RESERVOIRS ACT 1975

CERTIFICATE UNDER SECTION 10(6) AS TO THE CARRYING OUT OF SAFETY RECOMMENDATIONS

I, [redacted] of Jacobs, [redacted] being a member of the All Reservoirs Panel, appointed by Cheshire East Council to supervise the carrying into effect at the reservoir known as Poynton Lake reservoir, situated at SJ 923 845, of measures taken in the interests of safety recommended in a report made on 23rd August 2016 by [redacted] of Mott MacDonald, am satisfied that those measures have been carried into effect.

Signature of Engineer: ……[redacted] ………………………………

Date of Certificate: 5th December 2019

Note: various recommendations to the Undertaker and Directions to the Supervising Engineer are included in the Annex to this Certificate

1 Scope of document

This Annex to the Section 10(6) certificate under the Reservoirs Act 1975 summarises the works covered by this certificate, to provide information for future dam safety reviews.

2 Works carried out

Recommendations made in the interests of safety (MIOS) in the last Section 10 Inspection Report are shown in Table 1.

Table 1: Recommendations as MIOS in last Section 10 Report

<table>
<tr><th colspan="2">Recommendation</th><th>Description of works carried out</th></tr>
<tr><td>a)</td><td>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 of the inspection report as follows: “I recommend that an Emergency Drawdown Plan should be prepared for the reservoir. This must consider how the drawdown would be achieved including the methods to be used, the implementation of the plan, lines of communication and responsibilities.”</td><td>Plan prepared in dialogue with the Undertaker; copy attached</td></tr>
<tr><td>b)</td><td>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.</td><td>Flood study completed, and attached. See recommendations below and Directions to Supervising Engineer</td></tr>
<tr><td colspan="3">Recommendations to the Undertaker
The flood study has concluded that the spillway capacity does not meet the standards set out in the ICE Guide to Floods and Reservoir Safety (4th Edition). I therefore make the following recommendations to the Undertaker
a) Complete a feasibility study of options to increase the spillway capacity, within 18 months, and complete the works within four years of the date of this certificate, all under the supervision of an All Reservoirs (AR) Panel Engineer who should be appointed to agree the works to be carried out, oversee the works and then issue a description of the works and design criteria on satisfactory completion.
b) Regularly (at least annually) review the emergency drawdown plan, sometimes as a desk top exercise, and update as appropriate to ensure it provides an effective plan for lowering the reservoir in an emergency.</td></tr>
<tr><td colspan="3">Directions to the Supervising Engineer
a) If the works to upgrade the spillway are not completed within the time shown above, then use Section 12 (3) of the Act to recommend a S10 Inspection.
b) At least once a year check the drawdown plan remains available and up to date.</td></tr>
</table>

3 Key parameters of reservoir

These are given below. The 2019 survey of the manholes was limited to lifting the manhole covers and measuring depth to the base, so the pipe sizes are unreliable, and instead reliance has been placed on the cctv survey by Drain Doctor Plumbing (20th October 2019).

Table 2: Key parameters for reservoir

Feature	Units	Value	Source / comments
Date built		1750	Now forms amenity lake within Poynton Park
Surface area	m^2	68,000	Previous S10. Flood Study uses 65,500 measured from OS master map and online aerial imagery
Reservoir volume above EGL	m^3	130,000	Previous S10
Dam height	m	6.3m	Local maximum at incised valley. Generally, only 1.4m to A523 and 1.6m to downstream ground level.
Crest width	m	12	26m if including A523
Design Flood Category		B	A523 and properties alongside road (road up to 1m below TWL)
EA Risk designation		High Risk	

Table 3 Key levels (mOD)

Feature	Value				Source / comments
	2016 S10	2019 survey		2019 Flood study	
		Section 5 (highest)	Section 6 (typical)		
Dam crest	90.92	90.94	91.24	90.88	90.88 used in flood study
Water level at time of survey		90.714			
Top water level	90.711	90.55		90.55	Difference 2016 to 2019 presumably due to different datum
Freeboard (m)	0.21	0.39	0.69	0.33	
Footpath on east side of A523		89.31	89.85		
Downstream toe		84.64	88.62		

Table 4 Manholes as inferred from Jacobs 2019 survey and cctv

No	Location	Cover level	Manhole dimensions					
			Incoming			Outgoing		
			Jacobs		CCTV	Jacobs		CCTV
			pipe dia (mm)	*invert level (mAOD)*	pipe dia (mm)	pipe dia (mm)	*invert level (mAOD)*	pipe dia (mm)
Spillway	Upstream face		N/A	*N/A*	N/A	600	*89.5*	N/A
MH 1	Dam crest	91.1	900	*89.57*				
			UTM	*87.03*	450	UTM	*UTM*	450
MH 2	d-s face	88.11	900	*86.55*		UTM	*UTM*	300
			UTM	*UTM*	450	UTM	*UTM*	450
MH3	field	not measured	UTM	-1.48 bcl	450	UTM	-1.52 bcl	450
MH4	field	not measured	400	*-1.39 bcl*	300	400	*-1.41 bcl*	300

bcl = below cover level; UTM = unable to measure

4 Summary of flood studies

The rapid flood estimate in the 2005 S10 and the recent 2019 flood study are compared in Table 5. Part of the increase in flood inflows can be attributed to the 40% increase in catchment area. To better understand the reason for the differences a check was carried out on the 2019 estimates using the ICE rapid method, as shown on Figure 1, and this confirms the 2019 estimates are not unreasonable. It is concluded that the 2006 estimate appears to have been incorrect.

Table 5 Summary of available flood studies

	2005 S10 (repeated in 2018)	**2019 study**
Method	ICE Rapid	FEH 2013 rainfall FEH Rainfall Runoff model
Catchment area	1.4km^2 direct	1.96 km^2
SPR		32%
SAAR		897mm
Indirect	500mm pipe from 4.6km^2 indirect	490mm pipe form 4km^2 indirect
Spillway	4m long weir	3.16m long weir *Reduced to 2.7 in model to account for bars*
Downstream controls	600mm diameter pipe, invert 0.8m below weir	600mm diameter pipe, invert 89.50mAOD so 1.05m below weir. Then 450mm diameter from MH 1 (at high RWL control is pipe full flow on 450mm pipe from MH2)
Inflows (m^3/s)		
1 in 10,000		11.2
1 in 1,000	2.6 (0.3 PMF)	6.9
1 in 100		3.8
1 in 50		2.6
Indirect catchment		Around 0.3 in extreme floods
Peak water level for design flood (1 in 1,000 annual chance)	0.1m below embankment crest	0.19m above lowest point on crest

The routing curve for the spillway is shown in Figure 2, and it can be seen that the control changes from the weir to the pipe, once discharge reaches around 0.8m^3/s. Raising the crest would therefore not materially increase flow through the service spillway but if raised to a fixed level with a concrete marker should make overflow more uniform and reduce the risk of local scour.

It is noted that the current estimates do not allow for:

- spillway blockage – new guidance is being issued by the EA
- climate change

However, these are only some of the uncertainties, with the others being the methods of flood estimation for which a major research program is about to commence.

Figure 1 Peak inflow vs annual chance (direct catchment only)

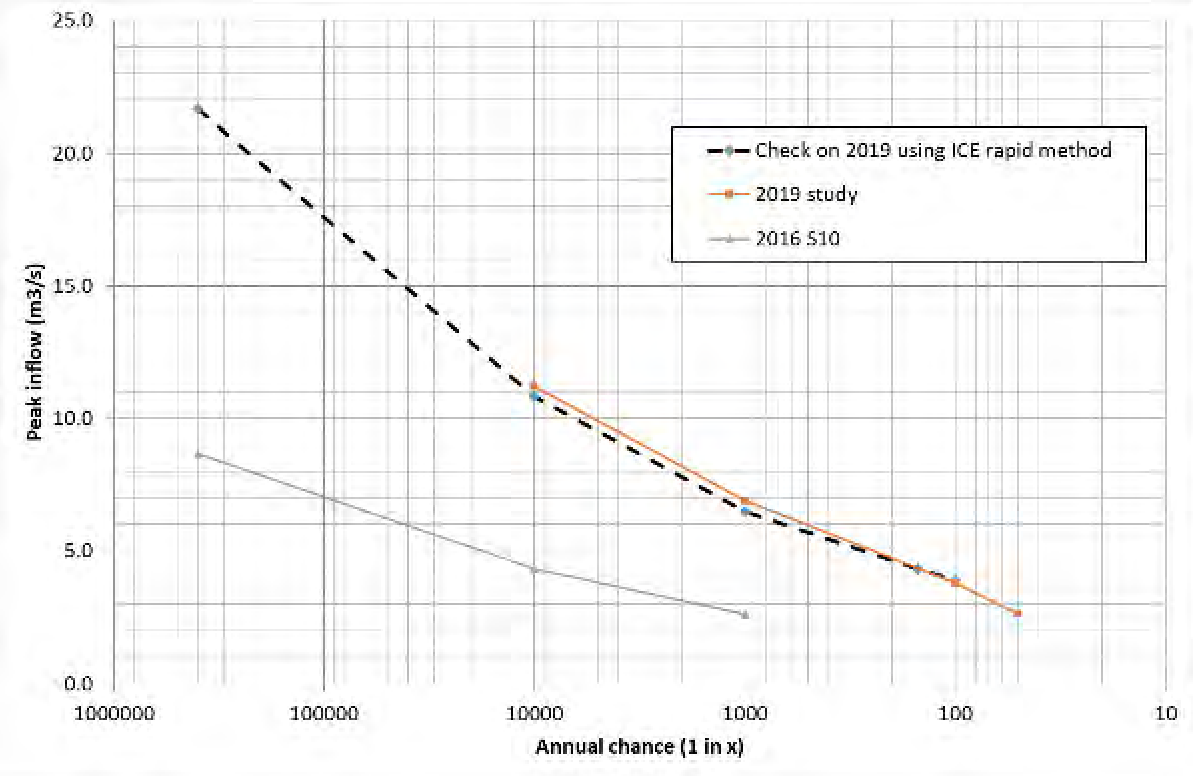
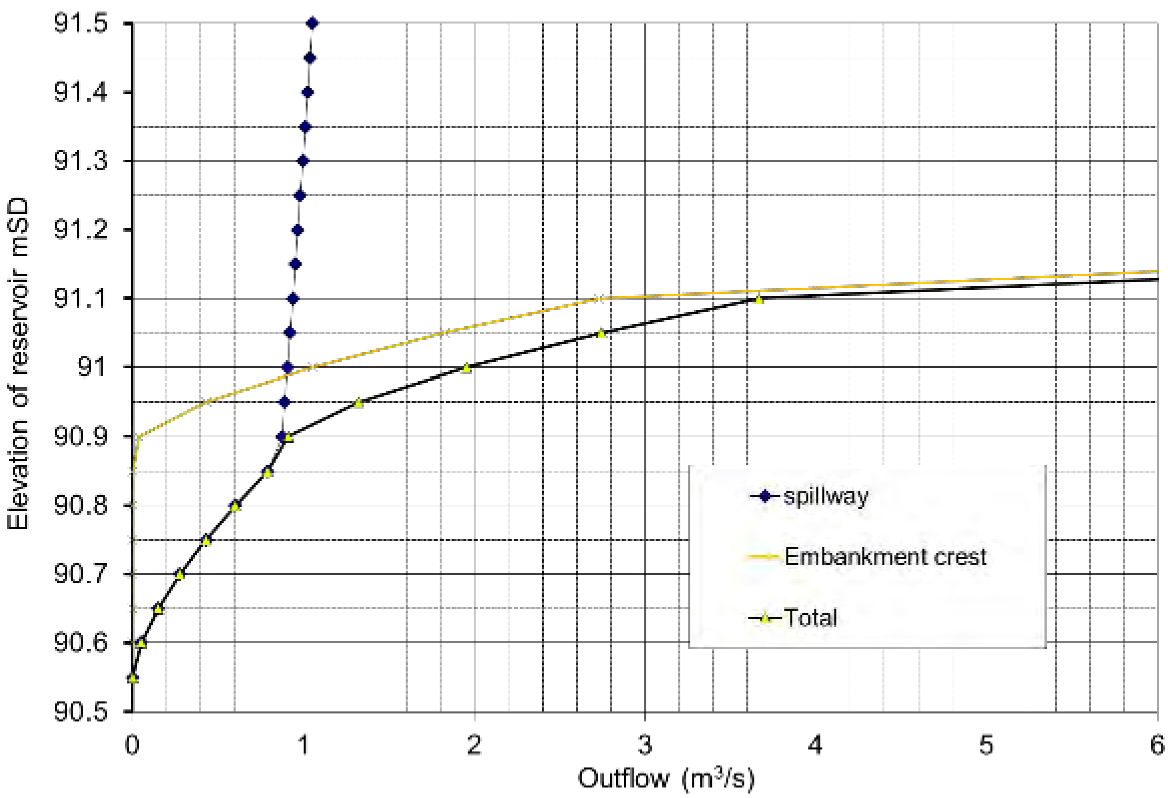


Figure 2 Outflow vs reservoir level



5 Guidance on standard for spillway upgrade

The 4th edition of Floods and Reservoir Safety (ICE, 2015) suggests that where an existing spillway does not meet the engineering standard, then a risk based approach may be adopted. This compares the cost of upgrading to the reduction in risk to life achieved.

At Poynton, to pass the 1 in 1,000 year flood through the service spillway, the overall pipe area would need to be increased by a factor of around seven (e.g. to $2m^2$ cross section), whilst to pass a 1 in 100 flood the overall area would need to be three times larger (e.g. to approx. $1m^2$ cross section). Both would involve a 30m long culvert including crossing the busy A523.

The alternative of providing some form of emergency spillway over the crest would in effect need to be in two parts, to spill onto the A523, and then a second spillway to collect flow over the road and pass it into the valley downstream. It should also be recognised that where there is a longitudinal fall on the road this will tend to direct overflow towards the low spot on the road, around Ch 660-680 (measured from south, as shown on 2019 survey).

In terms of the potential consequences of dam failure the reservoir safety team in Exeter have been contacted and the inundation mapping carried out to the 2009 RFM specification suggests a population at risk (PAR) of 5,700 and likely loss of life of 0.8. This is considered to be highly questionable for the following reasons:

a) when compared to the houses shown on the internet as being at risk most of the PAR appears to be in Cheadle 8km downstream (extracts in Section 6) which is already flooded in a 1 in 100 fluvial flood with no dam failure
b) the 2009 specification is known to be overconservative and for this reason the RFM maps are being redone by the Environment Agency to an updated 2016 specification which is likely to give much lower figures, especially for a wet day failure it will now consider the incremental effect of the dam failing on top of the 1 in 1000 fluvial flooding that would occur anyway with no dam failure.

Other factors that are likely to reduce the chance and consequences of a catastrophic release of water are:

Consideration	Comment
Initial breach	Likely to be limited to dam above A523, which is typically less than 1.3m high (locally 1.8m), with 12m wide crest
Breach of full height of dam due to overflow	This would require complete loss of 10m wide tarmac road i.e. highly unlikely
Erodibility of dam	S10 describes the geology at the dam as permo traissic mudstones. At the time the dam was built in 1750 the embankment fill is likely to have been dug locally so it is likely to be clayey and thus relatively non-erodible.
Time for complete breach	Flood study shows overflow for 3 hours. The rate of erosion would depend on the depth of overflow, and the extent to which tree roots inhibit breach (the dam crest is heavily wooded)
Catchment area of watercourse into which any breach would flow	Any breach of the reservoir would pass into Poynton Brook which has a catchment area around $18km^2$ and so at times of flood would already have a significant flow even without the flows from release of the reservoir, so the impact on people would depend on how much the flow was increased by the breach. There is a river level monitoring station in Poynton (https://flood-warning-information.service.gov.uk/station/5201)

The standard required for the spillway upgrade will be a matter for the Panel AR Engineer appointed to oversee the works and may involve simplified rapid dambreak using the methods in RARS, and a feasibility study to identify options, cost them and assess the reduction in risk to people.

