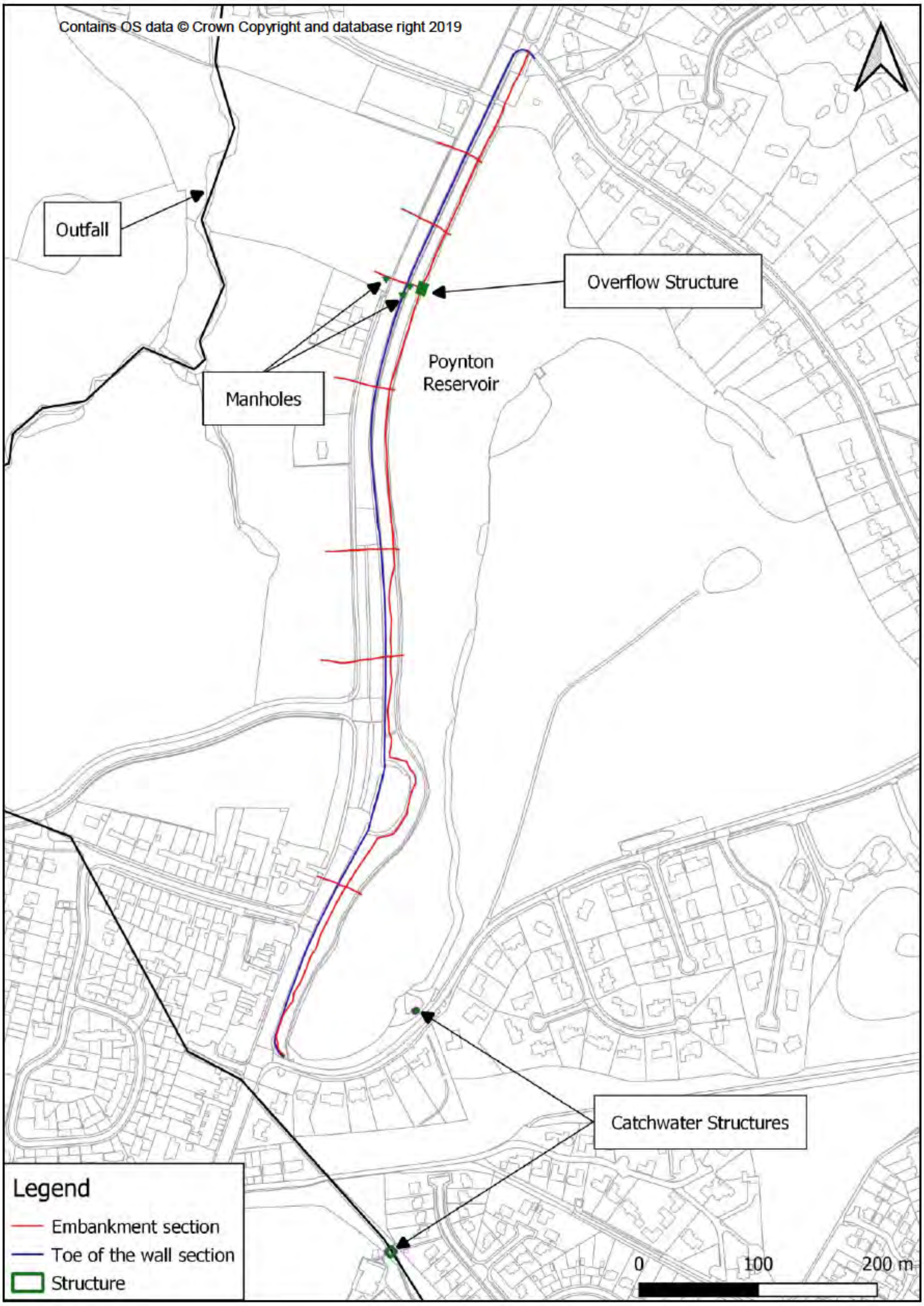
The survey results were provided relative to national spatial datum and to Ordnance Datum. A full list of deliverables for the topographical survey is provided in Table 3.1.

Note: The embankment crest is heavily vegetated, and its topography is variable. It is anticipated that the accuracy of the long section survey of the dam crest is therefore limited. Levels taken may somewhat underestimate the true crest of the dam. Whilst this is considered to be marginally conservative, and appropriate for baseline design flood assessment, sensitivity testing is required to determine the significance of dam crest level to safety check flood event.

**Table 3.1: Survey data available**

| Poynton Lake | |
|---|---|
| **File** | **Description** |
| POYNTON LAKE_3D TOPOGRAPHICAL SURVEY_Rev B.dwg | 3D CAD plan survey. Ordnance datum, national spatial datum. |
| Poynton Lake_Cross Section Survey Report.docx | Cross-Section Survey report, Poynton Lake, Cheshire East Council, 30th September 2019 |
| Poynton Lake Cross Sections.dat | Cross section database in Flood Modeller Pro format, with level in mAOD |
| Poynton Lake Embankment Long Section.dat | Long section database in Flood Modeller Pro format, with level in mAOD |
| Poynton Lake Retaining Wall Long Section.dat | Long section database in Flood Modeller Pro format, with level in mAOD |
| Poynton Lake Section 8, 9 & 10 (Weir Top & Bottom).dat | Cross section database in Flood Modeller Pro format, with level in mAOD |

**Figure 3-1: Topographic Survey**



## 3.2 Ground Elevation Data

LiDAR data (Environment Agency, 2019) with a 1m / 2m resolution grid size were available for the Poynton Lake Reservoir direct catchment and over the majority of the indirect catchment (https://data.gov.uk/dataset/).

## 3.3 Historic Reports

The following historic reports (as relevant for this flood study) were available:

- Report on an Inspection under Section 10(2) of the Act (Mott MacDonald, August 2016);
- Reservoirs Act 1975 Annual Supervising Engineer's Statement, Poynton Lake (Mott MacDonald, November 2018).

Key notes and extracts from these documents of relevance to the present study are provided below:

**Poynton Section 10 Inspection Report 2016 (Mott MacDonald)**

Section 11.2:

*"It was stated in [redacted] report of 2010 that there was no indication that a complete flood study assessment had been carried out previously. As part of his inspection therefore he prepared a 'Rapid Assessment' in accordance with the Floods and Reservoir Safety Guidelines (3rd Edition) that were in place at that time. This assessment included an assessment of the 30% Probable Maximum Flood (0.30 PMF), which is generally considered to be equivalent to the 1 in 1,000year flood event. The results of this estimation and of the routing of this flood hydrograph through the reservoir showed that the predicted flood surcharge would rise to within 0.1m of the embankment crest, and that there would be very little remaining freeboard margin to accommodate wave action on top of the flood. These results showed that the estimated maximum inflow to the reservoir at the peak of the flood would be 2.64$m^3$/sec, although there was no information provided on the amount of restriction to the incoming flow (if any) that had been taken into account to allow for the discharge capacities of the culverts that feed the water from the streams to the reservoir."*

Section 15.3:

*"In the Interests of Reservoir Safety I recommend that:*

a) *An Emergency Drawdown Plan shall be prepared for the reservoir to describe the methods to be used and the procedures to be followed in order to facilitate a lowering of the water in the reservoir by up to 300mm in the first 24 hours of an emergency situation. Further details of this requirement are given in Section 10.4.*

b) *A Flood Study Assessment shall be prepared for the reservoir. This shall include an estimation of the inflow hydrographs for the Design Flood and the Safety Check Flood, the hydraulic characteristics of the inlet works to the reservoir (direct and indirect catchments), discharge characteristics of the overflow weir and outlet pipe, and flood routing to determine flood surcharge levels. The study should also incorporate an estimate of wave heights and the potential for wave over-topping that could occur during the passing of these floods, as well as a topographic survey of the embankment crest."*

## 3.4 Gauged Data

The National Rivers Flow Archive (NRFA) website confirms that there are no gauging stations in the Poynton Lake Reservoir catchment. There is one gauging station: the Micker Brook at Cheadle (station number: 69011), approximately 9km downstream of the Poynton Lake Reservoir. The catchment area to this gauging station is $67.3km^2$. The Micker Brook at Cheadle gauge is a peak flow rated gauge with its QMED suitable for peak flow analysis but was closed in 2006. The gauge is assessed by the NRFA as having approximately 53% of the catchment being of high permeability bedrock with the remainder of the catchment being classified as moderate (6%) and mixed permeability bedrock (41%), which differs marked to that of the Poynton Lake Reservoir catchment. Since the gauging station has a catchment area much larger than that of the Poynton Lake Reservoir catchment, and differing permeability characteristics it has been assessed as an unsuitable donor for the possible transfer of information to refine the target catchment's rainfall-runoff properties.

A number of other nearby gauging stations have also been assessed for suitability as potential donors. This includes Goyt at Marple Bridge (station number 69017), Dean at Stanneylands (station number 69008) and Etherow at Compstall (station number 69015). These gauges are all on substantially larger catchments and have varying degrees of similarity of catchment descriptors. They have therefore not been used for the flood study.

No record of water level recordings in the Poynton Lake Reservoir were provided by East Cheshire Council for use in this flood study.

## 3.5 Flow Transfer

In addition to the inflows from a direct catchment, the Poynton Lake Reservoir also receives inflows from a tributary of the Poynton Brook (indirect catchment) via an intake which is located at NGR: SJ 922 838. The intake has been surveyed as part of this investigation in order to represent the flow transfer within the hydraulic model. The results of the modelling indicate that up to about $0.3m^3/s$ could be diverted into the Poynton Lake Reservoir from the indirect catchment. The remainder of the flow in the tributary of the Poynton Brook continues downstream before flowing into the Poynton Brook. Figure 2-2 shows the location of the tributary of the Poynton Brook intake and associated water transfer route into the Poynton Lake Reservoir.

# 4. Catchment Hydrology

## 4.1 General

The aim of the hydrological analysis is to produce inflow hydrographs that can be applied to the hydraulic model of the Poynton Lake Reservoir. As discussed previously, there is a direct reservoir catchment which drains the land to the east of the reservoir and an indirect catchment that provides water via a pipe to the reservoir from a tributary of the Poynton Brook. The inflow hydrographs represent the full flood flow from the direct catchment and part of the flood flow from the indirect catchment draining into the Poynton Lake Reservoir.

As described in the Floods and Reservoir Safety (FRS) guidance (ICE 2015), the FEH rainfall-runoff method is the recommended hydrological method for very rare events with return periods greater than the 0.1% AEP event (1,000-year return period). In principle, REFH1 is valid to an AEP event of 1.5% (150-year return period) but has been found to overestimate design flows when it is run with a storm duration much longer than the critical or recommended durations for the catchment (see EA, 2017 and WHS, 2016). ReFH2 is valid to an AEP of 0.1% (1,000-years return period) but is not applicable for the 0.01% AEP event (10,000-year return period). The ReFH1 and ReFH2 methods are also not fully accepted as a standard approach for reservoir flood studies at the time of this assessment. Therefore, the FEH rainfall-runoff method was considered the most appropriate method for this reservoir flood hydrology and used for both return periods investigated. This has the added benefit of consistency across the range of design events.

