Poynton lake

Spillway Upgrade: Initial options report

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Cheshire East Council

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Executive Summary

Poynton Lake is a 130,000m^3 reservoir retained by an 6m high embankment dam, to the north of Poynton in Cheshire. The reservoir was constructed around 1750 and is now used as a public park. The dam is unusual in that the A523 runs on a berm along the downstream face, so that the upper dam is up to 1.6m, with the dam below the road is typically only around one metre high, although it is locally around 5m high for around 30m length where it crosses a small gully.

A ten-yearly review of dam safety under Section 10 of the Reservoirs Act 1975 in May 2016 included a mandatory recommendation in the interests of safety to review the spillway capacity. This was completed in 2019 and a recommendation made in the 10(6) certificate to review the feasibility of increasing spillway capacity by 5th June 2020, and If works are proportionate then they should be designed and built by 5th December 2023. If these dates are not met, then a Reservoirs Act Section10 inspection should be called early by the Supervising Engineer.

[redacted],the road itself and houses downstream, for which the Institution of Civil Engineers recommends in Floods and Reservoir Safety 4th Ed. (ICE, 2015; FRS4) that the dam is Category B. To comply with the recommended standards, the spillway system should therefore be capable of passing a design flood of annual chance of 1 in 1,000 with no damage, and the dam itself should not fail and release the reservoir in a 10,0000 chance per year flood.

The current capacity of the spillway is around 0.8m^3/s, well short of the 1 in 1000 and 1 in 10,000 chance per year floods required for Category B dams which have values of 6.9 and 11m^3/s respectively. The current annual chance of failure is estimated as 1 in 250 per year, representing 140mm depth of overflow over the dam crest. When plotted graphically, the level of risk is therefore in the unacceptable zone where works should be carried out to reduce the risk.

To meet the recommended standard the usual engineering approach would be the design and construction of a new, larger culvert under the A road, both to reduce the overall chance of failure, and chance of flooding the road. In simple terms to pass a design flood (T1000) of 6.9m^3/s this would need to be 9 times larger than the current 450mm pipe, or around 1.4m diameter, whilst to pass the safety check flood of 11m^3/s it would need to be 1.7m diameter. If allowance was made for 35% increase in flows due to climate change this would need to be 1.6m and 2.0m diameter respectively. The estimated project cost of a pipe to pass the current T1000 is of the order of £1.3M.

However, FRS4 recommends that where an existing dam does not meet current standards, then the Owner and Panel Engineer may choose to adopt a risk based approach to assess the extent of upgrading to ensure this is proportionate. This report has considered the options in Table E.1 to increase resilience to overflow. This analysis shows that that currently regulating the crest, so any overflow is spread out uniformly is worthwhile (Option 3C Upper), whilst the other options are marginally proportionate. When allowance is made for climate change then all options are proportionate.

If the client wishes to comply fully with the engineering standard they would need to adopt Option 2 and 3C_upper combined. (£1.8M). In addition, it is recommend that 3C lower is also implemented to reduce the risk of scour damage to the [redacted], which increases the total cost to £2.1M

If the client wishes to adopt a risk based approach then the ARPE overseeing this reports considers that the minimum that would be acceptable is Option3C Upper. It is noted that an All Reservoirs Panel Engineer should be appointed to oversee the design and construction in the role of Qualified Civil Engineer (QCE). This oversight would then allow him to issue a Description of Works under the Reservoirs Act 1975 and thus avoid the need for an early Inspection under clause 6(1) (6) of SI 2013 No 1986.

The Undertaker responsible for the safety of the reservoir (Cheshire East Council) needs to decide whether to adopt a risk based or engineering standard, and thus which option they are adopting, confirm this in writing to the Supervising Engineer, and All Reservoir Panel Engineer. In simple terms the options are

- 2 and 3A – Increase pipe capacity by different amounts, no change to dam crest
- 3B – no works across road; provide emergency spillways on upper and lower embankments to reduce risk of damage to dam in floods but there would be a significant visual impact with loss of trees
- 3C upper – Increase resilience to extreme floods by regularise dam embankment crest to spread out overflow over longer length; locally raise crest to reduce risk of breach opposite vulnerable houses; no change in risk of flooding A523. Significant uncertainty in optimising works on the crest – see Table 5.5. preferred design should be developed in discussion with panel engineer and CEC Rangers
- 3C lower- emergency spillway on lower embankment reduces risk of damage to lower embankment, half of which is [redacted]

Clearly combinations of options are also possible. To assist in this decision the considerations affecting the choice of adopted option(s) are summarised in Table E.2.

The scope includes "Define scope for planning/outline design" and this will be produced once the decision on the proposed works has been provided by Cheshire East Council following consideration of the presented options.

Table E.1: Summary of economics analysis

Option	Works involved	Budget project cost (Appendix G)	Annual chance as 1 in x of (With 35% climate change in brackets – see Table: 2.2)		Cost to save a life: current (with climate change) (Note 2, 3)
			Onset of flood A523 (damage to embankment) (Note 1)	Dam failure with release of reservoir	
	Existing situation		55 (40)	250 (120)	n/a
2	Large culvert to pass Design flood (T1000) for FRS4 Flood Category B dam	£1.3M	1,100 (450)	6,000 (1,500)	6 (0.4)
3A	Additional pipe to mitigate climate change and increase pipe capacity	750k	120 (60)	500 (150)	7 (9)
3B	3B Increase resilience to overflow. Lower crest and form emergency spillway on embankment above A road, armoured to resist overflow; together with flatten downstream slope of embankment below A road, to reduce risk of catastrophic washout of supporting embankment at low point in road.	730k	as existing	1,000 (350)	3 (-0.7)
3C - Upper	Regularise dam embankment crest to spread out overflow over longer length; locally raise crest [redacted] to reduce risk of breach [redacted]	540k	as existing	10,000 (2,200)	-0.6 (-13)
3C lower	As above plus works to embankment downstream of A523	£340K	55 (40)	1,000,000 (> 1M)	N/ a see text 6.6.1

Notes.

1. Onset of damage to existing embankment and flooding of road in unlikely to lead to failure of the dam and release of the reservoir as the embankment is wide.
2. Cost becomes grossly disproportionate when cost to save a life exceeds £8.5M – see Appendix C.
3. Negative CSL is where benefits of reduced risk of £5M damage t[redacted]outweighs PV costs of scheme

Table E-2: Preliminary comparison of options to increase spillway impact (see Table E1)

Consideration	Option 2	Option 3A	Option 3B	Option 3C_upper	Option 3C_lower
	Culvert to pass T1000 flood	Smaller 0.6m pipe	Emergency spillway	Regularise crest	Flatten slope
Project Cost	£1.3M	£750k	£730k	£540k	£340
Cost to save a life £M/ life as EA 24th March 2021	6	7	3	0 (benefits outweigh costs)	see text section 6.6.1
CEC Reputation (flood risk management)	√√√	√	X	√	√√
Spillway capacity – dam failure	√√√	√√	√√	√√√	√√√
Onset of flooding A523	√√√	√	No change	√	√
Onset of damage to dam	√√√	√	√√	√√	√√
Risk of future dam safety works	√√	xx	√	√	√
Fluvial flood risk	xx	x	√√√	√√√	√√√
Heritage	√√	√√	xxx	x	
Visual impact	√√	√√	xx	x	X
Key (scale of 1 to 3): √ Beneficial, X Detrimental					

1. General

1.1 Purpose of the Report

Poynton Reservoir is located to the north of Poynton Village in Cheshire at National Grid Reference SJ 923845.

This report follows from a 10(6) certificate issued by [redacted] dated 5th December 2019, and which included the following direction 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.

And Directions to the Supervising Engineer which included

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.

This study is due to be completed by 5th June 2021 otherwise a S10 is due to be carried out.

A meeting was held with Cheshire East Council (CEC) on 9th March 2020, where the engineering and risk based standards in Floods and Reservoirs Safety (ICE, 4th edition 2015) were described and discussed, with the slides included in Appendix B.

CEC have now instructed this initial options report, to include

a) Works needed to meet the full engineering standard for a Flood Category B dam,

b) Lesser options that could be adopted if a risk based approach were adopted, if the cost to meet the engineering standard was disproportionate to the benefits in reduction in risk to life.

c) Whether reservoir safety works can be merged with a flood alleviation scheme to achieve efficiency. Refer to Appendix H.

The draft report was issued on 3rd March 2021, but the Environment Agency then provided the updated reservoir flood mapping on 23rd march, which modified (increased) the likely consequences of dam failure. A meeting was held on 13th April 2021, with a site visit to ground truth the various options on 19th May 2021, following which the report was updated and issued as final.

1.2 ALARP Assessment

The ALARP (As Low As Reasonably Practicable) approach is a tool for assessing whether the cost of the works is proportionate to the reduction in risk achieved, using quantitative risk assessment (QRA). The tests that can be used for determining whether the reduction is proportionate are described in Appendix B.

The methodology may be simplified as:

$$\text{Cost to save a life (CSL)} = \frac{\text{Cost of risk reduction works}}{\text{Reduction in “Likelihood of failure x likely loss of life”.}}$$

Where the cost to save a life is less than the "value of preventing a fatality", multiplied by a disproportionality factor in recognition that the risk of death from dam failure should be less than a vehicle accident, the cost is deemed proportionate and the works should be implemented.

For the purposes of this assessment any risk reduction measures will be considered disproportionate where the cost exceeds £8.5M per statistical life saved, being the product of CSL of £1.7M and a disproportionality factor of 5.

1.3 Available Data

Available date is summarised in the table below:

Table 1.1 – Summary of Available Data

Description	Date	Originator	Comment
Poynton Lake Reservoir – Report of an Inspection under Section 10(2) of the Act	August 2016	Mott MacDonald [redacted]	
Poynton Reservoir Flood Study Report	December 2019	Jacobs	
Certificate Under Section 10(6) as to the carrying out of safety recommendations	December 2019	Jacobs [redacted]	
BRJ10627/Topo/3d - Topographical Survey - Sheets 1 to 5	December 2019	Jacobs	Copy in Appendix A
Poynton Pool – CCTV Survey of spillway pipes	October 2019	Drain Doctor	
Utilities	No searches available		At this stage visual inspection from site visit and google
Land ownership	[redacted]		
Reservoir flood mapping	Dec 2019	Environment Agency	Summary sheet and six maps provided 24 March 2021

1.4 Topographic Survey

It should be noted that although crest levels from 0 to Ch 400 were on the high point, those from Ch 400 northwards were on the path as the remainder of the crest was too overgrown to survey. A site visit on 19th May 2021 identified that the actual crest levels from Ch 400 northwards appeared visually to be around 200 to 300mm higher than shown on Figure 2.2 and 2.3,

Similarly, the weir crest level was not shown on the survey, but the surveyors subsequently stated that it is 90.55mOD (i.e. 1.07m below the cover level of 91.62mOD). However, this seemed low relative to the quoted water level of 91.71mOD (implied flow 0.3m^3/s), especially as the survey was caried out from 23rd to 27th September 2019 when water levels would not normally be that high site.

The site visit on 19th May identified that the weir is 0.9m below the concrete cover (0.97 below manhole cover) so TWL: is actually 91.72mOD. Thus the weir level appears to be around 100mm higher than quoted by the surveyors, consistent with the zero datum on the gauge board being around 100mm below weir level (see Photo 1), and probably due to the chamfer on the sides of the weir crest – see Photos 3 and 4 in Section 2.3)

However, both of the above should not change the principles set out in this report, as the spillway capacity is governed by the pipe size when choked with head only having a modest influence. Lastly there was no detailed survey of the dam embankment below the A523, especially in the area downstream of the spillway where there are candidate works in terms of increasing resilience to overflow and/or new pipe outfalls. For this study the field survey was therefore supplemented by lidar data, but it is recommended that further survey is carried out prior to any detailed design to confirm the weir level and ground levels where works are proposed.

2. Site Characterisation

2.1 Introduction

A description of the reservoir and associated site is given in the last Section 10 Inspection report. This is summarised below, where necessary for this study of initial options for spillway upgrading.

2.2 Dam and Reservoir

The reservoir is understood to have been constructed in 1750 as an ornamental lake located within the grounds of Poynton Park. The embankment that impounds the reservoir is approximately 800m long and is orientated in a north to south direction. The crest of the embankment varies in width between 10 and 12m along most of its length widening to around 20m at its southern end. Minimum freeboard is around 0.4m, although being more generally 0.7m

The A523 road that connects Macclesfield and Stockport, occupies a berm on the downstream face of the embankment. At the upstream (eastern) side of the road there is a low masonry retaining wall that extends over much of the length of the embankment. The wall is approximately 1m high. At the northern end of the dam the wall has been discontinued and the downstream face is simply formed by a steepened slope at around 1 (V) to 2 (H) of height of up to 2m. The road varies in width from 11m at the north end to 22m, becoming two lanes with a central reservation along the southern 500m

The slope below the berm is typically only around one metre high, although it is locally around 5m high for a 30m length where it crosses a narrow valley near to the northern end of the reservoir, at the location of the spillway outfall.

The key parameters of the dam and the reservoir it retains are summarised below.

Table 2-1 Key parameters for reservoir

Feature	Units	Dimension as shown on		Source/ comment
		2016 S10	2019 Survey	
Reservoir Capacity	m^3	130,000	NA	2016 S10
Reservoir Area at TWL	m^2	68,000	N/A	2016 S10
Embankment Crest	m (AOD)	90.92	91.24 (Typical) 90.93 lowest	As shown on 2019 Topo survey, but see Note 1
Spillway weir crest	m (AOD)		90.55	Given on email from surveyor [redacted] dated 15 oct 2019 but see Section 1.4. WL on day of survey 91.71mOD
Lowest level on A road	m (AOD)	n/a	89.31	
Downstream Toe	m (AOD)		84.3 (section 5)	
Design Flood Category	B			
EA Risk Designation	High Risk			

Notes

1. Site inspection on 19th June 2021 noted that from around Ch 450 to 800 ground level in undergrowth downstream appeared to be around 200-300mm higher than levels on footpath (which is earth worn along reservoir rim- see Photo 4 In Appendix D). This is consistent with spot levels on Section 3 to 7 (see Figure 2.2) measurements

Figure 2-1 Cross section on embankment at maximum height (Section 5; existing spillway)

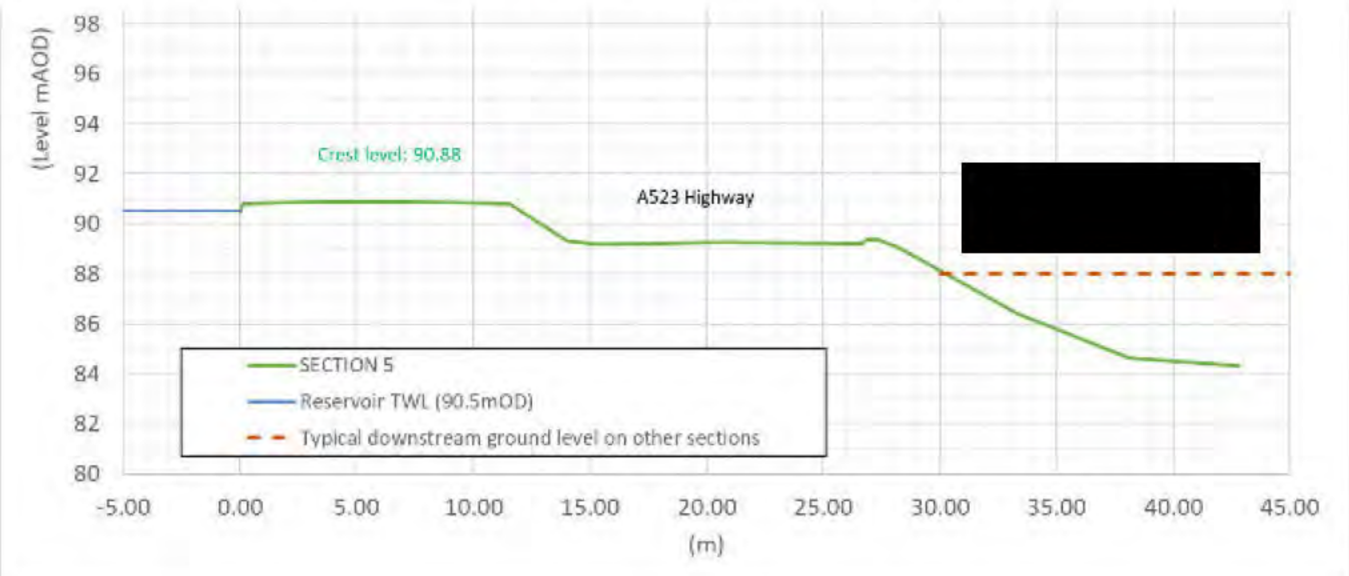


Figure 2-2 Long section on dam crest and A 523 (30 times vertical exaggeration)

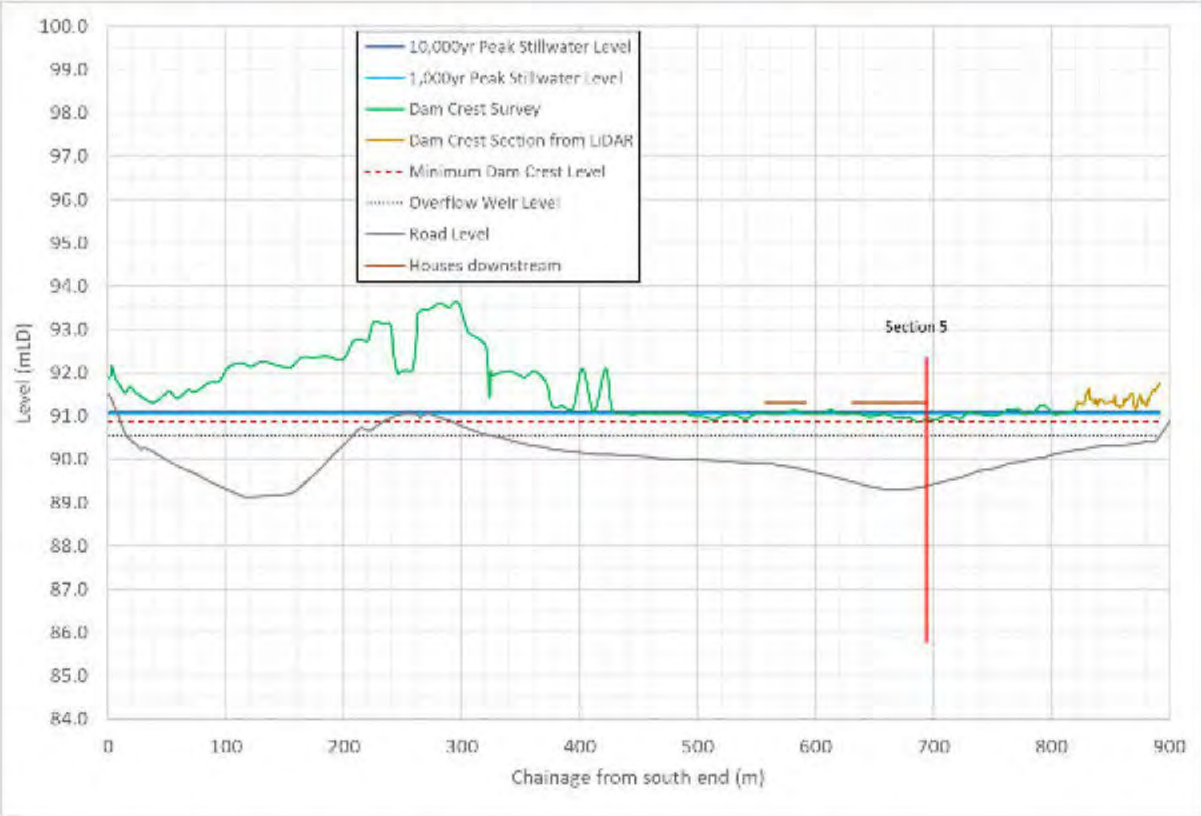
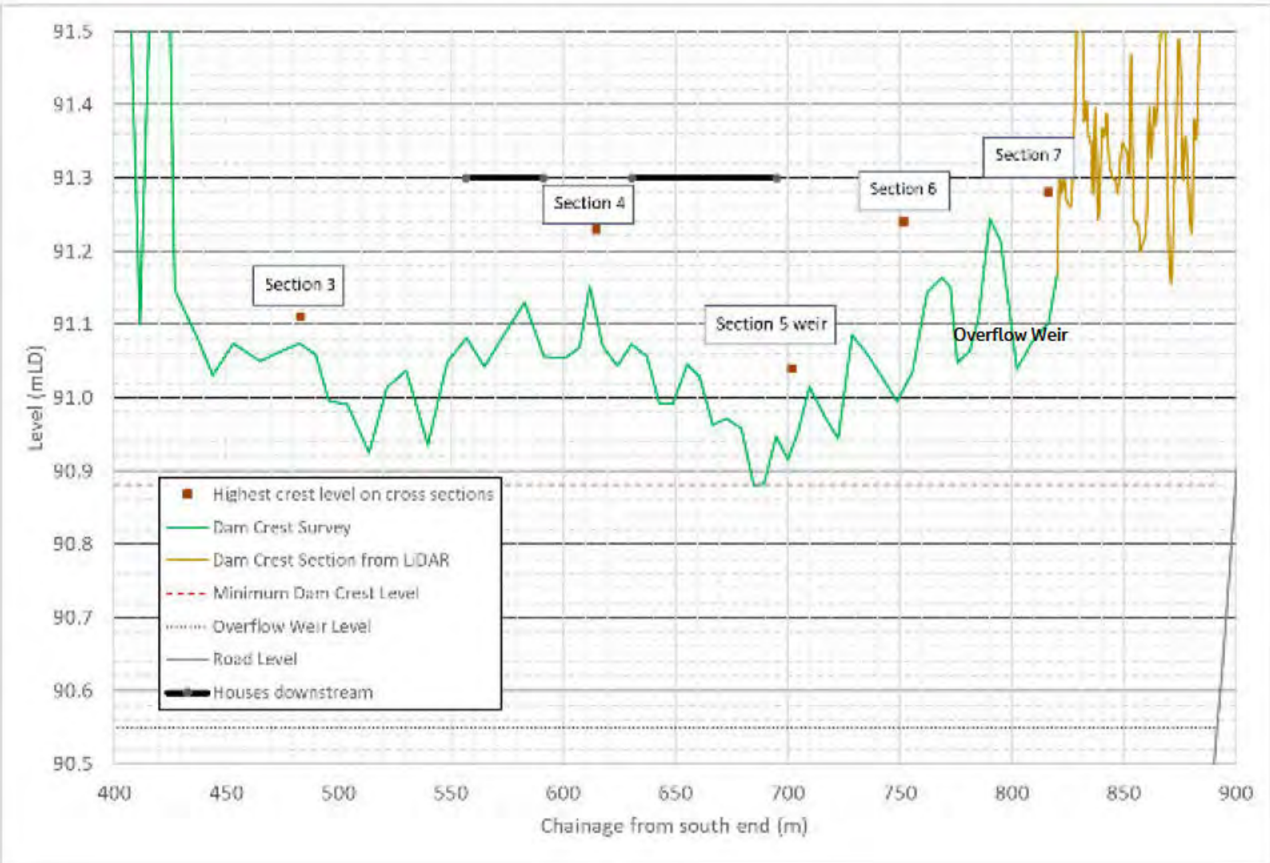


