

MAIDSTONE BOROUGH COUNCIL

Mote Park Lake

**Assessment of whether the risk of failure of the dam
during flood events is
“As low as reasonably practicable” (ALARP)**

April 2017

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Executive Summary for Undertaker

Mote Park Lake is a 200,000m³ reservoir retained by a 6m high embankment dam, in the middle of Maidstone. The reservoir was constructed in 1793 and forms an important part of Mote Park, a public park.

A ten yearly review of dam safety under Section 10 of the Reservoirs Act 1975 in June 2014 included a mandatory recommendation in the interests of safety that:

“The spillway capacity is increased to reduce risk of failure due to overtopping as low as reasonably practicable (ALARP)”

Stillwater Associates have undertaken a review of the existing spillway capacity and identified options for increasing the capacity.

The consequence of failure of Mote Park Lake would be that Turkey Mill Business Park and areas of housing along the River Len would be flooded. The dam retaining Mote Park Lake is therefore assessed as Flood Category A, where a failure would “threaten lives in a community”.

The current capacity of the spillway (with the sluice gates closed) as the dam starts to overtop is around 11m³/s which represents a flood with an annual chance of 1 in 100. This is well short of the PMF value of 336m³/s required as a safety check flood for Category A dams (and also a 1 in 10,000 chance per year flood of 119 m³/s which would be the safety check flood for Category B dams). If all four sluice gates were operational and fully open the capacity is doubled to 22m³/s, reducing the imminent failure capacity to around a 1 in 200 chance per year and this was presumably the historical capacity.

To meet the recommended standard the usual engineering approach would be the design and construction of a new, enlarged concrete spillway, both to reduce the risk of damage and the overall chance of failure. However, ICE 2015 recommends that where an existing dam does not meet current standards, then a risk based approach may be adopted to assess the extent of upgrading to ensure that the cost is proportionate.

The purpose of this report is to describe a range of practicable options and their costs and to assess which option best reduces risk against an “as low as reasonably practicable” (ALARP) criterion. This is a risk based approach assessing whether the cost of any works would be proportionate to the reduction in risk achieved.

The risk of failure of a dam (product of “annual probability of failure” times “consequences if it did fail”) can never be reduced to zero, but it should be reduced “as low as reasonably practicable” (ALARP) (www.hse.gov.uk/risk/theory/alarp.htm). This means that dams with higher consequences of failure are required to satisfy higher design standards.

Following a risk based approach undertaken here the consequences to the public if Mote Park Lake failed during a flood and triggered release of the impounded water have been assessed as that around 3 people would be killed and £5M of property damage. However, the fluvial flood itself would have caused damage even without dam failure, so the proportion due to dam failure is likely to be around half these figures.

Following HSE guidelines the level of risk is therefore in the upper part of the ‘as low as reasonably practicable’ (ALARP) zone where works should be carried out to reduce the risk, when the cost is proportionate to the reduction in risk achieved.

Guidance on how to decide what works are proportionate is given in Sections 10.3 and 10.4 of the Guide to Risk Assessment for Reservoir Safety Management (Environment Agency, 2013), with an example economic ALARP calculation in Appendix A.

In terms of the practicable options to reduce the likelihood of dam failure the economic assessment is summarised in Table E.1 below and this shows that:

- a) Two concrete auxiliary spillways (Options A2-40 and C2-50) with the crest of the non-overflow section 1.7m above the auxiliary spillway crest (freeboard) would pass a 1 in 60,000 chance per year flood and be just disproportionate in cost.
- b) However, the Council does not own the downstream face of the dam, and the landowner is likely to be resistant to Option A which would remove the existing tree backdrop to Turkey Mill Pond, used as a wedding venue by the downstream landowner. This leaves Option C which if constructed in concrete with a 1.7m freeboard (C2-50) would pass a 1 in 10,000 chance per year flood and be proportionate in cost.
- c) It could be argued that a concrete chute would be unacceptable in a Grade II listed park, which would leave a grasscrete option (C1-50) with 1.2m freeboard (the maximum depth of overtopping grasscrete can withstand) which could pass a 1 in 3000 chance per year flood and be proportionate in cost

The selection of what is “reasonably practicable” would be to construct as close to the engineering standard as possible, unless compelling evidence is provided which shows that this is not “reasonably practicable”. The tests include consideration of the reduction in risk to life which would be achieved by its implementation.

In conclusion some upgrading works are therefore unavoidable, and the following actions are required, otherwise the Environment Agency will commence enforcement proceedings against the council:

- **A decision as to which option/works will be implemented by 6th June 2017, to be confirmed in writing by the Council to the All Reservoirs Panel Engineer (and thus to the Environment Agency)**
- **Completion of the works within three years i.e. by June 2020.**

It is noted that the finalised layout must be approved by an All Reservoirs Panel Engineer who should oversee the design and construction in the role of Qualified Civil Engineer (QCE). This oversight would then allow him to issue the Section 10(6) certificate under the Reservoirs Act 1975 confirming that the recommendations have been implemented.