Due to established deficiencies in the FEH1999 depth-duration-frequency (DDF) rainfall model, the updated FEH2013 DDF rainfall model was employed. This is an improvement on the FEH1999 rainfall model, especially for high return period events.

## 4.2 Catchment Description

The Poynton Lake Reservoir is located within the town of Poynton in East Cheshire and is an artificial lake constructed around 1750 (Mott MacDonald, 2016)[2]. The Poynton Lake Reservoir has a direct catchment with an estimated area of 1.96km$^2$ and has a mixed land use consisting of predominantly agricultural land, woodland and sub-urban (residential) land use within the town of Poynton. The reservoir surface area is approximately 65,280m$^2$ and represents less than 4% of the direct reservoir catchment. The reservoir is situated at an altitude of approximately 90m AOD, with a generally gently sloping upstream catchment as shown in Figure 4-1 and Figure 4-2. The Macclesfield Canal dissects the upper catchment of the indirect catchment.

According to the Geology of Britain Viewer,[3] the bedrock geology of the direct reservoir catchment is composed of Manchester Marls Formations (mudstone), Collyhurst Sandstone Formations and Pennine Middle and Lower Coal Measure Formations (mudstone, siltstone and sandstone). The bedrock is overlain by predominantly till deposits (Devensian - Diamicton) with small areas of glaciofluvial deposits (Devension) including sands and gravels. Overall the geology has been assessed as being predominantly composed of moderate permeability bedrock with a narrow strip of high permeability bedrock being present in the vicinity of the reservoir.

The bedrock geology of the indirect reservoir catchment is composed of similar geology to the direct catchment (Manchester Marls, Collyhurst Sandstone and Pennine Middle and Lower Coal Measure Formations) with the addition of Chester Formation (sandstone, pebbly (gravelly) in the lower south west area of the catchment.

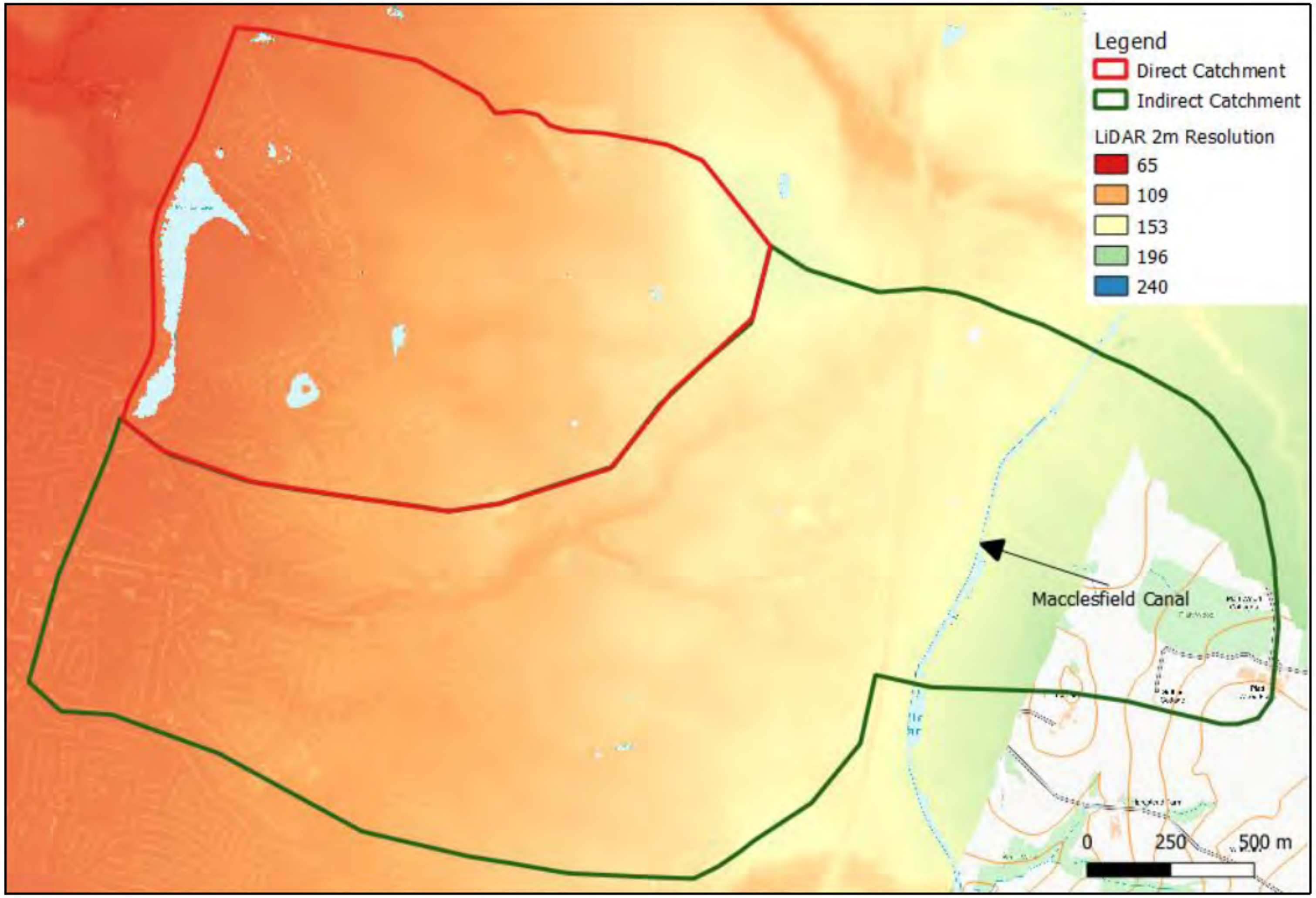
Based on the Soil Maps of England and Wales Scale 1:250,000 (Soil Survey of England and Wales, 1983) the soils within the direct reservoir catchment have been assessed as being composed of predominantly 711m – Salop (54%) and 541r – Wick 1 (46%) soil types. The soils for the indirect reservoir catchment have been assessed as being composed of predominantly 711m – Salop (95%) with small areas of 541r – Wick 1 (5%) soil

[2] Reservoirs Act 1975, Poynton Lake Reservoir, Report of an Inspection under Section 10(2) of the Act (Mott MacDonald, 2016)
[3] geology from the Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html

types. The soil type 711m – Salop is assessed as being a generally slowly permeable and seasonally waterlogged soil whereas 541r – Wick 1 being classified as being generally a deep well draining soil.

No water level recordings for the Poynton Lake Reservoir were provided by East Cheshire Council at the time of assessment. There are no continuous level or flow gauges in the catchment.

Figure 4-1: Catchments with 2m LiDAR Data

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**LiDAR data was not available for the full indirect reservoir catchment.*

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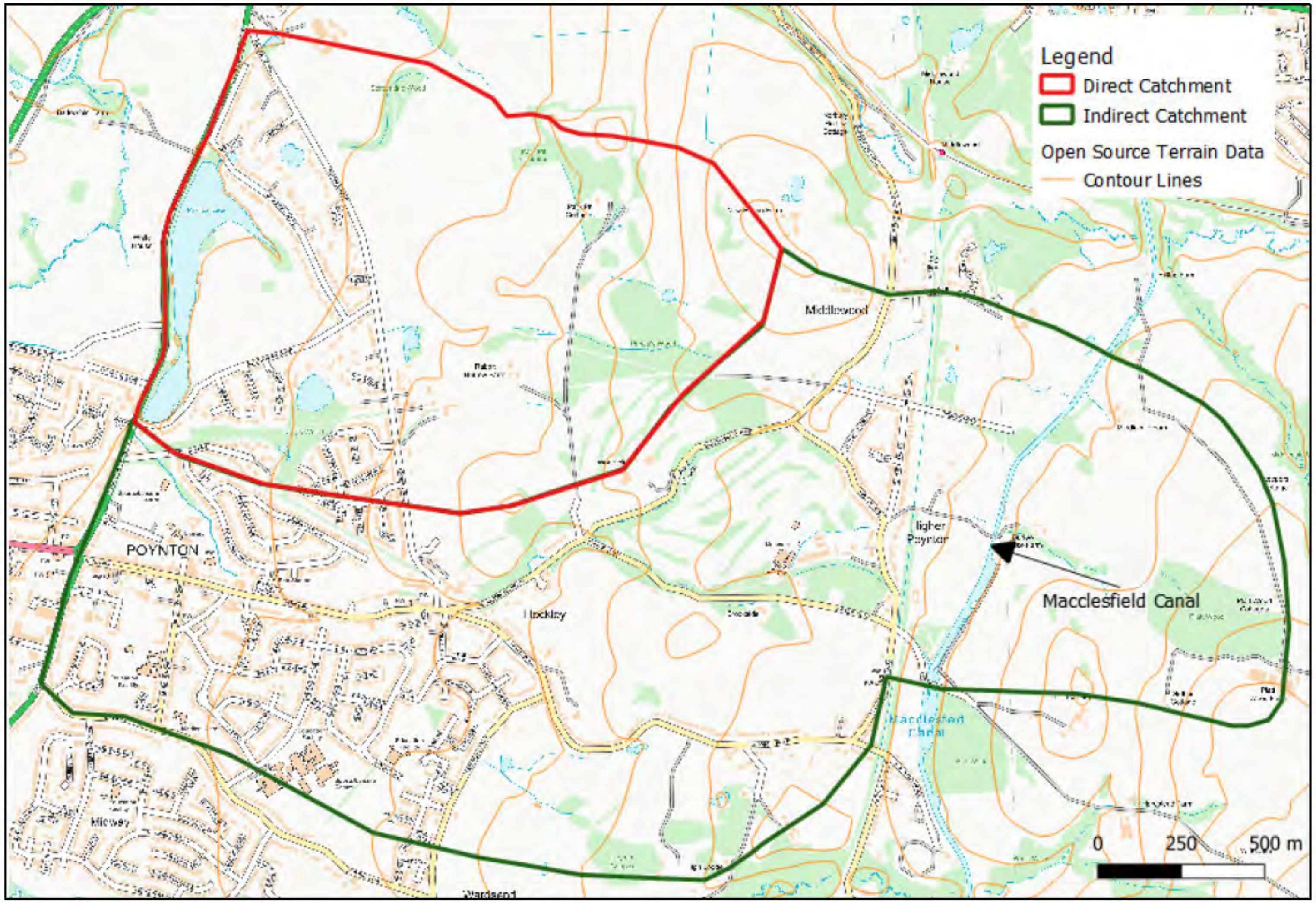
**Figure 4-2: Poynton Lake Reservoir and contributing catchments**

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## 4.3 FEH Catchment Descriptors

### 4.3.1 Introduction

This chapter discusses the derivation of catchment descriptors required for the FEH rainfall-runoff method.