If a risk based approach was adopted, then works that could be carried out to improve the resilience to dam break due to overflow could include

1. an ALARP assessment may show that it would be disproportionate in cost to have a major upgrade of the service spillway. Such an assessment is probably best carried out first as a simple screening assessment, followed if appropriate by more detailed assessments of potential consequence and annual probability of failure.
2. works to protect the houses downstream, including both locally raising the dam crest upstream of the houses and potentially works on the A road (local property protection)
3. some form of level crest to form an emergency overflow spillway, say 100m plus in length so overflow is spread out evenly over a long length
4. works to improve the resilience of the downstream face of the upper embankment to overflow, such as flattening the downstream slope
5. works to improve the resilience of the embankment below the A road in the incised valley (Ch 660-680), probably importing clay fill to flatten the slope and some form of slope protection

6 Attachments

Appendix A Reservoir inundation maps as shown on internet Nov 2019
Topographic survey (sheet 5 only – sections at highest point)
Drawdown plan
Flood study

Document control sheet

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Document Title: Poynton 10(6) certificate
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Date: 05-12-19
Client Name: Cheshire East Council
Client No:
Project Manager: [redacted]
Author: [redacted]
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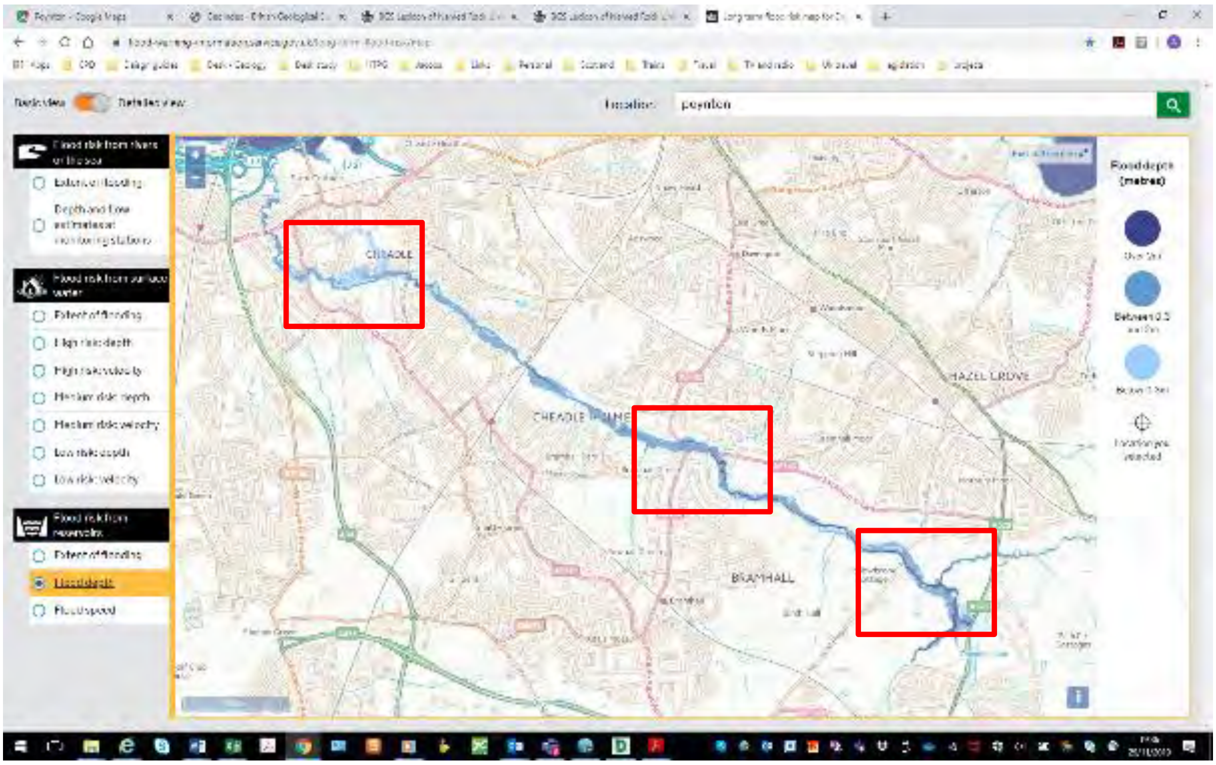
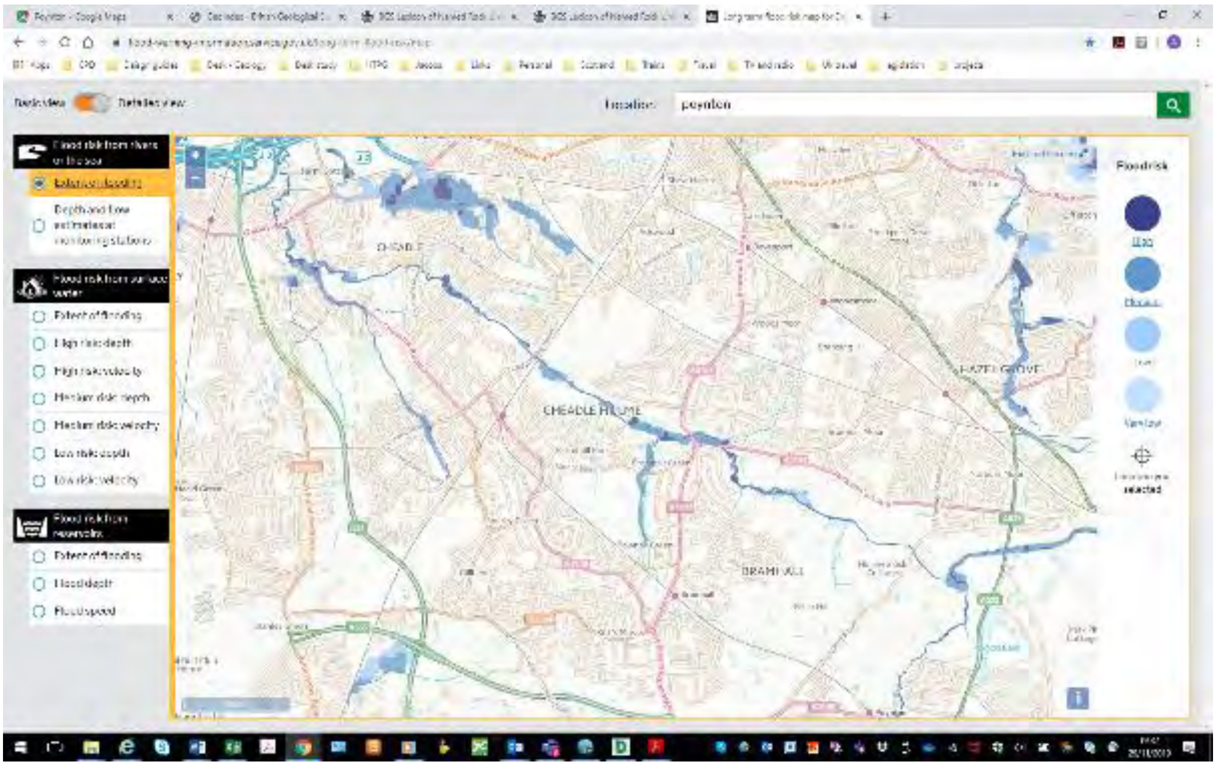
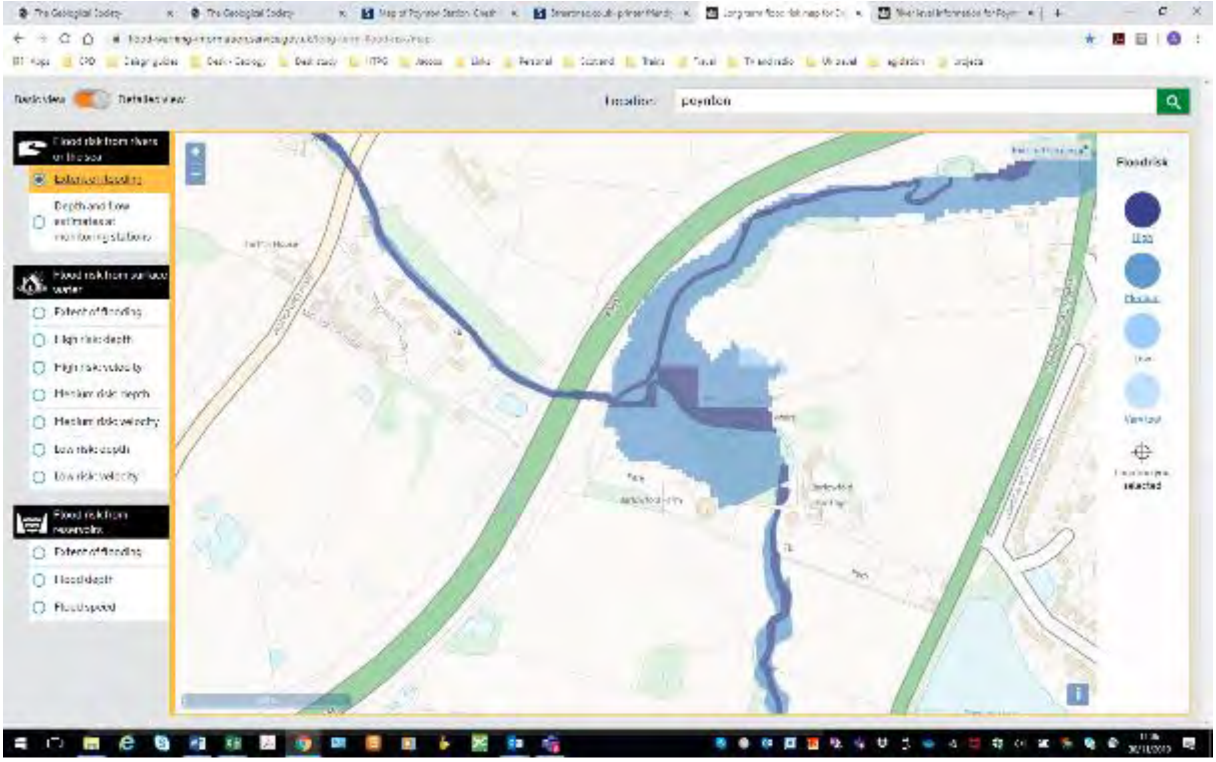
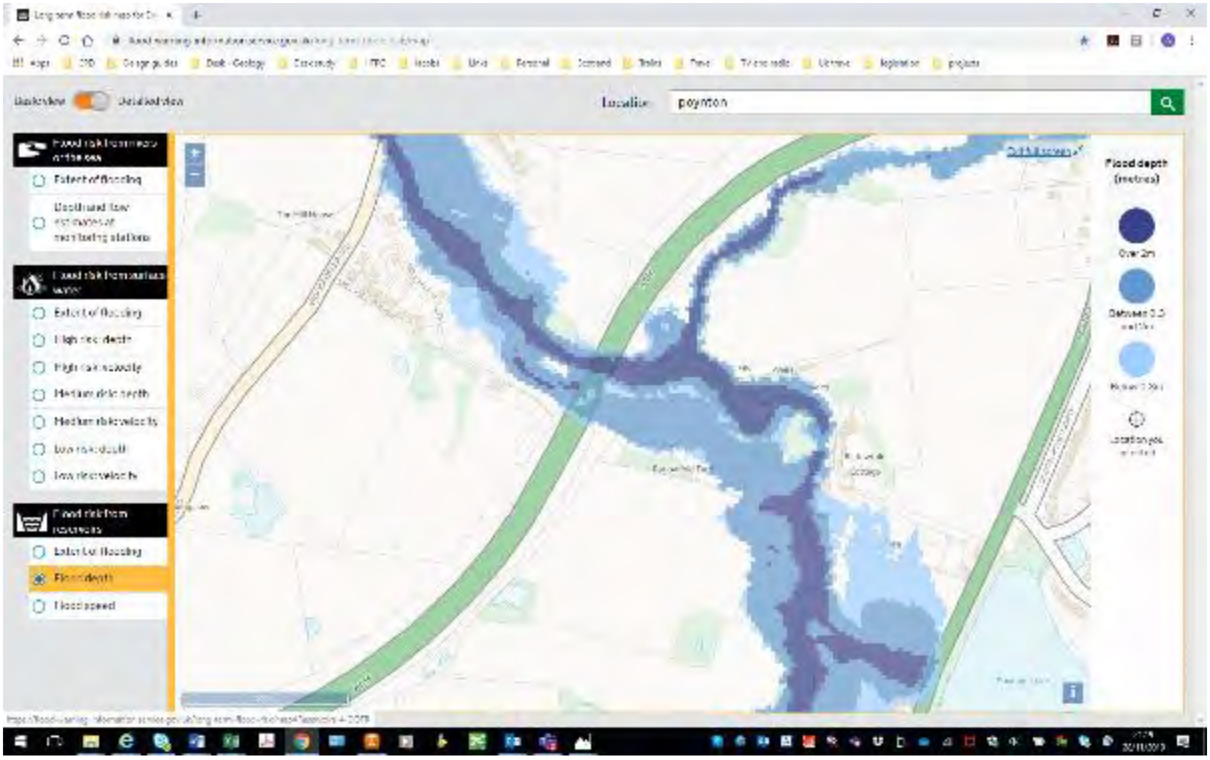
0	29/22/2019	Draft Issue	[redacted]	[redacted]	[redacted]
1	5/12/2019	Final issue	[redacted]	[redacted]	[redacted]

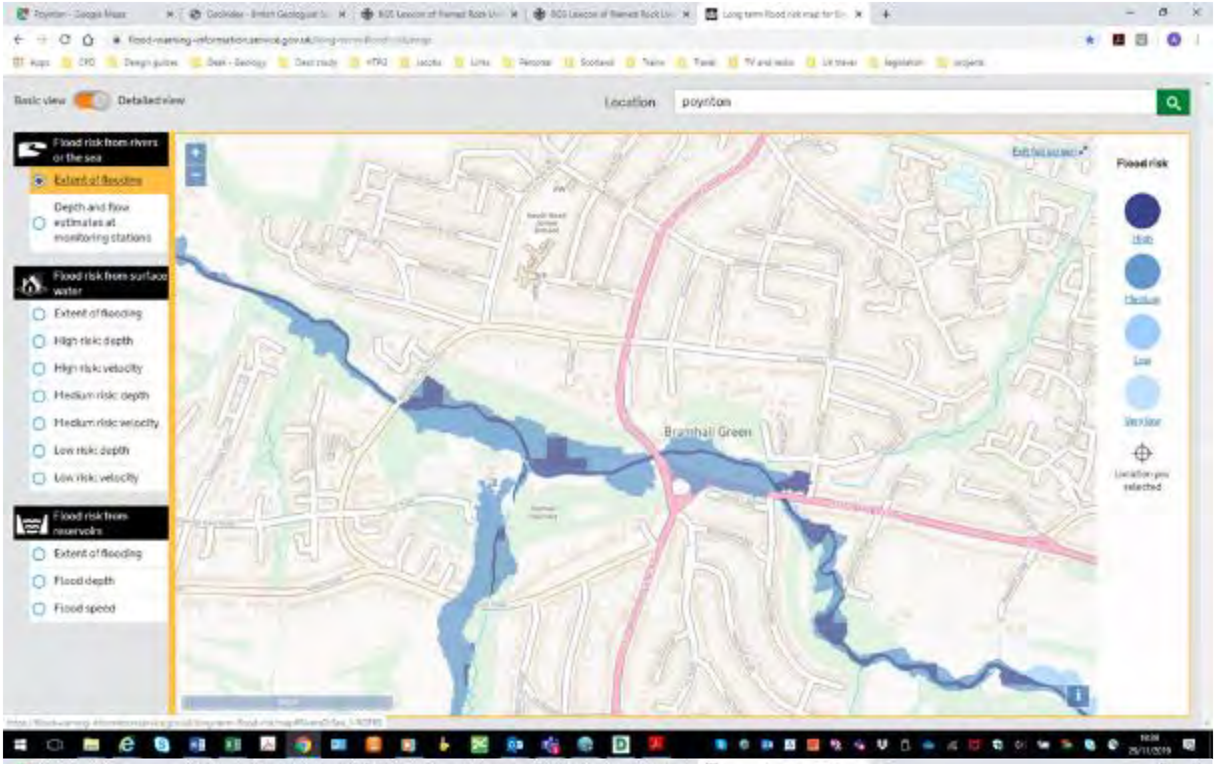
Appendix A – Extracts from flood maps shown on Internet in Nov 2019 (river or surface water flooding on top, reservoir inundation (2009 mapping) below i.e. RFM not yet updated for 2016 spec)