Figure 2-3 Long section on dam crest and A 523 (600 times vertical exaggeration)



2.3 Existing Spillway

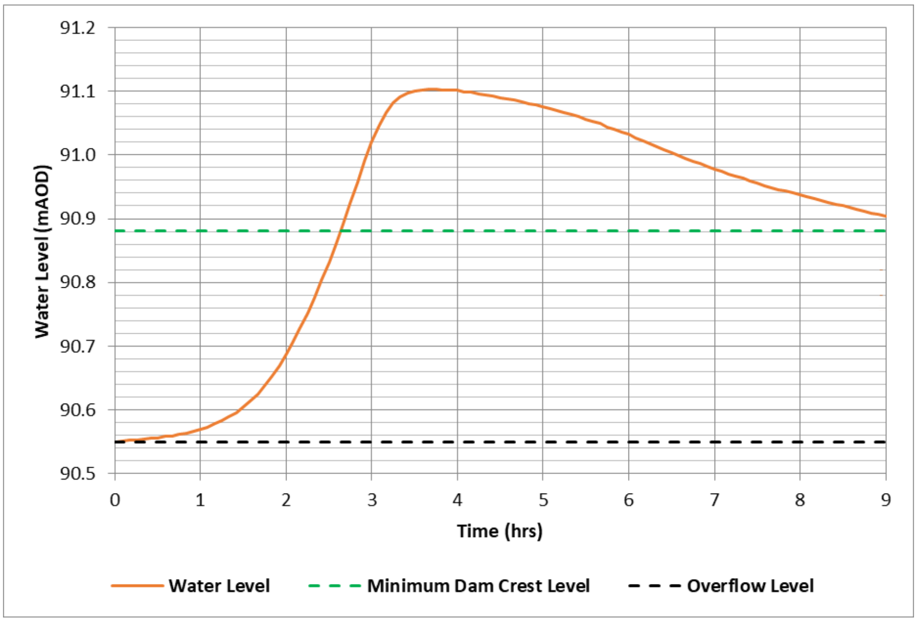
The existing overflow arrangement consists of a concrete intake structure with screens housing an overflow weir. It is located towards the northern end of the embankment. The length of the weir is 4m and it is formed in two equal sections with a dividing pier. The two screens are hinged at the top to allow them to be opened to give access to the weir.

Flow capacity of the spillway is governed by the pipes downstream of the weir, with available details on the pipes below. The downstream pipes comprise a high level 600mm diameter pipe across the main crest, a drop manhole and then a low level 450mm pipe under the A523. Screening calculations suggest that the flow capacity is governed by the 450mm pipe in MH 1 on the downstream side of the crest.

Table 2-2 Detail of pipes controlling capacity of existing spillway.

Feature	Pipe			Comment
	Día (mm)	Invert level at upstream end	Length (m) approx.	
Weir	4m crest length	90.55	n/a	
Inlet chamber to MH 1	600	89.5	12	MH 1 on dam crest
MH1 to MH2 (under A road)	450	87.03	22	MH 2 in downstream face of embankment (D/S of the A523)
MH 2 to MH3/4	MH3 -450		95	
	MH 4- 300			
MH3/ 4 to watercourse			70m to Poynton Brook	
1. Manhole details taken from Table 4 in 10(6) certificate				

Figure 2-4 0.01% AEP (10,000-year return period) stage hydrograph with critical dam structure levels (Fig 6.2 of flood study)



2.4 River and Flood Flows

2.4.1 Current

The direct catchment is 1.96km². Flood flows are given in the 2019 flood study and summarised in Figure 2.5.

There are also inflows from a 4km² indirect catchment, with the intake some 234m away with an average longitudinal gradient of 0.02%. The pipe intake is 490mm diameter and the estimated inflow is 0.3m³/s in the 1 in 10,000 per year chance flood.

2.4.2 Climate change

The scope of this study includes commenting on the resilience of selected options to climate change based on published government forecasts. These are given on https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances , with values for the North West given in Table 2.3 below. For civil engineering works the 2080's values would be appropriate, and it is suggested that **"Higher central" based on the 70th percentile would be appropriate, namely 35% increase in flow**s. This value is used in the options assessment later in the report.

Figure 2-5 Peak inflow vs annual chance (direct catchment only)

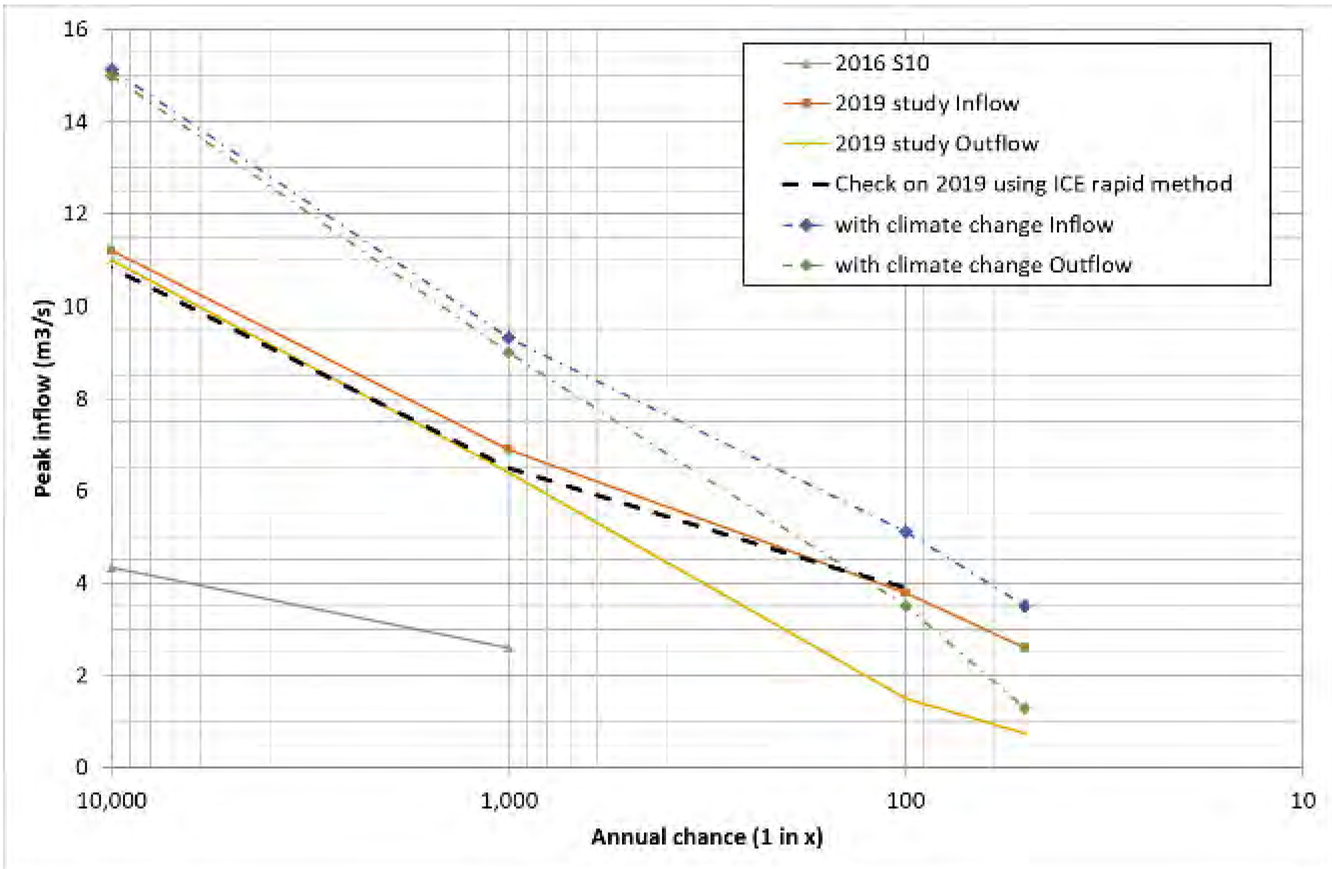


Table 2-3 Peak river flow allowances for North west river basin district (based on a 1961 to 1990 baseline) given on www.gov.uk

River basin district	Allowance category	Total potential change anticipated for the		
		'2020s' (2015 to 2039)	'2050s' (2040 to 2069)	'2080s' (2070 to 2115)
North west	H++ (extreme scenarios)	25%	45%	95%
	Upper end (90th percentile)	20%	35%	70%
	Higher central (70th percentile)	20%	30%	35%
	Central (50th percentile)	15%	25%	30%

2.5 Ground Conditions

GeoIndex shows ground conditions at the reservoir as glacial till, although with river terrace deposits present along the watercourse downstream (a side channel reaching up to [redacted]. Glaciolacustrine deposits are shown to the west in the area between the confluences between the two tributaries of Poynton brook.

Underlying Solid deposits are shown as follows

- from the spillway north MANCHESTER MARLS, Lithology is *Red marl (calcareous mudstone and siltstone) with thin beds of fossiliferous marine limestone and dolomite; locally green; sandy in places especially in top part; local breccias and pebbly beds.*
- South of the spillway: Chester Formation: Lithology described as *Northwards into the Worcester Basin, West Midlands, Staffordshire (Steel and Thompson, 1983), Cheshire and Leicestershire, the formation comprises conglomerates and reddish brown, cross-bedded, pebbly sandstones with subordinate beds of red-brown mudstone.*

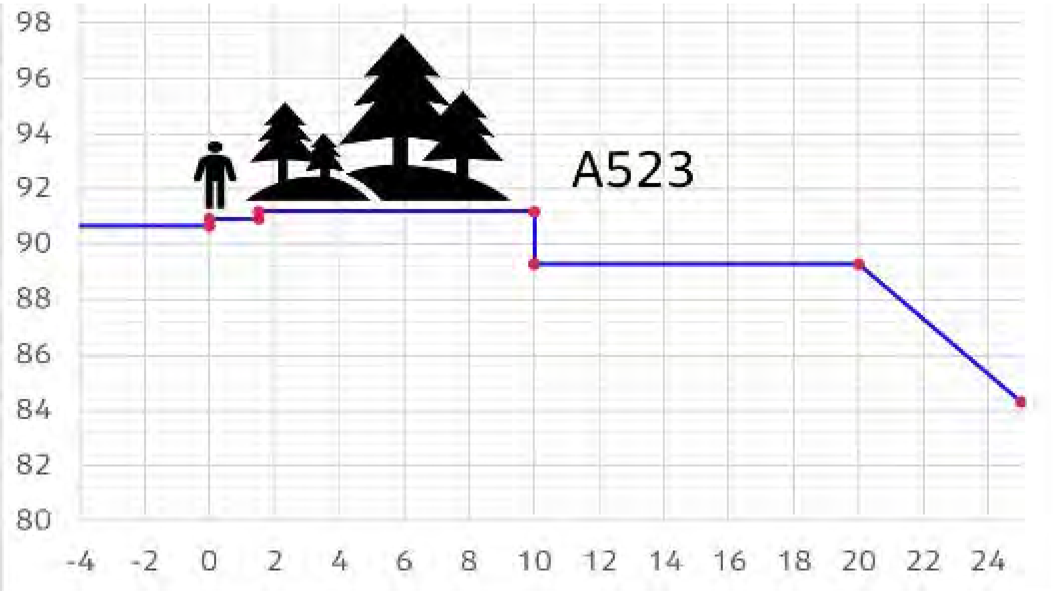
Once a preferred option has been chosen by CEC then the requirements for ground investigation can be carried out, areas that would be considered for Ground investigation for the lager option may include the Dam Crest, The A523 and the valley downstream of the low spot in the road.

2.6 Site Constraints on Upgrading Works

2.6.1 Dam Crest

This is seen by the Rangers as a "wood", with the vegetation deliberately left unmanaged and thus generally inaccessible. Use by the public has worn a earth path along the lake side, this varies in width from 1.2m to around 2m. chippings and other debris has been placed in the "wood" in places, so the ground level within the "wood" is irregular.

Figure 2-6 Schematic cross section on embankment



2.6.2 A523 on berm along downstream face

The A523 is a main road connecting Macclesfield and Stockport so it will be important that any upgrading works take this into consideration in both the long and short term. Although at the northern end where the road is straight the centreline is higher than the edges, from Ch 700 south there is a superelevation with the outer edge of the road on the west side higher than the east.

2.6.3 Utilities

Initial checks on potential utilities located in the overall vicinity of the works, particularly the A523 London road North have identified the following utilities that will be impacted by the proposed works at Poynton Reservoir. The utilities identified are as follows.

- 4 No 33kV electricity cables along centreline of A532 (Depth ~900mm)
- 4 No LV electricity cables in centre of northbound carriageway (Depth Unknown)
- 1 No LV overhead electricity cable feeding the 4 propertied and street lighting to west of road.
- United Utilities Distribution main along western verge and pavements (Assumed 25mm diameter at 750mm depth.
- Carriageway drainage on eastern side of A523.

Prior to any detailed design works a comprehensive utilities check will be required.

[redacted]

Chainage (see Figure 2.3)	[redacted]	Comment
830m	[redacted]	At extreme northern end of reservoir
620m	[redacted]	On south side of valley where dam highest and spillway outfalls. [redacted]
530m	[redacted]	
310m	[redacted]	
115m	[redacted]	
0 -115m	[redacted]	

[redacted]

[redacted]	Ground level by property threshold (mOD)	Road level (mOD) – see Notes	
[redacted]		Centreline	Gutter
[redacted]	N/A	89.26	89.23
[redacted]	88.98	89.46	89.55
[redacted]	89.14	89.54	89.70
[redacted]	89.22	89.65	89.70
[redacted]	89.30	89.72	89.80
Notes All levels taken from lidar. Accurate survey recommended prior to detailed design [redacted]			

Figure 2-7 Base of valley downstream of service spillway indicated by red dashed line

[redacted]

2.6.5 Control of reservoir level during upgrading works

There are no draw down facilities at the reservoir therefore there is no means of controlling reservoir levels during the upgrading works. The only means of control is the existing spillway arrangement. Any upgrading works will need to use cofferdams, and pumps or siphons to control inflows.

2.6.6 Time to refill reservoir after any lowering

The flow gauging station at Micker Brook in Cheadale, on the stream from Poynton does not include any flow data. The nearest gauging station with daily flow data is therefore Station 69008 on the Dean at Stanneylands, which has a Q50 of 0.54m^3/s. When this is reduced in proportion to catchment area (4 / 58.9 = 0.068) this gives an indicative Q50 inflow of 37 l/s into Poynton Reservoir. If the reservoir was lowered by one metre with average inflows it would take 3 weeks to refill, whilst in a dry period with Q95 inflows it would take three months to refill.

3. Probability of damage and failure of dam due to floods

3.1 Introduction

This has been evaluated by carrying out an approach similar to that in a Tier 2 risk assessment, with the approach used to estimate annual probability of failure described in this section.

3.2 Frequency and Magnitude of Overflow from Dec 2019 Flood Study

The key findings from the flood study, relevant to the probability of failure due to overflow are

Table 3-1 Key findings from 2019 flood study

Aspect	Shown in flood study	Comments
Embankment crest levels	Figure 7.1 Min level 90.88 at Ch 685	• levels taken along path north of Section 3 (Ch 450) so unlikely to be fully representative of ground level in overgrown area downstream. • sensitivity of raising level by 0.2m (Section 8) gave small reduction in outflow.
Duration and depth of overflow	T1000 Figure 6.4	0.19m above lowest point (peak 91.07mOD), for 6 hours Note that overflow only over 68m for 1 in 100 flood (Table 6.4).
	T10,000 Figure 6.2	Peak 91.103mOD (0.223m depth of overflow)
	T10000 sensitivity Table 8.1	Peak WL 91.298 mOD for 0.2m higher crest (0.218m depth of overflow).
Inflows and outflows	Table 6.4	Existing spillway capacity only 0.8 m^3/s, compared to peak inflow in 100 year of 3.8 m^3/s.

3.3 Potential Failure Modes

Understanding the potential failure modes when overflow occurs over the dam is necessary to be able to understand and quantify the annual probability of failure.

In a detailed risk assessment this would include event trees to evaluate the probability of failure modes progressing to failure. As the crest width is around 10m, as well as the depth of overflow the other key issues are the duration of overflow and erodibility of the dam fill. There is currently no guidance on the latter, so one approach could be to carry out a sensitivity study of detailed dambreak analyses using proprietary software. However, this would require accurate representation of the issues in Table 3.2, so would be to some extent a research exercise. Another option would be a workshop of key stakeholders to agree an event tree describing the failure process, and probability of each step.

The approach adopted for this report has been to consider a credible sequence of events leading to release of the reservoir as shown in Tables 3.3 and 3.4.

Modes of failure relating to internal erosion, for example along the outside of the spillway pipe, have been neglected as being less likely, due to the modest hydraulic gradient, and the drop manhole on the downstream side of the crest forming a barrier to continuous seepage along one pipe.

Table 3-2 Information required for detailed assessment of breach development

Data required	Comment
Inflow hydrograph	Could use output from 2019 flood study e.g. Figures 6.2 and 6.4.
Erodibility of embankment fill	a) No information in last S10. b) Would need ground investigation. c) Assuming that the dam was formed of material excavated from the reservoir basin, this would be expected to be largely clayey but possibly with zones of sand / gravels. d) Likely to be variable over 800m length.
Erodibility of tarmac layers (road, footpath)	Currently there is no guidance on how to model these in dambreak software.

Table 3-3 Key steps in progression of failure for this screening study, for overflow of upper dam

	Step in failure process	Comment
1	Inflows exceed spillway pipe capacity and water starts to overflow lowest point on dam crest i.e. 90.88mOD	Onset of flooding of A523 Figure 3.1 suggests that this would occur on average around once in 55 years (1.8% annual probability).
2	Local erosion at downstream face of dam above A road	Initiation of damage but would not cause release of the reservoir.
3	Erosion gully cut back to perimeter footpath	
4	Erosion gully widens and creates a constant slope from footpath to road pavements	
5	Erosion undermines footpath, leading to collapse	Will depend on flow concentration on face, and variability/ vulnerability of crest path to overflow.
6	Erosion cuts back to reservoir, leading to uncontrolled release of reservoir	
Notes: 1. Clearly actual performance may vary from above, for example due to effects of tree roots, preferential erosion along services crossing the crest etc, but this is considered a credible sequence leading to release of the reservoir.		