The following catchment descriptors are required:

- AREA – catchment area (km$^2$);
- DPLBAR – average drainage path length (km);
- DPSBAR – average drainage path slope (m/km);
- SAAR – standard average annual rainfall for the period 1961 – 1990 (mm);
- SPRHOST – standard percentage runoff based on the HOST dataset (%);
- PROPWET – proportion of the time the soil is wet (i.e. soil moisture content is at or within 6mm of field capacity);
- URBEXT1990 (urban extent) – fraction of urbanisation in the catchment in reference year 1990.

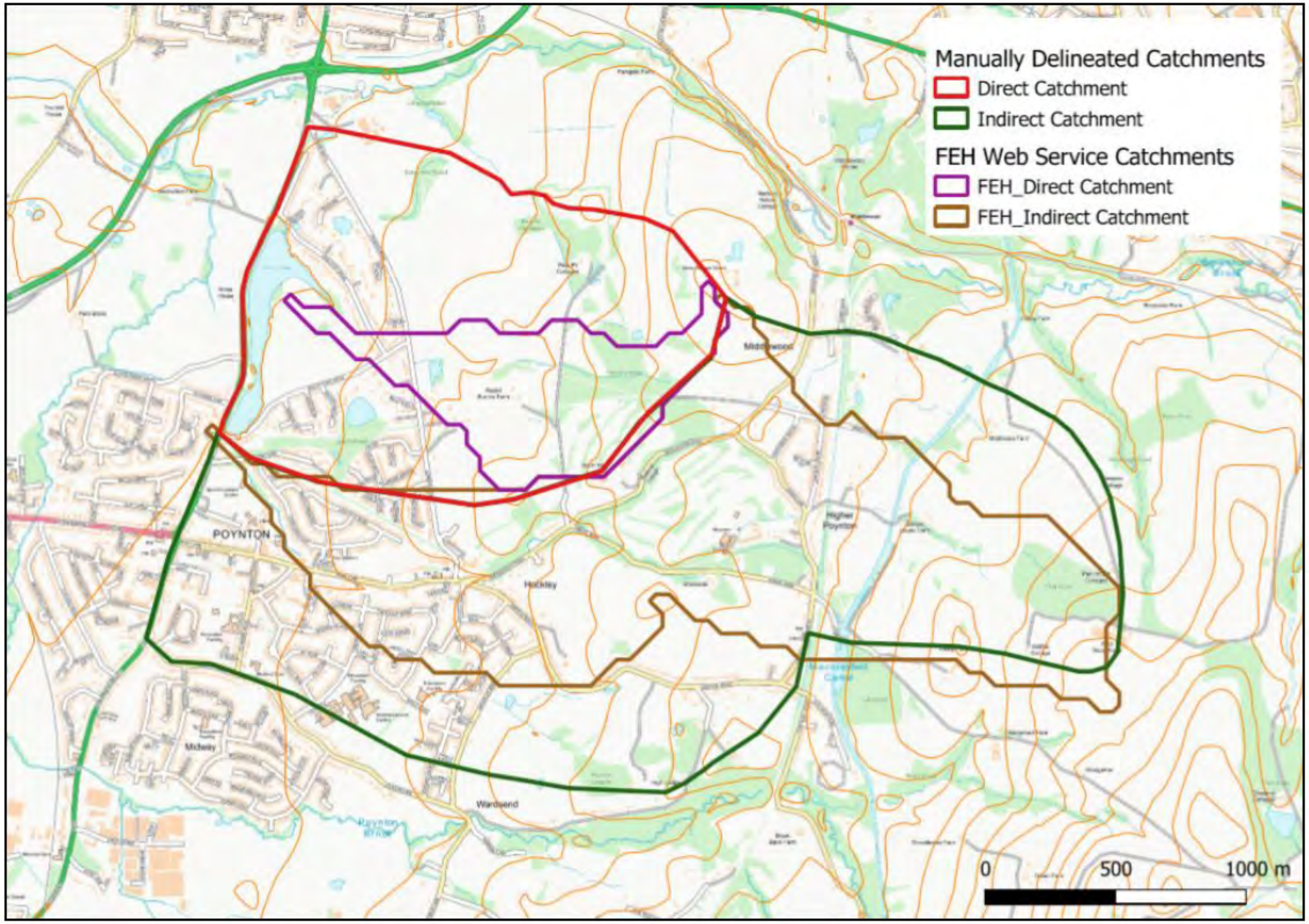
### 4.3.2 Catchment Areas

The FEH Web Service catchment boundaries were reviewed, however, it became apparent (based on the observations during the site visit and mapped drainage) that the software does not accurately delineate either the direct reservoir catchment or the indirect catchment. Manual delineation of the catchment boundaries was therefore required, using available Open Source Ordnance Survey mapping data, LiDAR data and site visit observations. The resulting catchment boundaries, alongside the FEH Web Service catchment boundaries are shown in Figure 5-3.

### 4.3.3 Other Catchment Descriptors

The principal source of catchment descriptors is usually provided by the FEH Web Service. However, given the FEH Web Service does not correctly delineate the catchments it was necessary to manual derive some of them (DPLBAR, DPSBAR, SPRHOST, BFIHOST and URBEXT1990) using the methods outlined in FEH Volume 5. The SAAR and PROPWET values obtained from the FEH Web Service for the direct and indirect reservoir catchments were assessed and found to be acceptable to use for this study.

Figure 4-3: Poynton Lake Reservoir Manually Delineated and FEH Catchment Boundaries

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The catchment descriptors for the direct and indirect catchment areas were developed as follows:

- DPLBAR (average drainage path length) was estimated using the DPLBAR equation (DPLBAR = AREA$^{0.548}$) as per FEHv5.
- DPSBAR (average catchment slope) was derived using the FEHv5 methodology based on catchment altitude and drainage path length.
- SPR (standard percentage runoff) was calculated using FEH v5 methodology based on soil types within the target catchments as shown in the Soil Map of England and Wales Scale 1: 250,000 (Soil Survey of England and Wales, 1983).
- PROPWET is constant across both FEH catchments (0.52), so that value was adopted.
- The SAAR values from the FEH Web Service catchments were assessed and found to be suitable to use for this assessment.
- URBEXT1990 values were calculated using standard FEHv5 methodology based on urban extent calculated using a 1: 50,000 Ordnance Survey map.

### 4.3.4 Standard Percentage Runoff (SPR)

SPR values for a range of 29 different soil classes (Hydrology of Soil Type [HOST] Classes 1–29) were derived from a multi-variable regression analysis described in IH Report 126 (Boorman et al. 1995) and are available for any catchment area in the UK as the SPRHOST catchment descriptor from the FEH Web Service.

The soils within the direct and indirect reservoir catchments were assessed using the Soil Map of England and Wales Scale 1: 250,000 (Soil Survey of England and Wales, 1983). From this the areal coverage of each soil unit in each catchment was determined and the proportion of each HOST class then determined from which the catchment SPRHOST value was calculated. The area of the reservoir and urban areas were excluded from the SPRHOST calculation. Table 5.1 shows the soil classes for both the direct and indirect catchments.

**Table 5.1: Soil types within the Direct and Indirect Reservoir Catchments**

| Catchment | Soil Types / Percentage Cover and HOST Classes | | | | | |
|---|---|---|---|---|---|---|
| | Soil Type 1 | Percentage | HOST Class | Soil Type 2 | Percentage | HOST Class |
| Direct | 711m/Salop | 54 | 81% HOST Class 24<br>19% HOST Class 18 | 541r/Wick 1 | 46 | 75% HOST Class 5<br>25% HOST Class 7 |
| Indirect | 711m/Salop | 95 | 81% HOST Class 24<br>19% HOST Class 18 | 541r/Wick 1 | 5 | 75% HOST Class 5<br>25% HOST Class 7 |

In the past some reservoir engineers have expressed a concern that HOST Class 4 may significantly underestimate the runoff. Therefore, a check was also undertaken to assess whether any HOST class 4 soils were present within the direct or indirect catchments. However, no HOST Class 4 soils were identified.

#### 4.3.5 URBEXT 1990

URBEXT1990 was calculated for both the direct and indirect catchments by calculating the urban extent of the catchments using an up to date 1:50,000 Ordnance Survey map. The standard FEHv5 equations were then employed to calculate URBEXT1990 based upon the current urban extent within the catchments.

Based on this assessment the direct reservoir catchment was assessed as 'slightly urbanised' and the indirect reservoir catchment was assessed as 'moderately urbanised'. The 75% winter profile was therefore assessed as appropriate to use in the FEH Rainfall-Runoff boundary to derive design peak flows and hydrographs.

#### 4.3.6 Resulting Catchment Descriptors

The final catchment descriptors for the direct and indirect reservoir catchments are presented in Table 5.2.

**Table 4.2: Adopted Catchment Descriptors**

| Catchment | Catchment Descriptor | | | | | | |
|---|---|---|---|---|---|---|---|
| | Area (km²) | DPLBAR (km) | DPSBAR (m/km) | SAAR (mm) | SPRHOST (%) | PROPWET | $URBEXT_{1990}$ |
| Direct | 1.96 | 1.45 | 35.8 | 897 | 32.3 | 0.52 | 0.0470 |
| Indirect | 4.00 | 2.14 | 29.5 | 918 | 40.1 | 0.52 | 0.0745 |

## 4.4 Design Rainfall

The design rainfall profiles were derived for the following scenario:

- The 0.1% AEP and 0.01% AEP (1,000-year and 10,000-year return period respectively) events based on the FEH 2013 DDF rainfall model.

The Flood and Reservoir Safety (FRS) guidance (ICE 2015) specifies in Appendix 2 that if the reservoir surface area exceeds 5% of the direct catchment, the rainfall falling on the reservoir surface needs to be accounted for explicitly. As the Poynton Lake Reservoir makes up less than 4% of the direct catchment to the reservoir outfall, a direct rainfall unit for the reservoir was not required. Rainfall was therefore calculated for the direct reservoir catchment (including the area of the reservoir) and for the indirect catchment.

#### 4.4.1 Design return period (T-Year) Rainfall

The design rainfall depths were based on the FEH2013 rainfall DDF model. As this rainfall model is not directly implemented in the FEH rainfall-runoff unit in ISIS/Flood Modeller Pro, the design rainfall depths were extracted manually from the FEH Web Service and implemented in the units as observed rainfall totals, so that the appropriate rainfall profile (75 percentile winter) could be fitted to the FEH2013 rainfall depth.

In order to do this, point rainfall for the direct reservoir catchment was extracted from the FEH Web Service and scaled by the appropriate catchment Areal Reduction Factor (ARF). This was undertaken as the FEH Web Service does not pick up the catchment areas correctly.