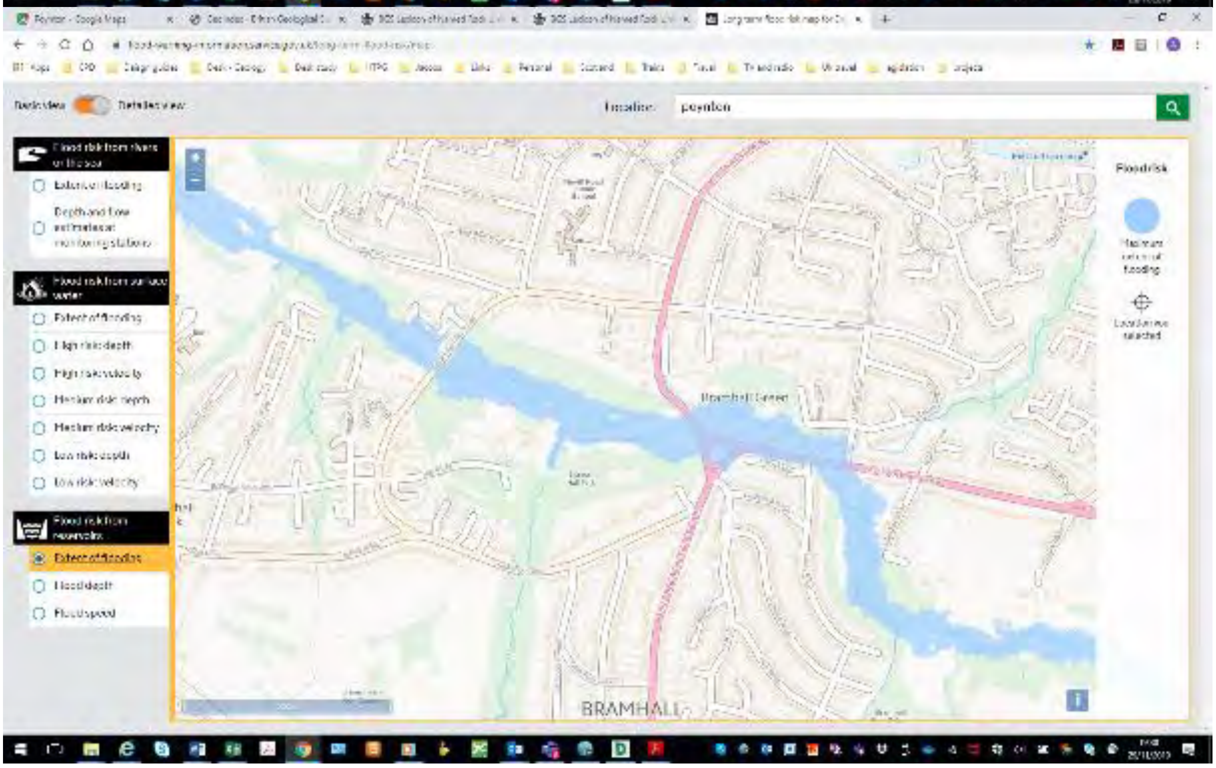


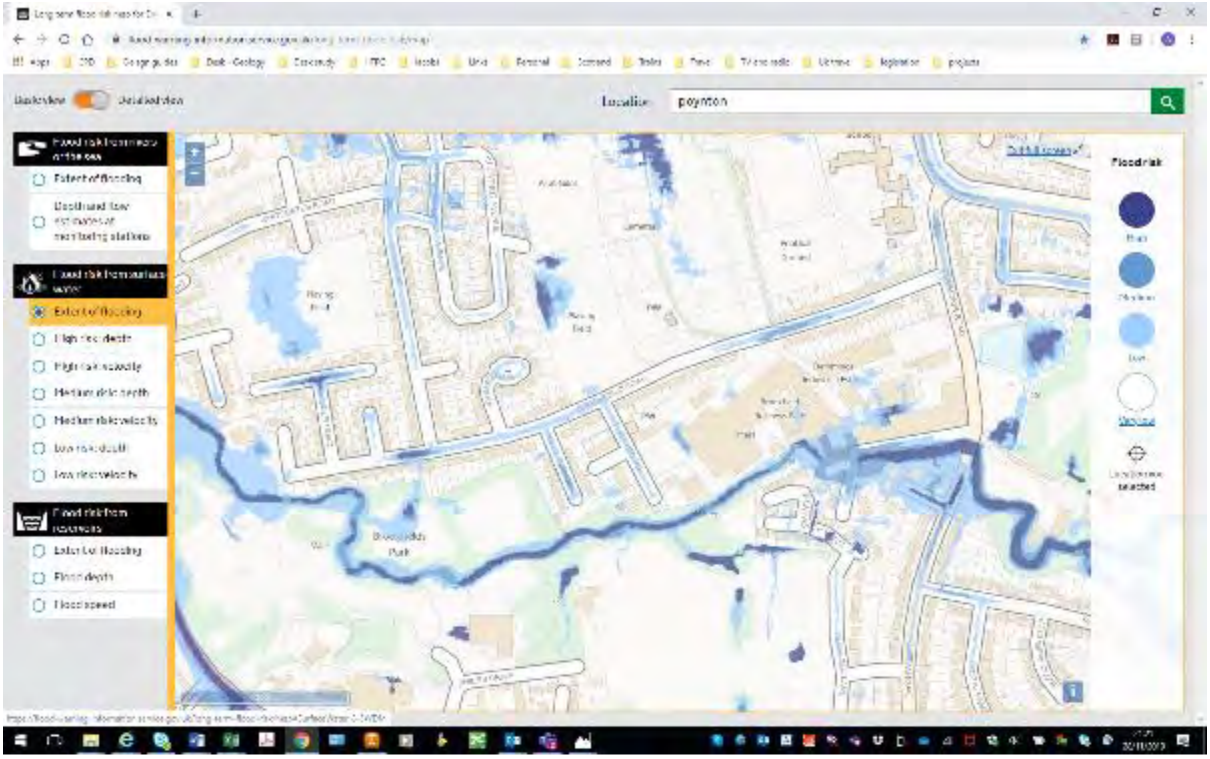


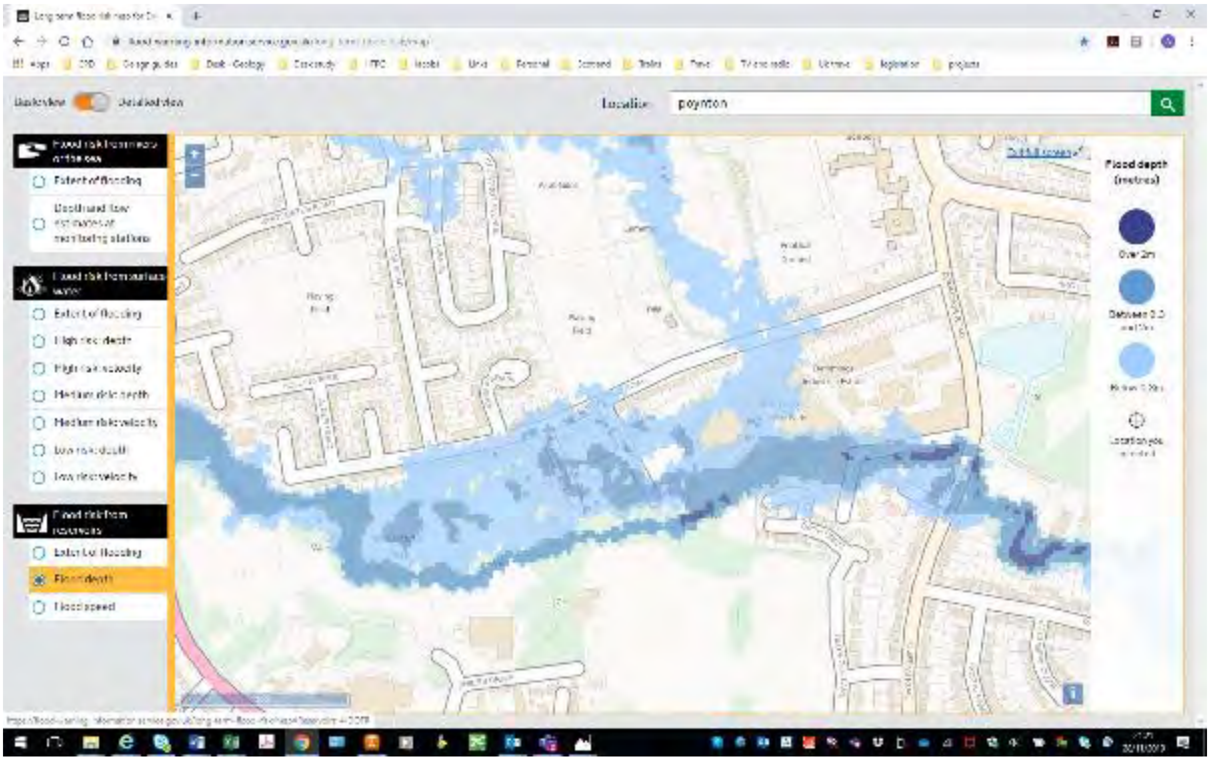

Basic view
Detailed view
Localise
poynton
Flood risk from rivers or the sea
Flood risk from surface water
Extent of flooding
Flood risk from reservoirs
Flood depth
Flood speed
Flood risk


Basic view
Detailed view
Localise
poynton
Flood risk from rivers or the sea
Extent of flooding
Flood risk from surface water
Flood risk from reservoirs
Flood depth
Flood speed
Flood depth (metres)
Over 2m

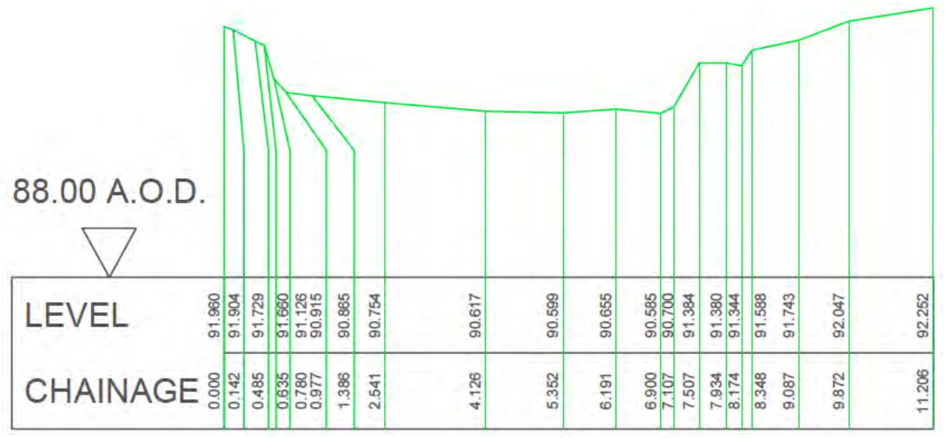

Basic view
Detailed view
Location
poynton
Flood risk from rivers or the sea
Extent of flooding
Depth and flow estimates at monitoring stations
Flood risk from surface water
Extent of flooding
High risk: depth
High risk: velocity
Medium risk: depth
Medium risk: velocity
Low risk: depth
Low risk: velocity
Flood risk from reservoirs
Extent of flooding
Flood depth
Flood speed
Bramhall Green
Flood risk
High
Medium
Low
Very low
Location you selected


Basic view
Detailed view
Location
poynton
Flood risk from rivers or the sea
Flood risk from surface water
Flood risk from reservoirs
Extent of flooding
Flood risk
Bramhall Green
BRAMHALL

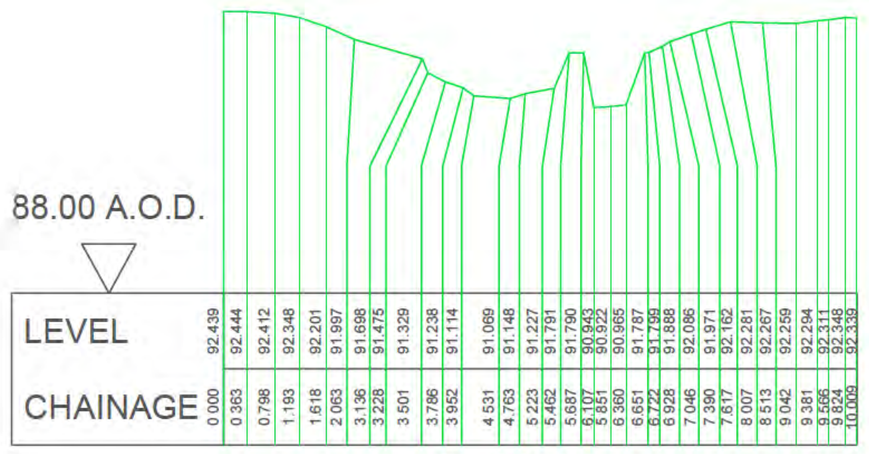

Localise
poynton
Flood risk


Localise
poynton
Flood depth (metres)

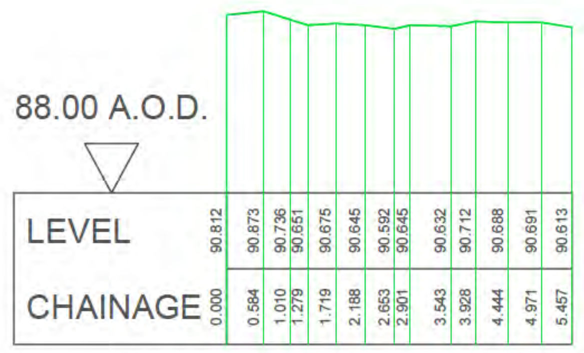
Drawing number / Rev
BRJ10627/Topo/3d/Rev 0



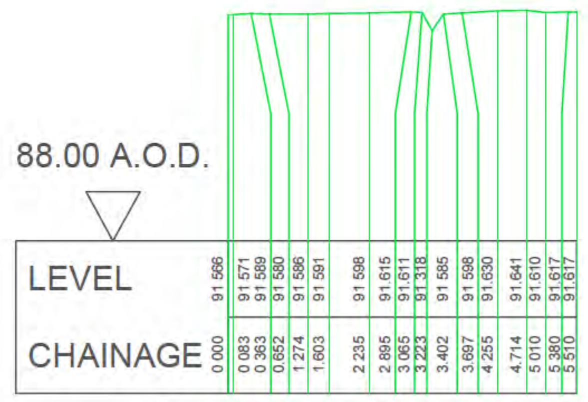
Section 8
Horiz. 1:100 Vert. 1:100



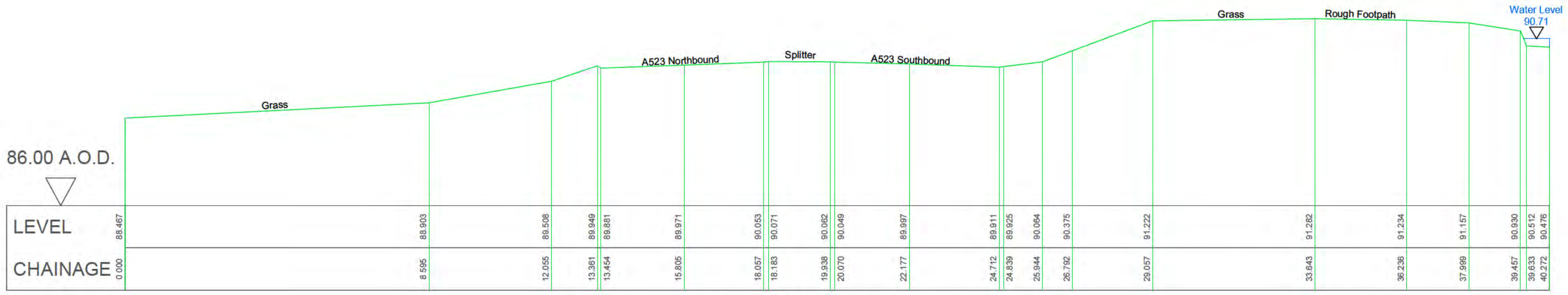
Section 9
Horiz. 1:100 Vert. 1:100



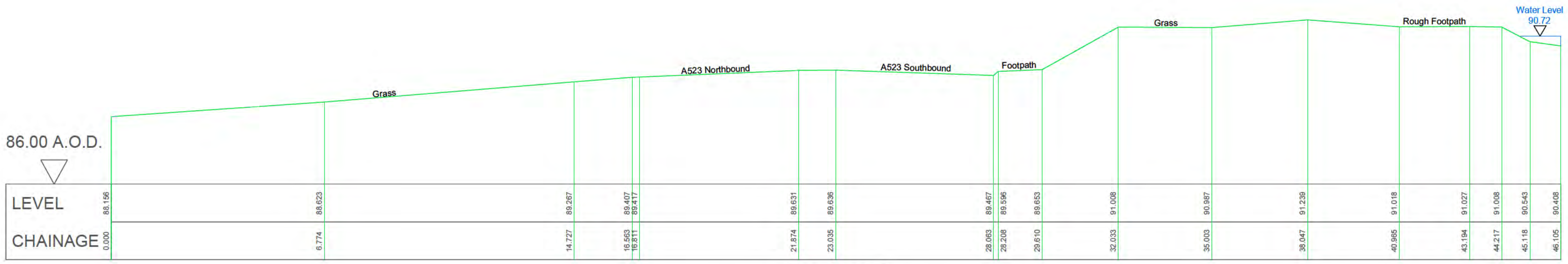
Section 10 (Weir Bottom)
Horiz. 1:100 Vert. 1:100



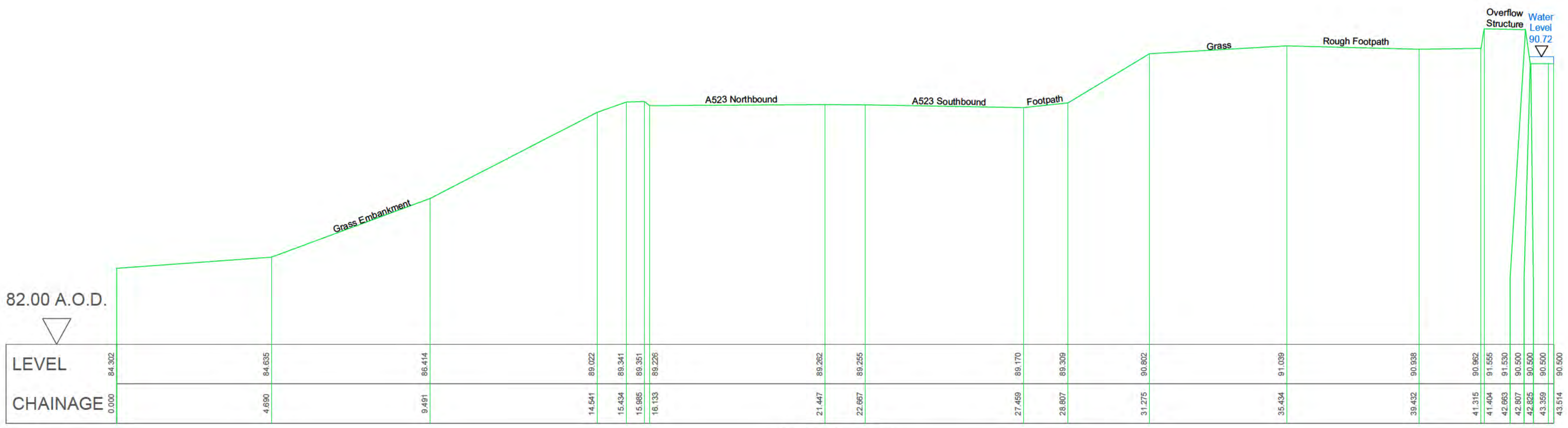
Section 10 (Weir Top)
Horiz. 1:100 Vert. 1:100



Section 7
Horiz. 1:100 Vert. 1:100



Section 6
Horiz. 1:100 Vert. 1:100



Section 5
Horiz. 1:100 Vert. 1:100

Notes:

This drawing is to be read in conjunction with the accompanying survey report, Poynton Lake_ *Topo Survey Report.pdf*. This report contains details of the height datum and coordinate control for this survey work.

Rev	Rev. Date	Purpose of revision	Drawn	Checkd	Rev'd	Apprv'd
B	15.10.19	Revision B	JD	NJH	NJH	PR
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Client
Cheshire East Council

Project
Poynton Lake

Drawing title
3d Topographical Survey
Sheet 5 of 5

Drawing status
FINAL

Scale	1:100 @ A1	DO NOT SCALE
Jacobs No.	BRJ10627	
Client no.		

Drawing number: BRJ10627/Topo/3d
Rev: 0

OFFICIAL SENSITIVE

Cheshire East Council

EMERGENCY DRAWDOWN PLAN FOR POYNTON LAKE RESERVOIR

Rev 2
December 2019

Document control sheet

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0	20/09/19	Incomplete draft issued by Jacobs for client INPUT	[redacted]	[redacted]	[redacted]
1	27/09/19	Updated with comments from CEC Facilities	[redacted]	[redacted]	[redacted]
2	04/12/19	Updated for final Section 10(6) certificate	[redacted]	[redacted]	[redacted]

Contents

1 Introduction and Reservoir / Plan Details

1.1 Need for plan

While it is not currently mandatory for all reservoirs to have an emergency on-site plan it was identified in the 2016 Section 10 inspect report that:

“There is no outlet works at this reservoir and no permanent means by which water can be abstracted from the reservoir. For a small reservoir this is not so unusual and need not be a problem. However, in the case of an emergency there might be a need to lower the water level in the reservoir in order to reduce the hydrostatic load on the embankment.”