Table 3-4 Key steps in progression of failure for this screening study, for overflow of embankment below A523

	Step in failure process	Comment
1	Inflows exceed spillway pipe capacity, water overflows onto road, spills longitudinally, collects next to houses and starts spilling over downstream bank	Likelihood as for upper dam.
2	Local erosion of downstream face of dam above A road	Initiation of damage but would not cause release of the reservoir.
3	Erosion cuts through grass cover and erodes around manhole/ pipe from spillway	
5	Spillway pipe fractures: water accelerates erosion of slope	
4	Combined effect of overflow and fractured spillway pipe lead to widening of erosion gully and undermines A523	
5	Erosion extends back under A523 with swallow hole opening up on upstream side of A523	
6	Erosion cuts back to reservoir, leading to uncontrolled release of reservoir	

3.4 Current Probability of Failure Due to Overflow

3.4.1 Upper embankment

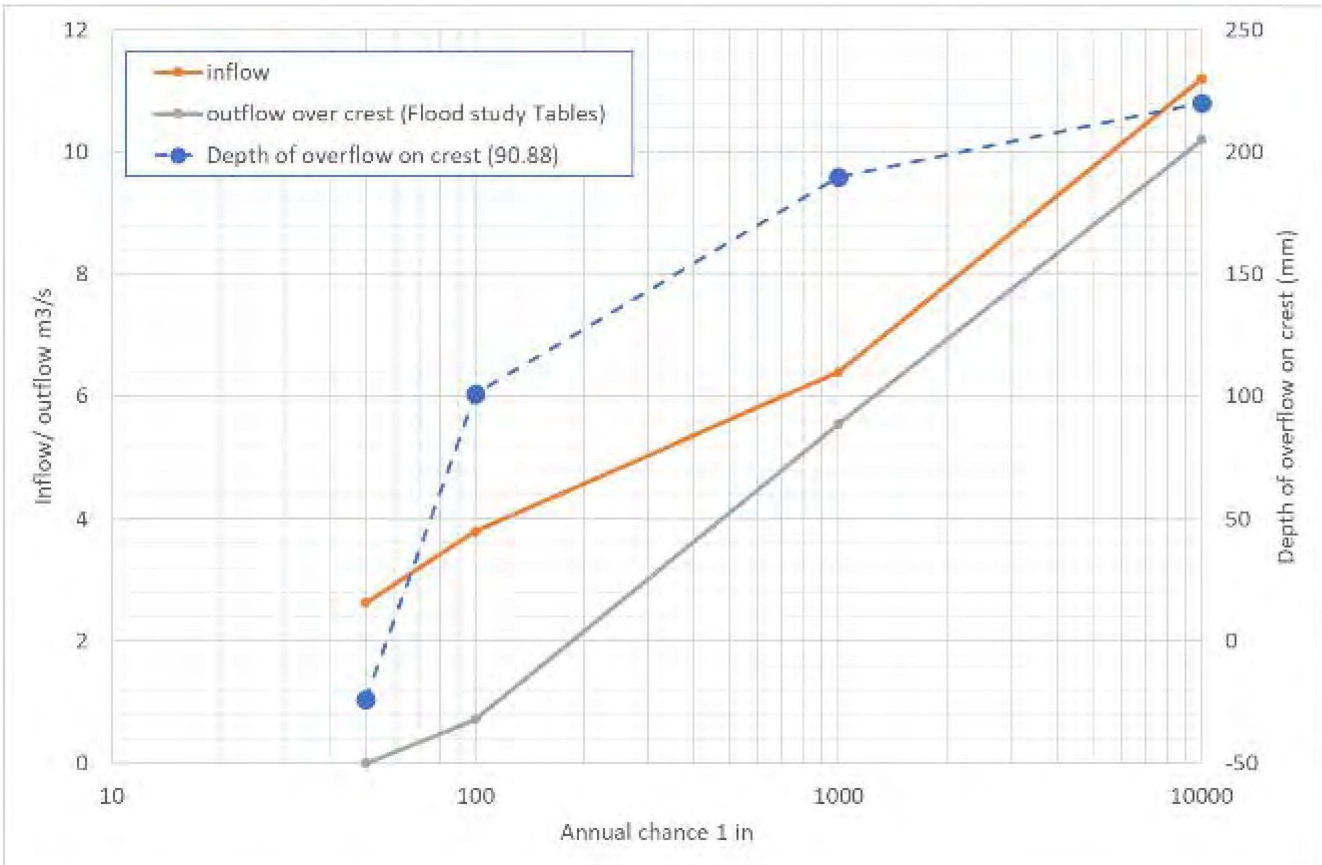
For this screening study the probability of failure of upper dam is taken as steps 3-4 of Table 3.3, when the upper dam has eroded back to the path and velocities down an assumed slope exceed allowable velocities for plain grass. This is quantified by making the further simplifying assumptions

a) Allowable velocity for duration of whole inflow hydrograph.

b) velocity on downstream face to progress to breach is 1.25 times the allowable velocity for plain grass as given in Figure 6.3 of FRS4.

From Fig 6.3 of FRS 4 for a 6 hour duration storm (see Figure 2.4) on plain grass with poor cover, the allowable velocity is 1.8m/s; equivalent to 100mm depth of overflow. 1 .25 times this is 2.25m/s, equivalent to 140mm overflow on the crest. The estimated depth of overflow in its current condition is shown in Figure 3.1, and for 140mm depth of overflow taken as the failure condition the current annual chance of failure is taken as 1 in 250 chance per year. This is consistent with the age of the reservoir of around 250 years, and no reported failure. Figure 3.1 also shows that the associated overflow of the crest at failure is around $3m^3/s$.

Figure 3-1 Depth of overflow vs annual chance as shown in Jacobs 2019 flood study



3.4.2 Embankment supporting A523

One approach which could be used is as for the upper embankment, where the breach probability is taken as the flow which gives a multiplier of the allowable velocity on grass. Here as the grass is intact, and significant further erosion is needed for erosion to progress to upstream of the A523, if the critical velocity for release of the reservoir is taken as twice the allowable velocity for average grass i.e. 5 m/s. For a 2H;1V slope this corresponds to a depth on the crest (weir) of 0.34m, and unit discharge of 0.3 $m^3/s/m$. Spreading this out over 10m gives a total breach discharge of 3 m^3/s, which is the same as the critical flow estimated for the upper dam. In this case the probability of failure of the upper dam could be used for the ALARP calculations.

If the resilience of the upper dam were increased, but not the lower embankment, then an alternative approach would be to consider the depth of overflow on the road necessary to cause breach. The Environment agency speciation for reservoir flood mapping suggest that this could be around 1m (Table 13 of specification, road > 4m wide), giving a water level on the road of 90.3mOD. However this would give deep flooding [redacted] so a depth of 0.5m above the gutter level is used, namely 90.05mOD. if an effective crest length of 60m were used (preliminary approximation based on Figure .3.3) then the overflow at failure would be 54 m^3/s. This is significantly higher than the PMF, so a annual probability of 1 in a million is adopted for the economic analyses. Clearly there would be flooding of [redacted] and damage to the bank downstream of the road, including the [redacted]

3.5 Options to Reduce Probability of Failure Due to Overflow (Hydraulic Dimensions)

3.5.1 Introduction

In broad terms there are three groups of options to reduce the probability of failure, as shown below. For this report they have been considered as separate options, but combinations could be selected. The text in the following sections describes the hydraulic dimensions that could be adopted, and the reduction in probability of failure that would be achieved i.e. how the numbers in the table have been calculated. The engineering options and implications are discussed in Chapter 5.

Table 3-5 Potential options to reduce probability of failure

Type of options to reduce probability of failure	**Options considered for this study**			**Total outflow m^3/s (Annual chance as 1 in: current/ with 35% Climate change)**	
	Option no.	Description	Comment	Flood A523	Dam failure
Outflow required in order to comply with engineering standards				6.9 (1,000)	11 (10,000)
Existing situation				0.8 (55/ 40)	3.8 (250/ 120) Note 3
Increase pipe sizes	2	Large enough to pass the current design flood (1 in 1000)	Would need to be larger if to allow for climate change and/or safety check flood	7 (1,200 /450)	10 (6,000/ 1,500)
	3A	3A – Additional 600mm pipe		1.9 (120/60)	4.9 (1 in 500/ 150)
Increase resilience to overflow	3B	Create preferential flow paths over embankment		as Existing	6.6 (1,000/350)
Regularise crest level (upper embankment)	3C	Regularise crest level, to spread out overflow		1.0 (70/50; Note 2)	11 (10,000/ 2,200)
2+3C	Combination of options as described above			7 (1,200/ 450)	17 (80,000/ 20,000)
3A + 3C	Combination of options described above			3 (200/ 85)	12 (15,000/ 3000)

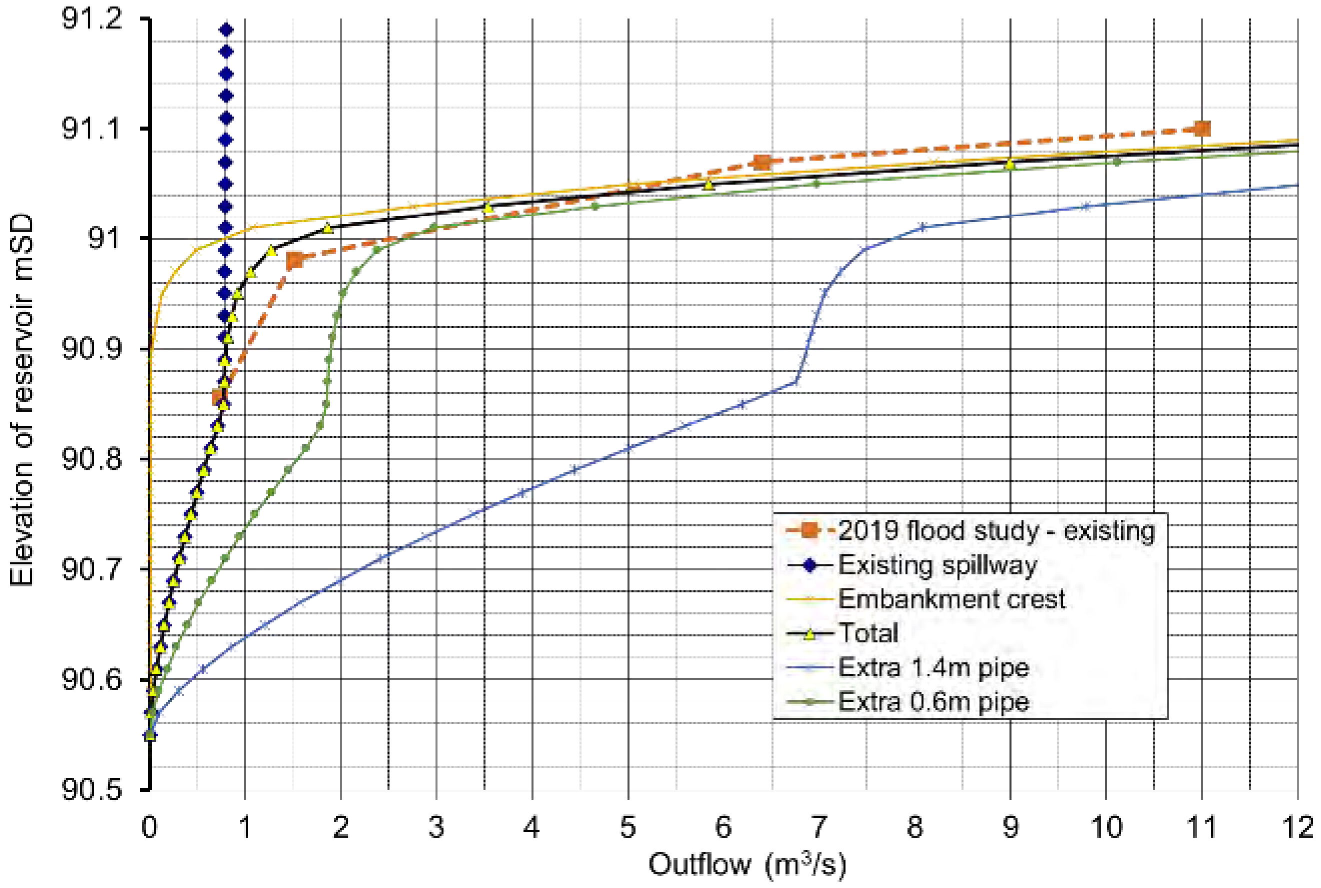
Notes

1. AEP of damage and failure for options 3B and 3C is given in Table 3.7.
2. Nominal amount less than current, as infilling low spots will give slightly higher flow in pipe, and if 10mm above 150m long path were considered as onset of damage this would be equivalent to about $0.2m^3/s$.
3. Existing AEP of failure of lower embankment and erosion across A road, leading to catastrophic release of reservoir taken as 1 in a million – see Section 3.4.2.

3.5.2 Options 2, 3A : Increase size of spillway pipes under A523

The relationship of outflow to reservoir level is shown in Figure 3.2. The thresholds for flooding of the A523 and dam failure are the same as for the current arrangements, namely when the reservoir level reaches 90.88 and 91.02mOD respectively. The outflow which would cause onset of flooding of the A523 has been determined from Figure 3-2 as the flow correlating with a water level of 90.88mOD, whilst the overflow to cause failure is taken from Figure 3.1. Figure 2.5 is used to infer the annual probability. The flows and probabilities derived in this way are presented in Table 3-5.

Figure 3-2 Outflow vs reservoir level for different pipe sizes



3.5.3 Option 3B : Provide Emergency Spillways

3.5.3.1 Introduction

The second group of options comprise works to make the embankment more resilient to overflow, by constructing emergency spillways on both the upper and lower parts of the downstream face (above and below the A523). Their purpose is to increase the quantity of overflow before the dam breaches, from the current estimate of around $3m^3/s$. However, the opportunity for an emergency spillway is significantly limited by the A523, as this means two emergency spillways would be needed one on the upper dam and a second on the dam below the A523.

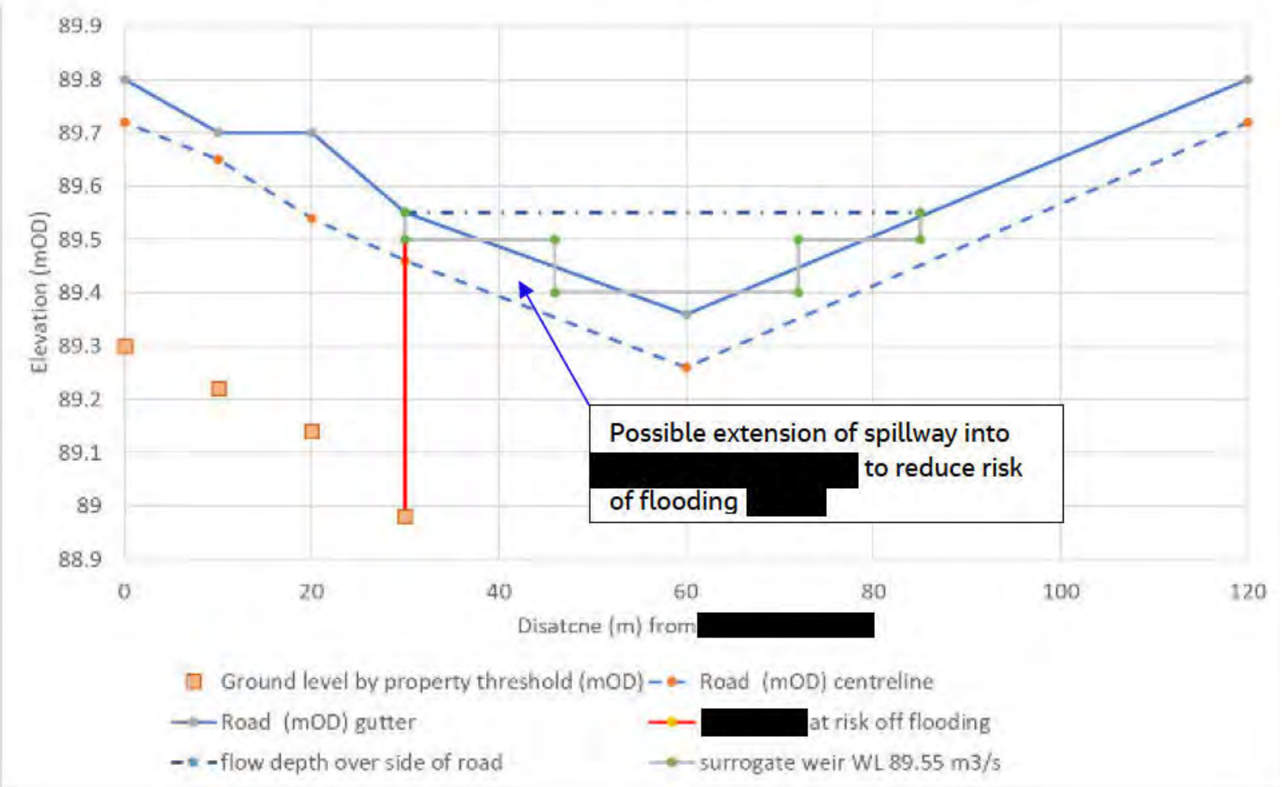
The scenarios considered for this report are summarised in Table 3.7 and discussed in the following text.

3.5.3.2 Lower embankment emergency spillway (below A532)

As the level of A523 is not constant but falls to a low spot at Ch 690 (Figure 2.2) the lower emergency spillway would need to be sited at Ch 690m.

However, the head that is available below th[redacted]s limited as described in Section 2.6.3, and shown graphically on Figure 3.3, being a maximum of 200mm flow depth in order to keep the width of overflow from impacting on [redacted] Using the simplified surrogate weir section on Figure 3.3 this is equivalent to a total flow of around 2.8 m^3/s. Thus, it is assumed that an emergency spillway of this capacity would be provided on the upper dam, and the lower dam strengthened to not fail before the upper (sum of emergency spillway and crest breach flow). Assuming that the downstream slope was flattened to 4H:1V and flow was at least 10m wide then the terminal velocity on the downstream slope would be around 4 m/s, which would require open mat reinforcement of the grass to withstand 6 hours overflow. Alternatively, it could be flattened to say 6H:1V when good grass may be sufficient, depending on the design criteria. However, for the purposes of this study it will be assumed that a 4H:1V slope is used, to shorten the length for any pipe options

Figure 3-3 Sketch long section of key levels on west side of A523, at possible lower emergency spillway



3.5.3.3 *Upper embankment emergency spillway (above A532)*

For the two design cases considered above, then the weir length and crest levels could be as follows. At detailed design the exact combination would be selected to achieve the best compromise of minimising damage to trees whilst minimising the length of crest path that needs raising. For this report a 35m length weir is adopted, to avoid works to avoid flooding [redacted]

Table 3-6 Preliminary dimensions for emergency spillway on upper embankment (Option 3B)

Parameter	**Units**	**Value, to match capacity of lower embankment spillway – see Section 3.5.3.2**		**Justification/ comment**
		No flooding of houses on London Road	[redacted] - **see 3.5.4.2**	
Design flow	m^3/s	3	11	
[redacted]	mm	Not required	200	
Crest level	mOD	90.9	90.9	Current lowest crest level
Level of remainder of crest	mOD	91.05	91.05	Minimise crest raising.
Weir length required to pass design flow	m	35	140	Plain grass/ average cover should be sufficient to resist scour under overflow.

Table 3-7 Scenarios considered to improve resilience of embankment to overflow (reinforced spillway in bold)

Scenario	Upper spillway: Embankment above A523				Lower Embankment below A523			
	Slope/ width	Overflow (m^3/s) for		Source/ comment	Slope	Overflow (m^3/s) for (Note 1)		source/ comment
		No damage	Breach			No damage	breach	
Existing	Existing variable, includes walls	Nil overflow	3	See section 3.4.1	Existing 2H:1V	Nil overflow	3	See Section 3.4.2
3B – emergency spillway	**4H:1V spillway 35m width as Table 3.6**	**3**	**6**	**Assume breach is as existing plus spillway; would be increased if regularise crest. No damage flow limited to avoid [redacted] on [redacted]**	**4H:1V open mat grass reinforced spillway**	**2.8**	**Say 1.25 times allowable i.e. velocity of 6.5 m/s, which gives around 11 m^3/s**	**see Section 3.3.2**
3C - Increase Resilience to Overflow	As existing	Nil overflow	11	See Section 3.5.4	**as above. See Note 3**			

Notes

1. These flows on the lower embankment assume that side bunds to contain flow are present to above the estimated flow depth and are reinforced to the same standards as the main downstream face.
2. Total outflow is overflow plus flow in spillway pipe (around $0.8m^3/s$ in existing situation).
3. For the purposes of this study it is assumed that the lower embankment is flattened to a slope of 4H;1V, and reinforced against overflow with concrete blocks, to reduce the risk of damage from livestock, which is likely to give a higher breach flow than shown above depending on the quality of construction and maintenance of the concrete blocks. However, at overflow of more than $2.8m^3/s$ water is likely to spill over the road into the [redacted] (see Section 3.5.3.2); at this stage no allowance is made to mitigate this as for Options 3B and 3C the annual probability is less than 1 in 1000.

3.5.4 Option 3C:Increase Resilience to Overflow

3.5.4.1 Upper embankment

The current crest level is irregular with the profile adopted for the purposes of the 2019 flood study shown in Figure 2.3. Attention is drawn to the comment in Section 1.3 that the crest levels from Ch 400 north were on the crest path, and that the site visit in June suggests the ground level in the undergrowth to the west of the path is higher by around 200 to 300mm.

There are two reasons that the crest could be modified

a) regularised to spread out overflow uniformly along the crest, and thus be able to tolerate a larger overflow before breach occurs
b) locally raised so that overflow is directed away from [redacted] (A523)

For the purposes of this study the following assumptions will be made in defining this option used to estimate the probability of failure. It is assumed that the low section of crest would be capable of resisting overflow, but that the higher section [redacted] would be a simple earth bund on the downstream side of the path.

The physical works required to deliver these hydraulic requirements are described in Section 5.6

Table 3-8 Preliminary dimensions of works to upper embankment for Option 3C (crest regularisation)

Parameter		Value used in analysis	Justification
Minimum crest level	Crest level	91.10 mOD	see section 1.4. actual value would be selected once reliable survey of ground levels in woodland available.
	Tolerance	±6mm	As SHW series 1100 clause 1101. 5
	Chainage/ length	Ch 420 to 820 less below	Ch 0 to 400 already above level [redacted]
Opposite houses	Minimum crest level	0.3m higher	Flood rise in 1 in 10,000 flood.
	Chainage/ length	2 lengths of 40m (Ref Table 2-4)	
Effective overflow length at minimum level		150m	Assume that crest locally high in undergrowth over around 50% of length.
Incipient breach flow		$11m^3/s$	Uniform overflow of 140mm so whole length at minimum level at imminent failure.
Notes This option assumes works only to crest path. Options with some form of armoured overflow path are considered under Option 3B.			

3.5.4.2 Implications for Lower embankment

For Option 3C at breach up to say 11 m^3/s overflow could occur over the upper dam. This would result in,

a) a flood level on the road of say 89.75mOD, which is around 200mm above the road outside [redacted]), and at the threshold of the next [redacted]
b) velocities of around 7m/s on a 4H;1V (flattened) slope which means to avoid damage the grass surface would need to be reinforced with reinforced grass concrete blocks, such as Dycel or Armourflex.

The proposed approach is set out in table 3.7.

3.5.5 Combination of options

In principle combinations of increasing the size/ number of pipes from the spillway could be combined with options which increase the resilience of the embankment to overflow. The effect of this on probability of failure is included in Table 3.5.

4. Consequences and existing risk of failure

4.1 Introduction

The consequences are relevant to both assigning the flood category, and economic calculations if an ALARP assessment was adopted. Preliminary comments on the likely consequences of failure were given in Section 5 of the 10(6) certificate and these are built upon here. A rapid dambreak analysis has been carried out using the method in RARS and is described below, with supporting information in Appendix D. Subsequent to the draft of this report the Environment Agency provided inundation extent reservoir flood maps produced to the 2016 spec, namely separate and a composite map for each of two breach locations, for each of dry and wet days scenarios. The original independent screening assessment ash been retained, but the EA analysis added for completeness.