In accordance with the FEH rainfall-runoff method the return period for the rainfall event is the same as for the flood event for both the 0.1% and 0.01% AEP (1,000 and 10,000-year return period) events. The design rainfall depths for the direct catchment are presented in Table 5.3. These were also used for the indirect reservoir catchment.

Additional T-year rainfall depths were taken from the FEH Web Service FEH2013 DDF data for the simulation of selected events in order to estimate the capacity of the reservoir overflow system.

Table 4.3: Direct catchment rainfall (Areal Reduction Factor applied)

| Rainfall Depths in mm | | Storm Duration (hours) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Flood Return Period (year) | Rainfall Return Period (year) | 2.3 | 2.5 | 2.7 | 2.9 | 3.1 | 3.3 | 3.5 | 3.7 | 3.9 | 4.1 | 4.3 | 4.5 |
| 0.1% AEP event | 0.1% AEP event | 89.5 | 90.9 | 92.3 | 93.5 | 94.7 | 95.8 | 96.8 | 97.8 | 98.7 | 99.6 | 100.4 | 101.2 |
| 0.01% AEP event | 0.01% AEP event | 133.5 | 135.2 | 136.6 | 138.0 | 139.3 | 140.4 | 141.6 | 142.6 | 143.6 | 144.5 | 145.4 | 146.3 |

## 4.5 Critical Storm Duration

FEH Volume 4 (IH 1999a) describes how the critical storm duration (D) in a catchment without a reservoir can be estimated from the storm Time-to-Peak Tp(t) associated with a catchment:

$D = Tp(t)\ (1+SAAR/1000)$ (without reservoir)

where SAAR is the Standard Average Annual Rainfall (mm).

Tp(t) is a function of the Time-to-Peak of the instantaneous unit hydrograph Tp(0) and the adopted hydrological model time step $\Delta t$ as follows:

$Tp(t) = Tp(0) + \Delta t/2$

In the hydrological analysis a time step $\Delta t$ of 0.1 hour (6 minutes) was adopted.

For ungauged catchments Tp(0) can be estimated from a regression equation based on DPSBAR, PROPWET, DPLBAR and URBEXT (provided in FEH Volume 4).

$Tp(0) = 4.270*(DPSBAR^{-0.35})*(PROPWET^{-0.80})*(DPLBAR^{0.54})*(1+URBEXT)^{-5.77}$

The critical storm duration was established using the hydraulic model simulation of the response over a range of storm durations and selecting the storm duration that resulted in the highest reservoir water levels. The critical storm duration was found to vary with annual exceedance probabilities. The resulting critical storm durations are tabulated in Table 6.1.

## 4.6 Antecedent Catchment Wetness

The saturation of the soils at the start of the storm is quantified by the Catchment Wetness Index (CWI). In T-year design runs, the CWI is a non-linear function of SAAR as shown in FEH Volume 4 (Figure 3.7). For the direct catchment draining to the Poynton Lake Reservoir the CWI is simulated by Flood Modeller to be approximately 122.8mm with the indirect catchment being simulated to have a CWI of approximately 123.2mm.

# 5. Hydraulic Modelling

## 5.1 Model Schematisation

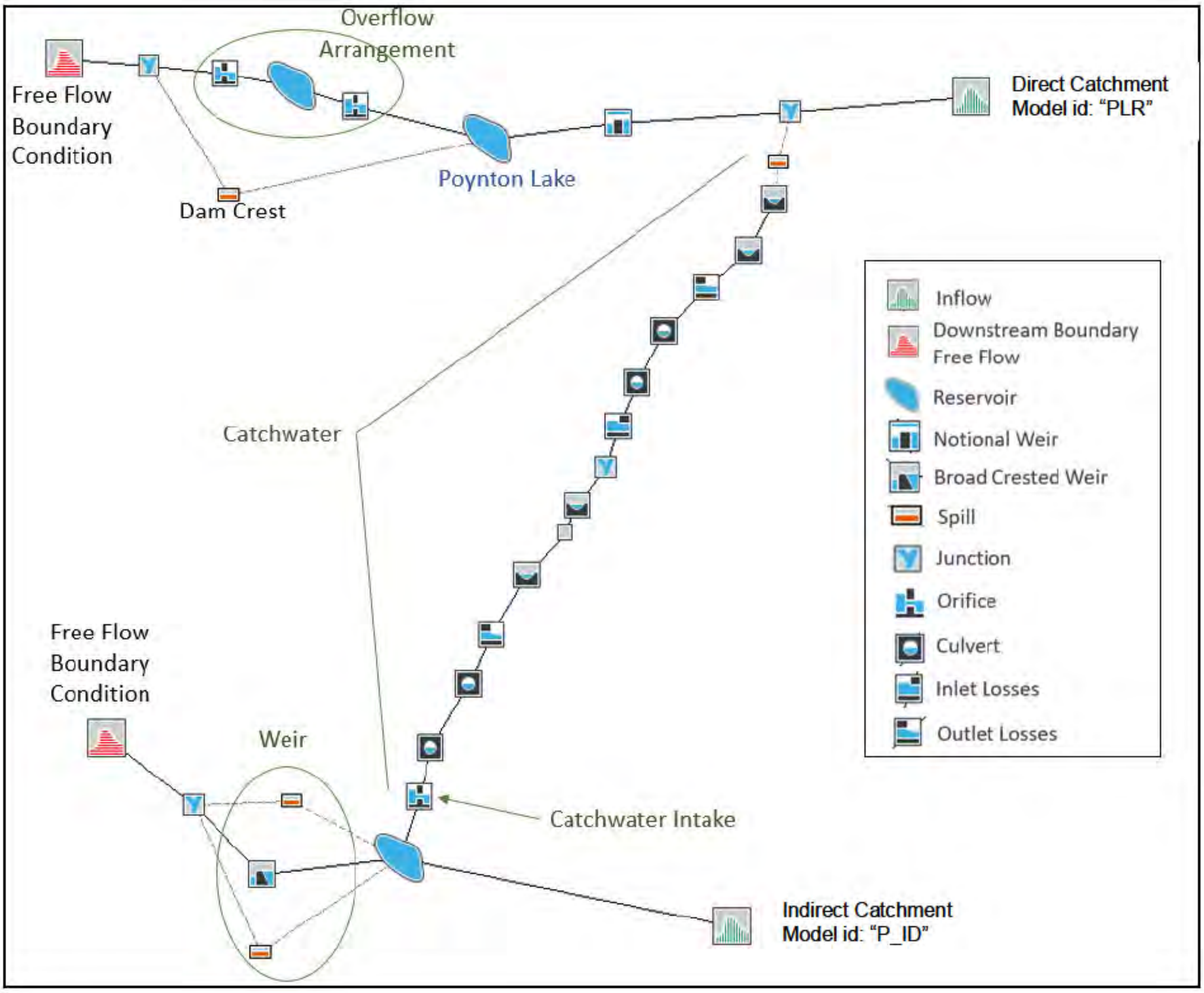
Version 4.4 of the Flood Modeller Pro hydraulic modelling software package (Jacobs, 2018) was used for modelling the Poynton Lake reservoir. The model schematisation is shown Figure 5-1. Key features of the reservoir system can be seen in Figure 1-2.

The model consists of the following components:

- Direct inflow from the reservoir direct catchment;
- A representation of the catchwater structure which diverts flows from a tributary of Poynton Brook to Poynton Lake reservoir (see Section 5.2);
- A reservoir stage-area storage curve (see Section 5.3.2); and
- A representation of the overflow arrangement (see Section 5.3.3).

Diversions of flows from the catchment by drains or sewer systems (shown in Figure 1-3) have been assumed negligible or non-functioning.

**Figure 5-1: Flood Modeller Pro Schematisation**



## 5.2 Poynton Catchwater

The Poynton Catchwater intercepts and diverts flows from a tributary of Poynton Brook, which naturally bypasses the lake to the south and passes under the A523 through a culvert. About 170m upstream of the A523 crossing, there is an intake structure which diverts some of the flow to the Poynton Lake Reservoir. The bi-furcation is formed by a weir across the tributary, with an intake to the catchwater pipeline situated in the upstream weir pool. The catchwater structure comprises a culvert under Woodside Lane, followed by an open channel and a culvert under South Park Drive which discharges into Poynton Lake Reservoir. The intake and the outfall of the catchwater have been surveyed, as well as the weir levels across the tributary (Jacobs, 2019). Other dimensions have been taken from site measurement and from OS Master-map data. Dimensions of the weir across the tributary and of the different elements constituting the catchwater are given in Table 5.1 and Table 5.3 respectively.

The bifurcation has been modelled using an orifice unit representing the pipe inlet, along with weir and spill units for the inline weir. A reservoir unit with nominal dimensions is used for the weir pool, linked to the tributary inflow. The weir across the tributary has been modelled using a broad crested weir with an average level of 90.69mAOD. The weir has been extended using side spill units with levels interpolated from the surveyed cross-sections of the tributary. These features are assumed to spill with free discharge, which is appropriate considering the estimated capacity of the downstream channel, which would not cause the weir to drown.

From upstream to downstream the catchwater comprises 122m of culvert, 64m of open channel and 48m of culvert, for a total length of 234m. The average slope is 0.02%, which is relatively flat. Losses have been added to account for contraction and expansion along the catchwater.

**Table 5.1: Dimensions of the Weir across the Tributary**

| Parameter | | Value |
|---|---|---|
| Elevation of crest | mAOD | 91.60 |
| Breadth of crest | m | 5.51 |
| Length of weir (inline) | m | 0.77 |

**Table 5.2: Poynton Catchwater Dimensions**

| Parameter | | Value |
|---|---|---|
| Intake pipe upstream invert level | mAOD | 90.850 |
| Intake pipe diameter | mm | 490 |
| Inlet pipe length | m | 122 |
| Open channel width | m | 1.3 |
| Open channel length | m | 64 |
| Outlet pipe downstream invert level | mAOD | 90.802 |
| Outlet pipe diameter | mm | 900 |
| Outlet pipe length | m | 48 |

Figure 5-2: Catchwater Intake and Outfall

| Catchwater Intake | Catchwater Outfall |
| --- | --- |
|   |  |

## 5.3 Poynton Lake Reservoir storage

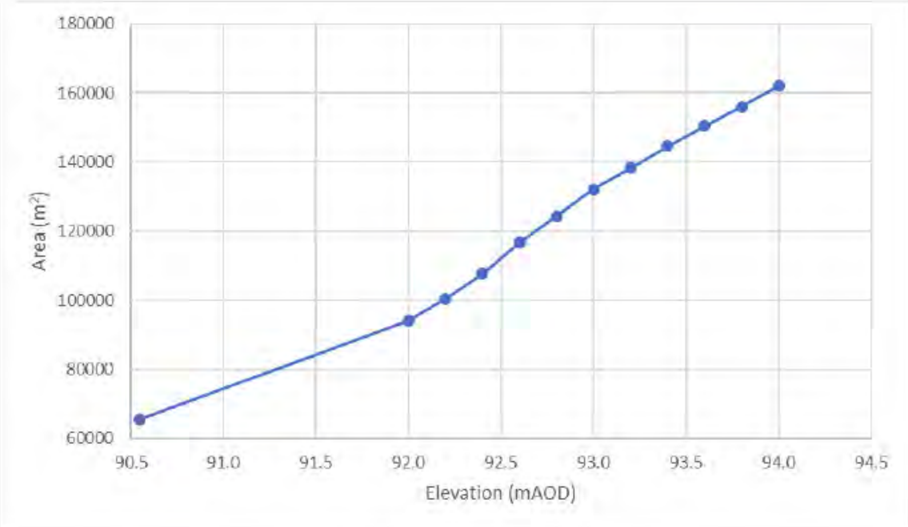
### 5.3.1 Model Inflow

Poynton Lake reservoir is supplied by its direct catchment inflow, represented as a point inflow (Figure 5-1) connected to the reservoir unit.