If there was a structural failure leading to sudden catastrophic release of the stored water, then the resulting flood wave has the potential to affect the A523 immediately downstream and a few properties alongside the road. As result it was recommended in the section 10 inspection report that in the interests of reservoir safety:

“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. “I recommend that an Emergency Drawdown Plan. should be prepared for the reservoir. This must consider how the drawdown would be achieved including the methods to be used, the implementation of the plan, lines of communication and responsibilities.””

It is not expected the reservoir could be lowered during extreme floods so the emergency drawdown plan is prepared to cover lowering of the reservoir water level during a ”wet day” event inflow.

1.2 Purpose of Plan

This plan supplements existing Cheshire east emergency management plans and protocols to define:

- What constitutes a reservoir safety incident?
- Actions that would be taken in response to an incident
- Information to be provided to facilitate those actions

1.3 Structure of Plan

The plan is intended to be usable by those unfamiliar with the dam and therefore includes key site information in the appendices.

1.4 Reservoir Type, Location, Use and Details

The reservoir is an ornamental lake and provides a local amenity for walking, birdwatching and angling within Poynton Park. Key data on the reservoir is covered in Table 1

Table 1 Key data on reservoir

Reservoir type	Online impounding reservoir
Reservoir location	NGR SJ 923 845
Reservoir Flood Category (as defined by ICE Floods and Reservoir safety 4 h ed, 2015)	B
Year Built	Around 1750
Reservoir Features	Volume: 130 000 m^3 (taken from S10 report 2016) Typical dam height: 2 to 3m Maximum Dam height: Approximately 7m (above narrow valley near to the northern end of the reservoir) Dam crest length: Approximately 800m Dam Construction: Unknown but likely to be heterogeneous clay due to age of embankment

1.5 Site location

A location plan for the reservoir is shown on Figure 1.

1.6 Maintenance of plan

Details are given in Appendix G, and may be summarized as:

Frequency	action	Responsibility
Annually	Check phone numbers	Facilities Management, Cheshire East Council
Annually	Check plan available at short notice	Supervising Engineer
Three yearly	Review whole plan, and update as appropriate	Facilities Management/Emergency planning officer, Cheshire East Council
Ten yearly	Review from a dam safety perspective, as part of Section 10 Inspection under Reservoirs Act.	Inspecting Engineer

Figure 1: Site plan for Poynton Lake Reservoir

2 Reservoir Incident: Identification and response

2.1 Identifying a reservoir safety incident

A reservoir safety incident is any occurrence which has the potential for a catastrophic failure of the dam leading to rapid release of the reservoir water.

There are four levels of incident each of which may warrant a different response, as shown in Table 2.1. The table also shows examples of features that are likely to be symptomatic of a serious structural problem during a "wet day" event and a typical level of escalation of notification within the CEC management and externally, although this may vary on an incident specific basis. This list is not exhaustive and other signs may be observed. If these are of concern, they should be discussed with the Supervising Engineer.

Table 2-1: Link between symptoms and level of concern

	Level of Incident	**Advisory**	**Alert**	**Alarm**	**Imminent failure**
	Key action	**Engineering advice**	**Precautionary works**	**Emergency works**	**Evacuation downstream**
	Internal escalation	Facilities Management	Countryside Management	CEC Emergency Planning	External Police
	See also	Table 2.4		Table 2.5	
Initiation	**Failure mode**	**Observations on site**			
Leak through dam / foundation	Internal erosion	New leakage	Leak carrying dirty water	Sinkhole on dam crest / face	Whirlpool in reservoir

2.2 Declaring the severity of a reservoir safety incident and escalation

A reservoir safety incident will be declared by one or more of the personnel listed in Table 2.2, who would normally collaborate in assessing the incident. Once declared, the incident will be escalated as shown in Table 2.3. Personnel contact details are included in Appendix A.

Table 2-2: Personnel who would normally be involved in declaring a reservoir incident

Order	Role	Name	Organisation
1	Facilities Management	[redacted]	CEC
2	Countryside Management	[redacted]	CEC
3	Reservoirs Act Supervising Engineer	[redacted]	Mott Macdonald Ltd

Table 2-3: Escalation of notification once a reservoir incident is declared

Level of incident	Action required by	Organisation	Level
Advisory	Personnel familiar with dam and Head of Service	See Table 2.2	Not app
Alert	Director for Environment	Cheshire East Council	
Emergency works (Flood warning)	Local Authority (Emergency Planning)	Cheshire East Council	Silver and Bronze
Evacuation	Emergency Services	Police	Gold

2.3 Deciding response to a structural problem at a dam

The level and type of response to manage any structural problem at a dam would be made by those listed in table 2.2, and a first step normally includes obtaining advice from an All Reservoirs (panel AR) Engineer under the Reservoirs Act.

If the problem could lead to uncontrolled release from the reservoir the next step is commonly to lower the reservoir, which is the subject of this plan. This plan does not cover other potential mitigation measures or actions that would be taken, that would be covered in a full “On-site plan”.

The actions that would need to be taken in respect of emergency drawdown are shown in the appendices, namely

A. Contact details for notifying CEC management

B. Number of pumps required vs drawdown rate

C. Pump suppliers

D. Access to site – information to give pump suppliers

E. Where pumps are to be located on site

F. Actions to stop inflows from indirect catchment

A plan showing the location of services at the dam that could be affected, for example, by installing and operating pumps, or other options to manage a dam safety incident is shown in Appendix I.

It is also assumed that CEC would put in place appropriate controls to manage health and safety of personnel involved in implementing an emergency drawdown. This would be recorded by a RAMS prior to commencement of the emergency works.

2.4 Co-ordination and control arrangements

Once an incident has been declared serious enough to involve the emergency services (see previous section) the CEC Emergency Planning Officer (or nominated alternative) would contact the Police, to inform them of the problem and ask for assistance. For a serious incident the Police would require assistance from other emergency services, and would mobilise and manage these through a Local Resilience Forum (LRF), which includes:

- Category 1 responders – Police, Local Authority, Ambulance, Fire & Rescue
- Category 2 responders – Utility companies, NHS trusts.

Depending on the potential geographical extent of the impact, and magnitude of resources required to mitigate its impact, the Police would establish a command structure of strategic/tactical/operational (Gold/Silver/Bronze) groups, corresponding broadly to Regional/County and Borough. Generic off-site plans for dealing with a reservoir incident are prepared by the local authority emergency planning officer.

Potential actions will depend on the seriousness of the situation, as shown in Tables 2.4 to 2.7. Further information on the emergency services is available on the Cheshire East Council website.

The following appendices should be included with information provided to the Police / LRF as briefing information:

1 Key data on reservoir
2 Features at risk of flooding in the event of dam failure
3 Safe access to site

Following discussions between the undertaker (CEC) and appropriate members of the LRF, the Undertaker (if practicable) will initiate the on-site response at a designated location near the incident scene; the emergency control centre. This location ideally would be suitably equipped for use as a Bronze Control by the Category 1 responders. For this the emergency control centre is to be equipped with rooms with desks, multiple fixed telephone and wifi, toilets, rest and refreshment rooms, as appropriate.

This could be in the council offices or could be set up on the flat ground adjacent to the reservoir with a mobile incident centre provided by the LRF.

Table 2-4: Action & Trigger Levels at Standby Stage (dam breach possible)

STATUS	DETAILS OF TRIGGER(S)	UNDERTAKERS ACTIONS	LOCAL RESPONDERS' ACTIONS
ADVISORY	Behaviour of the dam outside normal limits, and advice sought from a Qualified civil Engineer (Inspecting/ Supervising Panel Engineer)	a) Arrange immediate visit by Supervising and / or Inspecting Engineer. b) Increase frequency of readings and surveillance. c) Consider notifying the Police and LA when there is any on-site activity related to a significant potential problem	a) Police/LA may contact all relevant partners to place on standby.
ALERT	A precautionary drawdown is to be carried out to reduce the likelihood of failure to an acceptable level.	a) Undertaker reports details of incident to Police. b) Undertaker attends scene and provides updates to local responders and Supervising Engineer. c) Undertaker implements On-site Plan and relevant actions in conjunction with relevant Engineer.	a) Police contact relevant Cat 1 (Local Authority, EA etc.) and Cat 2 partners to place on standby. b) Police, in conjunction with Cat 1 partners, consider possible activation of Tactical (Silver) Control on precautionary basis to review procedures, undertake relevant forward planning including evacuation, public information and warning. c) Police, in conjunction with Cat 1 partners, consider possible activation of Strategic Coordinating Group (Gold Control) on precautionary basis to review procedures and undertake relevant forward planning including evacuation, public information and warning.

Table 2-5: Site Action & Trigger Levels at Implementation Stage (dam breach imminent/ in progress)

STATUS	DETAILS OF TRIGGER(S)	UNDERTAKER'S ACTIONS	LOCAL RESPONDERS' ACTIONS
ALARM	An emergency drawdown is required to avert failure of dam structure.	a) Undertaker attends scene and provides updates to local responders and Supervising/ Inspecting Engineer. b) Undertaker implements On-site Plan and relevant actions to mitigate failure or limit impact in conjunction with the relevant Engineer.	a) Police activate and implement Off-Site Plan in conjunction with partners and undertake all necessary migratory actions. b) Police implement all relevant multi-agency command and control arrangements with participation of Undertaker and relevant Engineers.
IMMINENT FAILURE	Control of the reservoir has been lost and failure is inevitable.		
FAILED	The dam has failed and large uncontrolled release of water has occurred.		

Table 2-6: Site Action & Trigger Levels at Stand Down Stage (floodwaters subsided or return to properties permitted)

STATUS	DETAILS OF TRIGGER(S)	UNDERTAKER'S ACTIONS	LOCAL RESPONDERS' ACTIONS
POST – STANDBY	Serious problem averted.	Undertaker agrees and implements any urgent recommendations from the relevant Engineers.	a) 'All clear' given. b) Cat 1s review plans in light of response and any ensuing recommendations.
POST-IMPLEMENTATION	Water flows from the reservoir are minimal and efforts are focussed on consequences in zones of total and partial devastation.		Cat 1s focus on ongoing response and recovery operations affecting the needs of local populations, buildings, critical infrastructure etc.

2.5 On-site response to reduce likelihood of failure

The Undertaker would appoint an Incident Controller (IC) – see duties in Table 2.7. It should be noted that the Incident Controller can only authorise activities on Cheshire East Council land. They should not take any offsite actions, which should be carried out by the Police or personnel authorised by the Police.

The IC will appoint personnel to cover particular site roles. Individuals may be asked to cover more than one role. Additional members may be needed to be drafted into the area to undertake roles. If a role cannot be filled locally the IC should resource it from a third party.

An important part of responding to an incident is assessing what could be done to reduce the risk of failure. This should include health, safety and environmental risks associated with implementing what is proposed. Examples of assessment forms and possible on-site actions are given on a reservoir specific basis in Section 2 of this document.

Table 2-7: On-site Roles & Responsibilities (may be used as Action card on site)

Role	Typically undertaken by	Responsibilities Include
Incident Controller (IC)	Reservoir Owner	a) Direction of ALL people associated with any aspect of the incident response. b) Assesses the problem. c) Agrees technical solutions with Panel Engineer d) Agreement of overall response & recovery strategy with Incident Manager (Police to appoint). e) Implement a range of measures to avert failure including the on-site plan. f) Providing ongoing surveillance and situation assessments. g) Communication with Incident Manager. h) Overall H&S of all personnel addressing incident. i) Notification of and liaison with emergency service leaders on site. j) Maintaining a log. k) Record keeping.
Panel Engineer	Supervising/ Inspecting Engineer	Provide technical advice to IC on engineering aspects of managing the incident
Marshalling Officer (MO)	The IC or their appointee	l) All people arriving on site must 'check in' with the marshalling officer and 'check-out' when leaving. m) Checking on continuity of response i.e. that if someone is leaving site their responsibilities are either fully executed or properly handed over. n) Providing safe approach routes to the emergency services. o) Assists in information flows to incident team via the Information Officer. p) Establishing an emergency control location and monitoring personnel requirements.
Practical works	Undertaker staff or External Contractor	a) Erect barriers, control traffic, prevent public access, b) Arrange over-pumping, digging ditches and other works as instructed by MO or IC.
Incident manager	CEC/ Police	As defined for Silver/ Gold Control

2.6 Dealing with the media

All questions posed by the media regardless of type (local, national etc.) shall in the first instance be directed to the Communications Manager for Cheshire East Council (listed in Appendix A).

Reservoir Panel Engineers employed by CEC will **not** provide information or comment to any form of the media unless specifically requested to do so by CEC management.

All technical queries that CEC has relating to the reservoirs should be directed to the Reservoir Panel Engineer(s) who are assisting with the incident.

DATA SHEETS

Appendix A Key Contact Details & Plan Distribution

Role	Name	Office	Other Phone	Email	Base location	Copy of this plan?
Cheshire East Council						
Poynton Park Ranger	[redacted]	[redacted]		[redacted]		Yes
Facility Management Officer for Poynton Reservoir	[redacted]	[redacted]	[redacted]	[redacted]		Yes
Facilities Manager	[redacted]	[redacted]	[redacted]	[redacted]		Yes
Head of Estates	[redacted]		[redacted]	[redacted]		
Head of Countryside Management	[redacted]	[redacted]	[redacted]	[redacted]		
Head of Highways	[redacted]	[redacted]	[redacted]	[redacted]		
Emergency Planning Officer	[redacted]	[redacted]	[redacted]	[redacted]		Yes
Communications Manager	[redacted]	[redacted]	[redacted]	[redacted]		
Reservoir Panel Engineers						
Supervising Engineer	[redacted]	[redacted]	[redacted]	[redacted]	Mott MacDonald Ltd	Yes
Alternate Supervising Engineer	[redacted]	[redacted]	[redacted]	[redacted]	Mott MacDonald Ltd	No
All Reservoirs Panel Engineer (ARPE)	[redacted]	[redacted]	[redacted]	[redacted]	Jacobs – Reading	No
Emergency Services						
Police	Emergency Only 999 / Non-emergency 101 (24 hours a day, 7 days a week.)					No
Fire	Emergency Only 999 / Local fire station (Poynton): 01625 877295					
United Utilities	Water and wastewater emergencies: 0345 672 3723					
Environment Agency	Flood Line: 0845 988 1188					No
Local Authority (Emergency Planning) – As above for CEC						

Role	Name	Office	Other Phone	Email	Base location	Copy of this plan?

Notes

1. Escalation to Silver / Gold would be arranged by on the local Emergency Planning Officer (EPO) and Police, and if serious enough may involve the "Cheshire Emergency Support Group (ESOG)".Up to date guidance to councils and other responders is given on www.gov.uk
2. It may be possible to get mutual aid from Environment Agency local flood risk team
3. Landowner contact details given in Table A2

Table A2 Address for key contacts

Role	Address	Main office telephone (use individual phones in previous page first)
Dam owner (Undertaker)	Cheshire East Council Estates Service Municipal Buildings Earle Street Crewe Cheshire CW1 2BJ	[redacted]
Supervising Reservoir Panel Engineer	Mott MacDonald Ltd [redacted]	Office tel.: [redacted]
Local authority emergency planners	Cheshire West and Chester Council HQ 58 Nicholas Street Chester, Cheshire CH1 2NP Email: EmergencyPlanning@cheshiresharedservices.gov.uk	Tel: [redacted]
Landowners immediately downstream of Reservoir	[redacted]	Phone numbers available for facilities Manager, see Table A1

Appendix B Planning Emergency Draw-Down of reservoir

B1 General

The equipment required to drawdown the reservoir depends on:

- The inflows into the reservoir (pass through flows)
- The amount it is wished to lower the reservoir (lowering rate)

In general, “wet day” event inflows may be estimated by using the published distribution of flows over a year at the nearest gauging station. An alternative approach is to use the general “English Formula” suggested by Hinks (2009, Dams and Reservoirs, 19(1)) of:

Q_{10} (m^3/s) = 0.035 A(km^2).

Where A = Catchment Area

The “Guide to drawdown capacity for reservoir safety and emergency planning” (Environment Agency, 2017) recommends drawdown rate of 5% of dam height/day, and this rate will be used here. Note this rate is more onerous than the 300mm/day recommended in the Section 10 Inspection Report.

A calculation is given overleaf for the pumping capacity required to achieve these drawdown rates, but the assumptions made would need to be reviewed in the light of conditions prevailing at the time of any incident as different assumptions may be appropriate.

The Guide to drawdown capacity also notes an additional criteria that load on the dam should be reduced by half (water depth lowered to 70% of maximum depth). This means that if consideration was given installing a permanent outlet capacity it would if practicable be capable of lower the reservoir by at least this amount.

B2 Supporting information on capacity of pumps/siphons

Indicative pump capacity (from Guide to drawdown capacity for reservoir safety and emergency planning)

Pump size	Maximum flow rate (l/s)	Typical mobilisation time in an emergency from hire depot (excludes journey time)
150mm (6 inches)	55	Around 4 hours
200mm (8 inches)	95	
250mm (10 inches)	250	Not commonly in stock, therefore around 2 days

Note – Fire & Rescue High volume pumps have a maximum capacity of around 117 l/s (https://www.hwfire.org.uk/about-us/the-fire-service/our-vehicles/high-volume-pumping-unit/) However it is likely that the maximum capacity will not be achieved so an assumed efficiency of 50% for the top 1m of drawdown is assumed.