Table E.1: Summary of shortlisted options

Option	Works involved	Budget project cost (Appendix E)	Annual chance of dam failure with release of reservoir	Cost to save a life (Note 1)
	Existing situation		100	n/a
A1-40	40m wide grasscrete auxiliary spillway, on embankment with 1.2m freeboard	£1.2M	2000	3.3
A2-40	40 m wide reinforced concrete auxiliary spillway on embankment with 1.7m freeboard	£1.7M	8,000	4.5
B	Strengthen crest to inhibit breach	£0.7M	1,500	1.9
C1-50	50m wide grasscrete auxiliary spillway, on abutment with 1.2m freeboard	£1.4M	4,000	3.6
C2-50	50m wide concrete auxiliary spillway, on abutment with 1.7m freeboard	£1.9M	20,000	5.1
A2-40, C2-50	Both concrete options	£3.6M	80,000	9.3
Notes.				
Cost becomes grossly disproportionate when cost to save a life exceeds £8.5M – see Appendix A				

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1 GENERAL

1.1 General

Mote Park Lake is formed by a dam across the River Len which is a tributary of the River Medway. The Ordnance Survey grid reference of the dam retaining the lake is TQ 77433 55347, and the postcode of the site entrance is ME15 7SX. The dam is immediately upstream of Maidstone and effectively provides some flood attenuation to Maidstone; there is around 2m of flood freeboard in the lake which will attenuate floods up to around a 1 in 100 chance per year.

The Wikipedia description, based on the Historic England listing of the Park states that:

Between 1793 and 1800 the original Mote House was demolished and a new mansion constructed, designed by Daniel Asher Alexander.[12] At the same time the River Len was dammed to form a lake. The addition of internal roadways, walls, a boathouse and a bridge (the 'Great Bridge') over the lake stretched the financial resources of Charles Marsham, 3rd Baron Romney.[4] Eventually the family gathered enough funds to expand the property and the park reached the size it is today, approximately 180 hectares. The Great Bridge was demolished and the lake itself expanded to around 30 acres (120,000 m²) (to its current size, believed to be soon after 1938 based on information with HLF application)

1.2 Purpose of report

The 2014 Inspection report of the lake under Section 10 of the Reservoirs Act identified that the reservoir does not meet current standards for safety, as defined in Floods and Reservoir Safety 4th Ed. issued by the Institution of Civil Engineers. The Section 10 report recommended matters in the interests of safety that comprised:

- a) *A topographic survey is carried out to establish the as-built levels of the 2012 works, including the openings of the culverts under the park access road and also the profile and extent of the land downstream of the fence along the crest (owned by Turkey Mill Investments)*
- b) *The spillway capacity is increased to reduce risk on failure due to overtopping as low as reasonably practicable (ALARP). This should be carried out as a study to identify options and define/agree what works would be proportionate in cost to the reduction in risk achieved, followed by the works.*

The timescale to complete these recommendations was that the ALARP analysis study should be completed within two years of the Section 10 report (which was issued on 6th June 2014) i.e. by 6th June 2016, and the physical works completed within three years of the study i.e. by June 2019. The Panel AR Engineer has suggested that these both be deferred one year, and it is understood that the Environment Agency have accepted this.

It was agreed that Stillwater Associates would carry out an ALARP feasibility study to determine options to reduce risk of overtopping failure, and whether the cost would be proportional to the reduction in risk. This report comprises that study, carried out in accordance with Stillwater Associates letter of 2nd August 2016. This report is important not only as a basis of the current assessment of tolerability of flood risk, but because in future ten yearly safety reviews it will form a benchmark for review of adequacy of safety of the dam.

A preliminary list of options to reduce the probability of failure due to overtopping was given in Table 5.6 of the Inspection Report and is extended in this report.

This report is limited to consideration of failure modes due to extreme floods, and does not consider other failure modes, such as earthquake or sunny (dry) day failure modes.

This report was produced in draft and discussed at workshops with council staff, and both senior officers and councillors before being finalised.

1.3 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 A.

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” (defined by EU and HMG), 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 the CSL of £1.7M and a disproportionality factor of 5.

1.4 Available data

Available data is summarised below:

Table 1-1: Summary of available data

Date	Originator	Description
April 2010	Ian Farmer Associates	Exploratory holes at various locations around Mote Park. BH2, 2A, WS4 and WS5 near upstream end of spillway channel
2010	MBC	HLF Grant application for Mote Park Regeneration. In several volumes.
Sept 2016	J C White	Topographic survey (extend existing surveys)
October 2016	Stillwater Associates	Flood study

The following information is understood to exist, but is no longer available and could not be used in this study:

Date	Originator	Title Description
2010	JBA	Report on basis of fluvial model
2011	Environment Agency / JBA	Computer model of River Len, including Mote Park Lake

2 Site characterisation

2.1 Introduction

This section summarises the environment in which the dam and reservoir are located, in terms of geology, hydrology and environmental considerations.

2.2 Geology

The published geology (BGS geology of Britain viewer, GeoIndex) is summarised below. There are no nearby boreholes on BGS GeoIndex. An extract from the published 50k geological map is included in Appendix B.