### 5.3.2 Reservoir Storage

Storage in the reservoir above the overflow weir level (90.55mAOD) is defined in Flood Modeller Pro using an area/elevation relationship taken from the 1m resolution LiDAR data (Environment Agency, 2019) and entered into a Reservoir Unit (see Figure 5-3). Due to poor LiDAR data quality in the vicinity of the reservoir, the reservoir surface at the starting water level was extracted from the OS Mastermap data and checked against aerial photography. The reservoir is assumed to be full when the flood event arrives.

Figure 5-3: Area vs Elevation Plot for Poynton Lake Reservoir




### 5.3.3 Reservoir Overflow

The general arrangement of the overflow is given in Figure 1-4. The overflow structure comprises an overflow box with screens housing an overflow weir. The box discharges into a 600mm diameter pipe intake situated approximately 1.05m below the weir level. The overflow pipeline passes through the embankment and discharges downstream to Poynton Brook. It is noted from the manhole survey (Drain Doctor 2019) that the subsequent pipeline goes through a number of dimension changes. However, the outlet level of the first manhole downstream from the intake is shown to be 2.5m below the main intake level. It is therefore assumed that the overflow pipe capacity is inlet controlled, and that the intake operates as an orifice for high flows.

NOTE: due to access constraints, the topographic survey was only able to determine the invert level of the intake with 50mm accuracy.

The overflow weir has been modelled using an orifice unit with the dimensions of the box opening and the level of the weir. To account for the blockage due to the grid, the area of the box opening has been reduced. The box has been modelled using a reservoir unit which discharges into an orifice unit representing the pipe inlet.

The overflow dimensions are given in Table 5.3.

Table 5.3: Poynton Lake Overflow Dimensions

| Parameter | | Value | Data Source |
|---|---|---|---|
| Weir level | mAOD | 90.55 | Jacobs 2019 topographic survey |
| Weir length | m | 3.16 | Jacobs 2019 topographic survey |
| Box opening – without screen | $m^2$ | 2.59 | Jacobs 2019 topographic survey |
| Box opening – with screen | $m^2$ | 2.21 | Jacobs 2019 topographic survey |
| Box area (footprint) | $m^2$ | 2.78 | Jacobs 2019 topographic survey |
| Pipe intake invert level | mAOD | 89.50* | Site measurement |
| Pipe intake diameter | mm | 600 | Jacobs 2019 topographic survey |

*Direct survey not possible due to access issue, level measured accurate to ±0.05m

**Figure 5-4: Poynton Lake Overflow**

| Overflow box | Pipe inlet |
|---|---|
|  |  |

### 5.3.4 Downstream Boundaries

The invert level of the overflow pipe in the overflow box is 89.50mAOD. In manhole MH1, 9.9m downstream of the overflow box, the invert level of the pipe is 87.03mAOD, i.e. 2.5m lower than the inlet invert level. Analysis of the downstream pipeline system supports the assumption for a free discharge condition downstream of the overflow orifice intake.

The model downstream boundary condition, applied downstream of Poynton Lake reservoir overflow, is represented with a stage/time (H/T) boundary unit, set with a constant nominal level, allowing a free flow discharge which means that there is no downstream control.

# 6. Hydraulic Model Results

The 0.01% AEP (10,000-year return period) event flood is specified by ICE Floods and Reservoir Safety 4th Ed, as the "Safety Check Flood", this is the standard up to which a Category B Reservoir such as Poynton Lake reservoir should be assured to be safe. In addition to this the 0.1% AEP (1,000-year return period) flood is specified as the "Design Event" which should be passed with appropriate freeboard.

## 6.1 Summary of Results

The 0.1% AEP and 0.01% AEP (1,000-year and 10,000-year return period) events were simulated for the critical scenario for Poynton Lake Reservoir using a 75% winter storm profile as appropriate for a relatively rural catchment (direct catchment URBEXT1990: 0.047). The critical storm duration for these T-year events was established using an optimisation method, by sequentially testing a range of relevant storm durations (and associated rainfall) in order to establish which duration gives the highest peak overflow from the reservoir. Table 6.1 gives the critical storm duration for each event together with the hydraulic model results for peak reservoir inflow, peak outflow and maximum reservoir level.

**Table 6.1: Critical storm durations for T-Year events at Poynton Lake reservoir**

| Event (year) | Critical Storm Duration (hrs) | Inflow ($m^3/s$) | Outflow ($m^3/s$) | Maximum Reservoir Water Level (mAOD) |
|---|---|---|---|---|
| 0.01% AEP<br>(10,000-year return period) | 2.9 | 11.2 | 11.0 | 91.10 |
| 0.1% AEP<br>(1,000-year return period) | 4.1 | 6.9 | 6.4 | 91.07 |

## 6.2 Poynton Lake Reservoir 0.01% AEP (10,000-year Return Period Event)

The peak 0.01% AEP event inflow, outflow and maximum stillwater levels are provided in Table 6.2, together with the calculated wind wave surcharge levels (see Section 9). Reservoir inflow and outflow hydrographs, for the critical 0.01% AEP event are presented in Figure 6-1, and Figure 6-2 shows the reservoir stage hydrograph with the critical dam structure levels.

The model results show that the 0.01% AEP event peak pass forward flow from the reservoir is 11.0$m^3/s$. Peak flow through the overflow culvert is 0.85$m^3/s$. The dam crest is exceeded by the peak stillwater level by 0.22m, and the majority of the pass forward flow is conveyed over the dam crest.

The 0.1% AEP event peak stillwater level is plotted against the dam cross section in Section 7 (see Figure 7-1).

Table 6.2: Poynton Lake Reservoir 0.01% AEP (10,000-year return period) key model results

| Parameter | 0.01% AEP Event |
|---|---|
| Main overflow weir crest level (mAOD) | 90.55 |
| Storm duration (hrs) | 2.9 |
| Peak total inflow ($m^3/s$) | 11.2 |
| Peak direct catchment inflow ($m^3/s$) | 10.9 |
| Peak Indirect catchment inflow ($m^3/s$) | 0.3 |
| Peak total outflow ($m^3/s$) | 11.0 |
| Peak culvert outflow ($m^3/s$) | 0.85 |
| Peak dam crest overflow ($m^3/s$) | 10.15 |
| Peak stillwater flood level (mAOD) | 91.10 |
| Stillwater flood rise (m) | 0.55 |
| Minimum dam crest level (mAOD) | 90.88 |
| Available freeboard to crest (m) | -0.22 |
| Minimum clay core level (mAOD) | Not available |
| Available freeboard clay core (m) | N/A |
| Mean annual significant wave height (m) | 0.31 |
| **FRS 4th Edition assessment of dam freeboard** | |
| Mean wave overtopping discharge (l/s/m) | Not applicable - Dam overtopped |
| Peak flood & wave surcharge level (mAOD) – with ICE recommended minimum surcharge allowance of 0.6m | Not applicable - Dam overtopped |
| Freeboard to dam crest (m) – with flood rise and ICE recommended minimum wind-wave surcharge of 0.6m | Not applicable - Dam overtopped |

Figure 6-1: Poynton Lake winter 0.01% AEP (10,000-year return period) inflow and outflow