Constraints on use of suction pumps

	Potential problems	Comment	Options for mitigation
1	Insufficient head on intake, so start to get vortices/air entrainment	Required submergence of the intake hose depends on flow rate and pipe size and could be several metres	Intake to extend well below water level to stop air entrainment
2	Loss of suction, once highest point on intake pipe is more than one atmosphere above water level	If the intake pipes go over the crest then the lowest level that suction pumps can work could be 7m below the crest. The efficiency of the pumps will dramatically drop after drawdown of the first 1-2m	(1) Change to submersible pumps
3	Cavitation damage due to insufficient back pressure	Unlikely to be an issue at these lift heights.	(1) Discharge at the same level as the pump (into open channel/ pipe of larger diameter than pump pipe). Valve on downstream end of outlet pipe to ensure some back pressure. (2) Change to submersible pump, where the pump body would be at a lower level and thus have higher back pressure

B3 Template to estimate number of pumps/siphons (including example)

	Item	Units	EXAMPLE: to achieve outlet capacity suggested in current guidance		Template for use in incident	
			Value	Source	Value	Comment
H	Height of dam (in metres)	m	7	Report of an Inspection under Section 10(2) of the Act [2016] (above lowest adjoining valley level)		
A	Direct catchment area	km^2	1.96	Report of an Inspection under Section 10(2) of the Act [2016]		
a	Area of reservoir (in hectares)	m^2	68,000	Report of an Inspection under Section 10(2) of the Act [2016]		
Q_P	Pass through flow	m^3/s	0.052	Poynton Reservoir Flood Study (Jacobs, 2019)		
		m^3/day	4493			
	Inflows from Indirect catchment	m^3/s	Nil	Assume diverted by block inlet – see Appendix E and F		
d	Target drawdown rate	% ht/day	5	Takes precedent as more onerous		
	S10 report recommended minimum drawdown rate	mm/day	300			
Q_R	Lowering rate (conservative as neglect side slopes), neglect inflows	m^3/day	23 800	$a(m^2) \times d(m)$		
Q_L	Total outlet capacity required	m^3/day	28 290			
	Available permanent drawdown facility	m^3/day	Nil			
	Type of pump/siphon		6" pump			
	Assumed capacity of each pump/ siphon	l/s	55	As a conservative assumption the value from the drawdown guidance is used as this is lower than the typical value for F&RS pumps		
		m3/d	4752			
	Number pumps/siphons	m^3/day	6	Q_L / 4752		
	Conclusion		**Require six 6-inch pumps but could be achieved through fewer larger pumps**			

Appendix C Contact Details for pumps, equipment and contractors

In the first instance Cheshire Fire and Rescue would provide any pumps required for use in emergency drawdown See contacts details in Table C1.

Table C1 Contact details for pump provider

Organisation	Cheshire Fire and Rescue
Address	Sadler Road Winsford Cheshire CW7 2FQ
Non-emergency number	[redacted]
Fax	[redacted]

Contact details for Other equipment and resources
This would be dependent on framework contractor at the time; at the moment all maintenance work is through Engie – but if this is specialist it may be that we use CE Highways

Organisation	Engie ServicesCE Highways
Address	c/o Delamere House, Crewe
Non-emergency number	[redacted]
Emergency (24/7) number	[redacted]
Email	[redacted] (no out of hours email)

Appendix D Access to site (information for pump suppliers and other assistance)

Give directions from nearest main road for large lorries
Sate car park at north end – space for 12 cars

<u>Directions from M60</u>

- At junction 3, take the A34 exit to Cheadle/Wilmslow
- Keep left and merge onto Kingsway/A34
- Continue to follow A34
- At the roundabout, take the 2nd exit onto Wilmslow - Handforth Bypass/A34
- At the roundabout, take the 2nd exit and stay on Wilmslow - Handforth Bypass/A34
- At the roundabout, take the 1st exit onto the A555 slip road to Poynton/Bramhall
- Merge onto Manchester Airport Eastern Link Rd/A555
- Turn right onto London Rd N/A523
- Destination will be on the left (Poynton Lake, Poynton, Stockport SK12, UK)
- Site car park at north end – space for 12 cars (see Figure D1)

Figure D1 Location of north car park

Appendix E Locations for pumps and hoses

1. Assume up to two pumps could be placed by outlet and discharge into Manhole 1. This would discharge into the 600mm outlet pipe. Analysis of the downstream pipeline system supports the assumption for a free discharge condition downstream of Manhole 1. Pumps to be craned by HIAB from southbound carriageway on A532 which would require a temporary closure of one carriageway. This could be managed by stop-go boards with warning signs in both directions.

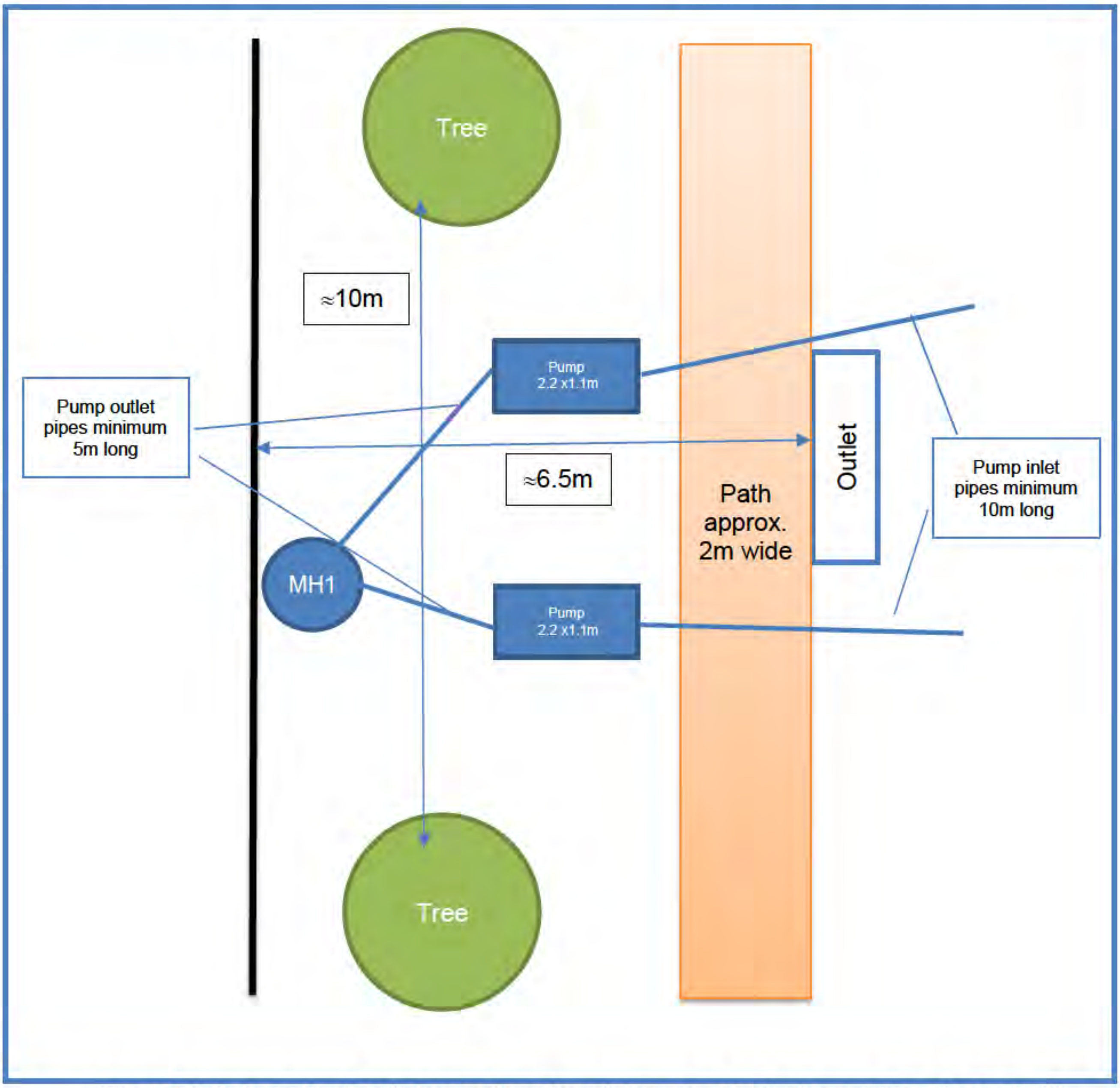

Figure E1 Schematic of initial pump locations (shown are typical pump sizes for up to 8" pumps)

Figure E2 Photograph showing proposed locations of initial 2 no. pumps

2. The second phase of drawdown (Pumps Nos 3-6) could either be provided by placing additional pump in the site (north) car park (Option A) or by closing the southbound lane of A532 and placing pumps within carriageway footprint (Option B), the choice depending partly on where outlet hoses would discharge to. Closure of the carriageway would require traffic lighting.

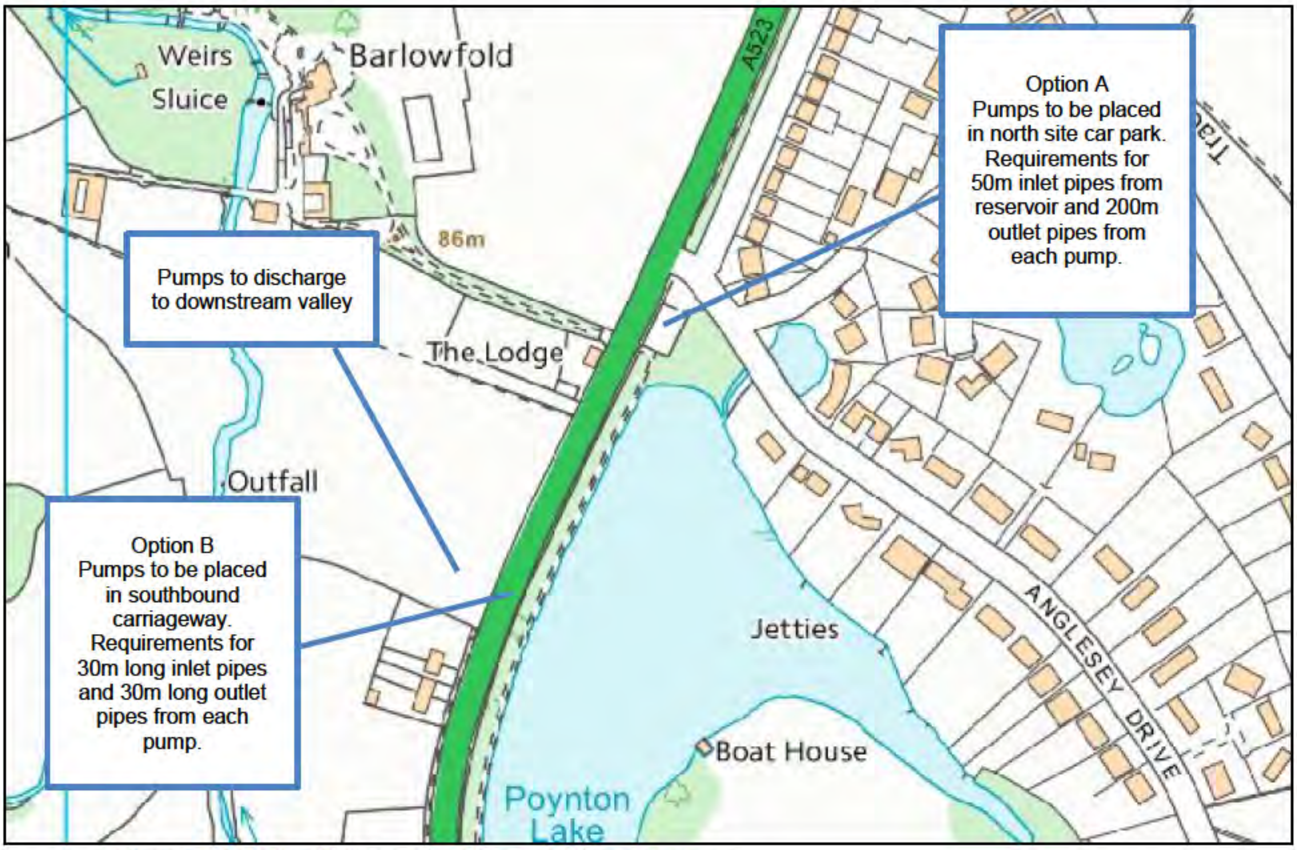

Figure E3 Potential location for additional pumps

Figure E4 Photograph of location for additional pumps (Option A)

Figure E5 Photograph of pump outlet pipe route for Option A

Figure E6 Photograph of outlet discharge location for additional pumps into deep downstream valley (white circle)

Appendix F Inflows to reservoir

There are two sets of inflows to the reservoir, from a direct catchment, and indirect catchment, as follows

Feature	Units	Direct	Indirect
Catchment area	Km^2	1.96	3.83
SAAR	mm	897	918
Method of bringing flows into reservoir		Fluvial system and surface water drainage network	Weir at NGR SJ 92241 83826 into 600mm pipe, which passes into open channel, then 900mm pipe under South park Drive outfalling at NGR SJ 92260 84026
Daily flows			
Q10 (wet day i.e. exceeded on 10% of days in a year	m^3/s	0.052	Not calculated
Q50 - median	m^3/s	0.014	Not calculated

If the reservoir was lowered by say 1m, then the time to refill assuming Q50 from both catchments could be of the order of 55 days

Appendix G Actions to stop inflows from indirect catchment

<u>Equipment required</u>

- 5 Number 2"x4" timber (minimum lengths 1.3m)
- 9 number sandbags
- Wire cutters
- 600mm pipe bung

<u>Actions</u>

To control flow from the indirect catchment stoplogs are placed in the existing stop log groove. See Figure E1. Use to 2" x 4" timber cut to length of groove. These should be stacked up to the height of the groove. These would be backed by sand bags stacked downstream of stop logs to provide additional stability. Access to the weir would be via 4x4 with trailer off Woodside Lane, see Figure E2. Access to weir is through wire fence that will require cutting with wire cutters. An alternative procedure would be to install a 600mm diameter pipe stopper (bung) fixed into position at the culvert inlet.

Figure E1 Photograph of stoplog groove

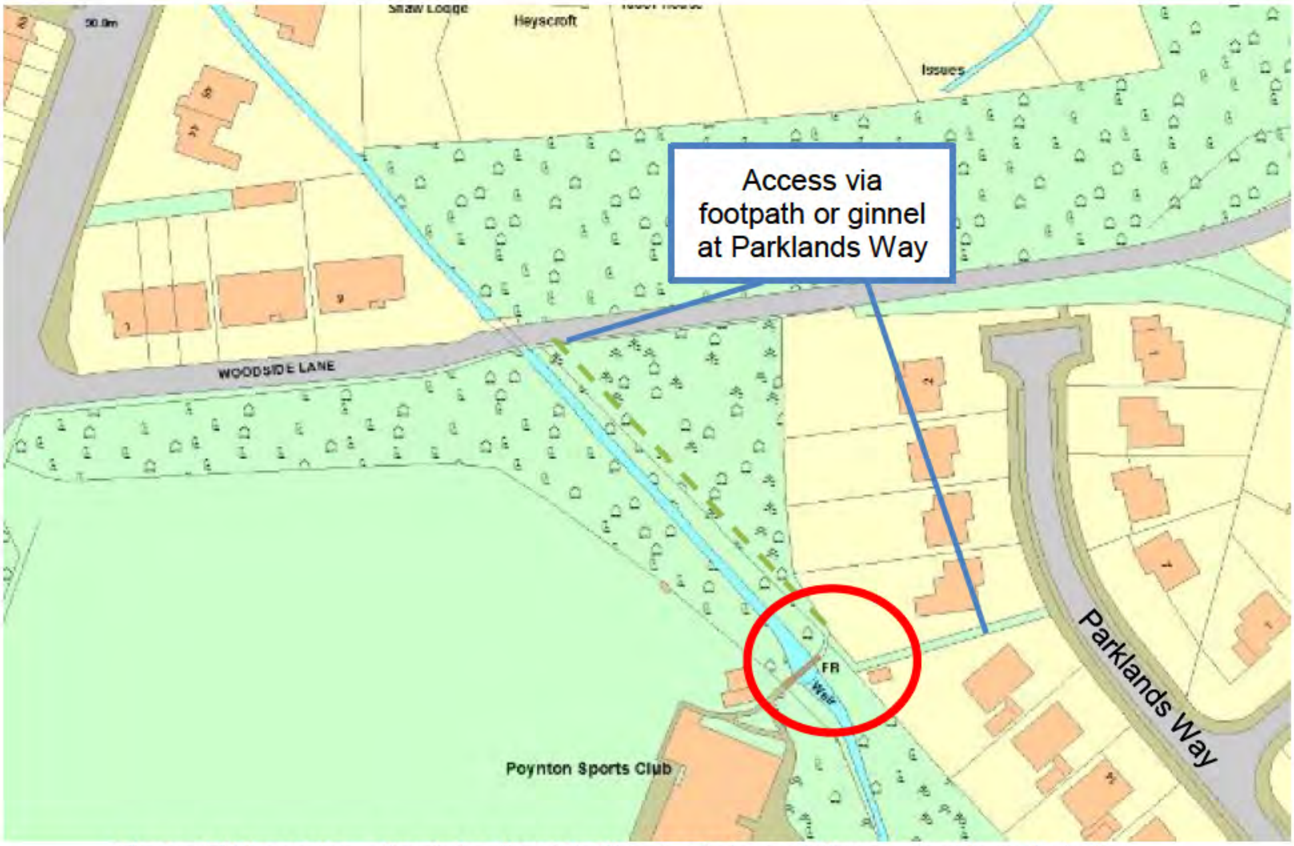

Figure E2 Location of weir to divert indirect catchment flows – inside red circle

Appendix H Source and Maintenance of On-Site Plan

Source of Plan

This plan has been produced to cover relevant sections of the following:

a) Draft Engineering Guide to Emergency planning, KBR, 2006 (available on http://www.britishdams.org/reservoir_safety/default.htm#historical)

b) Defra Guidance on Reservoir Emergencies (August 2009), as defined in

- On-site plan for reservoir dam incidents (guidance on production)
- Preparation Guide; On-site plan for reservoir dam incidents (supplement to above, typical content of plan).

It should be periodically reviewed and updated to ensure it conforms to current requirements and guidance. If and when elements of the Flood and Water Management Act 2010 are brought into force, the Supervising Engineer has various duties to approve production and direct exercising of an on-site plan. This emergency drawdown plan would form part of that work but it would also need to cover additional emergency scenarios. It should be noted that the Cabinet Office have produced a corresponding series of guidance for off-site plans, for use by the local authority emergency planners and Local Resilience Forum.

Routine Maintenance

The plan should be checked and updated as appropriate at the frequency shown below, all updates being recorded by the amended version having a new issue number with this recorded in the document history at the front of the plan. The responsible person is shown in Section 1.5.