Formation	Description in BGS Lexicon	Thickness	Location at the dam
Alluvium			Strip around 100m wide down middle of valley
Hythe (lower Greensand)	In Kent and eastern Sussex the formation comprises, alternating sandy limestones ("Ragstone") and glauconitic sandy mudstones (Hassock). The base is taken at the incoming of fine- to medium-grained sands above the silty or sandy clays of the Atherfield Clay, and is generally sharply defined.	18 to 100m.	Upper part of abutments
Atherfield Clay (lower Greensand)	Generally massive yellowish brown to pale grey sandy mudstone throughout most of its outcrop, with an impersistent phosphatic pebble bed with vertebrate bones, gritty sandstone or very shelly sandy mudstone with glauconite, at the base	Thins eastward to about 10m around Sevenoaks and Maidstone.	Abutments
Weald Clay	Dark grey thinly-bedded mudstones (shales) and mudstones with subordinate siltstones, fine- to medium-grained sandstones, including calcareous sandstone (e.g. Horsham Stone Member), shelly limestones (the so called "Paludina Limestones") and clay ironstones.	240m south of Maidstone	Base of valley, below alluvium

2.3 Catchment

The catchment parameters are as follows. The geology of the catchment, as shown on the online gauging station entry, is Lower Greensand, Folkestone and Hythe Beds, the catchment running along the south side of the North Downs.

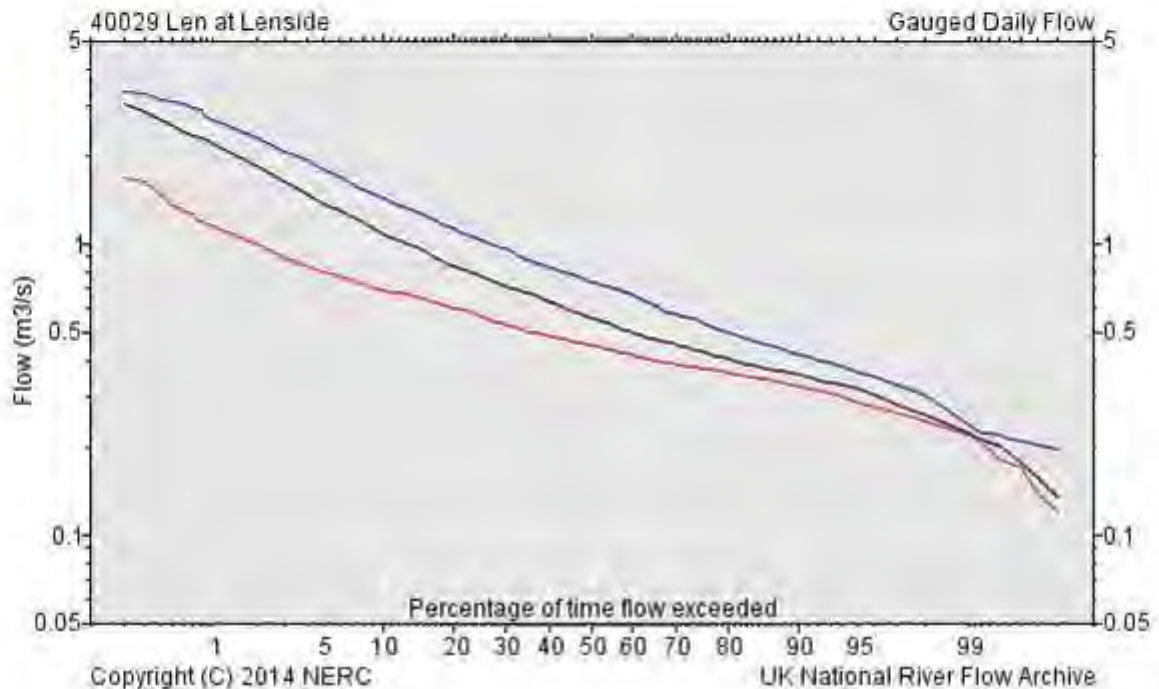
Table 2-1: Catchment parameters from FEH cd

Parameter	Symbol	Units	Value	Source
Catchment area	AREA	km ²	64.7	FEH cd
Standard runoff %	SPRHOST	%	20.1%	
Annual rainfall	SAAR	mm	711	
Urbanisation of catchment (2000)	URBEXT2000	%	7	

2.4 Normal river flows

There is a gauging station around one kilometre downstream (under Sainsbury's car park), with the daily flow duration curve shown below (data 1984 to 2015). It can be seen that the normal (Q50) flow is 0.5m³/s, whilst the winter wet day flow (Q10) is 1.5m³/s.

Figure 2.1: Flow duration curve for nearest EA gauging station (River Len at Lenside)



2.5 Flood Estimates

Flood estimates are shown in Table 2-2 and Figure 2.3, being taken from the 2017 Stillwater Flood Study Report. Flood hydrographs are given on Figure 2.2.

Maximum recorded daily flow over the 30 years of record at the gauging station just downstream was 4.4m³/s on 31st Dec 2001. The CEH file states “*Most flows contained but most peaks truncated at ~3.8 cumecs... Dec 2002 maximum generated by notable storm but flow is indicative only*”. This is similar to the estimated peak flows in a 1 in 10 chance per year flood, and provides an indication that the 1 in 1000 peak flows reported by URS as 12m³/s are too low i.e. inconsistent with regional growth curves.