This section concludes by assessing the existing risk, and its tolerability, using the current probability of failure given in the previous section.

4.2 Description of the Downstream Valley

The downstream conditions, as evident from an on-line search, are summarised in Table 4.1 and Figure 4.1.

Table 4-1: Description of valley downstream of the dam

Distance d/s (km) approx.	Feature at start of reach	Remarks	[redacted]	
			River flooding	Dam break
0	[redacted]		0	[redacted]
0.1	Confluence with river		n/a	N/A
0.5	A555	Flow in deeply incised channel.	0	[redacted]
0.8	Woodford Road		0	
3.5	Bramhall Green (and A5102 and A5143)	[redacted]		as river
5.3	Railway	In viaduct so unlikely to significantly attenuate flows.	0	0
6.8	Demmings Road/Queens Road	River culverted under factory. Flooding downstream of industrial estate and two residential closes.	0	[redacted]
7.5	Brookfields Park		0 [redacted]	Incl with Demmings Road
7.8	A5149 Wilmslow Road			as river
8.4	A560 Gatley Road		0	[redacted]
9.0	River Mersey flood plain	Passes under another railway viaduct and M60 motorway.	0	
		Total	[redacted]	

[redacted]

4.3 Breach Flow

The breach flow at Poynton is complicated by the wide berm (road) along the downstream face, and variable ground level at the downstream toe. Three different scenarios are summarised in Table 4.2. It is considered that the 10m wide tarmac road on the downstream face means that breach due to overflow would occur in two stages, first down to the road, and then to downstream ground level. It can be seen that this refinement means that peak breach flow is likely to be significantly less than the value used to produce the maps on the internet.

Table 4-2 Wet day Breach flow scenarios

	Scenario	Ht (m)	Reservoir volume (m^3)	Peak breach flow (m^3/s)	Comments
B1	Breach down to A road	1.6	70,000	30 (Note 1)	
B2	Breach from A road to lowest downstream ground level	4.3	84,000	109 (Note 1)	
B3	Used in 2016 EA flood risk mapping (national screening)	3.07	176,000	101 (time to peak 24 minutes)	See Note 2.

Notes.

1. Peak breach flow from Froelich. This is similar to peak flow from Xu and Zhang breach flow. 2009, Equation 20 if dam assumed to be high erodibility.
2. 2016 spec assumed reservoir level 0.67m above dam crest and have assumed failure to full dam height in one stage.
3. Beak flows due to sunny day failure (e.g. piping) could be higher if the breach initiated within the part of the dam below the A road.

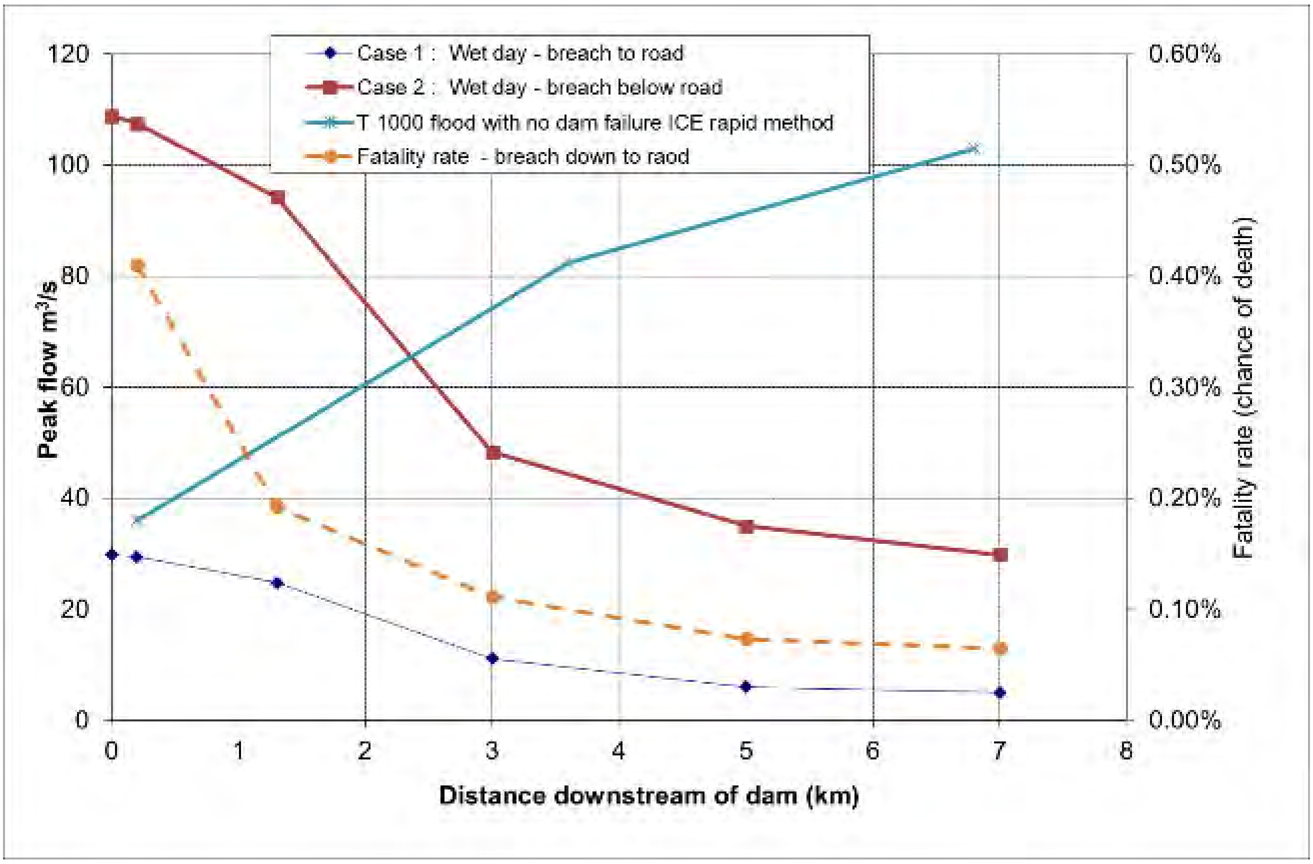
4.4 Attenuation of Breach Flows Down Valley

The other important factor in evaluating the potential impact of dam failure on a wet (relevant to spillway capacity) is the flood that would be happening anyway even with no dam failure, and this is summarised in Table 4.3. and Figure 4.2.

Table 4-3 Screening estimate of T1000 floods with no dam failure at different points in downstream valley

Location Watercourse	position	NGR	Distance downstream (km)	Catchment area A Area (km2)	source	T 1000 flood Hydrological estimate	pro-rata to Poynton res	ICE rapid method	Comment
Park Lane Stream	Intake to Poynton reservoir	SJ 922 838		4	Jacobs, 2019		14	8	
Poynton reservoir	reservoir		0	1.96	Jacobs, 2019	6.9			Jacobs 2019 table 4.2, 6.3
Poynton brook	confluence with outflow from reservoir		0.2	17.1	Note 1		60	36	
Lady brook	Barnhall Green		3.6	39.1	Note 1		138	82	[redacted]
Micker brook	Dennings road		6.8	48.9	Note 1		172	103	ICE is 0.3 PMF

Figure 4-2 Screening estimate of flood and breach flows downstream of the dam



4.5 Flood Mapping and [redacted]

The extent of additional flooding downstream due to dam failure is difficult to predict, as it will depend partly on timing of the dam failure compared to the timing of the peak natural flows with no dam failure. [redacted] are summarised in Table 4.1 and shown on Figures in Appendix D.

A screening estimate has been made using the screening methods in RARS, plus with valley sections from lidar with key data and the calculations shown in Appendix E and summarised in Table 4.4. In the final version of the report the updated Environment Agency values are also shown.

For the purposes of the economics ALARP assessment the base case likely loss of life for the upper dam will be assumed to be as given by the Environment Agency, namely 1.04. However **0.6 will be used for the lower dam as the proportion of warning to no warning; as the Jacobs screening** on the basis that before the lower part of the dam below the A road failed the warning would have been raised and people evacuated from downstream.

Table 4-4 Screening estimate of risk to life (wet day)

Source	Scenario	[redacted]	Time averaged population at risk	Likely loss of life		Source comment
				No warning	with warning	
Flood maps on internet (2009 spec)	River flooding	[redacted]				Table 4.1
Environment Agency 2016 dambreak	Dry day		184	0.12		
	Wet day		2246	1.97		
	Incremental wet day		1306	1.04		
Jacobs rapid dambreak (Note 2)	**Incremental damage in wet day failure (T1000 flooding downstream) (see Note 4)**					
B1	Breach down to A road	[redacted]	75	0.13	0.09	
B2	Breach from A road to lowest downstream ground level	[redacted]	207	0.54	0.31	As above but add [redacted]

Notes

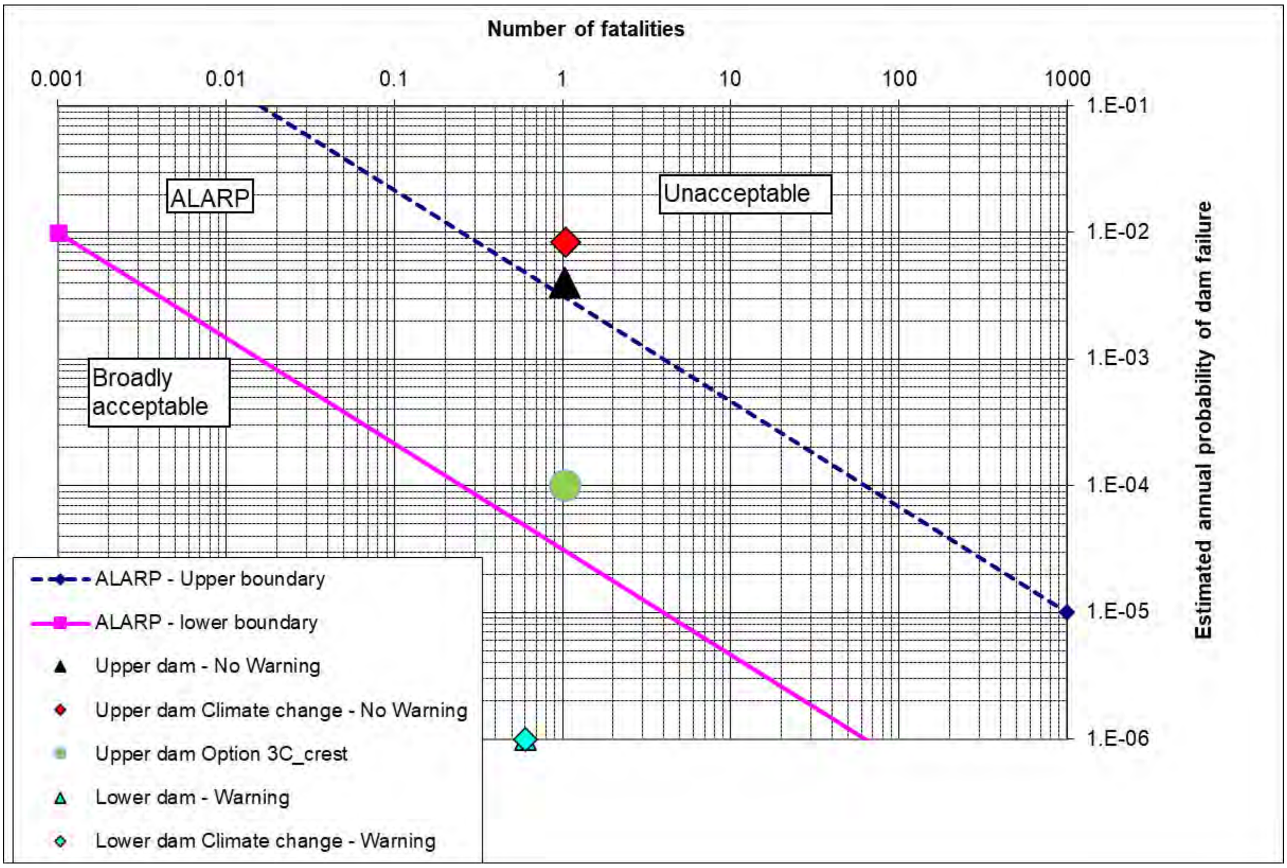
1. Only includes [redacted]s beyond that breach flow much less than T1000 flows.
2. Calculations only indicative and given in Appendix E.

4.6 Tolerability of Risk (Guide to drawdown section 8.3.3)

The estimated LLOL and index probabilities of dam failure were plotted on an FN chart, see Figure 3.2. This indicates that the current risk for the upper dam lies within the unacceptable zone.

Thus some works are necessary. Option 3C upper would reduce risk into the ALARP zone; which is the range where individuals and society are willing to live with the risks so as to secure certain benefits, provided that they are confident that they are being properly managed, and that they are being kept under review and reduced still further if and as practicable. Within this zone, HSE guidance is to implement mitigation options where the reduction in risk is proportionate to the costs. HSE guidance is similar if the current risk was in the 'Broadly acceptable' zone.

Figure 4-3 FN Chart plot of societal risk



4.7 Review of Flood Category as Defined in Floods and Reservoir Safety (ICE, 2015)

It is considered that the dam should remain a flood Category B, as

a) [redacted] a would be inundated in event of a breach of the dam [redacted]

b) the A road would justify Category B, as the FRS4 sates Category B applies "*to the severing of main road… or other critical infostructure such as gas mains and transformers*"

Although up to [redacted] could be affected in the valley downstream (Table 4.4) , which superficially would justify categorisation as Category A, it is suggested that the modest depth of flooding of around 0.3m and low fatality rate of 0.1% mean that Flood Category B is more appropriate.

5. Options for Spillway Upgrading (Works Required)

The Options that have been considered as part of this study are as follows.

- Option 1 – Discontinuance
- Option 2 – Create New Overflow to Pass Design flood (T1000) in FRS4
- Option 3A – Add Additional Pipe to Increase Service Spillway Capacity
- Option 3B – Increase Resilience to Overflow
- Option 3C – Regularise Embankment Crest, and Locally raise [redacted] of Road

The above options were defined in terms of hydraulic dimensions and reduction in probability of failure in section 3.5. This section describes the engineering works that would be required, with sketch plans and sections included in Appendix G. Costs are given in Section 5.8.

5.1 Option 1 Discontinuance

The option to discontinue the reservoir, has been considered as it removes any risk of dam failure and the ongoing costs of maintaining the reservoir throughout its life. It would involve removing the dam at its maximum height and constructing a culvert or bridge to carry the A road over the excavated valley.

However, this option has been discounted on the grounds that that Cheshire East Council are unlikely to want to remove the lake as its currently serves as an amenity lake for the local community.

Table 5-1 – Option 1 Discontinuance

Element	Adopted	Comment/Actions
Discontinuance of the Reservoir	This option was considered but discounted early on.	The Council are unlikely to want to remove the lake as it currently serves as an amenity lake for the local community.
Method of Construction	N/A	N/A
Option Cost (£)		N/A

5.2 Option 2 – Create New Overflow to Pass Design Flood in FRS4

To meet the recommended standard the usual engineering approach would be the design and construction of a new, larger culvert under the A road, both to reduce the overall chance of failure, and chance of flooding the road. In simple terms to pass a design flood (T1000) of 6.9m^3/s this would need to be 9 times larger than the current 450mm pipe, or around 1.4m diameter, whilst to pass the safety check flood of 11 m^3/s it would need to be 14 times larger or around 1.7m diameter. If allowance was made for 35% increase in flows due to climate change this would need to be 1.6m and 2.0m diameter respectively. Our pricing has assumed the works in Table 5.1.

Table 5-2– Option 2 Create New Overflow to Pass Design Flood (T1000) in FRS4

Element	Adopted	Comment/Actions
Additional Overflow Structure	Form overflow/control structure from sheet piles leading into pipe/culvert. To pass T1000 6m^3/s with existing crest levels require 22m long weir	Form new overflow from sheet piles, piles left high during construction period then burn off to TWL. Footprint and depth would need to be increased to accommodate the increased pipe dimensions. Weir length could be reduced if dam crest level raised, or TWL lowered. Pipe Intake needs to be at least 2.5m deep to avoid limiting flow. Existing overflow maintained to provide some capacity, faciality for dealing with stream flows during construction and facilitate future maintenance /inspection
Pipe through dam and Road	Install new pipe to form overflow Pipe invert 88.5mOD i.e. 2.4m below crest level.	Install new pipe /culvert of approx. sizes 1.4m for 6.9m^3/s or 1.6 to allow for this plus 35% climate change allowance. Construct drop manhole at edge of road to minimise gradients and excavation depth. Priced on traditional construction methods, i.e. open trench however there may be a cost saving to using trenchless technology to install the new pipe.
Downstream of the Road	New Impact control structure at downstream toe to control flows. 600mm low flow pipe to control "normal flow" from overflow structure to convey flows to the downstream watercourse. Approx. 150m in length.	Flatten the slope from the road to the valley to approximately 1 in 4 to increase resilience to overflow. Minimise the length of the new pipe by discharging at an outlet structure at or just beyond the downstream toe in the valley in the downstream field. This leaves a risk that the [redacted] ould flood during the design flood event of 0.1% AEP event. Install 600mm diameter pipe from impact control structure to river to allow discharge of normal flows over overflow. Raise the existing manhole on downstream face to suit flattened slope.
A523	Road closure, wholly or partly required while trench across road	Traffic management during construction of the works.
Method of Construction	Discussions with Contractor	Open trench to install the new pipework. Trenchless Technology – is a possible option to reduce impact on the road, however initial investigation suggests that large drive and receiver pits/shafts will be required. Existing services (diversions). Encountering ground water.

5.3 Option 3 – Risk Based Options – General

These are options which increase the resilience of the dam to overflow, and thus reduce the chance of release of the reservoir. They do not reduce the chance of overflow or the A road being inundated. Key hydraulic dimensions and the reduction in probability of damage and failure of the dam are given in table 3.7.

5.4 Option 3A – Add Additional Pipe to Increase Service Spillway Capacity

Original overflow configuration retained and supplemented with one additional pipe. It is anticipated that the additional pipe would be of similar size to the existing 600mm diameter pipe. The additional pipe might say double the capacity but would not be large enough to pass the design flood event. As the additional pipe would only achieve a percentage of the peak pass forward flow there is a still a risk of failure of the embankment, albeit reduced. Its main advantages are that it would mitigate climate change, reduce the risk of flooding the A523, with two spillways provide flexibility for inspection/ maintenance and being a smaller scale of works should be more straightforward to construct.

Table 5-3 – Option 3A Add Additional Pipe to Increase Service Spillway Capacity

Element	Adopted	Comment/Actions
Additional Overflow Structure	Form overflow/control structure from sheet piles leading into pipe/culvert. 5m long duckbill overflow structure.	Form new overflow from sheet piles, piles left high during construction period then burn off to TWL. Footprint and depth would need to be increased to accommodate the increased pipe dimensions.
Install Additional Overflow Pipe	Retain the existing overflow arrangement and supplement with additional 600mm pipe to increase the overflow capacity.	Installing larger pipe would increase the capacity but also increase the amount of excavation in the embankment and road, however a smaller pipe diameter will minimise the depth of any excavations. Construct drop manhole at edge of road to minimise gradients and excavation depth. Priced on traditional construction methods, i.e. open trench however there may be a cost saving to using trenchless technology to install the new pipe. Invert level of Existing pipe inlet 89.50m. Invert of new pipe intake 88.5mOD
Downstream of the road	New outlet structure.	New outlet structure new pipework positioned at or close to the downstream toe, after regrading. Existing 450mm pipework retained. Regrade embankment as for Option 2
Impact on A523	Road closure, wholly or partly required while trench across road	Traffic management during construction of the works.
Other method of Construction	Discussions with Contractor	Open trench to install the new pipework. Trenchless Technology – is a possible option to reduce impact on the road, however initial investigation suggests that the large drive and receiver pits/shafts will be required. Existing services (diversions). Encountering ground water.

5.5 Option 3B – Emergency Spillways

This comprises existing overflow retained, and new emergency spillway conveys flood flows that the pipe cannot taken, with hydraulic dimensions given in section 3.5.3.

5.5.1 Upper embankment above A532

Two options are presented for the upper embankment in Table 3.6, a 35m wide spillway to pass flows that avoid flooding [redacted] and a 140m wide spillway to pass 11m^3/s (safety check flood). At this stage the narrower spillway is taken forward to costing.

5.5.2 Lower embankment below A425

The existing steep slope needs to be flattened from around 2H;1V to 4H;1V to reduce the risk of failure under overflow (see table 3.7). Unfortunately, this steep downstream face extends [redacted] (see photo 7 in Appendix D), So if reasonably practicable the earthworks would need to extend into [redacted] This could be part of the spillway, thus lengthening the crest and reduce the head rise over the spillway, which would reduce the risk of flooding [redacted] (see Figure 3.3).

An important issue is access for maintenance and surveillance of the toe of the dam within [redacted] Strictly this should already be occurring, as it forms part of the dam. It is suggested that a design which minimises the need for maintenance and inspection would be preferred, so that it is perhaps only inspected once a year when the Supervising Engineer visits. Thus, scour protection on the downstream, face would need to be agreed, but some form of concrete blocks infilled with topsoil and grass would probably be most appropriate.