Frequency	Scope of Check
Annual, before Section 12 visit	a) All contact details in the On-site Plan are correct b) Check out of hours number works by phoning at 2300 hours one weekend
Three yearly	a) All equipment and materials referenced in the On-site Plan are available in an emergency b) All personnel named as having a role in the On-site Plan are fully aware of their role and what it entails c) Communications with contractors regarding required equipment / material to confirm availability during an emergency d) Review by the Reservoirs Act Supervising Engineer, and update as appropriate
Every ten years, as part of Section 10	Review by Reservoirs Act Inspecting Engineer that dam safety aspects up to date and reflect current good practice in the dams industry.

Staff training

All full time site and associated office management staff should be familiar with all aspects of this emergency drawdown plan.

An induction workshop should be held on first issue of this plan, and as agreed thereafter, to discuss the arrangements, confirm their practicality and identify improvements that could be made to increase the value of the plan.

Exercising

Exercising of plans has been shown to be valuable in refining and improving the plan. However, it is costly and needs careful organisation.

Technical papers summarising exercising of emergency plans for dams are available as listed below and should be read as part of training by those who may be involved.

Date	Authors	Title	Comment
		Operation Triton	Exercise by United Utilities at Dovestone Reservoir by Greater Manchester Resilience Forum. Presented at Sept 2016 conference and to be written up in *Dams and Reservoirs* publication
2017	Environment Agency	Guide to drawdown capacity for reservoir safety and emergency planning. On internet	Section 2.3 covers regular exercising of valves.
2012	Windsor	Pebley Reservoir (Derbyshire) Emergency Drawdown Exercise. Proc BDS Conference. Leeds. Pp193-206	Exercise was carried out by the Canal & River Trust in November 2011
2010	Brown et al	Exercising of Emergency Draw-down Plans. Thomas Telford Ltd. Proc BDS., Strathclyde.	Exercise by Canal and River Trust at Foulridge Reservoir, with 8 inch electric submersible pump

Appendix I Services Plans

JACOBS

Poynton Reservoir Flood Study

Cheshire East Council

Poynton Flood Study Report

D03 | V01

December 2019

Document history and status

Revision	Date	Description	By	Review	Approved
D02	29th Nov 2019	Draft for client comment	[redacted]	[redacted]	[redacted]
D03	5th Dec 2019	Final report	[redacted]	[redacted]	[redacted]

Distribution of copies

Revision	Issue approved	Date issued	Issued to	Comments
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D03	[redacted]	5th Dec 2019	Cheshire East Council	No revisions required

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 4th 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

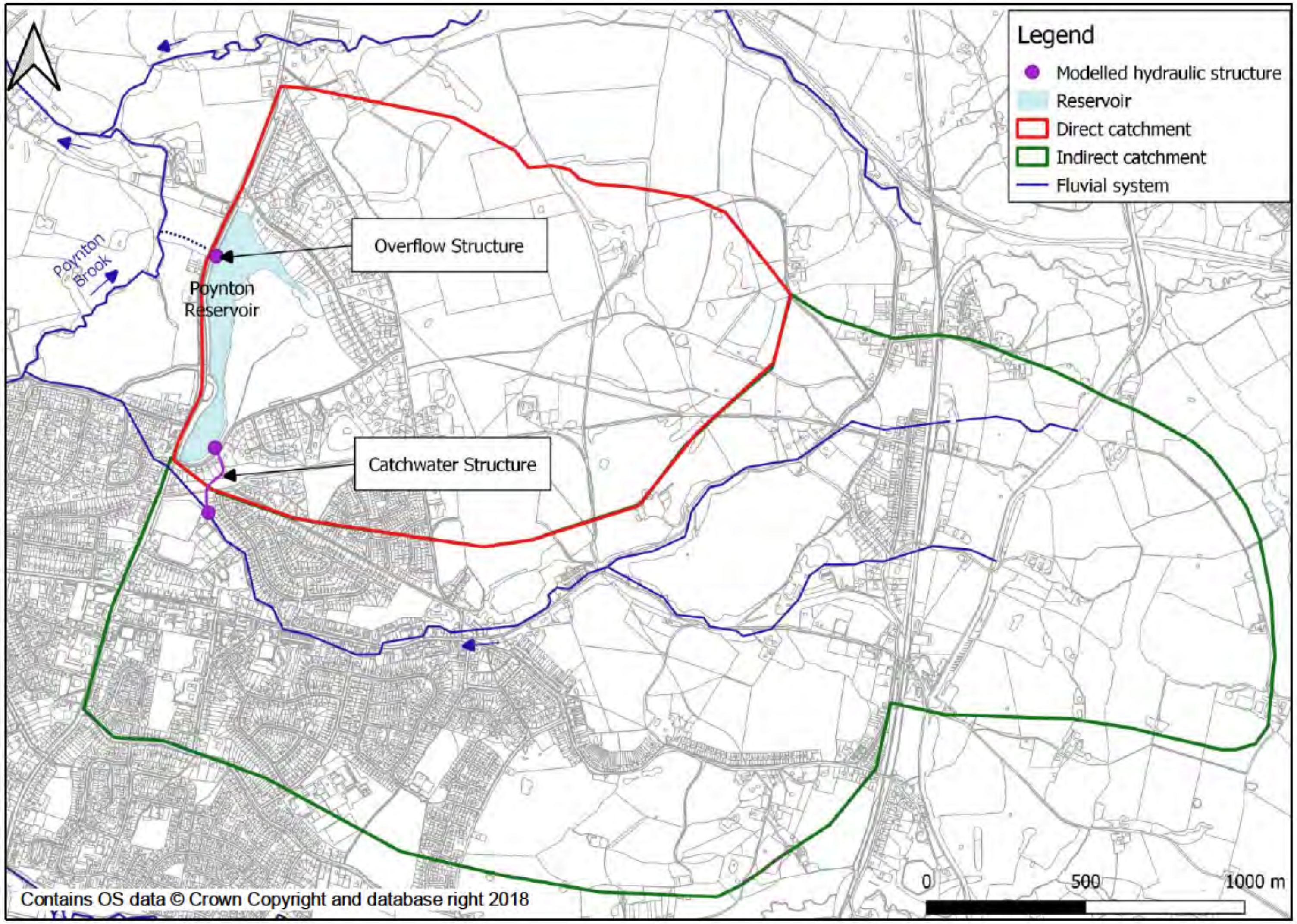

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Figure 1-3: Surface Water Drainage Network at Poynton Lake Reservoir

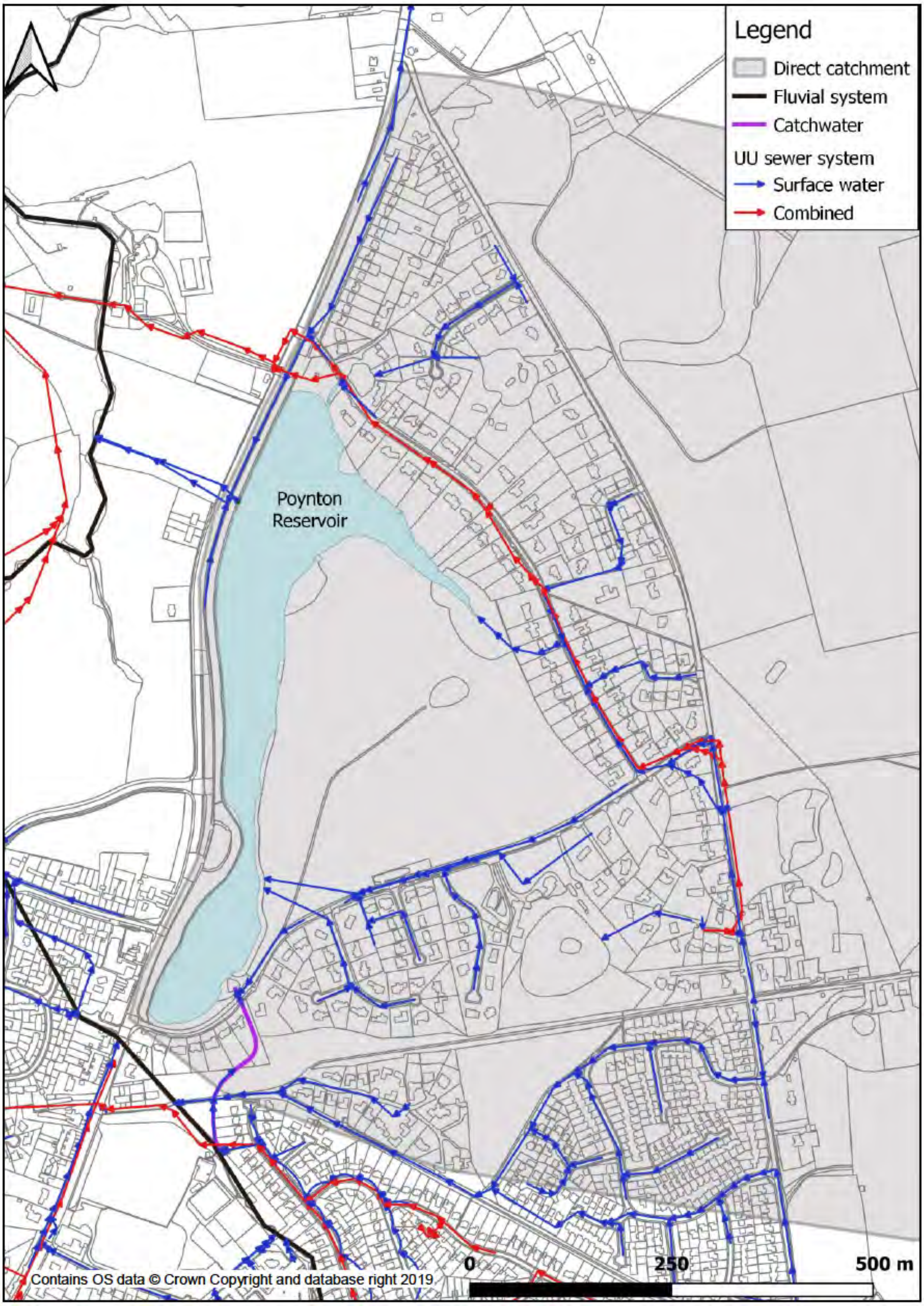

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Figure 1-4: Poynton Lake Reservoir Overflow Arrangement, photo inserts show (a) overflow box and (b) internal view of culvert inlet.

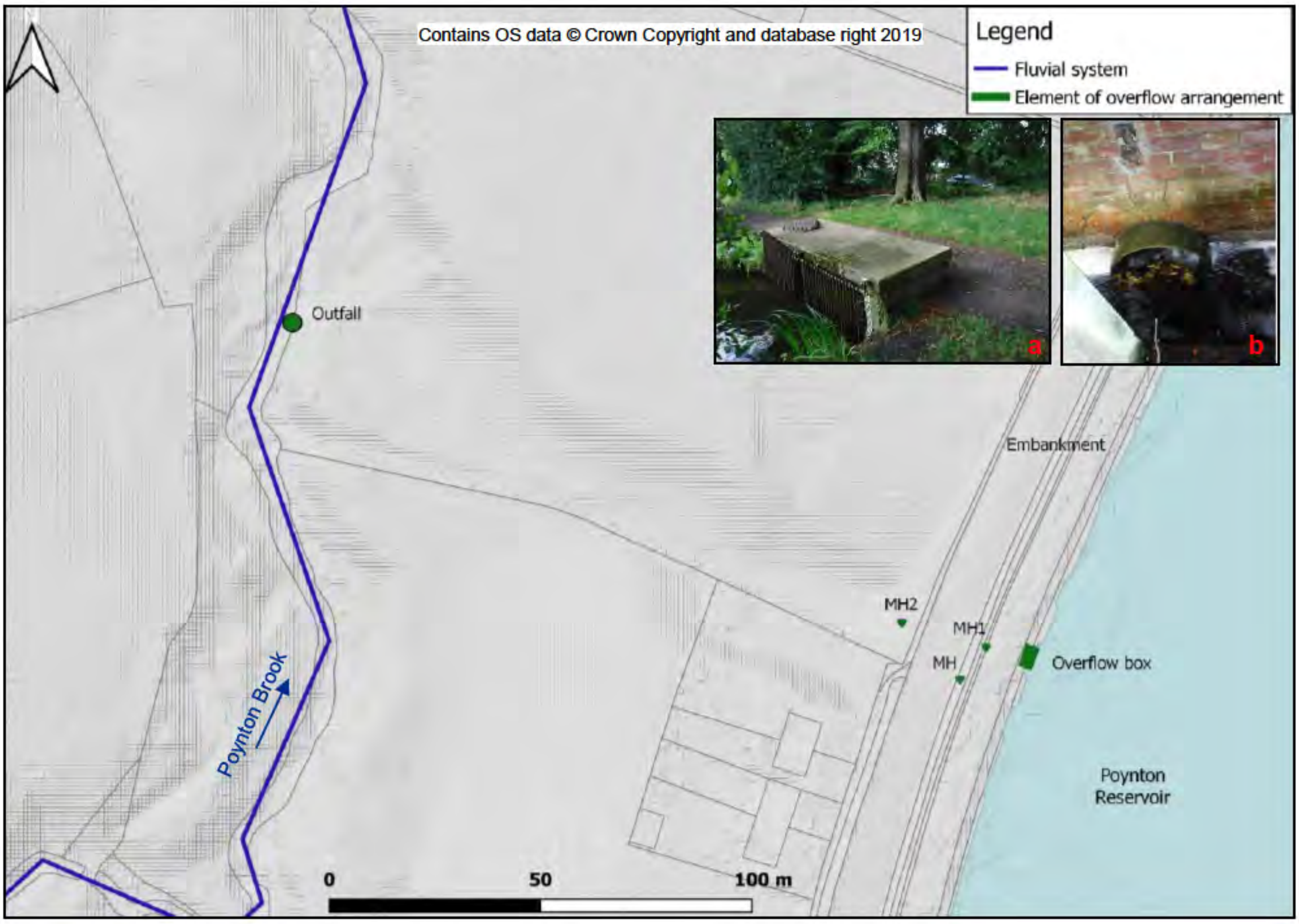


Table 1.1: Reservoir Details

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.

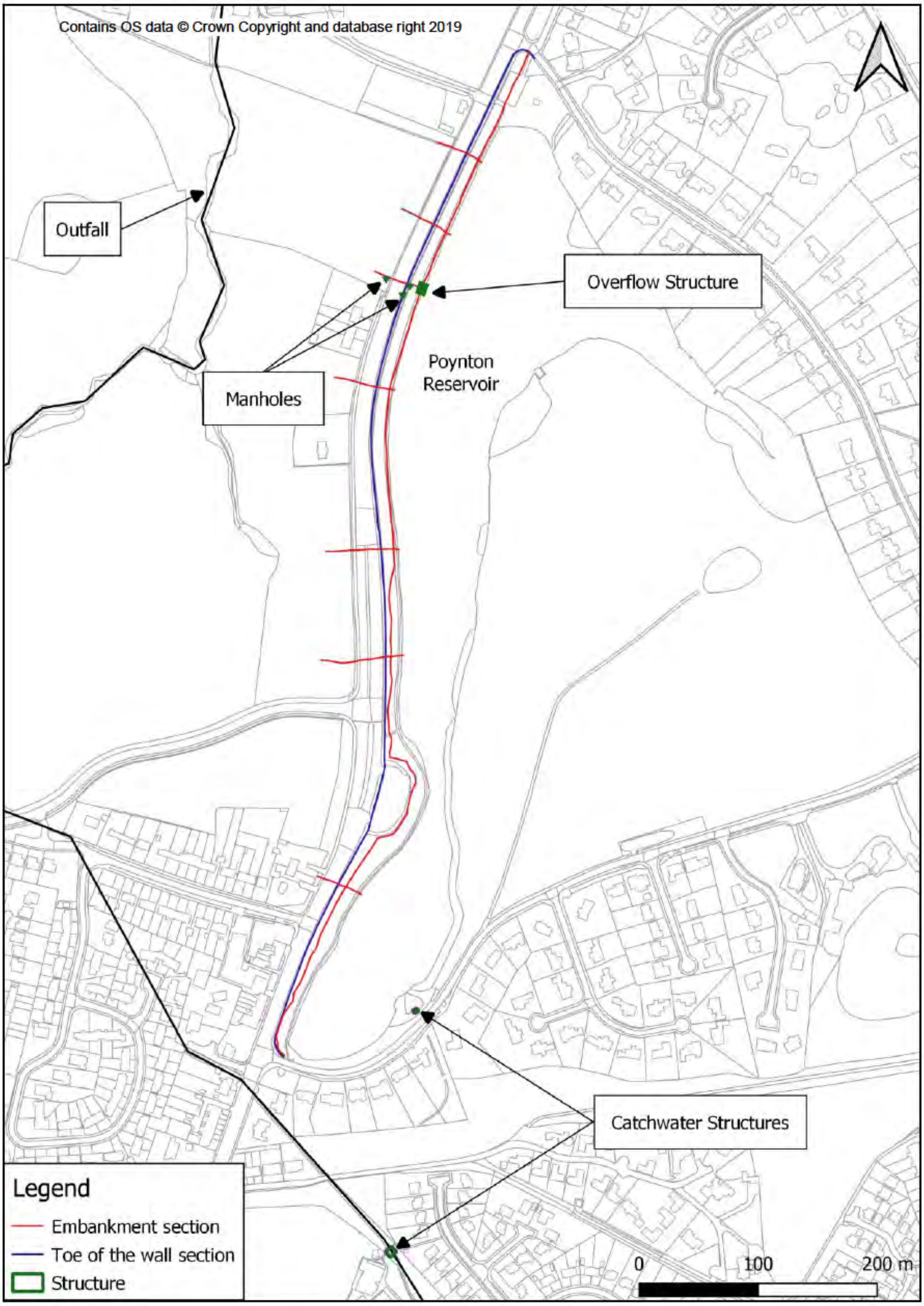
The survey results were provided relative to national spatial datum and to Ordnance Datum. A full list of deliverables for the topographical survey is provided in Table 3.1.