Support for (relatively) low flows up to say the 1 in 100 event, with significant increase for more extreme floods is provided by consideration of the reservoirs at Leeds Castle, where the catchment is around one third of that at Mote Park Lake. The embankment for the access road into the castle forms a dam across the valley, which retains a shallow reservoir (The Great Water) and provides 60,000m³ of flood storage. That dam is overtopped and no longer attenuates natural floods for floods of the order of 1 in 150 chance per year and higher, when pass forward flows would increase. The same effect is likely to happen at other infrastructure embankments which cross the river.

Table 2-2: Summary of estimates of magnitude of floods

Event, methodology	Rainfall depth	Net runoff volume	Peak catchment inflow	Peak spillway channel flow	Peak dam crest overflow	Peak flow along dam crest track	Outflow (sum)	Maximum flood level	Freeboard to lowest point on dam crest profile (21.69 mOD)
	mm	m3x106	m3/s	m3/s	m3/s			mOD	m
Summer PMF:	231	7.02	336	26.7	294	15.5	336.2	23.43	none
Winter PMF:	130	5.98	255	24.0	219	12.2	255.2	23.24	none
10,000 year flood - FEH2013 rain – SPR = 33.6%:	177	5.04	164	21.1	134	8.6	163.7	22.99	none
10,000 year flood - FEH2013 rain:	177	3.65	119	20.0	92.4	6.7	119.1	22.84	none
1000 year flood - FEH2013 rain:	125	2.28	74.9	19.1	51.1	4.8	75	22.65	none
150 year flood - FEH2013 rain:	98	1.59	47.7	18.2	26.2	3.3	47.7	22.49	none
150 year ReFH2 – FEH2013 flood – 11.25hr event:	64	1.43	14.5	11.6	0.0	0.0	11.6	21.63	0.06
10 year flood - FEH2013 rain 9.25 hour event:	49	0.65	22.0	15.4	1.8	0.9	18.1	22.08	none
10 year flood - FEH2013 rain 13.25 hour event:	54	0.74	22.1	15.8	2.7	1.1	19.6	22.14	none
10 year ReFH2 – FEH2013 flood – 11.25hr event:	32	0.62	6.61	4.45	0.0	0.0	4.45	20.80	0.89