Table 5-4 – Option 3B Emergency Spillways

Element	Adopted	Comment/Actions
Upstream of road	Form emergency spillway in upper embankment to be at current minimum crest level from around chainage 670 to 705 (~35m).	This is just north of the overflow. The downstream face would need to be armoured and flattened to a 4H;1V slope to accommodate velocities generated by flows coming over the overflow. Plain grass with average cover should be sufficient to resist erosion.
	Existing overflow retained	New emergency spillway conveys overflow and prevents scour of the embankment.
	Tree clearance	The grass reinforced spillway chute should not be shadowed by trees, so trees would need to be removed both on the spillway and for say 10m either side.
Road	Removal/ relay the kerb on the downstream edge over length ~ 26m	This will allow passage of water to the downstream field. Level and extent of upper road kerb to be agreed and would depend on factors such as extent to which supporting embankment could be designed to resist overflow. Level could be 89.5mOD (Figure 3.5 in report), or if non-overflow 89.8mOD for Option 3C New traffic barrier installed along length of lowered section.
Downstream of road	Regulation and armouring of the downstream face	The drop into the valley is steep (1(v) :2.5(h)). Therefore, flattening to 4H:1V and armouring required over this length to provide protection from flows. The lidar suggests that the valley extends into [redacted] so land agreements/ access

Element	Adopted	Comment/Actions
		will need to be negotiated to extend this flattening into [redacted] with appropriate reinstatement of existing features.
	[redacted]	Reprofile pavement and downstream face of road, including replace fencing.
	Flooding of Properties	There may be some flooding of the lower half of [redacted] but this is considered acceptable as it would only happen in major flood events.
	Raise Manhole	The existing manhole on the flattened section downstream of the road will need to be raised to suit slope.
Impact on A523	lane closure while modify kerbs on west side	Minor roadworks, traffic management lane closures etc.
Other method of Construction	Discussions with Contractor Discussions with Environmental Specialists [redacted]	Tree/vegetation clearance on downstream face over length of new emergency overflow. Potential flooding and necessary land take to be [redacted]

5.6 Option 3C –Increase Resilience to Overflow

5.6.1 Embankment upstream of A523

Carry out the works described in Section 3.5.4, namely. Regularise Embankment Crest and locally Raise Upstream [redacted] on Downstream Side of Road.

In practice this is complicated by the features shown below.

Table 5-5 Features which need to be addressed for crest works in Option 3C upper

	Features (See Section 2.6.1)	Options	Assumed in Costing
1	The existing path is around 0.2 to 0.3m below ground level in the woodland area.	a) a dwarf wall around 300mm high, so top 50mm above highest ground level. b) import fill to raise ground level along path, so crest is a marker 50mm above ground level rather than wall.	Assume that the ground is raised along the crest, the installation of a concrete kerb as the crest marker.
2	Wall along upstream side of dam.	In some places the crest is protected against wave erosion by trees/ vegetation, but elsewhere there are short sections of wall.	No allowance for replace/ raise wall along water line. If path raised, then upstream edge is set back on line 2H;1V from base of upstream wall
3	Resistance to erosion under overflow.	Could be tarmac path, or as concrete edging on downstream side of existing path, in concrete bedding/ mass concrete trench below depth of desiccation cracking of fill. Top of marker preferable 50mm min above genal ground level with say 4h;1V slope down to general ground level.	Assume the path is to be tarmac path along the length as the existing path is described as an "earth path". Concrete kerb to be installed on upstream side of path as crest marker.
4	The path is too narrow for normal construction plan (min existing 1.2m).	Either accept cost of smaller plant or widen to say 3m wide.	Assume that the path will be widened to 3m. this will require vegetation and tree clearance over the length of the path (400m).
5	Two footpaths across crest	Ramp up to crest marker or establish constant level of path along the crest.	Assume that the new path along the crest will be set at a constant level.
6	There are many mature trees along crest, right up to edge of path.	This will complicate kerb/ dwarf wall construction, as if severed for construction then would damage trees. Insitu bedding/ wall could be specified but would cost more.	Assume that mature trees will require to be removed every ~10m over the 400m length. This gives a total of 40no trees.
7	Old tree stumps present in places, making path deviate in plan	Remove any stumps from the area.	Assume stumps ground down to general ground level where obstruct path, and formation level where on line of "kerb"
8	Local high ground in woodland	Some of these areas appear to be old chippings etc.	Allowance for local excavation/ removal.

Table 5-6 – Option 3C Regularise Embankment Crest and Locally Raise Upstream [redacted] on Downstream Side of Road

Element	Adopted	Comment/Actions
Crest Regulation	Form level crest to spread out overflow, to tolerance of ±6mm	We have priced this as the relaying of ~400m of path, and including for the practicable issues in table 5.3
Crest [redacted]	Raise and regulate the crest by ~300mm Locally to protect properties downstream	The crest would be raised over two key areas to provide protection to [redacted] of the dam. It is not necessary to do this at the southern end of the dam as the crest is already high enough here. Due to width of crest ~10m a bund could be formed set back from the existing path along the crest.
	Tree/vegetation clearance	This will need to take place to allow the formation of the raised section. The extent may be limited to minor tree felling and vegetation clearance. The price has been calculated as approximately 40no trees to be removed (including stumps)
Road	As for Option 3B	
Downstream of road	As for Option 3B	
Impact on A523	Lane closure while modify kerbs on west side	Minor roadworks, traffic management lane closures etc.
Method of Construction	Discussions with Contractor Discussions with Environmental Specialists [redacted]	Tree/vegetation clearance on crest and upper part of downstream face. Potential flooding and necessary land take to be discussed [redacted]

5.6.2 Works to embankment downstream of A523

These would be the same as for Option 3B

5.7 Key Quantities and Main Dimensions

The table below outlines the Key quantities and main dimensions for the options discussed above.

Table 5-7 Comparison of key work items between different options

Key work item/ Quantities	**Option**				
	1	2	3A	3B	3C crest
22m long Sheetpile overflow weir.	*N/A*	√	*x*	*x*	*x*
1.4m diameter concrete pipe (45m long).	*N/A*	√	*x*	*x*	*x*
2.5m diameter x 5.5m deep Manhole.	*N/A*	√	*x*	*x*	
6m x 5m x2.5m Impact structure.	*N/A*	√	*x*	*x*	*x*
0.6m diameter outlet pipe (150m long).	*N/A*	√	√	√	*x*
Flattened Downstream slope (310m^3).	*N/A*	√	*x*	√	√
6m long Sheetpile overflow weir.	*N/A*	*x*	√	*x*	*x*
Single 0.6m diameter concrete pipe (45m long).	*N/A*	*x*	√	*x*	*x*
2.5m diameter x 3.5m deep Manhole.	*N/A*	*x*	√	*x*	*x*
4m x 3m x1m Impact structure.	*N/A*	*x*	√	*x*	*x*
Section of embankment lowered by approximately 0.15m over 35m length.	*N/A*	*x*	*x*	√	*x*
Road edging modified to form overflow	*N/A*	*x*	*x*	√	√
Kerb removed and ground reprofile 45m long	*N/A*	*x*	*x*	√	√
Armouring of Downstream slope with Grasscrete/Armourflex open mat (240m^2).	*N/A*	*x*	*x*	√	√
New crest path/ edging to tolerable of ±6mm over 400m	*N/A*	*x*	*x*	*x*	√
Protective bund to raise embankment to peak flood level at 2 no location. Total length 80m.	*N/A*	*x*	*x*	*x*	√

5.8 Cost

High level estimated project costs are given in Appendix G and summarised below.

The costs are an indication, at pre-feasibility level, of activity costs sufficient to compare options and determine if the incremental cost of reservoir safety work is disproportionate. Allowances are made for preliminaries (30% of measured items) and professional fees for planning etc to arrive at a project cost for the purposes of the ALARP analysis.

The following table outlines the estimates of project cost for the four options that have been considered for the spillway upgrade works at Poynton Lake. A cost breakdown for each option is provided in Appendix G.

Table 5-8 Summary of project cost of options considered

Option	Option 2	Option 3A	Option 3B	Option 3C	Option 3C
	Pass T1000 flood	0.6m pipe	Emergency spillway	Crest Resilience	D/S Embankment
Enabling works	£69,800	£69,800	£134,500	£104,000	£18,666
Permanent works	£221,000	£69,000	£82,000	£102,000	£44,000
Temporary works	£226,000	£150,000	£71,000	£17,000	£71,000
Minor items	£104,000	£58,000	£58,000	£45,000	£27,000
Contractors preliminaries	£130,000	£73,000	£72,000	£56,000	£34,000
Construction contract value	£750,800	£419,800	£417,500	£324,000	£194,666
Professional fees, surveys etc.	£215,000	£145,000	£136,000	£79,000	£58,000
Optimism bias	£291,000	£180,000	£170,000	£130,000	£78,000
Total Project cost	**£1,261,000**	**£750,000**	**£730,000**	**£540,000**	**£338,000**
% of full engineering standard	100%	59%	58%	43%	27%

6. Evaluation and discussion of options

6.1 Introduction

This section assesses the costs of the candidate options and whether these are proportionate to the reduction in risk achieved. To avoid enforcement action by the EA, any works must be completed within four years i.e. by 5th December 2023.

The decision on what works should be carried out should be based on considerations including:

a) Compliance with engineering standards

b) Acceptability of damage to dam, and flooding of the A523

c) Economic calculation of costs for each option and their benefit in terms of reduced risk of failure to the public downstream (release of the reservoir).

d) Other considerations, including impacts of each option.

6.2 Buildability

The formation of a new overflow in both Options 2 and 3A would require excavations in the embankment and under the A Road. The installation of the pipe in the dam means that the provisions will need to be put in place to prevent the weakened section of dam failing during the works should a flood event occur. In addition to this there is the complexity of installing pipes under the busy A Road as extensive traffic management will be required for the duration of the works. However, it may be possible to install the pipes using trenchless technology, but this would require discussions with specialist contractors.

Options 3B and 3C are generally works comprising of earth moving and reprofiling and require little or no drawdown of the reservoir for the duration of the works. The works will have a minor impact on the road so traffic management will be required, albeit to a lesser extent and duration to those required for Options 2 and 3A. However, both require works on the embankment downstream of the road which includes works [redacted]

All options will require the removal of a number of trees on the downstream face.

6.3 Engineering Standards

The consequence of failure of Poynton Lake would be risks to lives in [redacted], the road and [redacted], for which the Institution of Civil Engineers recommends in Floods and Reservoir Safety 4th Ed. (ICE, 2015; FRS4) that the dam is Category B. For such a dam, the spillway system should be capable of passing a design flood of annual chance of 1 in 1,000 with no damage, and the dam itself should not fail in a 10,000 chance per year flood.

To meet current engineering standards the only acceptable option would be a new overflow and associated pipe, to reduce the chance of failure to that set out in FRS4. However, ICE 2015 also recommends that where an existing dam does not meet current standards, then the Undertaker may wish to adopt a risk based approach to assess the extent of upgrading (comment at end of Chapter 2, and process diagram in Appendix 3 of FRS4). This risk-based approach is presented below.

6.4 Acceptability of Flooding A523 and Damage to Dam

A separate consideration is the risk of flooding the A523 and damage to the dam, which is currently around a 1.8% annual probability (1 in 55 chance per year). The reduction in annual chance of flooding the A523 and damage to the dam due to the options considered is given in Table 3.5.

6.5 Risk Based Approach (ALARP Economic Evaluation, RARS Methodology Section 10.3)

6.5.1 Introduction

This method compares the cost of candidate options (to reduce risk) with the reduction in risk achieved. The cost is deemed proportionate (or "as low as reasonably practicable" i.e. ALARP) where the cost to save a life over a 100 year horizon is £8.5M or less (see Appendix C for further information).

6.5.2 Cost to save a life – Base case

Inputs into this calculation are given earlier in the report as follows

- Probability of failure in Table 3.5
- Consequences of failure in Table 4.4
- Costs of candidate options to reduce risk in Section 5.8

These have been used to assess the cost to save a life, and thus where the works are proportionate to the reduction in risk, as shown in Table 6.1. This shows that currently regulating the crest, so any overflow is spread out uniformly is extremely worthwhile, in that the benefits outweigh the costs, whilst the other options are marginally proportionate.

When allowance is made for 35% increase in flows due to climate change (see Section 2.4.2) then all options are proportionate

6.5.3 Cost to save a life – Possible Phasing

At Poynton there are a number of elements of works which could reduce the probability of flooding of the A523, and others which would have a bigger impact in reducing the risk of reservoir breach and thus life loss. The "cost to save a life" will vary with the sequencing of elements of work, and so for simplicity have not been included at this stage.

6.5.4 Cost to save a life – sensitivity

Although the estimate in this report is a best estimate it is only at concept design level. There are clearly several factors and assumptions that are made in carrying out the economic calculation. Some of these are set out below, to provide a measure of potential impact of the scale of works that would be proportionate.

Table 6-1 Risk and uncertainty in ALARP estimates

Risk (uncertainty)	Effect on case for works	Comment/ possible action to mitigate
Breach hydrograph – base case assumes fails in flood in two stages, to A road, then below.	Strengthen if fail to full height in one stage	
Consequences of failure could be higher if dambreak occurs such that peak of T1000 flood and dambreak coincide	Strengthen	Detailed dambreak could provide sensitivity analyses but would not eliminate the uncertainty in predicting timing of floods and dambreak, and magnitude of fluvial floods at time of dam failure.
Consequence – do people evacuate in preceding fluvial flood? EA assume they don't. our assessment has assumed they do.	Strengthen if use EA estimates	Emergency planning

Risk (uncertainty)	Effect on case for works	Comment/ possible action to mitigate
If weak zones present in upper embankment along which preferential erosion occurs, increasing Probability of failure/ breach	Strengthen	With 800m length of dam it is difficult to predict. Probably better to adopt robust approach to risk management.
Construction/ project costs underestimated	Weaken	Includes 30% optimism bias. Could ask ECI/ Framework Contractor to price.

Table 6-2: Summary of costs and benefits

Option	Works involved	Budget project cost (Appendix G)	Annual chance as 1 in x of (With 35% climate change in brackets – see Table; 2.2)		Cost to save a life: current (with climate change) (Note 2, 3)
			Onset of flood A523 (damage to embankment) (Note 1)	Dam failure with release of reservoir	
	Existing situation		55 (40)	250 (120)	n/a
2	Large culvert to pass Design flood (T1000) for FRS4 Flood Category B dam	£1.3M	1,100 (450)	6,000 (1,500)	6 (0.4)
3A	Additional pipe to mitigate climate change and increase pipe capacity	750k	120 (60)	500 (150)	7 (9)
3B	3B Increase resilience to overflow. Lower crest and form emergency spillway on embankment above A road, armoured to resist overflow; together with flatten downstream slope of embankment below A road, to reduce risk of catastrophic washout of supporting embankment at low point in road.	730k	as existing	1,000 (350)	3 (-0.7)
3C - Upper	Regularise dam embankment crest to spread out overflow over longer length; locally raise crest [redacted] to reduce risk of breach opposite [redacted]	540k	as existing	10,000 (2,200)	-0.6 (-13)
3C lower	As above plus works to embankment downstream of A523	£340K	55 (40)	1,000,000 (> 1M)	N/ a see text 6.6.1

Notes.

1. **Onset of damage to existing embankment and flooding of road in unlikely to lead to failure of the dam and release of the reservoir as the embankment is wide.**
2. Cost becomes grossly disproportionate when cost to save a life exceeds £8.5M – see Appendix C.
3. Negative CSL is where benefits of reduced risk of £5M damage [redacted] outweighs PV costs of scheme

6.6 Other Considerations

These include the factors listed in Section 10.4 of RARS (Environment Agency 2013), and include:

- The confidence and defensibility of the owner
- Balance between reservoir safety and the damage to downstream property.

6.6.1 Risk of flooding of A523 [redacted]

Flooding of the road is not a dam safety issue, but is an operational risk for CEC, and can be assessed against levels of services on other highways.

[redacted] but scour of the downstream embankment in [redacted] is related to spillway capacity, and thus CEC responsibility. It is suggested that [redacted] to explore the possibility of flattening and armouring the downstream embankment, noting that it would be in the self-interest of [redacted] [redacted] with the proposed works in 3C lower to improve the resilience to extreme flood.

6.6.2 Risk of Future Dam Safety Works

If a risk based approach is selected and the downstream population at risk increases considerably, then there is the risk that in future ten yearly Section 10 inspections (safety reviews) that the panel engineer may consider that further upgrades are proportionate in cost and require further upgrades in spillway capacity.

A further risk is that whatever option is selected is that in future decades, when climate change is better understood, that estimates of the magnitude of the "probable maximum flood" and " 1 in 10,000 chance per year flood" may increase, and also lead to the requirement to carry out further spillway upgrades.

6.6.3 Downstream Fluvial flood risk

Enlarged concrete spillway pipes would increase fluvial flood risk downstream as flows would be marginally higher. However, the reservoir would continue to provide flood attenuation, relative to the situation if the dam were not present. There would be slight mitigation by regularising the crest, as this would infill low spots and inhibit overflow.

The emergency spillway would have no adverse impact on fluvial flood risk, as the current frequency of overflow would not be changed.

6.6.4 Heritage and environment

The Lake is currently used as an amenity lake by the local community and as such any works carried out should be detailed to minimise impact on the area.

6.6.5 Uncertainties in assessment

There are uncertainties in any estimate of floods, likelihood of failure and consequences of failure. However, it is clear that the dam does not meet the standard for a Category B dam, such that an increase in spillway capacity is required.

Even if more detailed analysis of one or more elements of the risk assessment were carried out, this would not change the conclusion that spillway upgrading is required.

6.7 Discussions and Conclusions

If the Client wishes to comply fully the engineering standard, they would need to adopt Option 2 and 3C combined. (£1.7M); and this is proportionate when allowance is made for future climate change

If the Client wishes to adopt a risk based approach then the ARPE overseeing this reports considers that the minimum that would be acceptable is Option3C- upper, crest raising alone, although addition of strengthening the embankment below the A road is also recommended.

However, the Client may wish to do more than the basic minimum, and this is a complex decision including factors discussed in this chapter. In simple terms the options are

- 2 and 3A – Increase pipe capacity by different amounts, no change to dam crest.
- 3B – No works across road; emergency spillways on upper and lower embankments reduce risk of damage to dam in floods but have big visual impact with loss of trees.
- 3C upper – Increase resilience to extreme floods by regularise dam embankment crest to spread out overflow over longer length; locally raise crest to reduce risk of breach [redacted] so no change in risk of flooding A523 but regulated crest. Significant uncertainty in optimising works on the crest – see Table 5.5. preferred design should be developed in discussion with CEC Rangers.
- 3C lower- emergency spillway on lower embankment reduces risk of damage to lower embankment, half of which is in the [redacted]

Clearly a combination of options is also possible. To assist in this decision the options are summarised below with a preliminary qualitative comparison of the relative advantages and disadvantages of each option. The comparison is to some extent subjective, and **selection of the preferred option has to be a Client decision, based on the various considerations, and their relative weighting.** In addition to selecting the preferred option the Client also needs to set the design criteria for the selected option(s), for example any allowance for climate change, and if so what magnitude.

Table 6-3: Preliminary comparison of options to increase spillway impact

Consideration	Option 2	Option 3A	Option 3B	Option 3C upper	Option 3C lower
	Culvert to pass T1000 flood	smaller 0.6m pipe	Emergency spillway	Regularise crest	Flatten slope
Project Cost	£1.3M	£750k	£730k	£540k	£340
Cost to save a life £M/ life as EA 24th March 2021	6	7	3	0 (benefits outweigh costs)	see text section 6.6.1
CEC Reputation (flood risk management)	√√√	√	-	√	√√
Spillway capacity – dam failure	√√√	√√	√√	√√√	√√√
Onset of flooding A523	√√√	√	No change	√	√
Onset of damage to dam	√√√	√	√√	√√	√√
Risk of future dam safety works	√√	xx	√	√	√
Fluvial flood risk	xx	x	√√√	√√√	√√√
Heritage	√√	√√	xxx	x	-
Visual impact	√√	√√	xx	x	X
Key (scale of 1 to 3): √ Beneficial, X Detrimental					

7. References

ICE	2015	Floods and Reservoir Safety 4th Ed.
CIRIA	1987	Design of reinforced grass spillways. Report 116. 119pp
Environment Agency	2013	Guide to Risk Assessment for Reservoir Safety Management (RARS)
Gosden, Ambler, Courtnadge	2014	Improving the overtopping resistance of flood detention reservoirs. BDS Conf. Belfast pp426-437
HSE	2001	Reducing Risk protecting people
ICE	2004	Interim Guide to Risk Assessment of dams.

Appendix A. Topographic survey

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

Notes

This drawing is to be read in conjunction with the accompanying survey report, Poynton Lake_*Topo Survey Report.pdf*. This report contains deta is 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	[redacted]			
0	01.10.19	Revision 0				

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Project
Poynton Lake

Drawing title
Cross-Section Location Plan
Sheet 1 of 5

Drawing status
FINAL

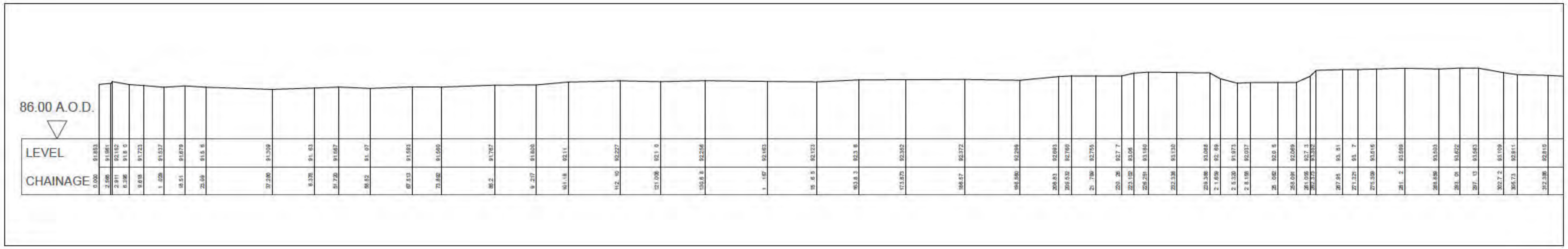
Scale	1:1250 @ A1	DO NOT SCALE
Jacobs No.	BRJ10627	
Client no.		