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 2.64m^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

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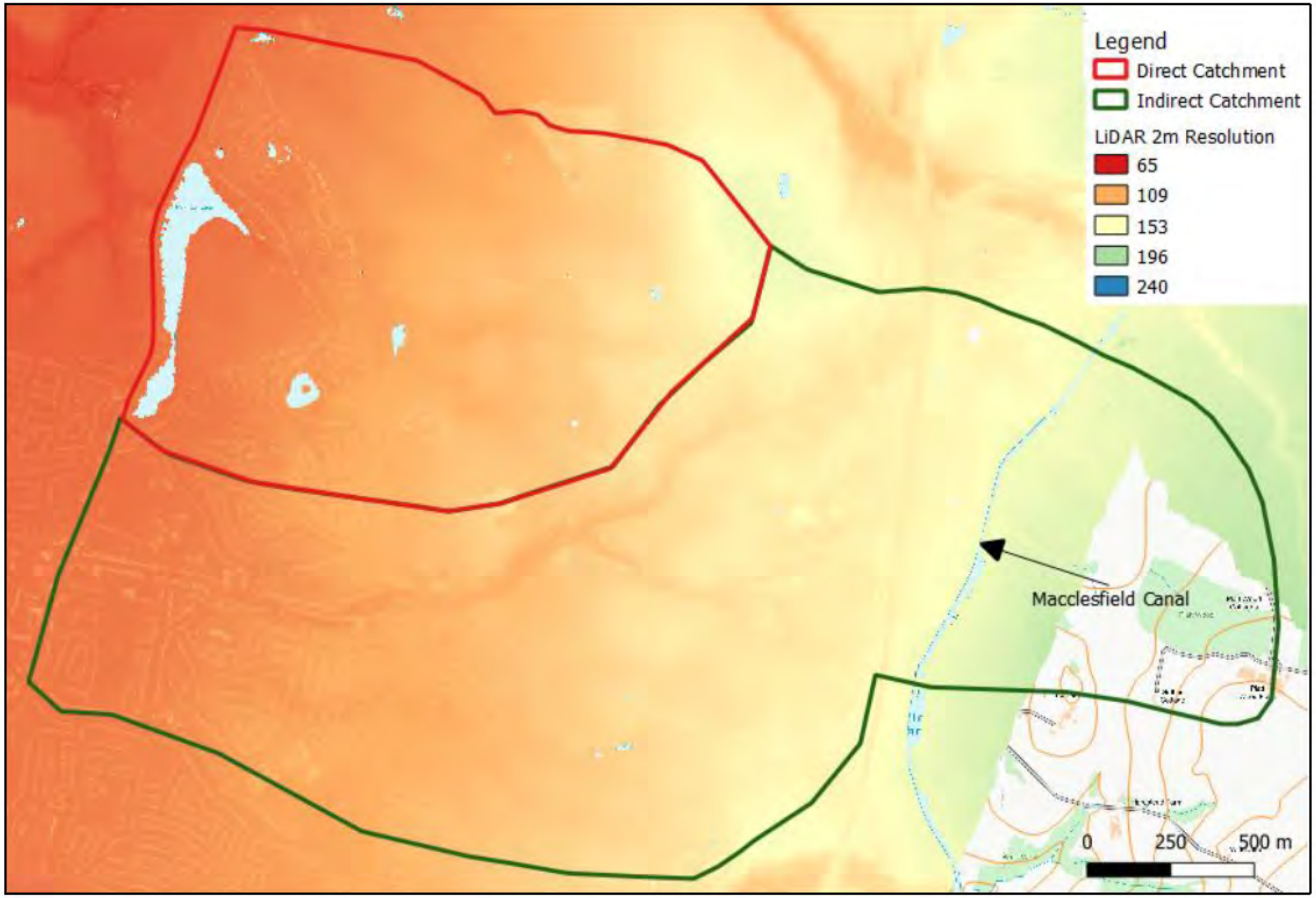
[2] Reservoirs Act 1975, Poynton Lake Reservoir, Report of an Inspection under Section 10(2) of the Act (Mott MacDonald, 2016)
[3] geology from the Geology of Britain Viewer http://mapapps.bgs.ac.uk/geologyofbritain/home.html

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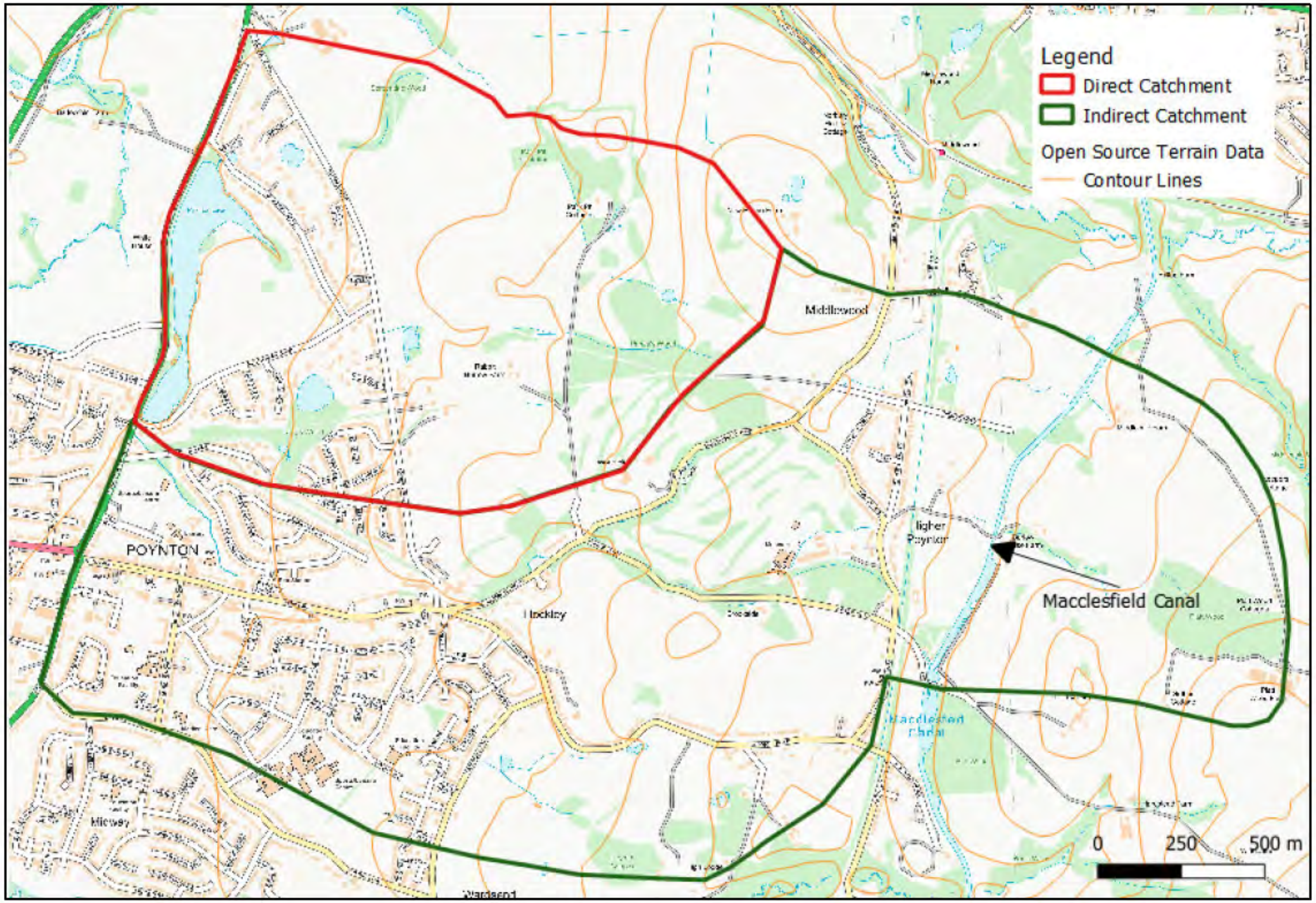
*LiDAR data was not available for the full indirect reservoir catchment.

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.

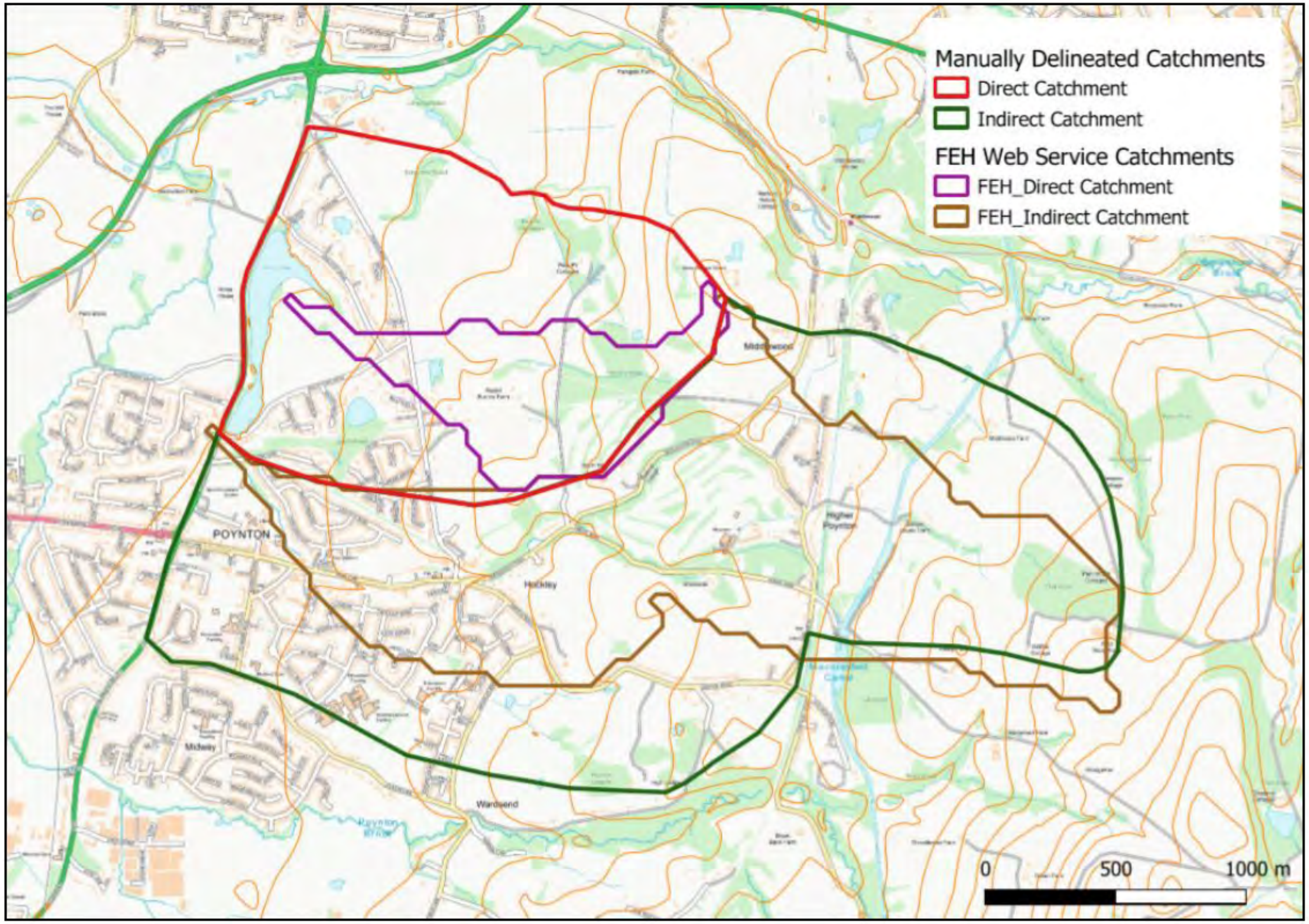
4.3.2 Catchment Areas

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 19% HOST Class 18	541r/Wick 1	46	75% HOST Class 5 25% HOST Class 7
Indirect	711m/Salop	95	81% HOST Class 24 19% HOST Class 18	541r/Wick 1	5	75% HOST Class 5 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 (km^2)	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:

$$D = Tp(t)\ (1+SAAR/1000) \qquad \text{(without reservoir)}$$

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 Δt as follows:

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

In the hydrological analysis a time step Δt 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

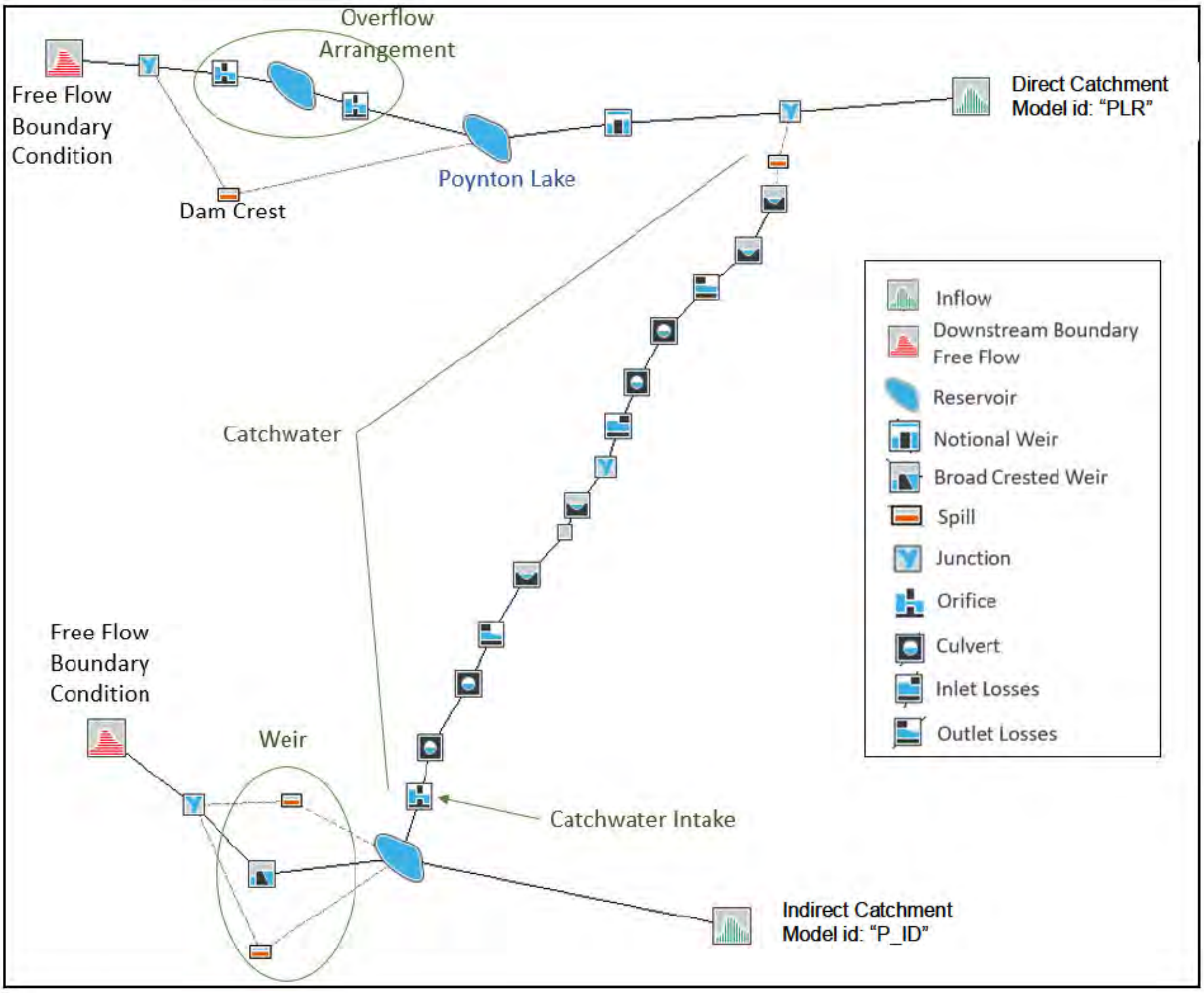
Version 4.4 of the Flood Modeller Pro hydraulic modelling software package (Jacobs, 2018) was used for modelling the Poynton Lake reservoir. The model schematisation is shown Figure 5-1. Key features of the reservoir system can be seen in Figure 1-2.