Figure 2.2: Beach hydrograph superimposed on T1000 flood hydrograph

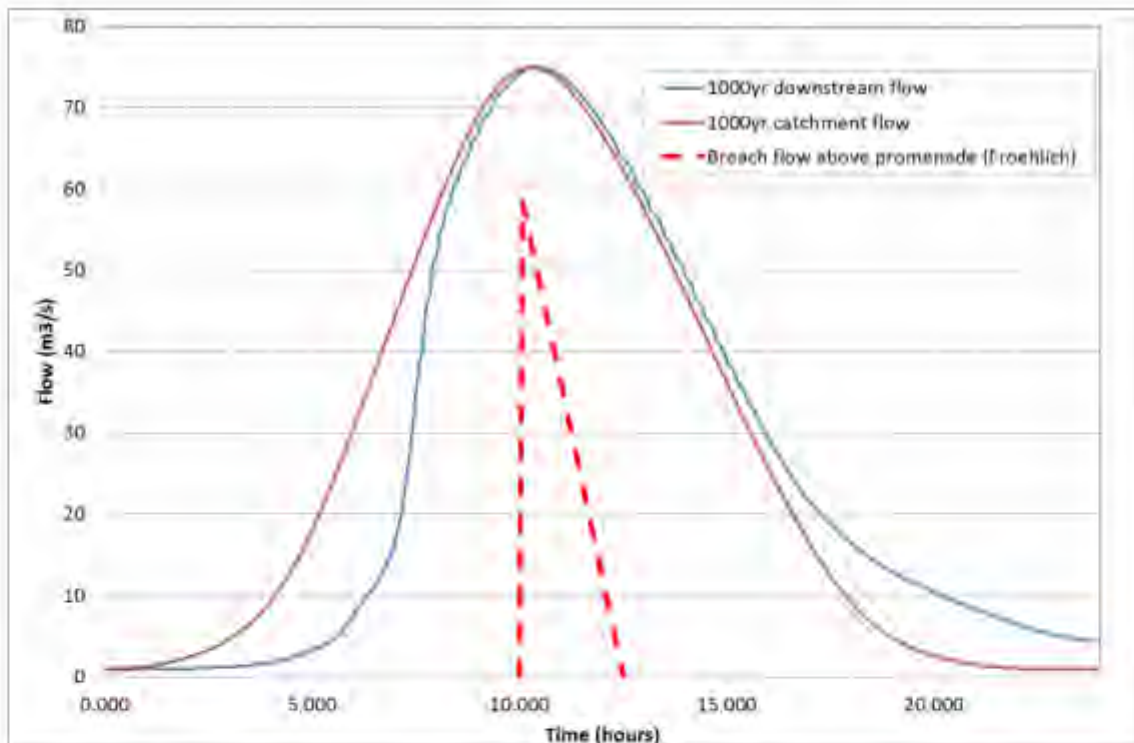
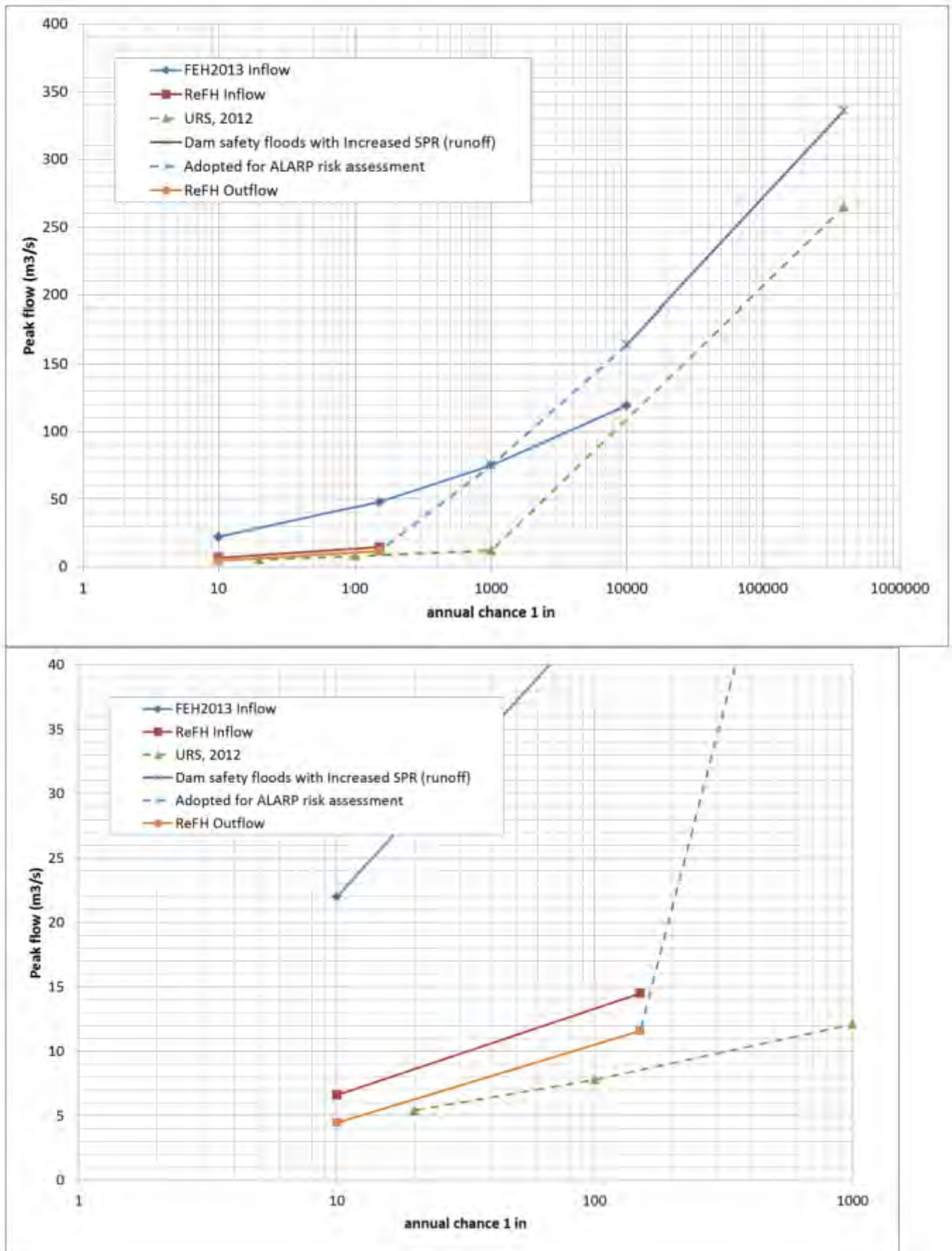


Figure 2.3: Peak flood inflow vs annual chance of occurrence



2.6 Valley Downstream of the dam

2.6.1 Turkey Mill Pond

The only available information is given in Section 3.9.2 and Appendix C of the 2014 Section 10 report. There are actually three water bodies:

Water body	Main dimensions	River control structures
Turkey Mill Pond	12,000m ³ reservoir as Table 3.8 of 2014 S10 (Height approx. 3.5m)	a) Into River Len - Unknown, in masonry building on right abutment b) Into Mill Race – photos 25, 26 in 2014 S10
Mill race (left abutment)	90m long	Photo 29 in 2014 S10. Believed to be disused
River Len	115m long x 15m wide channel, plus channel along downstream toe of Mill Pond	See photos 31, 32 in 2014 S10

Figure 2.4: Plan of water bodies at Turkey Mill



Approximate dimensions of the flow control structures, measured with a 3m tape on 4th April 2017 are as follows:

Location	Weir		Photos	Comments	Indicative dam critical flow (onset of damage)
	Crest length	Freeboard			
Turkey Mill Lake	6m (L shaped)	0.5 (viewing platform 0.48m above WL)	18	water 6cm deep on 04 April	
Gates on River Len	3 gates, 1.54, 1.48 and 1.54 opening	Crest of middle gate 18cm below "fixed" side gates	19	2cm water depth on sides 150mm freeboard to underside of bridge	
Culvert under mill	3.5 wide	Height not available (obscured by scaffolding)	20		

2.6.2 Maidstone

A description of the valley downstream is given in Table 3.7 with photographs in Appendix D of the last Inspection and this report. Further information is given in Section 5 of this report. The largest number of people at risk are those in the Business Park immediately downstream, with the various buildings shown in Figure 2.5.