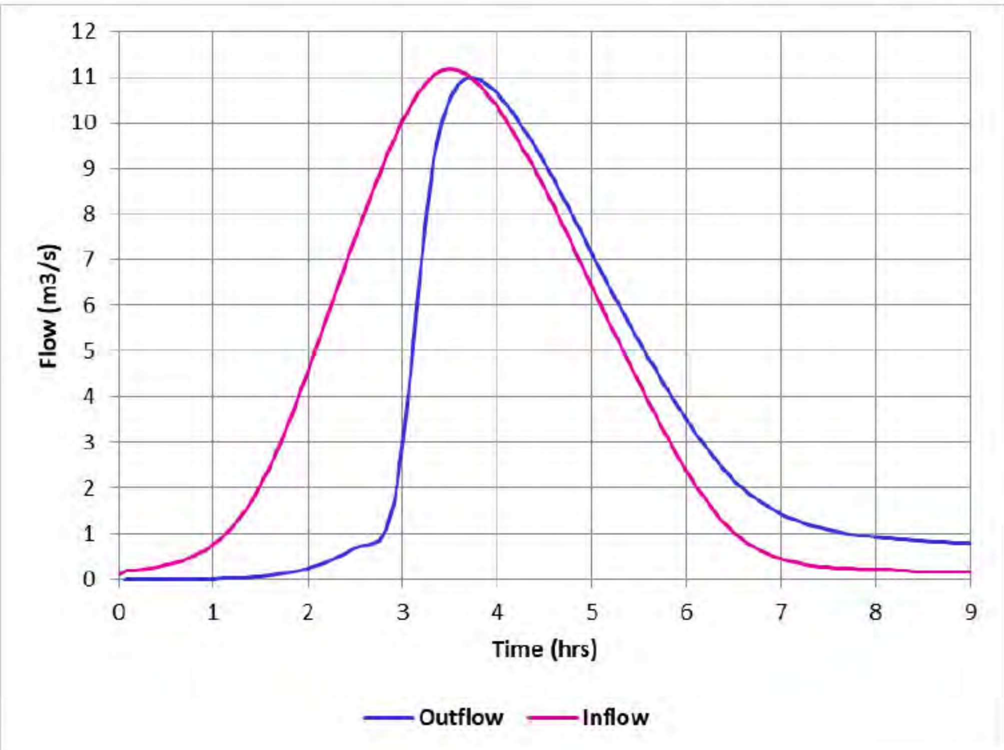
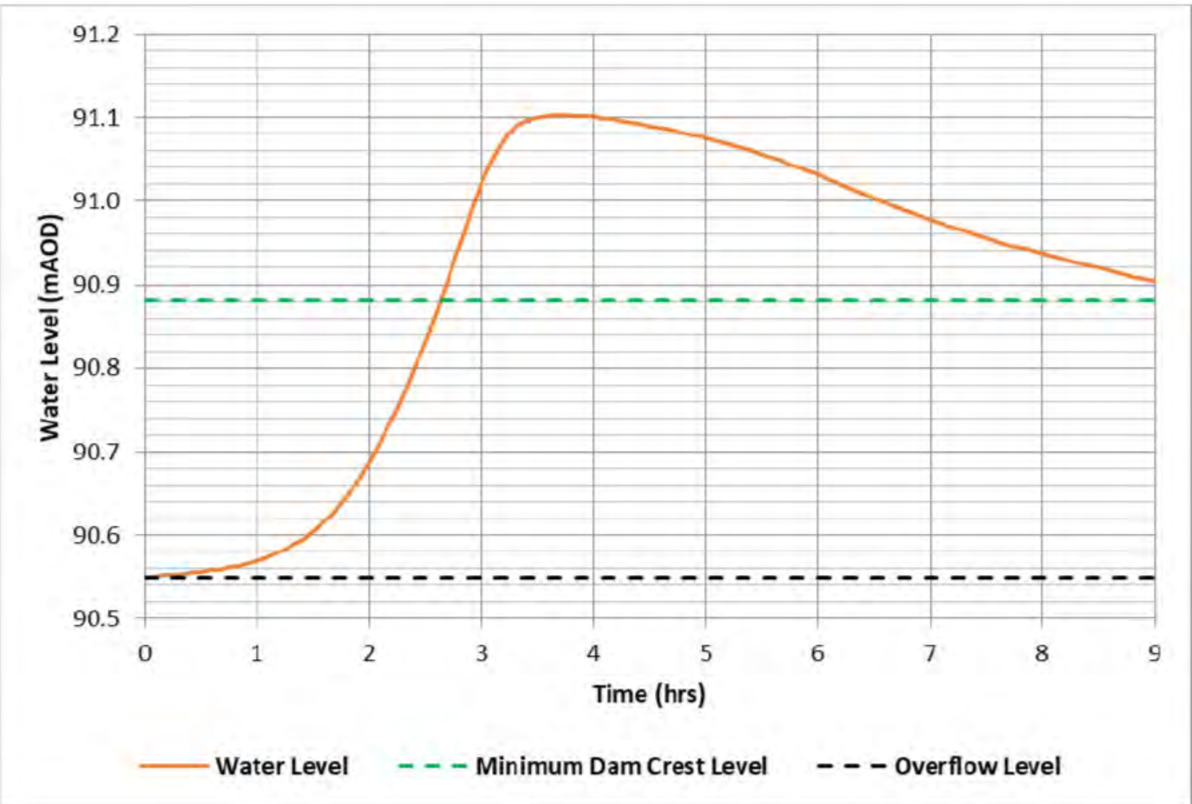


Figure 6-2: Poynton Lake winter 0.01% AEP (10,000-year return period) stage hydrograph with critical dam structure levels




## 6.3 Poynton Lake Reservoir 0.1% AEP (1,000-year return period event)

The peak 0.1% AEP event inflow, outflow and maximum stillwater levels are provided in Table 6.3 together with the calculated wind wave surcharge levels (see Section 9). Reservoir inflow and outflow hydrographs, for the critical 0.1% AEP event are presented in Figure 6-3, and Figure 6-4. Figure 6-4 shows the reservoir stage hydrograph with the critical dam structure levels.

The model results show that the 0.1% AEP event peak pass forward flow from the reservoir is 6.4m$^3$/s. Peak flow through the overflow culvert is 0.83m$^3$/s. The dam crest is exceeded by the peak stillwater level by 0.22m, and the majority of the pass forward flow is conveyed over the dam crest.

The 0.1% AEP event peak stillwater level is plotted against the dam cross section in Section 7 (see Figure 7-1).

Table 6.3: Poynton Lake Reservoir 0.1% AEP (1,000-year return period) key model results

| Parameter | 0.1% AEP Event |
|---|---|
| Main overflow weir crest level (mAOD) | 90.55 |
| PMF storm duration (hrs) | 4.1 |
| Peak inflow ($m^3/s$) | 6.9 |
| Peak total outflow ($m^3/s$) | 6.4 |
| Peak culvert outflow ($m^3/s$) | 0.83 |
| Peak dam crest overflow ($m^3/s$) | 5.55 |
| Peak stillwater flood level (mAOD) | 91.07 |
| Stillwater flood rise (m) | 0.52 |
| Minimum dam crest level (mAOD) | 90.88 |
| Available freeboard to crest (m) | -0.19 |
| Minimum clay core level (mAOD) | Not available |
| Available freeboard clay core (m) | N/A |
| Mean annual significant wave height (m) | 0.31 |
| **FRS 4th Edition assessment of dam freeboard** | |
| Mean wave overtopping discharge (l/s/m) | Not applicable - Dam overtopped |
| Peak flood & wave surcharge level (mAOD) – with ICE recommended minimum surcharge allowance of 0.6m | Not applicable - Dam overtopped |
| Freeboard to wave wall (m) – with flood rise and ICE recommended minimum wind-wave surcharge of 0.6m | Not applicable - Dam overtopped |

Figure 6-3: Poynton Lake reservoir winter 0.1% AEP (1,000-year return period) inflow and outflow

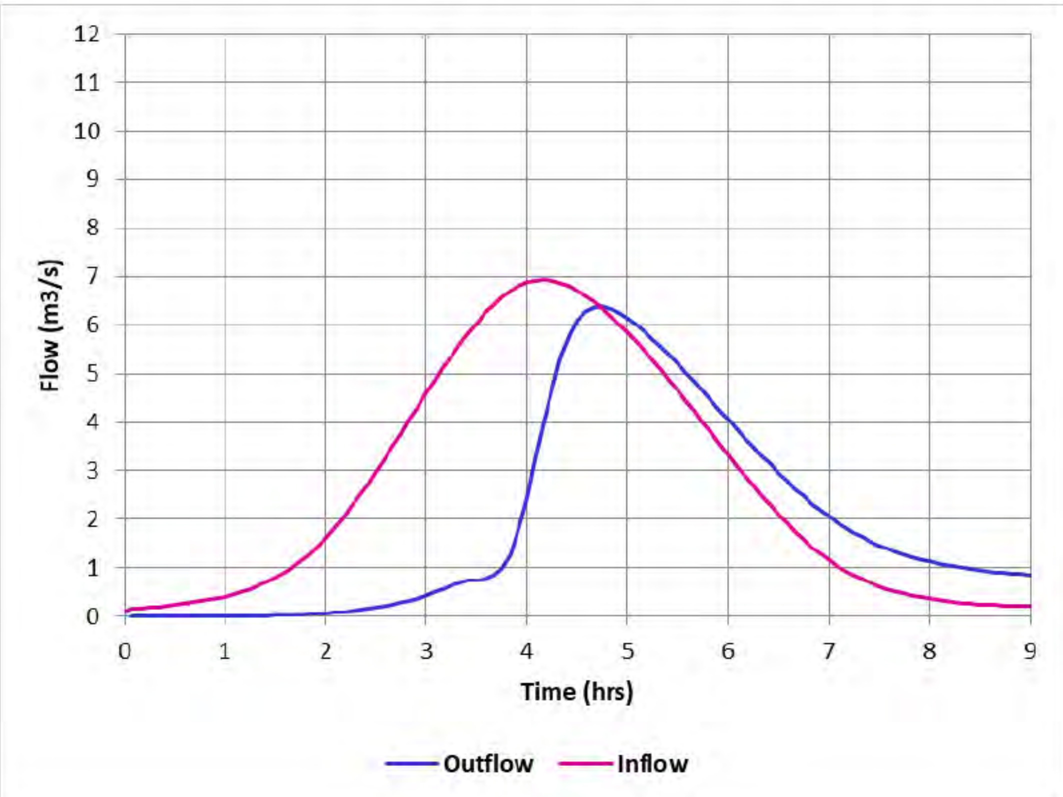
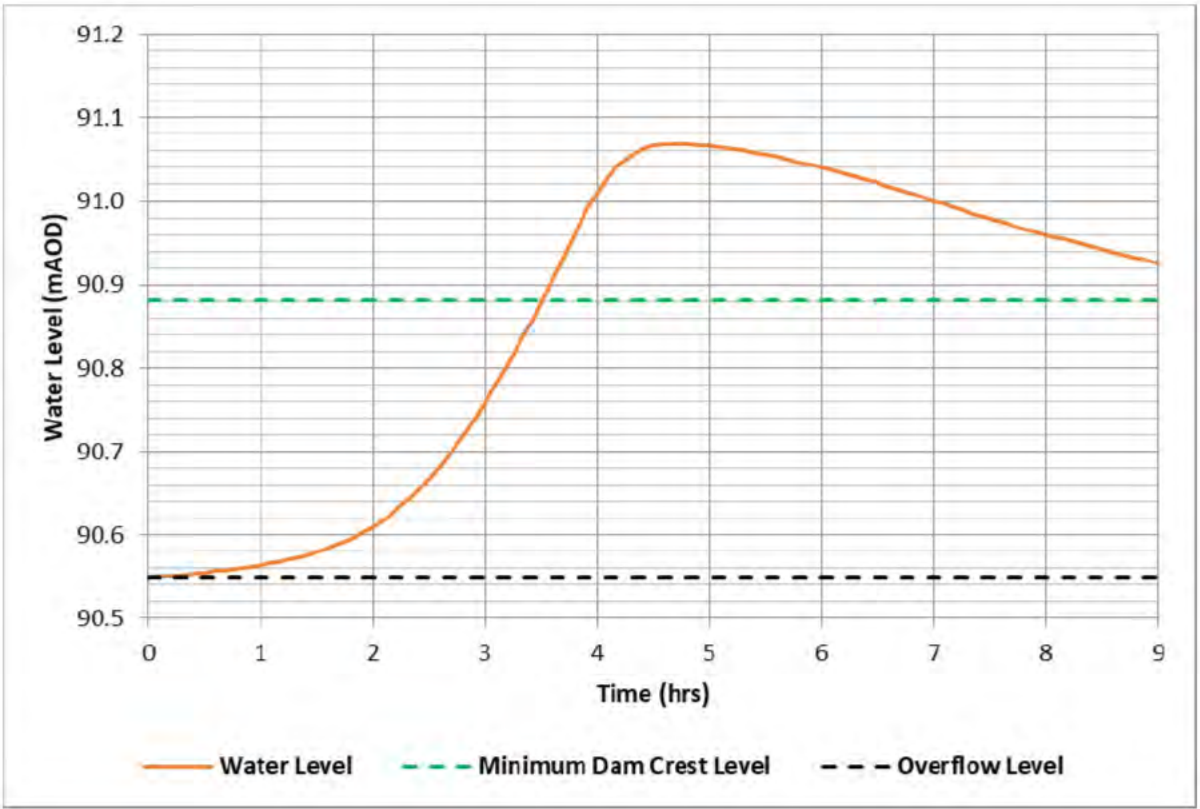


Figure 6-4: Poynton Lake reservoir 0.1% AEP (1,000-year return period) stage hydrograph with critical dam structure levels



## 6.4 Poynton Lake Reservoir Overflow Capacity Estimation

As it has been seen that the reservoir embankment is overtopped by the 0.1% AEP design storm and the 0.01% AEP Safety Check event, additional simulations were carried out for sequential storm event T-year return periods to determine which event first causes the overtopping of the reservoir embankment. The model inflows utilised the critical duration of 4.1hrs from the 0.1% AEP (1000 year) event, to provide an indicative standard of service for the reservoir. It is also seen that for these lower magnitude storm events that the capacity of the urban drainage network to bypass the reservoir is significant. A basic, pipe full Mannings calculation gives a network capacity of $0.6m^3/s$.