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



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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

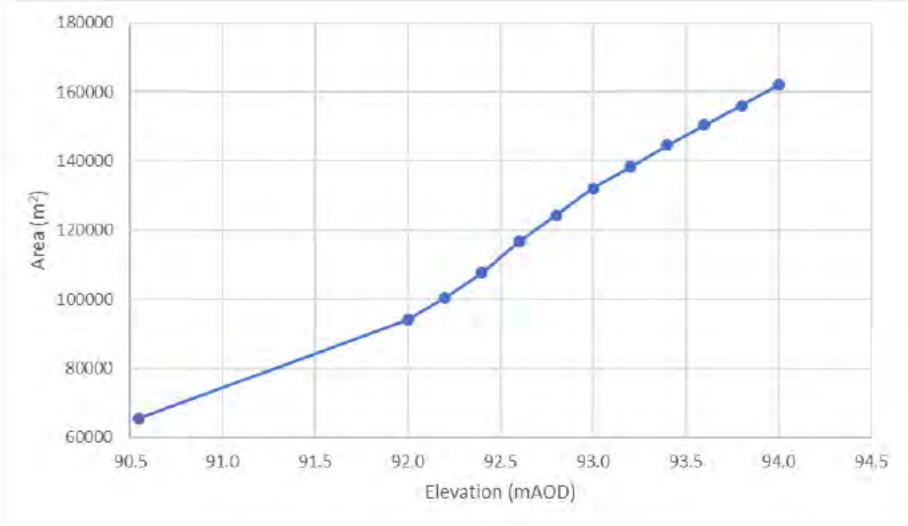
5.3.1 Model Inflow

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 (10,000-year return period)	2.9	11.2	11.0	91.10
0.1% AEP (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 11.0m^3/s. Peak flow through the overflow culvert is 0.85m^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

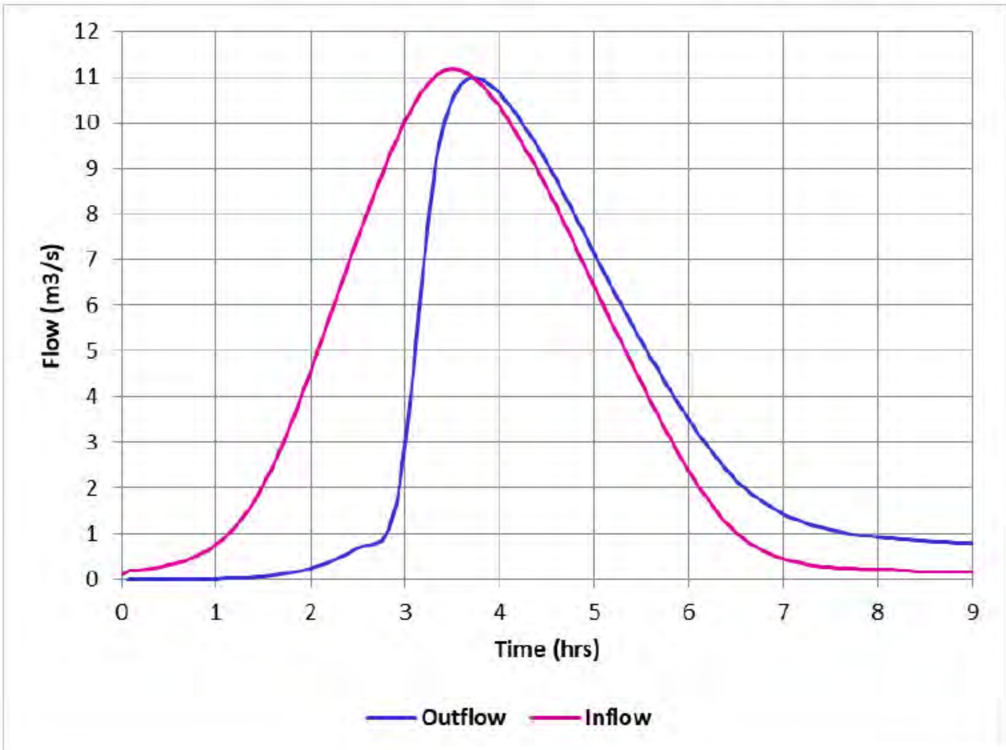
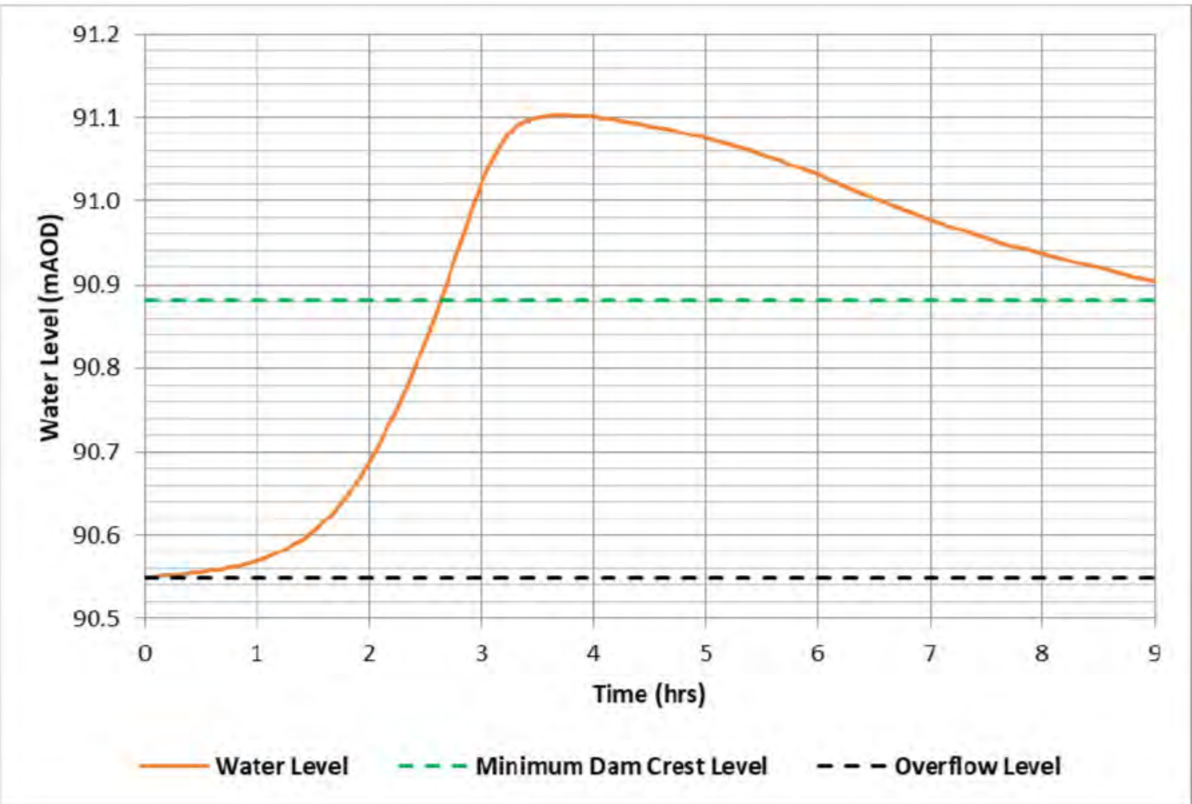


Figure 6-2: Poynton Lake winter 0.01% AEP (10,000-year return period) stage hydrograph with critical dam structure levels



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

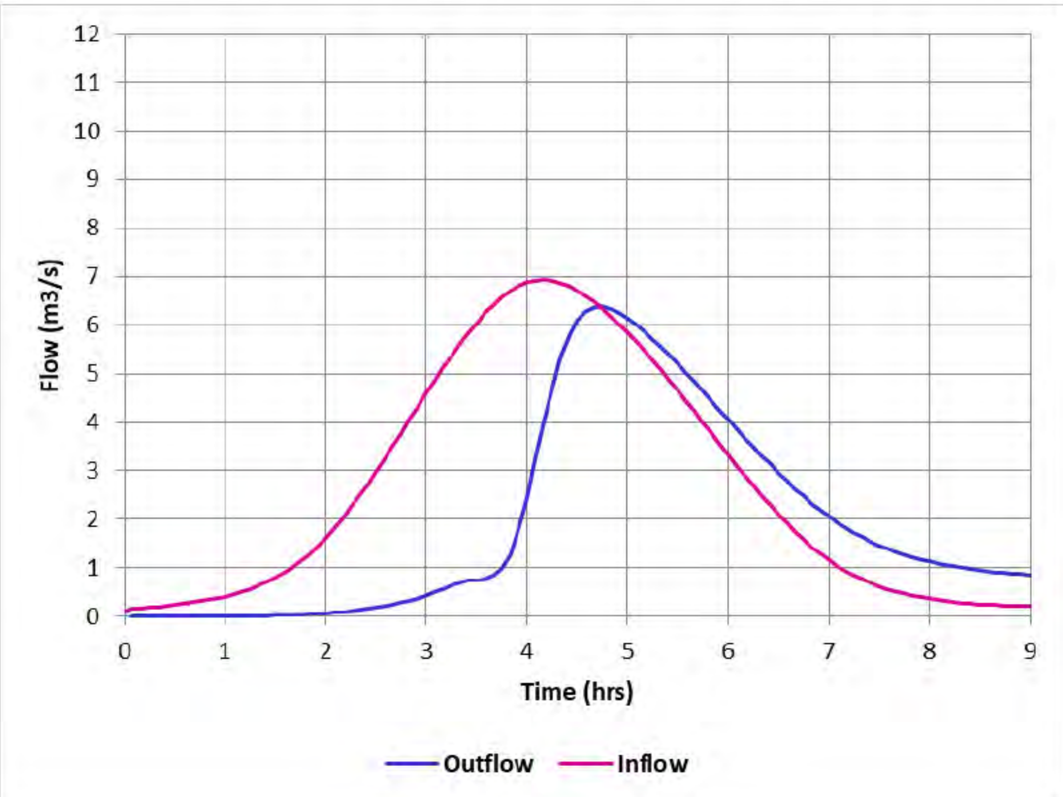
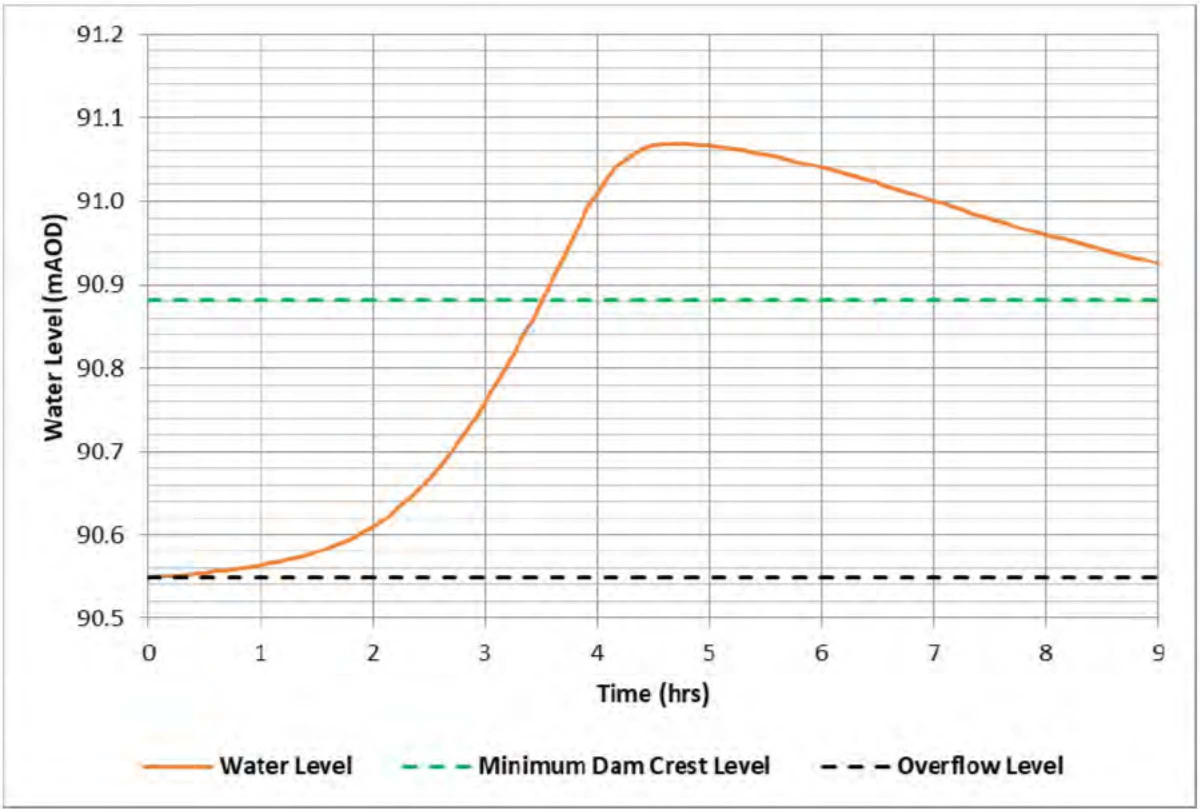


Figure 6-4: Poynton Lake reservoir 0.1% AEP (1,000-year return period) stage hydrograph with critical dam structure levels



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

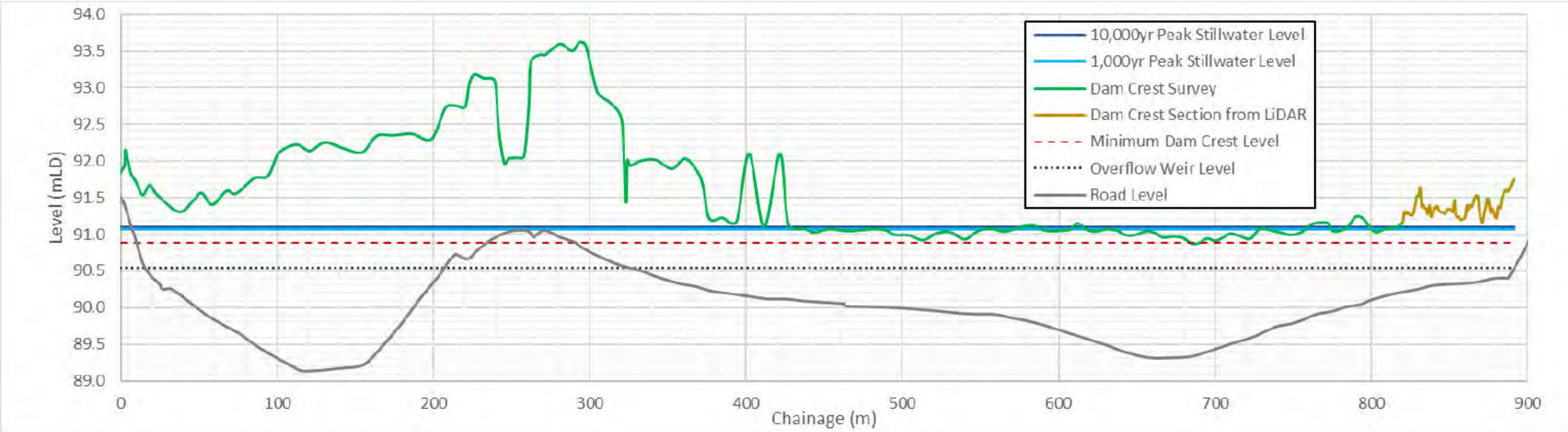
7. Dam Crest Profile and Cross Section

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 (<0.1m^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 1.63m^3/s. Peak water level in the reservoir increases by 0.01m.

The climate change flow uplift causes a more significant respons6e in the model results, with the peak pass forward flow increased by 3.37m^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 4th 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 [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 0.5m^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 6.9m^3/s, and the peak total outflow is 6.4m^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 11.2m^3/s, and the peak total outflow is 11.0m^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.

13. References

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Appendix A. Wind Wave Overtopping Calculation

Figure A1: Poynton Lake reservoir fetch length

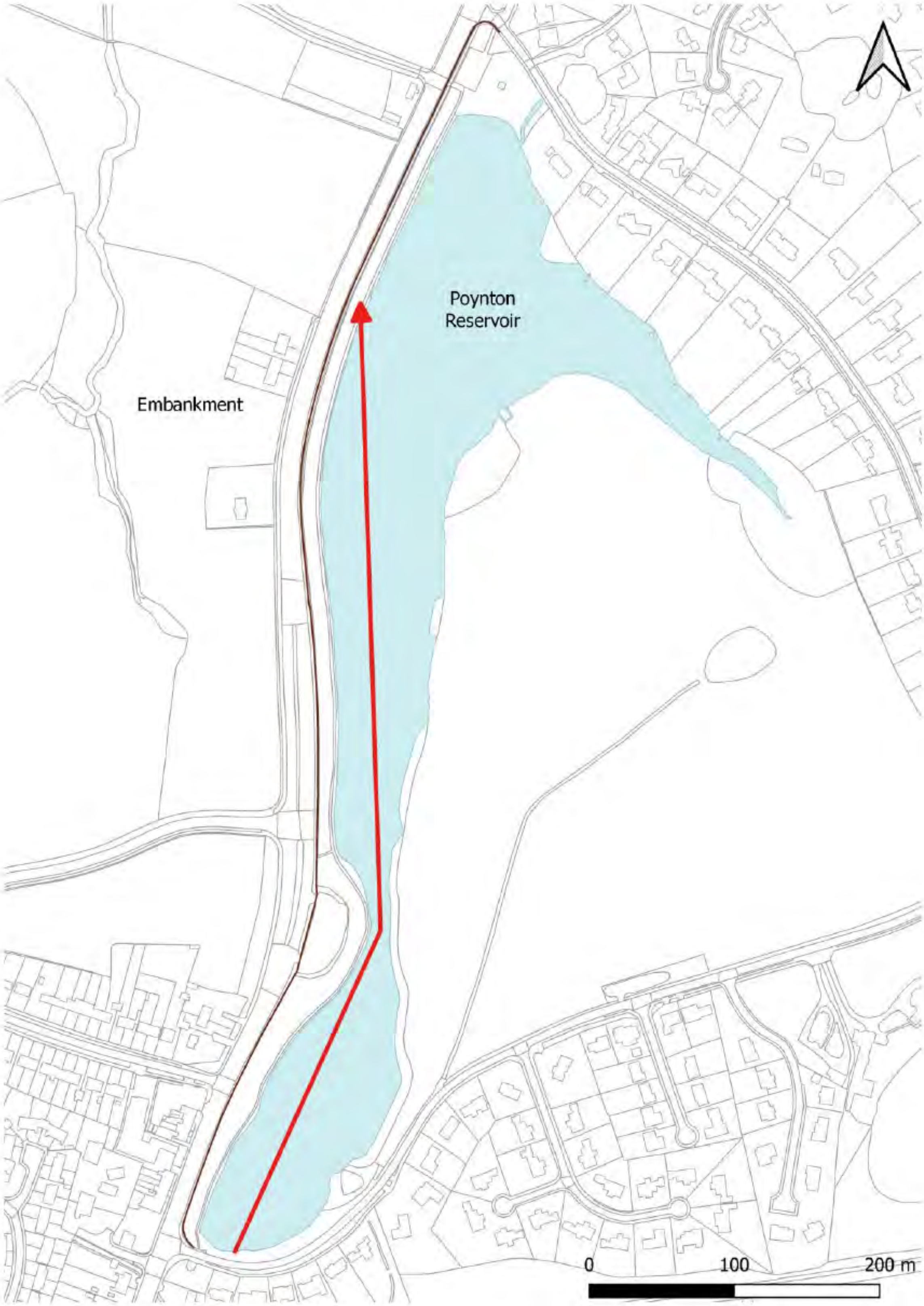

Table D1: Wave surcharge calculation data – Summer PMF Event

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 (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
H_S=	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. 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