Figure 2.5: Buildings in Turkey Mill Business Park



2.7 Site Constraints on upgrading works

2.7.1 Cultural heritage

The lake is entirely within the Grade II listing of Mote Park, whilst the house is Grade II*. The listing of the park also extends downstream to cover Turkey Mill Pond

The track along the crest of the dam is the old Ashford Road (former turnpike) and leads past the house.

Figure 2.6: Extract from gazetteer of features in park



Table 2-3: Features local to dam, as shown in Gazetteer with 2010 HLF application

Feature		Date built	Significance	Other/ comment
No	Name			
44	Old Ashford Road	1794	Regional	
49	Poll Mill (site of)	1585?	Regional	Bought in 1838 by Lord Romsey to allow north end of lake to be built
50	Overflow from Mote Lake to Turkey Mill	1838	Regional	Broken pipe into channel is from tapped spring at south end of park
52	Boathouse	1836-9	Regional	
55	Lane on east side of Turkey Mill	led to Poll Mill (49)	Local	
56	Mound	1830's	Local	area raised with arising from lake construction

2.7.2 Services

There are a number of services running along the dam crest and toe, but to date no information has been provided on these, other than a verbal report that there was a 33kV buried cable across the channel at the upstream end of the spillway channel. **It is recommended that a full services search is carried for the whole area of the embankment extending up to the spillway channel, to establish if there would be any clash with any of the options shown in this report.**

2.7.3 Downstream flood risk

With Maidstone downstream it is important any works do not increase the risk of flooding in Maidstone, in terms of the frequency and/or magnitude of floods up to the 1 in 100 chance per year flood. This is likely to be one of the governing criteria in option selection.

2.7.4 Land ownership

The downstream face and part of the crest of the dam was sold to the downstream land owner (Turkey Mill Investments Ltd) in 2014, the boundary being marked on site by a 1.8m high fence (no plan available). Therefore any works on the dam will need the agreement of the downstream land owner.

3 Description of dam

3.1 Introduction

The key parameters of the dam and the reservoir it retains are given in the last Section 10 report, with which this report should be read, and are reproduced below.

Table 3-1: Key dimensions relating to the reservoir

Feature	Units	Dimension	Remarks/ Source
Reservoir capacity	m ³	200,000	URS Flood study
Reservoir area at top water level	m ²	120,000	Wikipedia
Upstream Promenade	mOD	19.76 min	
Typical water level	mOD	19.61	2010 Culvert drawing
Embankment crest			
Low point on crest path	mOD	21.6	From topographic survey (CAD file) rises to 22.7mOD at left hand end of crest wall
Low point at West Ride bridge over spillway channel	mOD	21.7	URS 2012 report
Water level in Turkey Mill Pond	mOD	16.56	2010 Culvert drawing
Invert of outlet culvert	mOD	13.94	Table 3.6 of S10 report
Spillway			
Culvert invert(TWL)	mOD	19.29	Feb 2016 survey by JC White

3.2 Embankment

The composition of the dam is unknown, and may have built in stages, with widening (and raising) as part of later stages. The exact boundary between the embankment and abutment is unclear, but for the purposes of this study will be assumed to be from easting 577,360 to 577,500 (140m length).

Although overall height to lowest foundation is around 10m, water level from Turkey Mill Pond is impounded against the downstream face, such that the retained height of water is only around 4m, with 2m freeboard.