Table 6.4 below shows that the 2%AEP (50 year) event has modelled still water level that is just 24mm below the lowest point n the dam crest. This event therefore constitutes the modelled standard of service for the reservoir, overflow events of greater magnitude will cause overtopping of the dam. The 1%AEP (100 year) event causes significant over topping.

There are no anecdotal records currently available to compare the conditions at the lake during known major historic flood events, with the flood events simulated. It is not unreasonable that the reservoir might be subject to minor overtopping which has gone unnoticed, since constructed circa 1750.

**Table 6.4: Critical storm durations for T-Year, including drainage network bypassing of the reservoir.**

| Event (year) | Peak stillwater flood level (mAOD) | Reservoir Inflow ($m^3/s$) | Reservoir Inflow Volume ($m^3$) | Peak culvert outflow ($m^3/s$) | Peak dam crest overflow ($m^3/s$) | Length of crest overtopped (m) | Linear overtopping (l/s/m) |
|---|---|---|---|---|---|---|---|
| 1% AEP (100-year return period) | 91.981 | 3.79 | 41,300 | 0.80 | 0.71 | 68 | 10 |
| 2% AEP (50-year return period) | 90.856 | 2.61 | 27,400 | 0.74 | 0.00 | 0 | 0 |

Note: Minimum embankment crest level = 90.88mAOD

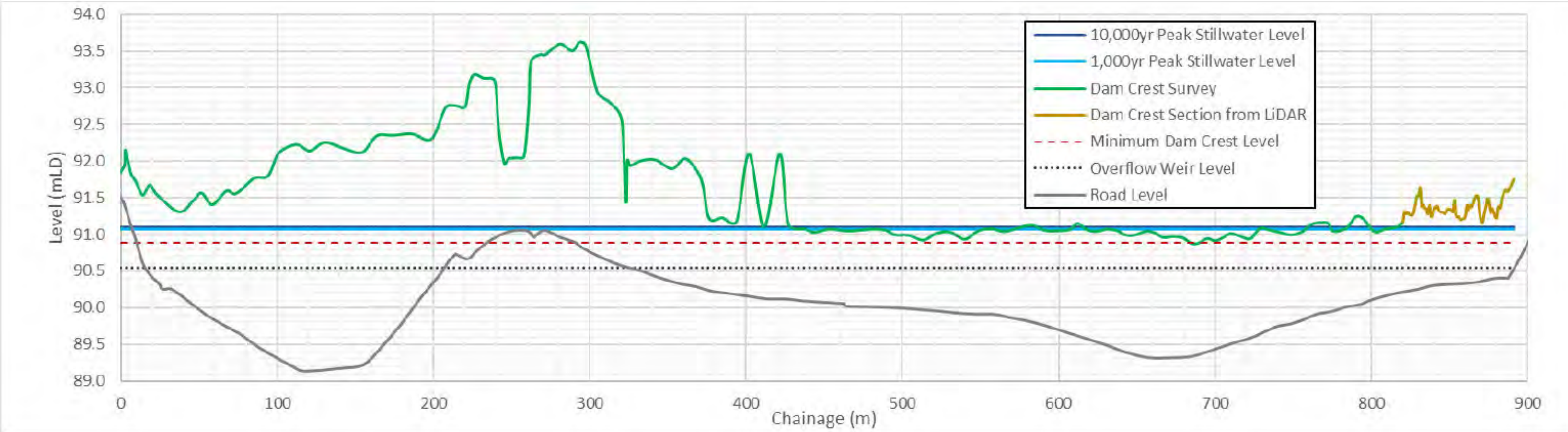
# 7. Dam Crest Profile and Cross Section

Figure 7-1 shows the dam crest profile looking downstream. The 0.01% AEP (10,000-year return period) and 0.1% AEP (1,000-year return period) event peak stillwater levels are also shown as well as the overflow weir level.

The elevations of the dam have been taken from the topographic survey (Jacobs, 2019). The surveyed dam crest data has been extended, using the available LiDAR data (EA, 2019) at the right-hand side where the dam ties into natural ground that rises up.

Results show that both the 0.01% AEP (10,000-year return period) and 0.1% AEP (1,000-year return period) event stillwater levels are above the dam crest and flows are spilling over the dam.

Figure 7-1: Profile of levels along the dam (from left to right looking downstream)



# 8. Sensitivity Analysis

The following sensitivity testing has been carried out on the hydraulic model for the 0.01% AEP event at Poynton Lake:

- Weir coefficient – weir coefficients were adjusted for the dam crest spill by +/-0.2;
- Standard Percentage Runoff (SPR) – the SPR value in the inflow units was adjusted by +/-20%;
- Soil type (HOST) Class 4 – an SPR value of 60% was assigned to HOST Class 4 to represent an extreme upper limit of the potential range of SPR values for HOST Class 4;
- Blockage –100% blockage was applied at the overflow pipe;
- Climate change - inflow was uplifted by 30% following the North-West England 2080s scenario central (50%tile) allowance (EA, 2016).
- Embankment Crest level - the vegetation cover and variable topography limits the accuracy of the dam crest survey. A test run was made with crest levels increased by 0.2m to check model response.

Maximum reservoir water levels and peak overflow discharges at Poynton Lake for the sensitivity tests are provided in Table 8.1 and compared with the summer PMF values.

**Table 8.1: Sensitivity Results**

| Sensitivity Case | Peak Outflow ($m^3/s$) | Difference from Baseline ($m^3/s$) | Stage (mAOD) | Difference from Baseline (m) |
|---|---|---|---|---|
| 0.01% AEP – baseline case | 10.981 | | 91.103 | |
| 0.01% AEP +20% catchwater roughness | 10.928 | -0.053 | 91.102 | -0.001 |
| 0.01% AEP -20% catchwater roughness | 11.050 | 0.069 | 91.103 | 0.000 |
| 0.01% AEP 100% blockage | 11.028 | 0.047 | 91.108 | 0.005 |
| 0.01% AEP +20% SPR | 12.611 | 1.630 | 91.113 | 0.010 |
| 0.01% AEP -20% SPR | 9.281 | -1.700 | 91.092 | -0.011 |
| 0.01% AEP 30% Climate Change | 14.352 | 3.371 | 91.122 | 0.019 |
| 0.01% AEP +0.2m dam crest | 10.323 | -0.658 | 91.298 | 0.195 |

An adjustment of the catchwater roughness by +20% and -20% has no significant impact on Poynton Lake flood still water level (<0.001m) and on reservoir overflow (<0.1$m^3/s$).

A blockage of the main overflow weir by 100% causes a small increase in reservoir water level of 0.05m but no significant change to the reservoir overflow. However, this does not significantly affect flood safety of the reservoir as the majority of the design storm and the safety check flood are discharged over the dam crest.

An increase of the standard percentage runoff (SPR) value in the catchments draining to Poynton Lake by 20% increases the peak pass forward flow by 1.63$m^3/s$. Peak water level in the reservoir increases by 0.01m.

The climate change flow uplift causes a more significant response in the model results, with the peak pass forward flow increased by 3.37$m^3/s$. Peak water level in the reservoir increases by 0.02m.

Increasing the effective dam crest level by 0.2m causes the peak water level in the reservoir to increase by a similar value, this creates flow attenuation and a small reduction in total pass forward flow.

# 9. Wind Wave Overtopping Analysis

Wind wave overtopping was calculated using the approach described in the Floods and Reservoir Safety 4$^{th}$ Ed guidance (ICE 2015). The calculation data for Poynton Lake reservoir is shown in Appendix A as well as the reservoir fetch.

The wave overtopping rate calculation is not applicable in this study as the 0.01% AEP event stillwater level exceeds the minimum dam crest level.

Calculation shows that south south-west winds over a fetch of 800m generate a mean annual significant wave height of 0.31m at Poynton Lake reservoir embankment.

# 10. Comparison with Previous Studies

No information on previous flood studies have been received for the Poynton Lake Reservoir. Some limited information was however, provided in the Section 10 Report (Mott MacDonald, 2016) which notes that a 'Rapid Assessment' in accordance with the Flood and Reservoir Safety Guidelines (3rd Edition) was undertaken by a [redacted] in 2010.

This assessment was reported as including an estimate of the 30% Probable Maximum Flood (0.3 PMF) design peak flow, which was reported as 2.64m$^3$/s. The 30% PMF was noted as being equivalent to the 0.1% AEP event (1,000-year return period). The predicted flood surcharge was reported as being simulated to rise to within 0.1m of the embankment crest (not including wave action).

It is noted that the 0.1% AEP event (1,000-year return period) reservoir inflow for the current study is more than double that suggested by the previous rapid method. It is also noted that the present study shows the reservoir embankment to overtop for discharge flows in the region of 0.8m$^3$/s, based on a 600mm orifice control. This capacity is considerably below the Rapid assessment discharge rate (2.64m3/s), which was quoted in the previous assessment to cause stage rise to 0.1m below the dam crest.

Given no additional details from the previous analysis are available for direct comparison, and only limited results are available from the 'rapid assessment', a direct comparison of previous studies with this flood study is not possible. The results from the 'rapid assessment' use an outdated high-level method, while the present flood study follows current guidance and a more detailed approach.

It is also noted that the present study, which is based on up to date topographic survey utilises an impounded top water level based on the surveyed weir level of 90.55mAOD. This differs from the value of 90.711mAOD quoted in the previous inspection reports, which is presumably sourced from older records. The top water level from the 2019 survey should be adopted in the reservoir Prescribed Form of Record.

# 11. Assumptions and Limitations

## 11.1 Overflow Weir Representation

The overflow 600mm pipe intake is assumed to act as an Orifice, analysis has shown this to be suitable for high flows in excess of 0.5$m^3$/s. At lower flows, reservoir water level is controlled by the overflow weir which is represented using a general weir equation. The screen is represented by shortening the weir length to account for the bars. This simplification will potentially underestimate head loss at low flows. but this is not significant for the modelled reservoir water level which is predominantly controlled by the spill over the crest and the orifice capacity.