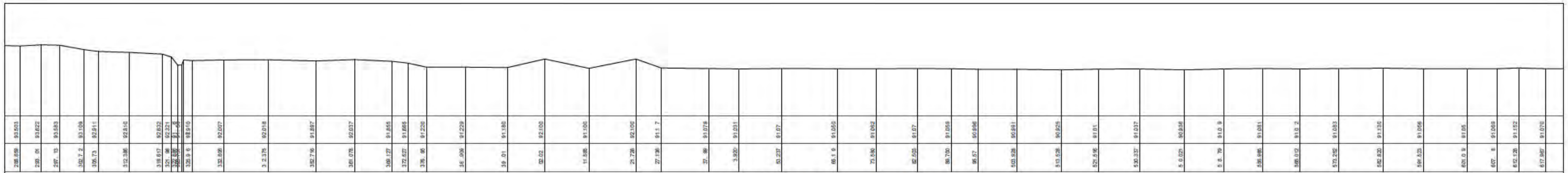
Drawing number
BRJ10627/Topo/3d

Rev
0

Drawing number / Rev

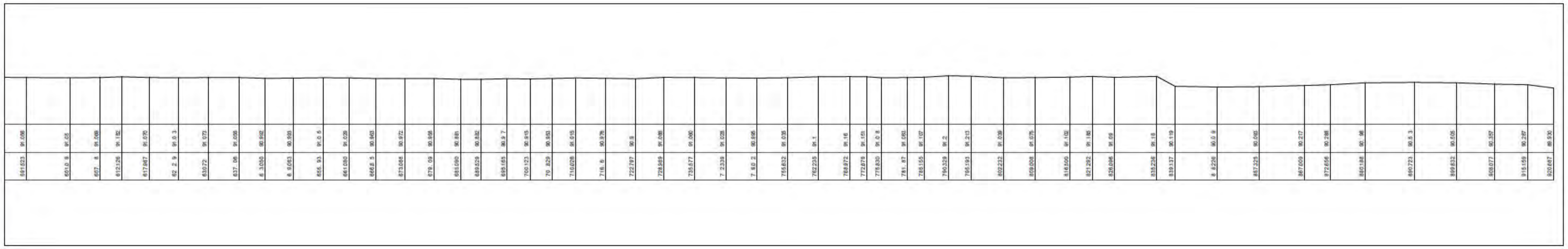
BRJ10627/Topo/3d/Rev 0





Embankment Long Section

Horiz. 1:500 Vert. 1:250



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	Check'd	Rev'd	Appr'd
B	15.10.19	Revision B	[redacted]			
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Drawing title

3d Topographical Survey
Sheet 2 of 5

Drawing status

FINAL

Scale	1:500 @ A1	DO NOT SCALE
Jacobs No.	BRJ10627	
Client no.		

Drawing number

BRJ10627/Topo/3d

Rev

0

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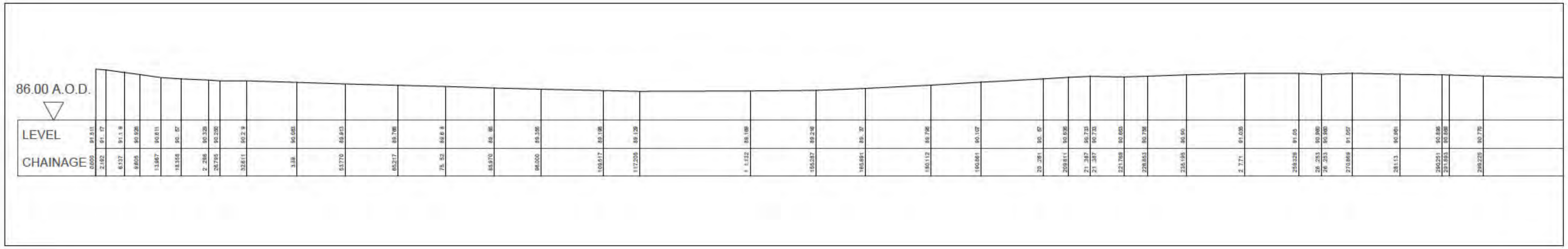
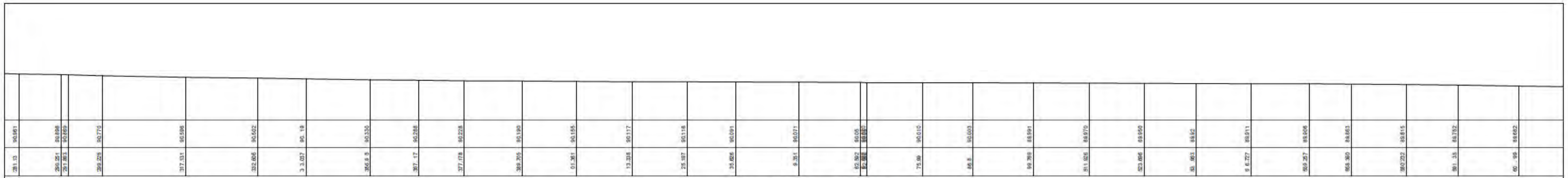
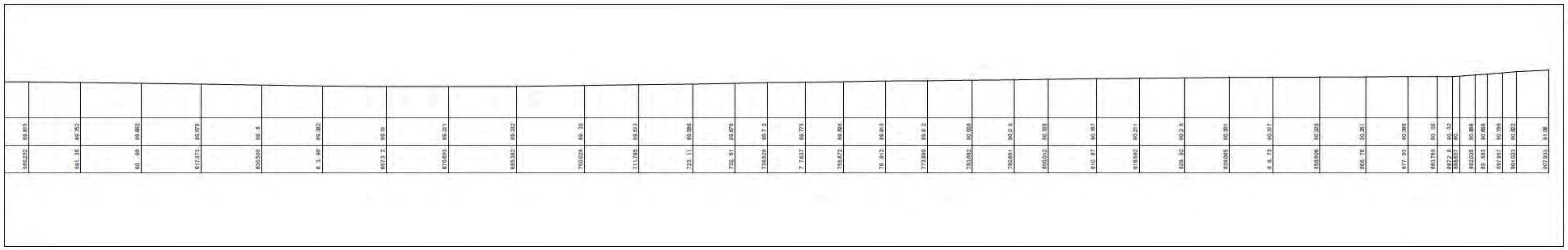
Drawing number / Rev
BRJ10627/Topo/3d/Rev 0





Wall Long Section
Horiz. 1:500 Vert. 1:250



Notes

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Rev	Rev. Date	Purpose of revision	Drawn	Checkd	Revwd	Apprv'd
B	15.10.19	Revision B	[redacted]			
0	01.10.19	Revision 0				

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Sheet 3 of 5

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Scale	1:500 @ A1	DO NOT SCALE
Jacobs No.	BRJ10627	
Client no.		

Drawing number
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Rev
0

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\\europe.jacobs.com\reading\Admin\Environment\PU35102 Dams Team\Bids and small jobs Europe\Cheshire East - Poynton lake\30 Incoming info\2019 Topo survey\POYNTON LAKE_3D TOPOGRAPHICAL SURVEY_Rev B.dwg - 04/03/2021 20:42:16 - A1-V Frame (2) - KellyRH

Drawing number / Rev
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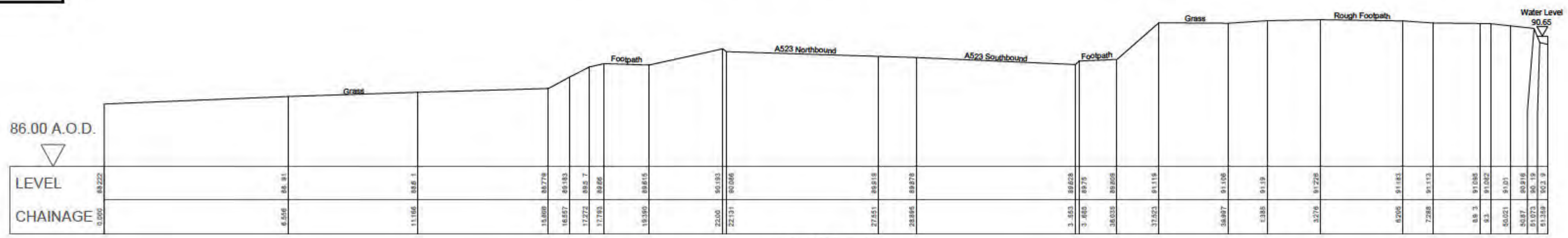

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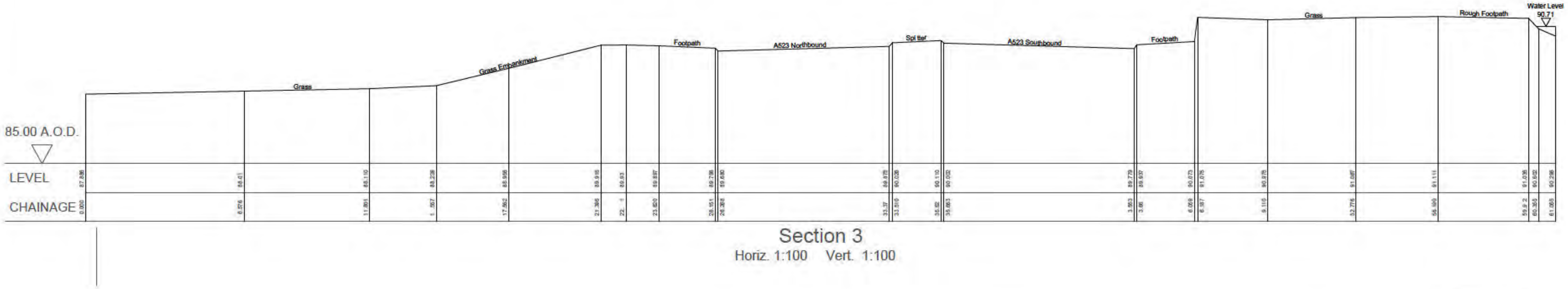

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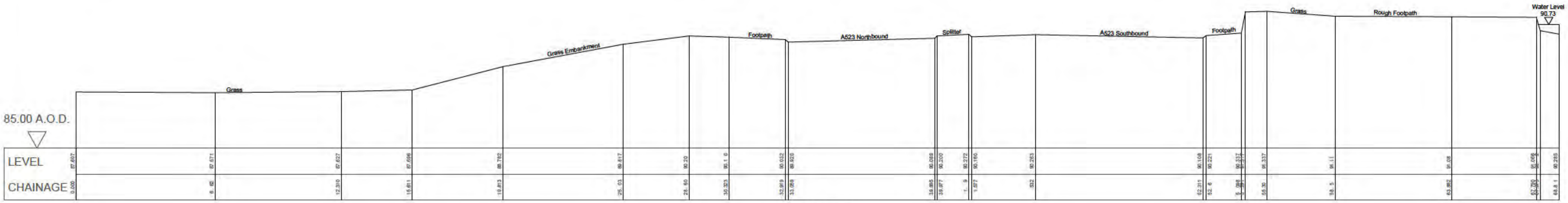
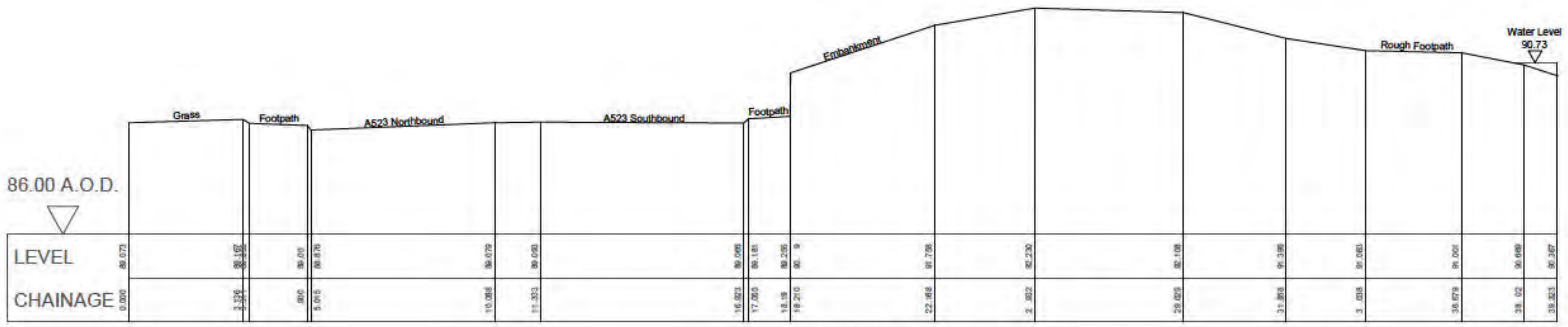
Horiz. 1:100 Vert. 1:100



Section 2
Horiz. 1:100 Vert. 1:100



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

Notes

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Sheet 4 of 5

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

Drawing number
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Rev
0

Drawing number / Rev
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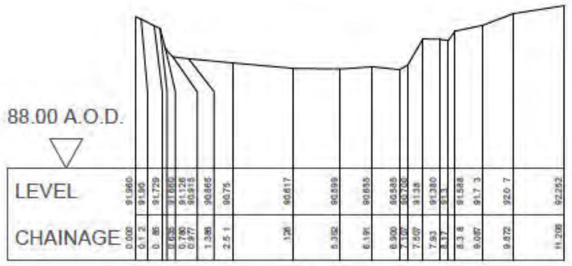
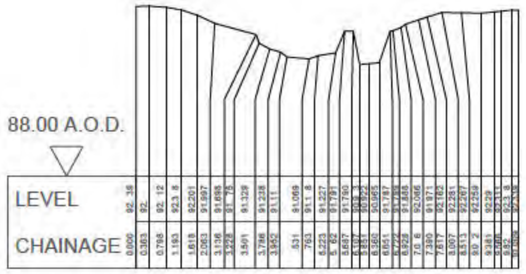
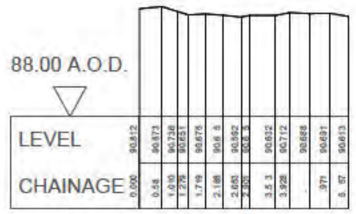


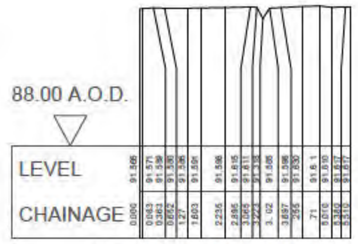
Section 8
Horiz. 1:100 Vert. 1:100



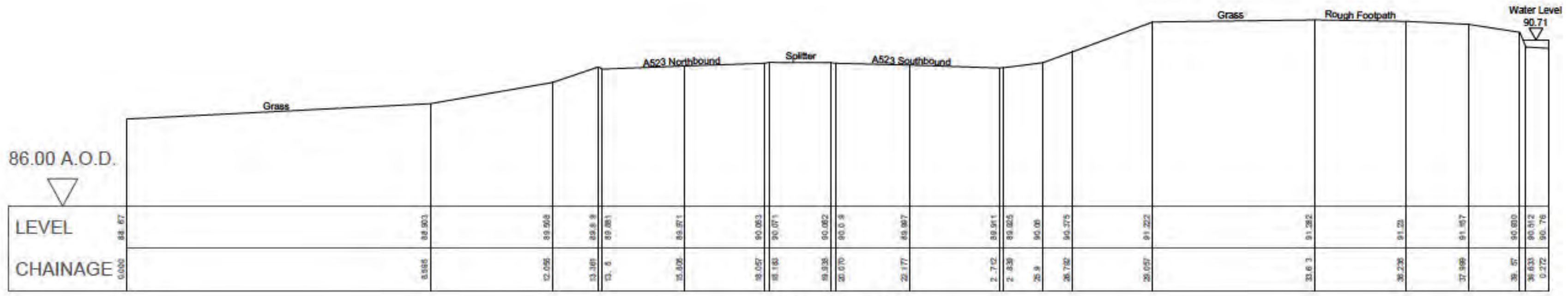
Section 9
Horiz. 1:100 Vert. 1:100



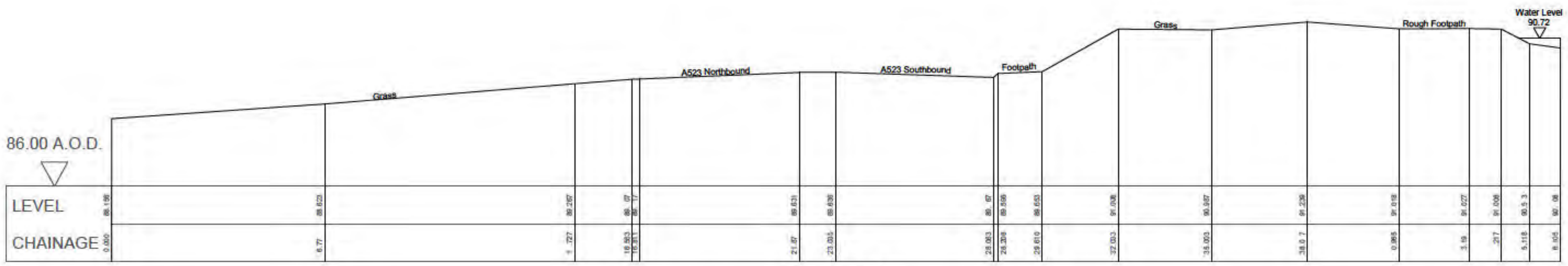
Section 10 (Weir Bottom)
Horiz. 1:100 Vert. 1:100



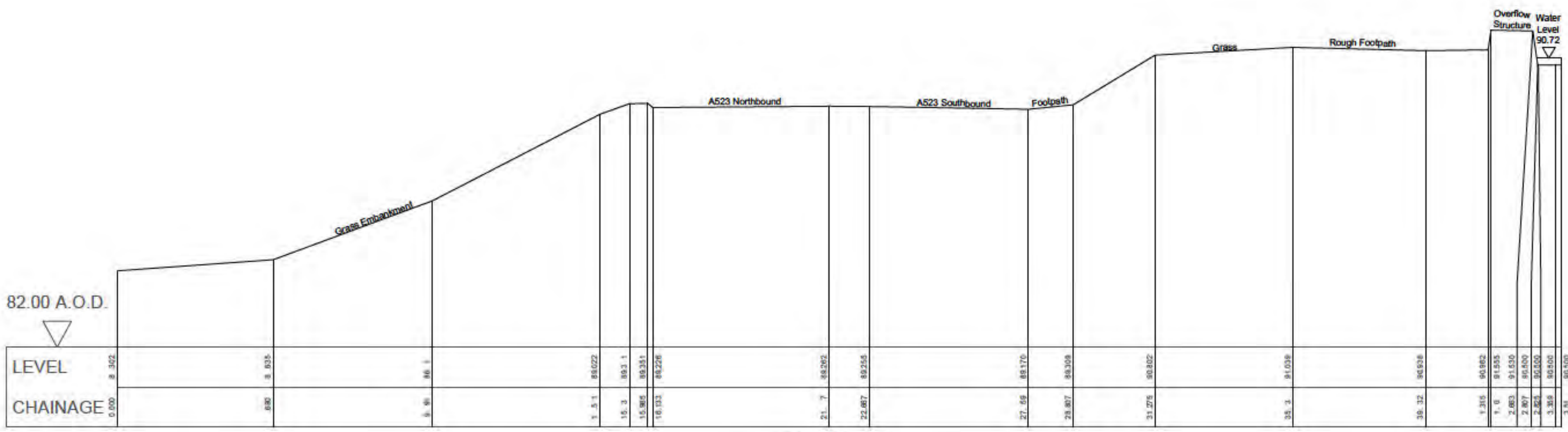
Section 10 (Weir Top)
Horiz. 1:100 Vert. 1:100



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

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Sheet 5 of 5

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

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

Appendix B. Notes and Slides of Meeting 9th March 2020, and scope for this study

Meeting Minutes

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Subject	**way forward to upgrade spillway**		
Project	Poynton Reservoir		
Project No.	BRJ1067	**File**	200309 Poynton Spillway upgrade Notes of meeting.docx
Prepared by	[redacted]	**Phone No.**	
Location	CEC offices Delamere House Crewe	**Date/Time**	9 March 2020
Participants	[redacted]		
Copies to	Those in attendance [redacted]	**Apologies**	N/A

Notes		Action
1	[redacted] presented 18 slides summarising current position regarding reservoir safety compliance.	Jacobs to draft scope for Options report
2	[redacted] raised the problem of Flood risk in Poynton town and whether Poynton Reservoir can be utilised to alleviate the risk. a) [redacted] noted that notionally surface water flooding, as not main river b) [redacted] queried whether flow from the stream to the south could be diverted into Poynton reservoir, to reduce surface water flood risk to properties downstream from the A523 c) [redacted] reports around 140 houses flooded twice in last five years. d) Agreed that there is no point in attenuating flows in Poynton reservoir (as no sensitive receptors downstream), but could use as a flow diversion route e) Noted possible new housing development on existing cricket pitch, which is to be moved west of A523	[redacted] check who produced reports on surface water flowing in Poynton town (Jacobs?)