Figure 3.1: Embankment cross section

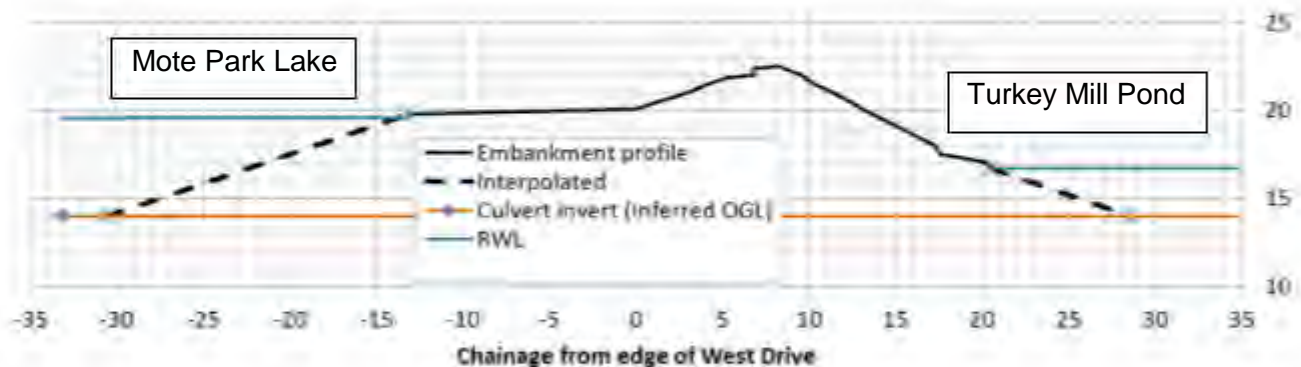


Figure 3.2: Plan of the dam

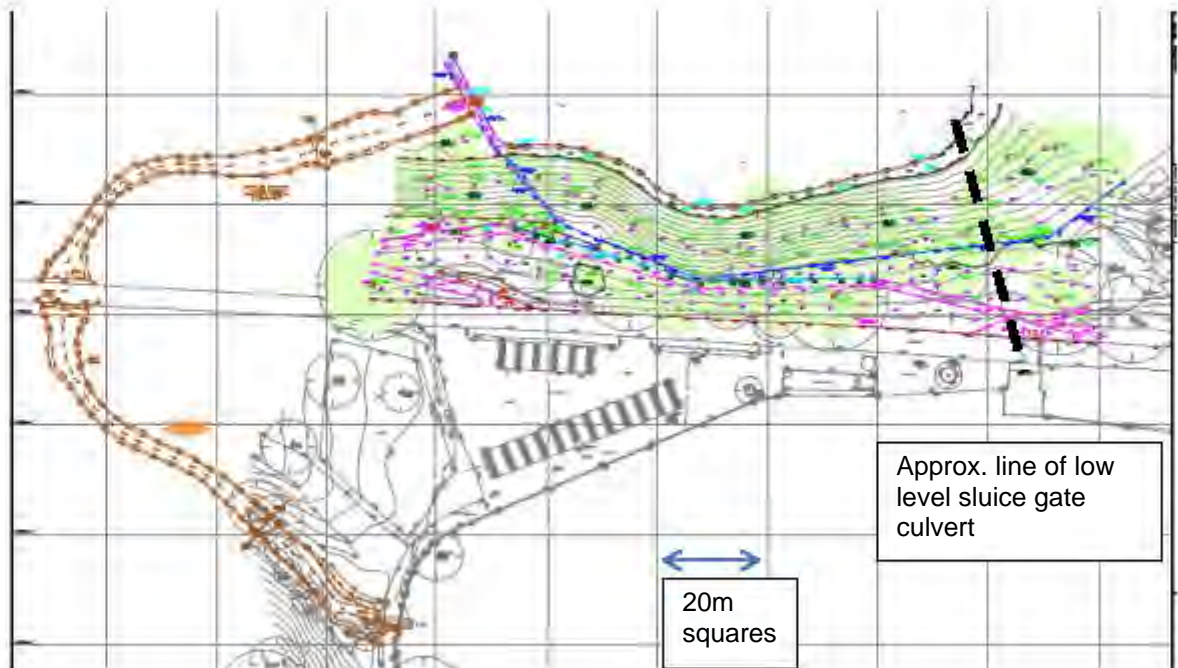
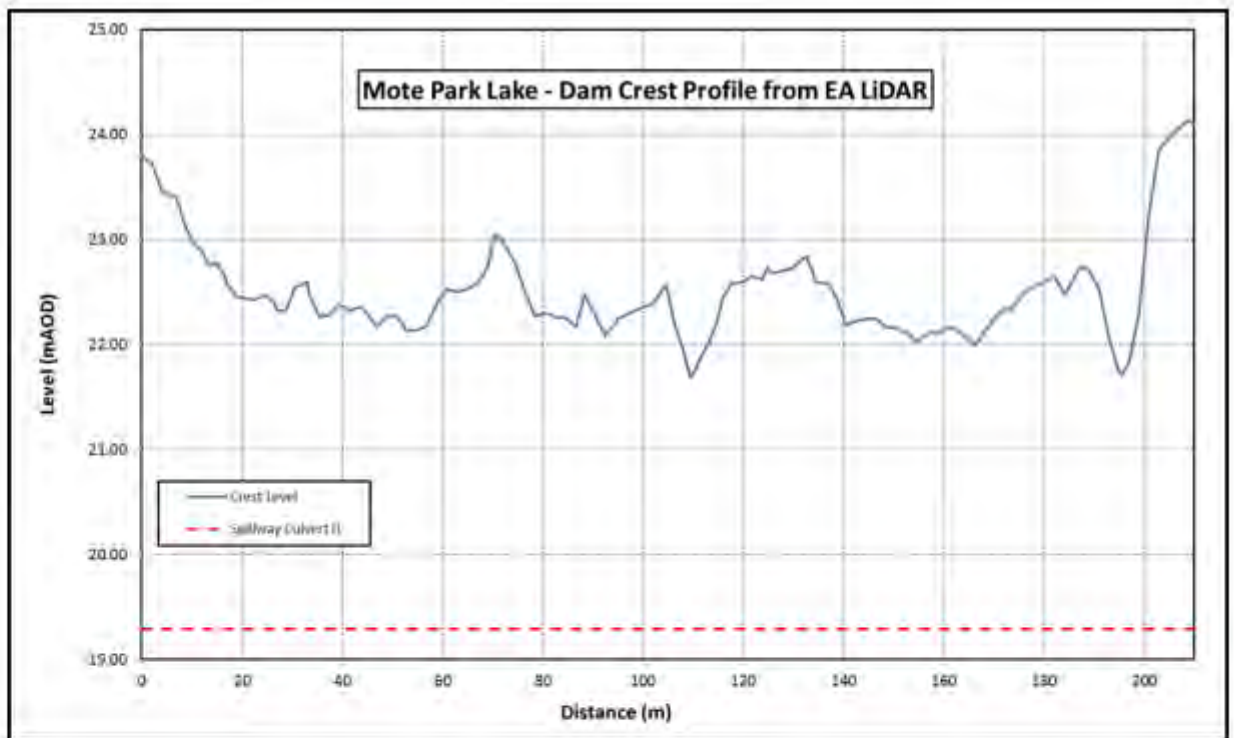