## 11.2 Hydrological Assumptions

The following assumptions and limitations were made in the hydrology study:

- The Poynton Lake Reservoir catchment is a small ungauged catchment. Flow estimates for small ungauged catchments are open to greater uncertainty than for larger gauged catchments.
- In order to account for potential impacts of climate change, a sensitivity analysis has been undertaken including an allowance for climate change for the 0.01% AEP events (1,000 and 10,000-year return periods). It is assumed the climate change uplift factors presented in Flood Risk Assessment: Climate Change Allowance (Environment Agency, 2016) are appropriate to use in the sensitivity analysis. The north-west region 2080s central allowance (30% uplift) has been selected for the Poynton Lake Reservoir and is assumed to be appropriate for this reservoir flood study.
- Historic flooding information could give verification data for the model. It is assumed that such information is not available.
- No allowance for canal breach has been included in this assessment.

Assumptions regarding the rate of water transfer from the indirect catchment will be discussed in the hydraulic modelling section.

## 11.3 Hydraulic Modelling Assumptions

The following assumptions and limitations were made in this study:

- LiDAR DTM data (EA, 2019) with a 1m resolution and OS MasterMap data were used to define storage available above the overflow weir level in the reservoir.
- Reservoir units assume a uniform, flat water surface with a travel time of zero for the propagation of the flood wave across the length of the reservoir. This is in line with the current industry best practice.
- Poynton Lake was considered to be full at the start of the design flood events, with the initial water level in the model is set to just above the overflow weir level so that the reservoir is spilling the base flow.
- The overflow pipe invert level drops by 2.5m over the 10m distance between the overflow box and the manhole “MH01”. It was therefore assumed that the intake is inlet controlled. The intake has therefore been represented in the model using an orifice unit.
- A simplified representation of the catchwater, which diverts flows from the tributary of Poynton Brook to Poynton Lake, has been used. More specifically, only the inlet and outlet pipes of the catchwater have been surveyed. The length, slope, dimensions and invert levels of the different elements constituting the catchwater have been assumed based on the OS MasterMap data or interpolated using the survey data.
- For the design storm and safety check simulations, no representation of or allowance for the surface water drainage network has been made. It is noted that three sewer mains could potentially bypass flows around the reservoir. However the capacity of the network will insignificant relative to these events.
- The crest level of the dam is heavily vegetated and highly variable, it is anticipated that the accuracy of the crest survey is necessarily limited. Sensitivity analysis has been carried out to assess this. The analysis shows that the key findings of the study are not affected.

# 12. Conclusions and Recommendations

An integrated hydrological and hydraulic model of the Poynton Lake reservoir and its direct and indirect catchments has been developed based on up to date topographic survey and Lidar data and utilising the current industry standard flood study methodologies.

The 2019 survey gives a reservoir weir level of 90.55mAOD. This top water level from the should be adopted in the reservoir Prescribed Form of Record.

The model has been used to estimate peak discharge flows and stillwater levels for the 0.01% and 0.1% AEP flood events (10,000-year and 1,000-year return period events).

The model results show that for the 0.1% Design Storm flood event, the peak inflow to the reservoir is 6.9$m^3$/s, and the peak total outflow is 6.4$m^3$/s. In this event, the peak stillwater level of 91.07mAOD exceeds the minimum dam crest level by 0.19m.

For the 0.01% Safety check flood event, the peak inflow to the reservoir is 11.2$m^3$/s, and the peak total outflow is 11.0$m^3$/s. In this event, the peak stillwater level of 91.10mAOD exceeds the minimum dam crest level by 0.22m.

An additional model simulation was carried out to estimate the effective capacity of the reservoir overflow system. This showed that with an allowance for surface drainage, then events in excess of approximately 1% AEP (50-year return period) will overtop the embankment.

The dimensions and make-up of the waterproof element of the embankment is not known. Investigation should be considered to determine the subsurface makeup of the dam, in order to better understand the risk of seepage through the dam.

The significant wave height of wind driven waves is calculated to be 0.31m. Any waves propagating towards the dam will increase the bulk overtopping. However, the floods and reservoir safety calculation is not applicable for water levels above the dam crest.

The results of the present study indicate a significant increase in flood risk compared to the previous “Quick Method”. It is considered that the present study adopts the latest industry standard methods along with detailed input data and this study supersedes the previous assessment.

The modelling results indicate that the overflow facilities at Poynton Lake do not safely pass the design or safety check flood event. Further investigation is therefore required to identify a suitable engineering solution. The investigation should be supervised by a Qualified Civil Engineer from the UK All Reservoirs Panel.

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# Appendix A. Wind Wave Overtopping Calculation

**Figure A1: Poynton Lake reservoir fetch length**

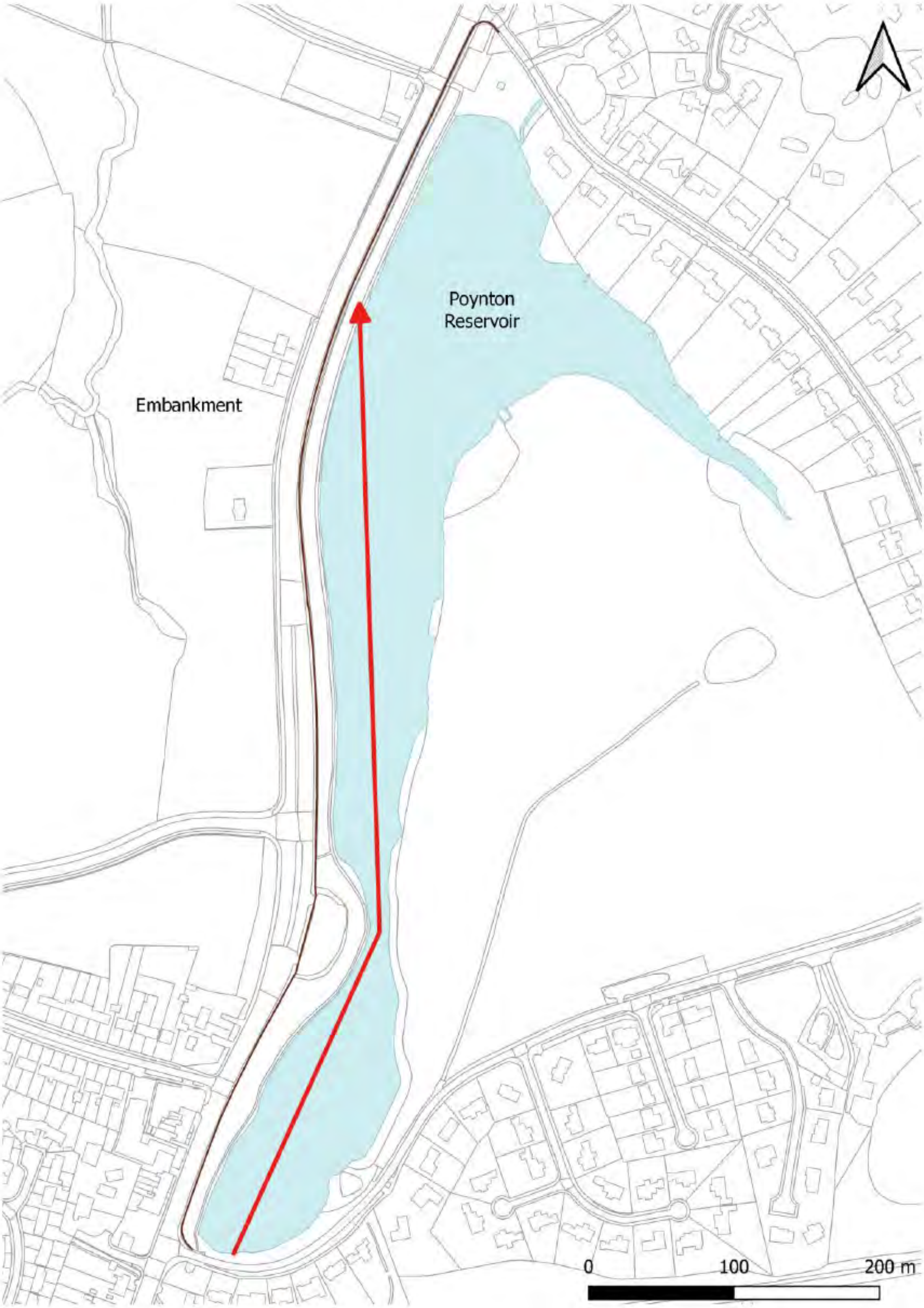

Table D1: Wave surcharge calculation data – Summer PMF Event

| | | | |
|---|---|---|---|
| **Water Level=** | 91.10 | mAOD | Summer PMF water level in the reservoir |
| **$U_{50}$=** | 22.5 | m/s | 50 year maximum hourly wind speed reduced to sea level (from Figure 3 of guidance presenting a wind speed map taken from BS6399) |
| **$f_T$=** | 0.79 | - | Adjustment factor for estimating the mean annual maximum hourly wind speed |
| **Altitude=** | 91 | mAOD | Altitude of embankment crest – from 2019 topographic survey |
| **$f_A$=** | 1.091 | - | Adjustment factor for altitude |
| **F=** | 800 | m | Fetch is determined from the point where there is maximum potential for breaching (lowest point of the top of the dam). Longest available fetch used (see Figure D1). |
| **$f_W$=** | 1.10 | - | Overwater adjustment factor lookup Table 4 of guidance. If fetch less than 1000m use 1.1 |
| **$f_D$=** | 1.05 | - | Duration factor (to convert the hourly wind speed to 10-20min duration for full development of waves - typical for UK reservoirs) - fixed value<br>(Source: CIRIA Special Publication No. 83/CUR Report SR 345) |
| **Fetch dir =** | 189.0 | deg | Fetch direction (degrees from North) |
| **$f_N$=** | 0.87 | - | Wind direction adjustment factor (from Table 5 of guidance, allows for the orientation of the principal axis of the reservoir with respect to 'general UK' wind direction). Regional data on wind direction could be used for each specific site. Lookup table based on guide |
| **U=** | 19.58 | m/s | Required wind speed |
| **Hs=** | 0.31 | m | Significant wave height for extreme conditions on the reservoir (mean height of the highest third of all waves) - Donelan/JONSWAP method. |
| The predicted significant wave height is exceeded by 14% of the waves.<br>Approx. 6% exceed 1.2Hs, and the maximum wave height may approach 1.67*Hs. | | | |
| | | | |
| **$T_p$** | **1.74** | - | Wave Period. Calculated according to F&RS 4th Ed. |
| **Mean wave overtopping rate** | **-** | l/s/m | Linear overtopping rate. Calculated according to F&RS 4th Ed |