Jacobs

Meeting Minutes

way forward to upgrade spillway
9 March 2020

	f) Diversion structure to the reservoir has an oversized 1200mm pipe installed underneath the [redacted]	
3	A phased project process was proposed, with an initial Options Report stage, to allow: a) Cabinet to make decision re-Engineering or risk based approach to reservoir safety compliance b) Decide if reservoir safety works can be merged with flood alleviation scheme to achieve efficiency This would be followed by a more detailed Feasibility Study.	
4	Other queries from CEC; a) Climate change? Jacobs - not included in current reservoir assessment b) What would Environment Agency do if the watercourse from Poynton reservoir were to be re-classified as main river? [redacted] would probably adopt full engineering standard, as they are also the Regulator	a) Include in Feasibility Study scope
5	Scope for operational flood risk. Agree at the Options Report stage just look at what proportion of flood could be diverted i.e. exclude; a) Looking at possible flood storage reservoirs up catchment b) Damages/ benefits/ economics of reduction in flood risk in Poynton town	Jacobs include in scope
6	[redacted] noted that process for internal approvals was not clear, and was probably a cabinet level decision	[redacted]
7	[redacted] noted that for the Risk Based Reservoir safety compliance: a) More study is potentially required to assess likelihood of failure (release of the reservoir), consequences etc plus b) There are several levels of possible analysis, commencing with the preliminary screening carried out in preparing the presentation, c) The level of detail selected for each element of the risk analysis depends on the level of robustness wanted to future challenge, potential change in outcomes etc	
6	Steps to achieve June 2021 signoff to selected option/ feasibility study: a) Options report, including ground investigation and any additional topo survey required b) Cabinet decision on strategy c) Feasibility study sufficient to define key dimensions for detailed design and make business case for funding	All

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Commission Estimate Approval and Authorisation for use with schemes.	**Come No & Rev: CEH-J470**
Scheme Title:	Poynton Lake: Spillway upgrade
ComE Title:	Poynton Lake: Initial options report

Discipline:	Architectural		Estates		M&E		Structures	
	Drainage		Geotechnical		Planning		Transport	
	Environment	x	Highways		PMCM			

CEC Project Sponsor: [redacted]	Task Order:
Ringway Jacobs Project Manager:	Ringway Jacobs Job Code:
Jacobs Project Manager: [redacted]	Jacobs Project Number: TBC

Reporting	x☐	Feasibility	x	Preliminary		Detailed Design		Site supervision	
Specialist Services	☐	Staff Provision		CDM Services		Geomatics		Site Inspection	☐
Inspection		Assessment		Architectural		Cost Estimate		Other	

This proposal is a direct follow on from the works previously carried out to address the measures in the interests of safety identified during the last Section 10 Inspection under the Reservoirs Act 1975 (CEH-J253).

Description of Works Required to be Undertaken

The last Section 10 Inspection was carried out by [redacted] of Mott Macdonald in August 2016 which recommended the following measures in the interests of safety:

- *Produce an emergency drawdown plan*
- *Produce a flood study assessment for the reservoir*

Jacobs were appointed to undertake these studies, which were completed in 2019 and certified by [redacted] under Section 10(6) of the reservoirs Act in a certificate dated 5th December 2019. An annex was attached to the certificate summarising the findings from the studies and making further recommendations.

The flood study identified that the spillway capacity does not meet the standards set out in the ICE Guide to Floods and Reservoir Safety (4th Edition), and as part of the Certificate Under Section 10(6) the following recommendation was made:

- *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.*

A meeting was held between Jacobs and CEC on 09/03/20 to discuss a way forward and it was agreed to adopt a phased project process, with an initial options report stage. This proposal covers production of this 'initial options report' which will consider:

- Whether the spillway upgrade should be designed to meet full engineering-based standards (e.g. the 4th Edition of ICE '*Floods and Reservoir Safety*' (FRS4) guidance), or whether to adopt a risk-based approach (e.g. following the principals in '*Risk Assessment for Reservoir Safety*' (RARS) guidance).
- Whether reservoir safety works can be merged with a flood alleviation scheme to achieve efficiency. The study will consider what proportion of flood flow could be diverted from the stream running to the south, into Poynton reservoir.
- Shortlist options which would comply with both the standards-based approach and the risk-based approach.
- Consider the need to make allowance for climate change

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- Identify requirements for any ground investigations and/or additional topographical surveys to inform the detailed design
- Recommend a preferred option for CEC cabinet board level agreement, defining key dimensions
- Define the scope for the outline planning design which will be carried out at the next stage

Methodology

Full Engineering Standard

We will identify the spillway upgrade works required to comply with the standards recommended in FRS4 guidance, i.e. to safely pass the 1,000year design flood with negligible wave overtopping and ensure that any wave overtopping in the 1 in 10,000-year safety check flood would not cause failure of the dam. As part of this approach the following shall be considered:

- Checks to confirm the capacity of the existing spillway.
- Review dam category, e.g. using simplified dam break methods from RARS
- Identify the best position and arrangement for the new spillway and assess impacts on the A523.
- High level costing exercise.
- Conceptual sketch(s).

Risk Based Approach

This element of the study will include:

- Further assessment of the likelihood and consequences of dam failure. For the purposes of this proposal it is assumed that a Tier 2 (basic quantitative) assessment will be carried out in accordance with RARS guidance.
- Identifying a range of ca[redacted] comprising combinations of the following elements:
  - Works to protec[redacted]ncluding both locally raising the dam crest upstream of [redacted] and potentially works on the A road [redacted]
  - 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
  - Works to improve the resilience of the downstream face of the upper embankment to overflow, such as flattening the downstream slope
  - 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
- High level cost estimates for candidate upgrade options
- Assessment of the cost in relation to the risk reduction
- It is assumed that a maximum of three main options will be considered
- Recommendation and conceptual sketch of the most feasible option which reduces risks to 'As Low as Reasonably Practicable' (ALARP)

Flood alleviation

As discussed at the meeting on 09/03/20, we will assess what proportion of flood flow could be feasibly diverted from the stream running to the south, into Poynton reservoir. This information will help inform the potential viability of the proposed scheme. The study would not quantify the flood risk benefits downstream, nor investigate alternative flood storage sites further up the catchment. To carry out a meaningful assessment of the damages/benefits/economics in relation to downstream flood risk would require a separate study including hydraulic modelling, which we would be happy to quote for separately if required.

Climate change

We will review the resilience of the selected options to climate change based on published government forecasts and make recommendations for any allowances.

Define scope for planning/outline design

Following client consultation and the cabinet boards descion on the preferred strategy, we will define the agreed concept option and identify:

- Key levels and dimensions
- Scope of any additional topographical surveys required
- Scope of any required ground investigation
- Scope of works/outline design philosophy.

Reporting

The above will be documented in an 'Initial options report'. Sketches will be included in the report as Figures.

Appendix C. Criteria to determine whether risk of dam failure has been reduced "as low as reasonably practicable" (ALARP)

B.1 Cost to prevent a fatality (CPF), and worked example

An ALARP approach calculates the cost to prevent a fatality (CPF), defined in Section 10.3 of the Guide to Risk Assessment for Reservoir safety (RARS) (EA, 2013) and is summarised as follows

$$\text{CPF} = \frac{\text{Cost of risk reduction measures} - \text{Present Value } (\Delta \text{ Pf x Damage})}{\text{Present value } (\Delta \text{ Pf x Likely Loss of Life (LLOL)})}$$

where Δ Pf is the change in annual probability of failure due to the proposed risk reduction works.

At its simplest where the CPF is less than the "value of preventing a fatality" (VPF) then the candidate works would be proportionate risk reduction measures; whilst where CPF exceeds VPF then the cost is disproportionate.

Costs should be estimated realistically; it is noted that it is recommended (Defra, 2003) that at prefeasibility stage an optimism bias of 60% is added to the best estimate of total cost, based on experience of total project outturn costs against the prefeasibility estimate. RARS notes in section 10.3 that using Treasury discount rates, the present value of recurring costs over a 100 year period is 30 times the annual value.

For the input values set out below the ALARP calculation equates to:-

$$\text{CSL} = \frac{\text{£}300{,}000 - 30 \text{ x } (5\text{E-}5 - 5\text{E-}6) \text{ x £}35{,}00\text{-},000}{30 \text{ x } (5\text{E-}5 - 5\text{E-}6) \text{ x } 32} \quad \frac{= 300{,}000 - 47{,}250}{=0.0432} \quad = \text{£}5.9\text{M}$$

Input values into above ALARP calculation

Parameter	Value
Cost of candidate works	£300,000
Present value	30 x annual value
Probability of failure - current	=1/20,000 = 5E-5
Probability of failure – after works	= 0.1 times above = 5E-6
Impact of failure: economic damage	32 lives
Impact of failure: economic damage	£35M

B.2 Value of preventing a fatality (VPF)

The value that should be assigned to VPF is a difficult decision and includes consideration of

- Direct costs (measurable) such as the earning potential of the victims, injury and long term health impairment of other victims not included in the LLOL value, and emergency services costs
- Indirect (business losses)
- Intangibles (psychological impact on people, environmental damage) – it could be argued that a value should be assigned to the Intrinsic Value of a Human Life (irrespective of age, health, education etc)

The Department of Transport publishes their assessed VPF for road and rail schemes on the internet, being updated for inflation, with the 2010 value being £1.7M (see RARS)

B.3 Gross Disproportion

However, HSE (2002a, para 25) notes that "gross" disproportion is required before ALARP is satisfied and defines a

$$\text{Proportion Factor (PF)} = \frac{\text{Cost to Prevent a Fatality (CPF)}}{\text{Value to Prevent a Fatality (VPF)}}$$

The purpose of a PF "grossly" greater than unity is to allow for the imprecision of estimates of costs and benefits and also to ensure that the duty holder robustly satisfies the ALARP principle.

HSE guidance on what constitutes a reasonable proportion factor is given in Table A.1.

For dams, where the risk to those in the potential inundation area is involuntary (in that the public are not generally aware of the risk from dams) it will be assumed that the PF should exceed 5 (i.e. product of "VPF and Proportion Factor") before the cost is considered disproportionate. Thus, where CSL is less than 5 x £1.7M = £8.5M it is considered proportionate to carry out the works.

Table A.1: HSE ALARP Suite (Expert Guidance) on Proportion factor

	Updated	Title	Extracts from HSE Guidance
1	2001	Principles and guidelines	26. Although there is no authoritative case law which considers the question, we believe it is right that the greater the risk: the higher the proportion may be before being considered 'gross'. But the disproportion must always be gross. 27. HSE has not formulated an algorithm which can be used to determine the proportion factor for a given level of risk. The extent of the bias must be argued in the light of all the circumstances. It may be possible to come to a view in particular circumstances by examining what factor has been applied in comparable circumstances elsewhere to that kind of hazard or in that particular industry.
2	2003	Assessing compliance with the law in individual cases and the use of good practice	
3	2003	Policy and guidance	
4	n/a	HSE principles for Cost Benefit Analysis (CBA) in support of ALARP decisions	• Rules of thumb adopted by D/Ds; • NSD takes as its starting point the HSE submission to the1987 Sizewell B Inquiry that a factor of up to 3 (i.e., costs three times larger than benefits) would apply for risks to workers; for low risks to members of the public a factor of 2, for high risks a factor of 10; • HID uses similar rules of thumb;
5		Cost Benefit Analysis (CBA) checklist	DFs that may be considered gross vary from upwards of 1 depending on a number of factors including the magnitude of the consequences and the frequency of realising those consequences, i.e. the greater the risk, the greater the DF
6		ALARP at a glance	

Appendix D. [redacted]

[redacted] 1 Datum on Gauge board at existing spillway May 2021,

Datum around 100mm above physical weir crest, probably because set at base of taper shown on Phots 3 and 4

Photograph 2 Existing spillway

Photograph 3 Pipe outlet from spillway chamber

Photograph 4 Downstream face of weir

Photograph 5 Example of view along crest path

[redacted]

[redacted]

[redacted]

[redacted]

Appendix E. RARS Tier 2 Screening Breach and Consequence Assessment

Comments

- Extract from flood maps on opengov.uk
- Lidar plan and sections
- Rapid dambreak and routing

Comparison of dam break (LHS) and fluvial (RHS) flood extent 3.5km downstream (Bramhall green)

[redacted]

Comparison of dam break (LHS) and fluvial (RHS) flood extent 6.8km downstream (Demmings)

[redacted]

D.2 Houses shown as flooded by river flowing

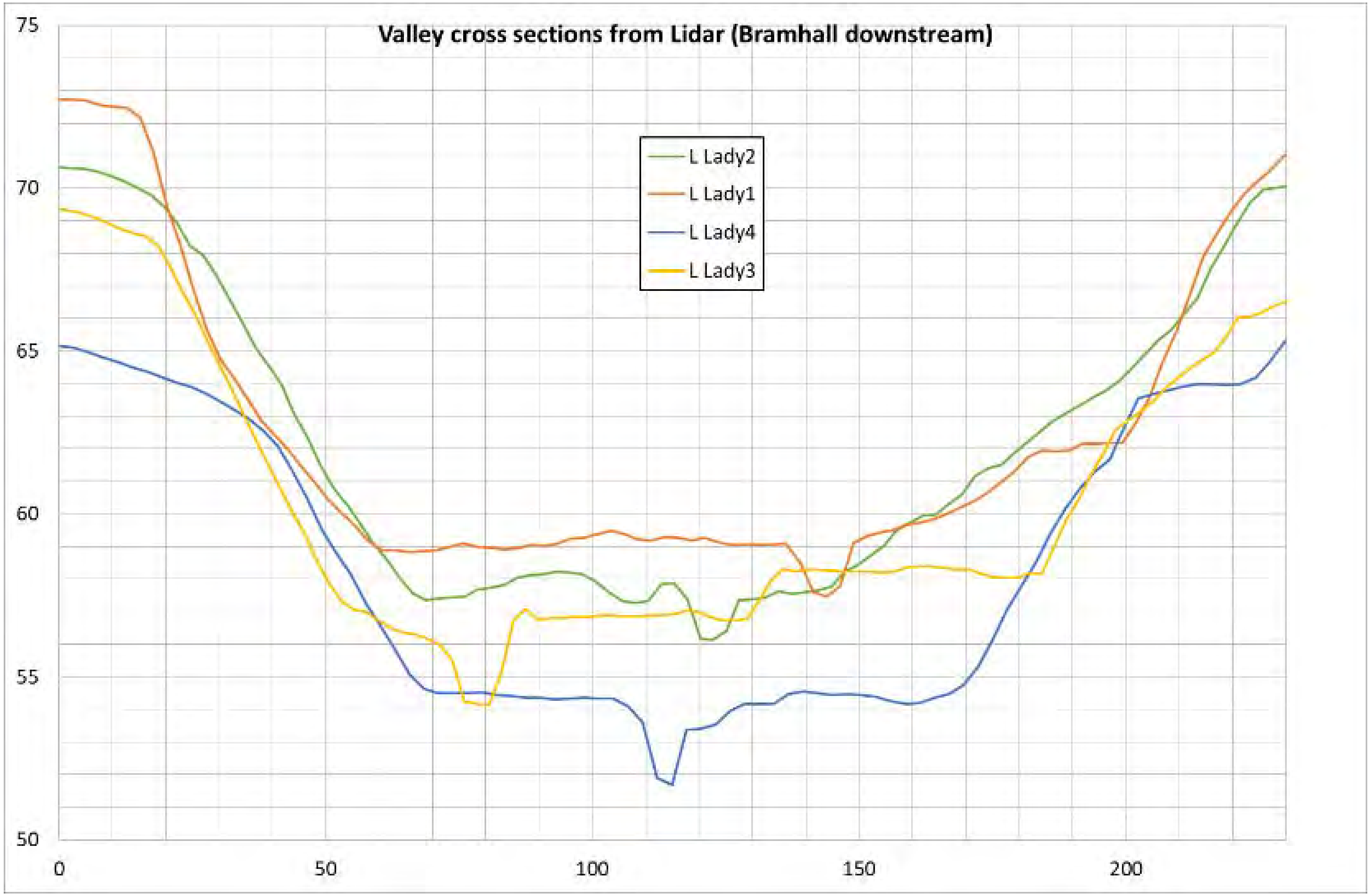
Maps ordered D/S to U/S

[redacted]

[redacted]

[redacted]

[redacted]


Valley cross sections from Lidar (Bramhall downstream)
L Lady2
L Lady1
L Lady4
L Lady3
75
70
65
60
55
50
0
50
100
150
200

STEP 2: RISK ANALYSIS
Sheet 2c(i): Dambreak Hydrograph

Dam name	Poynton Pool
Grid ref.	0
Calculation Number/ Description	2020 Flood Study

Return to main menu 

	Symbol	Units					
a) Determine subject dam breach hydrograph							
Failure conditions			**Sunny day**	**Rainy day**		**below road**	**EA 2009 (0.5m above**
Physical characteristics of subject reservoir (from 0_subject dam)							
Height of peak reservoir level above base of dam	H	m	1.2	1.6		4.31	6.5
Reservoir Capacity	V	m^3	43,000	69,813		94,102	188,500
Initial estimate							
Breach discharge as Froehlich, 1995							
Peak $Qp=0.607(V)^{0.295}(H)^{1.24}$	Q_p	m^3/s	18	30		109	223
Time base as RMUKR, Section 5.2.2							
Time to peak discharge, Tp=120(H)	Tp	sec	149	196		517	780
Time to end of discharge (so hydrograph vol. = reservoir vol.)	Te	sec	4,662	4,674		1,728	1,694
Where warning message in this row (i.e. Te < 2Tp) correct by one of the following							
1. Keeping Qp unchanged, reduce Tp (Te= 2Tp), where warning message, as Tp<40H	Tp	sec					
2. Assuming Tp=40H reduce Qp until volume of	Tp	sec					
flood hydrograph equals reservoir volume	Qp	m^3/s					
Adopted dam break hydrograph	Qp	m^3/s	18	30	(C/F to sheet	109	223
at subject dam	Tp	sec	149	196	(C/F to sheet	517	780
	Te	sec	4,662	4,674		1,728	1,694

STEP 2: RISK ANALYSIS
Sheet 2c(iii): Dambreak Attenuation Downstream

Dam name	Poynton Pool
Grid ref.	0
Calculation Number/ Description	2020 Flood Study

Return to main menu 

	Symbol	Units	0	1	2	4	5	6	7	8	9	Remarks
				Downstream end of Reach No								
OS Grid Ref			0									
Distance downstream of dam		km	0	0.2	1.3	3	5	7				Discussed with Paul Roberts on Screen and ta ked through locations the locations
River bank level (base of cross section)		mOD	89	80.3	70	62.5	55	47				
Feature defining end of reach			Dam	Poynton Brook	Norbury Brook	Upper Lady	Lower Lady	Denninsg road				
Topography of zone												
Note any other special feature in zone that would affect flow and/ or damage												
Length of zone	x	m		200	1100	1700	2000	2000	-7000	0	0	
Channel geometry of valley in each zone												
Average slope of base of valley that would be inundated	S_o	%		4.35%	0.94%	0.44%	0.38%	0.40%	-0.67%	#DIV/0!	#DIV/0!	
Manning's n	n			0.075	0.075	0.075	0.075	0.075				
Channel base width (trapezoid)	W_B	m		30.0	90.0	90.0	90.0	90.0				reach 1 by hosues
Channel side slopes H:V				10.00	10.00	10.00	10.00	10.00				

Estimated flow conditions

Case 1 : Wet day - breach to road

Reach Number	Symbol	Units	0	1	2	4	5	6	7	8	9	Remarks
Flooded width (adjust estimate until **ERROR** below is acceptable, or see workbook comment)	B1	m		39.1	97.6	95.6	94.4	93.9				
Attenuation factor k				0.01	0.1	0.1	0.05	0.05				
Attenuation length scale	La	m		14,115	6,517	2,122	3,303	11,117	#DIV/0!	#DIV/0!	#DIV/0!	
Discharge	Qp(x)	m^3/s	30	29	25	11	6	5	#DIV/0!	#DIV/0!	#DIV/0!	
	Q/w	$m^3/s/m$		0.8	0.3	0.1	0.1	0.1	#DIV/0!	#DIV/0!	#DIV/0!	
Time period at > half discharge	T_h (=T_e/2)	sec	2,331	2,364	2,799	6,236	11,425	13,677	#DIV/0!	#DIV/0!	#DIV/0!	
Max water depth (from Manning)	D	m	**0.8**	**0.5**	**0.4**	**0.3**	**0.2**	**0.2**	**#DIV/0!**	**#DIV/0!**	**#DIV/0!**	assume d<<width
ERROR - initial estimate of width (B1) as percentage of width implied by depth				0%	0%	0%	0%	0%	#DIV/0!	#DIV/0!	#DIV/0!	
Flood level		mOD	89.83	80.76	70.38	62.80	55.22	47.19	#DIV/0!	#DIV/0!	#DIV/0!	

STEP 2: RISK ANALYSIS

Sheet 2d(i): Estimate of Population at Risk, Average Social Life Loss & Direct Third Party Flood Damage

Dam name	Poynton Pool
Grid ref.	0
Calculation Number/ Description	2020 Flood Study

Return to main menu

	Symbol	Units	0	1	2	4	5	6	7	8	9
			Downstream end of Reach No								
Physical description and peak flow conditions (C/F from Sheet 8.5)			0	1	2	4	5	6	7	8	9
Distance downstream of dam		km	0	0.2	1.3	3	5	7	0	0	0
OS Grid Ref			0	0	0	0	0	0	0	0	0
Feature defining end of reach			Dam	Poynton Brook	Norbury Brook	Upper Lady	Lower Lady	Denninsg road	0	0	0
Note any other special feature in zone that would affect flow and/ or damage			0	0	0	0	0	0	0	0	0

	When occupied	Occupancy	Time averaged PAR			Assumed property values: Property destroyed	Inundation damage only
Residential: Number of PAR/ property	2.35	80%	1.88	/property	Residential (£k/property)	232	44
Non- residential: PAR as area (m2)/ occupant	14.5	25%	58	m2/ occupant	Non-residential (£k/m^2)	1.740	0.880

State which case on Sheet 8.5 is to be used — Case 1 — Wet day - breach to road

	Symbol	Units	0	1	2	4	5	6	7	8	9
Discharge	Qp(x)	m^3/s	30	29	25	11	6	5	#DIV/0!	#DIV/0!	#DIV/0!
Flooded width	B1	m		39	98	96	94	94	0	0	0
Max water depth (from Manning)	D	m	0.8	0.8	0.4	0.3	0.2	0.2	#DIV/0!	#DIV/0!	#DIV/0!
Other measures of forcefulness of flow											
Average velocity	V	m/s	2.8	1.4	0.7	0.4	0.3	0.3	#DIV/0!	#DIV/0!	#DIV/0!
Velocity x depth	VD	m^2/s	2.4	1.2	0.3	0.1	0.1	0.1	#DIV/0!	#DIV/0!	#DIV/0!
Discharge/ flooded width	Q/W	m^2/s		0.8	0.3	0.1	0.1	0.1	#DIV/0!	#DIV/0!	#DIV/0!
Time for peak dambreak flood to reach key points											
Time to travel reach		mins		2	17	51	94	113	#DIV/0!	#DIV/0!	#DIV/0!
Cumulative time to end of reach		mins		2	19	70	164	277	#DIV/0!	#DIV/0!	#DIV/0!

Number of properties vulnerable to Flood Damage in Each Zone

[redacted] From Table 9.3 for Aroad

	Units	1	2	4	5	6	7	8	9	Sub-total	Remarks
Residential Properties at Risk in Each Zone		No. of properties in each damage category:									
roperty destroyed	No.	0	0	0	0					0	
tion damage only	No.	4	4	20	0	0				28	
										28	No.
Non Residential I… Each Zone (build up in Table 1.6)		Total area (m^2) of non-residential properties in each damage category									
roperty destroyed	m^2	0	0	0	0					0	
Inundation damage only	m^2	0	0	200	0					200	
										200	m2

Population (pedestrians) at risk (water > 0.5m deep) (PAR)

	Units	1	2	4	5	6	7	8	9	Sub-total	Remarks
Number of residential properties	No.	4.0	4.0	20.0	0.0	0.0	0.0	0.0	0.0	28.0	
Total area of non-residential properties	m^2	0.0	0.0	200.0	0.0	0.0	0.0	0.0	0.0	200.0	
Residential properties: Max PAR (either accept equation, or overwrite)		9.4	9.4	47.0	0.0	0.0	0.0	0.0	0.0	65.8	
Non residential : Max PAR (either accept equation, or overwrite)		0.0	0.0	13.8	0.0	0.0	0.0	0.0	0.0	13.8	
On transportation routes (time averaged)		6.3		6.3		6.3				18.9	
Comment/Source											
Other open air e.g. playing fields, recreational areas (time averaged)										0.0	
Comment/Source											
Assessed total PAR (time averaged)		13.8	7.5	47.3	0.0	6.3	0.0	0.0	0.0	75.0	

Average Social Life Loss (ASLL)

	Units	1	2	4	5	6	7	8	9	Sub-total
Warning Scenario 1		All reaches receive no warning								
		First community, where warning raised:			N/A					
Has warning been given to the reach?		No warning	No warning	No warning	No warning	No warning	No warning	No warning	No warning	
Fatality rate - breach down to raod	%	0.4%	0.2%	0.1%	0.1%	0.1%	#DIV/0!	#DIV/0!	#DIV/0!	
ASLL		0.06	0.01	0.05	0.00	0.00	#DIV/0!	#DIV/0!	#DIV/0!	0.13
Warning Scenario 2		All reaches receive a full warning								
Fatality rate - breach down to raod	%	0.2%	0.1%	0.1%	0.1%	0.1%	#DIV/0!	#DIV/0!	#DIV/0!	
ASLL		0.03	0.01	0.05	0.00	0.01	#DIV/0!	#DIV/0!	#DIV/0!	0.09

Adopted likely loss of life — *Scenario 1* 0.13 — Warning time 1.5 hours: — *Scenario 2* 0.09

Estimated Cost of Damage

	Units	1	2	4	5	6	7	8	9	Sub-total
b) Per Residential Property per zone (average)		Cost of damage £k - Residential								
Total damage per zone	£k	176	176	880	0	0	0	0	0	1,232
c) Total per zone for non-residential property		Cost of damage £k - Non-residential								
Total damage per zone	£k	0	0	176	0	0	0	0	0	176
d) Total by reach										
Total damage per zone	£k	176	176	1,056	0	0	0	0	0	1,408

Sum of third party property damages (£k)	**1,408**
Other damages (Optional at discretion of user e.g. replacement cost of dam) (£k)	
Cost of emergency services (£k)	**79**
Value carried forward to Sheet 2d(ii) (£k)	**1,487**

Appendix F. Environment Agency Reservoir flood mapping (Dec 2019)

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

Appendix G. **Options for spillway upgrade: sketches and costs**

	Poynton Lake - preliminary costing of four options					
	Items	Cost (£)	Cost (£)	Cost (£)	Cost (£)	Cost (£)
		Option 2	Option 3A	Option 3B	Option 3C	Option 3C
		Pass T1000 flood	0.6m pipe	Emergency spillway	Crest Resilence	D/S Embankmen
	Enabling works					
E1	reroute BT cable	£3,300	£3,300	£0	£0	£0
E2	Sewer diversion	£11,000	£11,000	£0	£0	£0
E3	Gas Diversion	£22,000	£22,000	£0	£0	£0
E4	Services - provisional items	£5,500	£5,500	£5,500	£0	£5,500
E5	Remove trees	£25,000	£25,000	£122,000	£98,000	£12,166
E6	Stump grind	£3,000	£3,000	£7,000	£6,000	£1,000
	Permanent works					
P1	excavation	£16,000	£6,000	£5,000	£2,000	£2,000
P2	Dispose of unsuitable	£42,000	£14,000	£18,000	£7,000	£7,000
P3	Imported fill (clay/ granular)	£14,000	£0	£23,000	£12,000	£14,000
P4	Grasscrete/ armourflex	£2,000	£0	£16,000	£0	£2,000
P5	1.4m Pipe and backfill	£50,000	£0	£0	£0	£0
P6	0.6m pipe and backfill open Ground	£12,000	£0	£0	£0	£0
P7	0.6m and backfill through Dam and Road	£0	£11,000	£0	£0	£0
P8	Manhole @D/S toe for 0.6m pipe	£0	£7,000	£0	£0	£0
P9	Manhole @D/S toe for 1.4m Pipe	£18,000	£0	£0	£0	£0
P10	Modifications to Existing Manholes	£2,000	£0	£2,000	£2,000	£0
P11	Replace fencing around gardens of "Carnforth"	£7,000	£0	£7,000	£0	£7,000
P12	Tarmac +Road Surface and Make up	£8,000	£8,000	£10,000	£0	£10,000
P13	New 1.2m pedestrian pavement	£1,000	£1,000	£0	£70,000	£1,000
P14	Mass concrete to foundation	£1,000	£1,000	£0	£0	£0
P15	concrete base slab	£5,000	£2,000	£0	£0	£0
P16	Concrete side walls	£5,000	£3,000	£0	£0	£0
P17	Reinforcement (125 kg/ m3)	£8,000	£3,000	£0	£0	£0
P18	Formwork	£9,000	£3,000	£0	£0	£0
P19	Sheet piling 3m deep	£21,000	£10,000	£0	£0	£0
P20	New Kerb edging	£0	£0	£1,000	£9,000	£1,000
	Temporary works					
T1	site compound _ set up, fencing etc.	£6,000	£6,000	£6,000	£6,000	£6,000
T2	welfare units	£14,000	£14,000	£11,000	£11,000	£11,000
T3	Groundwater control in excavations	£18,000	£18,000	£0	£0	£0
T4	Pumping to lower/ control RWL	£30,000	£30,000	£0	£0	£0
T5	Pontoons for sheet piling in reservoir	£28,000	£28,000	£0	£0	£0
T6	Traffic Management	£130,000	£54,000	£54,000	£0	£54,000
		£516,800	**£288,800**	**£287,500**	**£223,000**	**£133,666**
Minor items		£104,000	£58,000	£58,000	£45,000	£27,000
Contractors preliminaries		£130,000	£73,000	£72,000	£56,000	£34,000
		£750,800	**£419,800**	**£417,500**	**£324,000**	**£194,666**
Professional fees, surveys etc.						
	Engineers - design a and construction support	£160 000	£90 000	£90 000	£70 000	£40 000
	Ground investigation	£15,000	£15,000	£6,000	£3,000	£3,000
	Ecological surveys/ inputs	£20,000	£20,000	£20,000	£5,000	£5,000
	Other fees (land agents etc.)	£20,000	£20,000	£20,000	£1,000	£10,000
		£215,000	**£145,000**	**£136,000**	**£79,000**	**£58,000**
		£970,000	**£570,000**	**£560,000**	**£410,000**	**£260,000**
		30%	30%	30%	30%	30%
	Optimis bias	£291,000	£180,000	£170,000	£130,000	£78,000
		£1,261,000	**£750,000**	**£730,000**	**£540,000**	**£338,000**
					Total 3C (1 +2)	£878,001

Appendix H. **Potential for diversion of flood flows from Park Lane stream into Poynton Reservoir**

The scope of this initial options study includes the following

Flood alleviation

As discussed at the meeting on 09/03/20, we will assess what proportion of flood flow could be feasibly diverted from the stream running to the south, into Poynton reservoir. This information will help inform the potential viability of the proposed scheme. The study would not quantify the flood risk benefits downstream, nor investigate alternative flood storage sites further up the catchment. To carry out a meaningful assessment of the damages/benefits/economics in relation to downstream flood risk would require a separate study including hydraulic modelling, which we would be happy to quote for separately if required.

No information has been provided on previous studies, so the only information is that available on the internet, and the Notes of meeting on 9th March 2020. The former includes the Section 19 report by CEC on the July 2019 Flood Event in the Catchments of: Poynton Brook, River Dean, River Bollin, Harrop Brook and tributary of Todd Brook. [redacted] and describes the watercourses as shown in table E.1 and Figure E.1 The description of flooding is consistent with the surface water flood risk maps shown on the Internet and reproduced in Figure E.2. The reports states (Section 1.2.2) that the reoccurrence interval for rainfall for the July 2019 event was variously 1 in 200, 1 in 78 and 1 in 153 years at the three rain gauges in the catchment areas

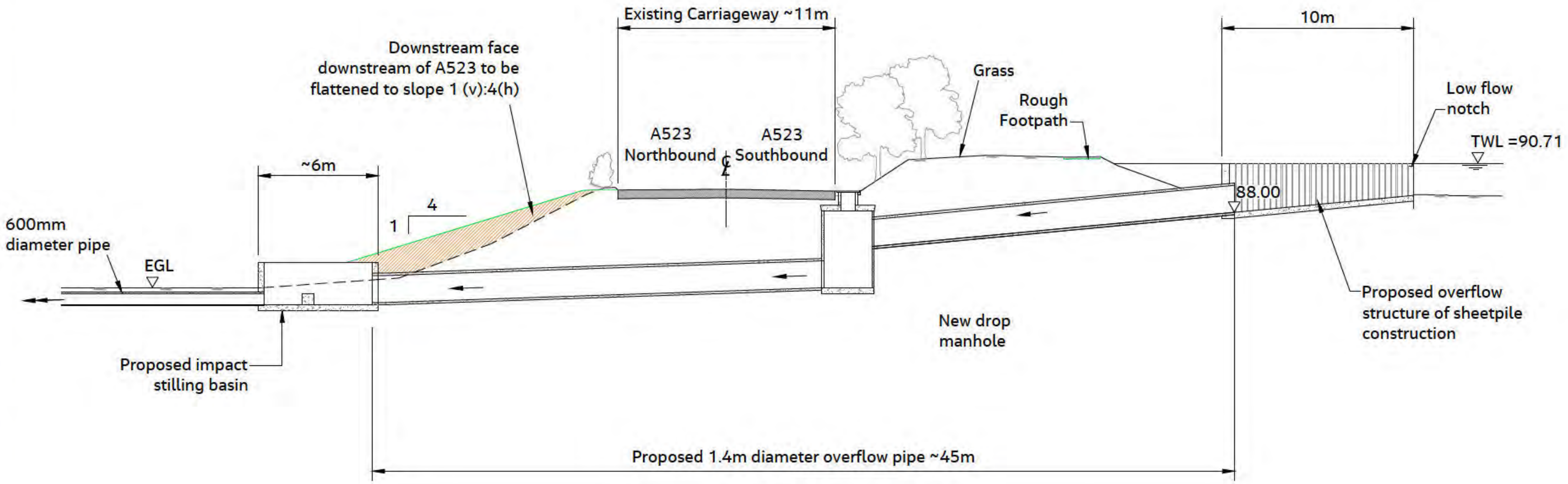
Table E-7-1 Sources of flooding in July 2019 event as reported in CEC section 19 report

watercourse	Type	
Unnamed in Reservoir catchment		described in Section 4.1.8 of report, with flooding shown on Figure 35 and described as "*In these areas flooding was reported from various mechanisms including surface water, sewer and ordinary watercourse. Teams from Cheshire East Highways and United Utilities are working with residents to resolve these issues.*
Park lane Stream	Ordinary w/c	Catchment is 4 km^2 at the point where diversion to Poynton Reservoir occurs Flooding on July 2019 description in section 4.1.6 (page 30-39) of section 19 report – with locations of flooding reported as follows • [redacted] • [redacted] • [redacted] At this location there is a bifurcation with flow into the reservoir (Figure 26). [redacted] ith route of ordinary watercourse shown on Figure 27 and 29
Coppice Stream	Ordinary w/c	Flooding described ins section 4.1.7
Poynton Broom	Main river	River level gauge where cross A523 https://riverlevels.uk/poynton-brook-bainbridge-poynton#.YDvNHWj7S70
Booth Green Brook	Main river	

A screening estimate of the magnitude of the T1000 flood was given in Table 4.3, and when adjusted pro rata to that at Poynton reservoir is around 14 m^3/s. The T100 flood would be about 60% of these, based on Table 3.2 of FRS4, so the 1 in 100 chance pr year flood on Park Lane stream at the reservoir offtake could be about 8m^3/s.

[redacted]

[redacted]



Section Through Proposed New Overflow

Rev	Rev. Date	Purpose of revision	Orig	Check'd	Rev'd	Apprv'd
0	Feb 21	For Information	RHK			

JACOBS

95 Bothwell Street Glasgow, G2 7HX

Client: Cheshire East Council

Project: Poynton Lake

Spillway Upgrade : Initial Options Report

Drawing title: Option 2 Create New Overflow to Pass Design Flood (T1000) in FRS4 Section

Drawing status: For Information

Suitability:

Scale: As Shown @ A3 — DO NOT SCALE

Jacobs No.: BRJ10627

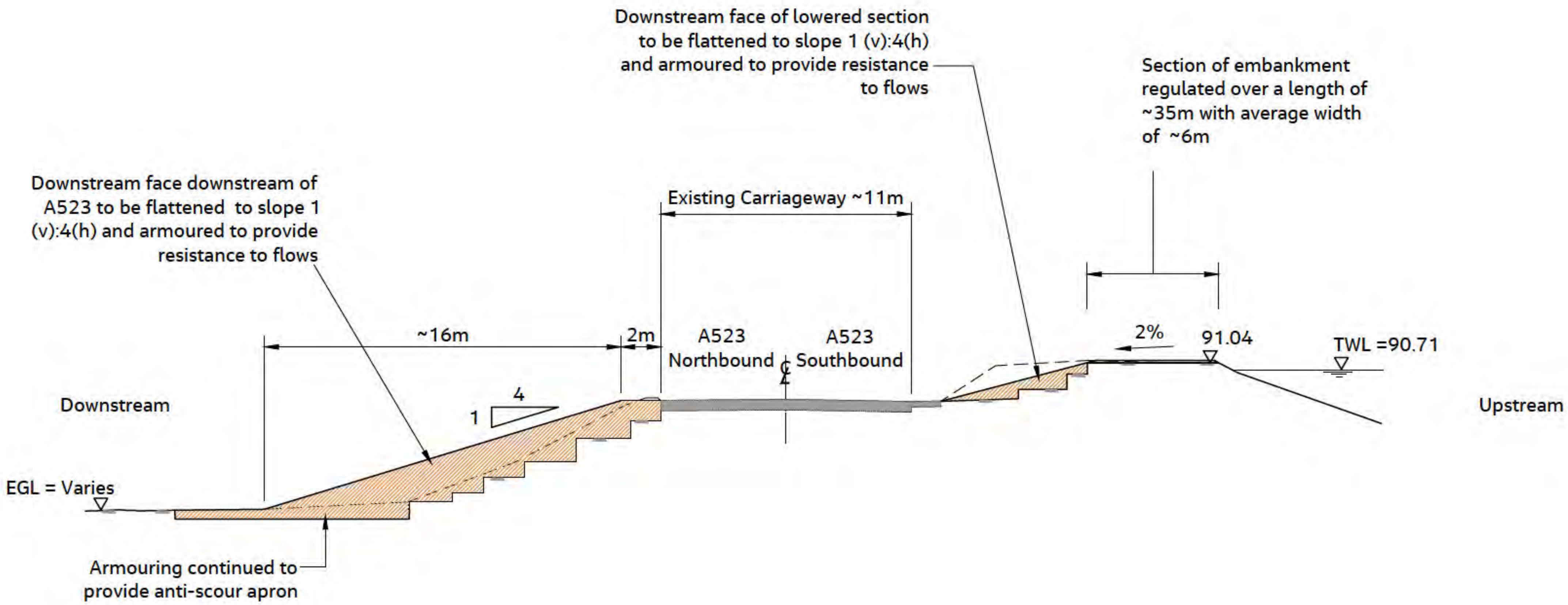
Client No.:

Rev: 0

Drawing number: BRJ10627-JAC-RP-DR-002

[redacted]

[redacted]



Section Through Proposed Resilience Option

0 5 10 15 20 25 m

SCALE 1 : 250

Rev	Rev. Date	Purpose of revision	Orig	Check'd	Rev'd	Apprv'd
0	Feb 21	For Information	RHK			

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Client
Cheshire East Council

Project
Poynton Lake
Spillway Upgrade : Initial Options Report

Drawing title
Option 3B
Increase Resilience to Overflow

Plan

Drawing status: For Information
Suitability

Scale: As Shown @ A3 — DO NOT SCALE
Jacobs No. BRJ10627
Client No.
Rev 0

Drawing number
BRJ10627-JAC-RP-DR-005

[redacted]

This can be compared with the current capacity of the offtake to Poynton reservoir as around 0.3 m^3/s (description of diversion system in section 5.2 of 2019 flood study, and diversion flows included with results in Section 6.) It is concluded that it might be practicable to say double this to 0.6m^3/s, but this would only be 4% of a T100 flood so is unlikely to be economic.

Figure E.1 Watercourses posing flood risk to Poynton

Figure 12 Indicative catchment of Poynton Brook

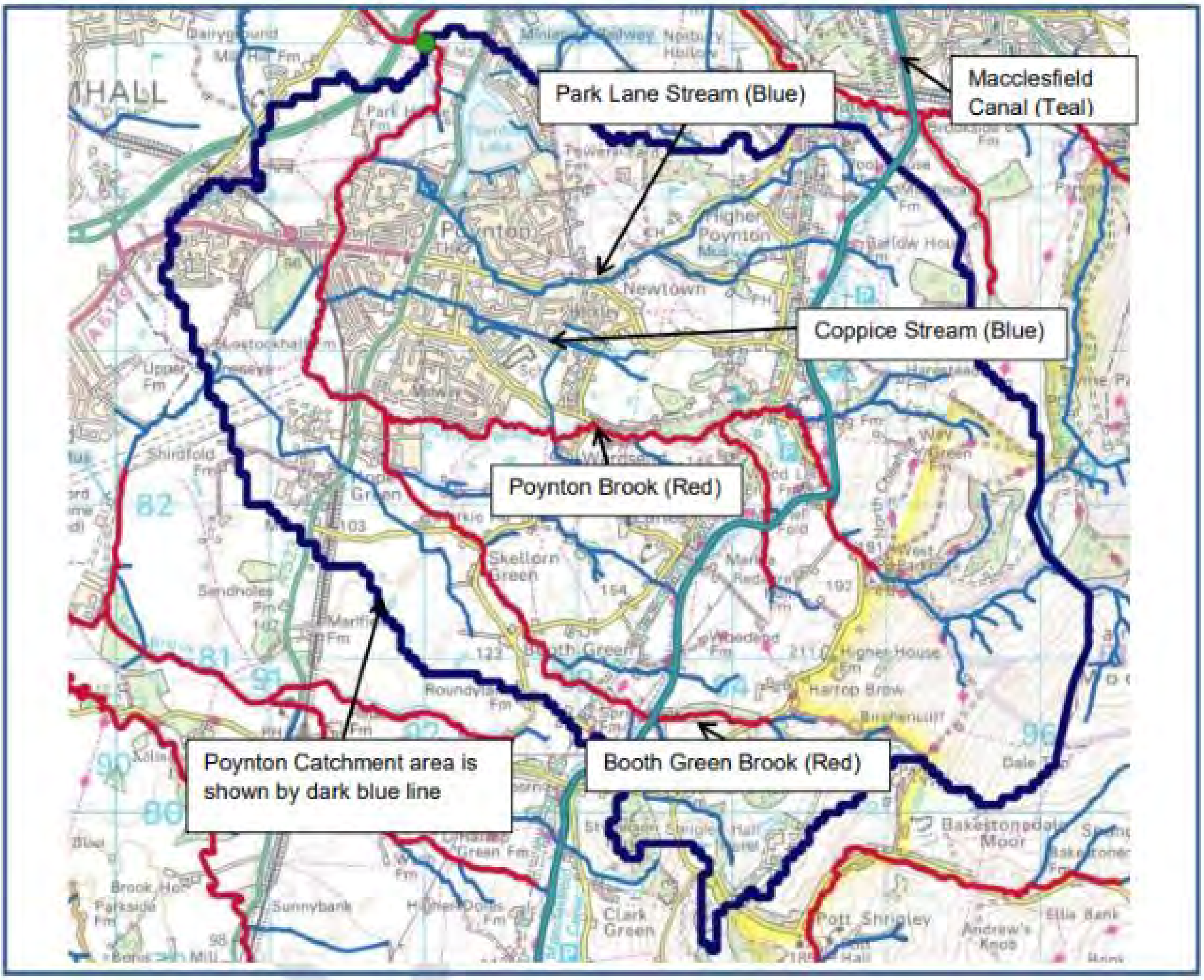

Figure E.2 surface water flood risk in Poynton town, as shown on opengov.uk

Flood risk

Low risk: depth

Location

poynton

[redacted]

Surface water flood risk: water depth in a low risk scenario

Flood depth (millimetres)

Over 900mm 300 to 900mm Below 300mm