Figure 3.3: Long section along embankment crest (looking downstream)



3.3 West Drive

This is one of the historic main entrances to the Grade II* mansion, established by 1750. (Historic England listing of park). The available topographic survey shows the drive falls from around 22.3mOD 100m west of the spillway to 20.1mOD at the promenade, a fall of 1.7% with an interpolated level at the spillway bridge of 21.5mOD. However, levels on a survey provided by URS suggest road level at the bridge is between 21.76 and 21.86mOD.

3.4 Gated outlet

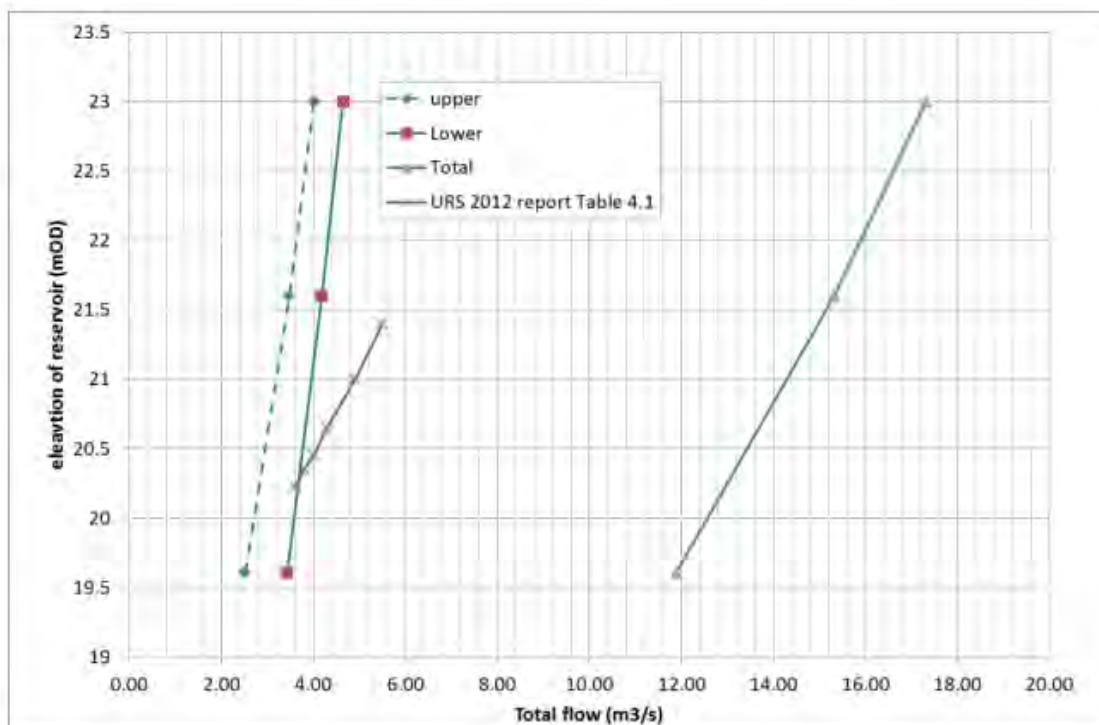
There are four sluice gates, each 0.8m square over an opening 0.6m wide with two gates vertically above each other. The gates are located upstream of a headwall to a culvert through the dam which discharges via a “boathouse” in Turkey Mill Pond. Scaling from the 1944 drawing suggests inverts are 17.4 and 15.6mOD respectively. Further detail is given in the Section 10 report.

Capacity is shown in Figure 3.4. It is understood that only one gate is working, and that this has been the situation since at least 1977 when the Youngs acquired Turkey Mill.

They are operated manually, and so can only be operated when the reservoir is below promenade level, so access is not underwater. One gate could empty the reservoir in around one day. It was reported that Turkey Mill Pond was formed to create a head pond for the turbines in the Mill, so the Mote Park gates may have been related to this, or may have been designed to pass floods, rather than for lowering the lake. It is understood that until 2014 these gates and the sluices at Turkey Mill were all operated by the Turkey Mill Water Bailiff.

It is also understood that gates at Turkey Mill Pond are used to create a “waterfall” used as a backdrop for weddings.

Figure 3.4: Estimated capacity of sluice gates



It is noted that Category A dams should have means of lowering the dam in an emergency, such that it is **recommended that at least one gate is kept in good working order.**

3.5 Spillway

The spillway comprises two culverts discharging into a modest earth channel, with the control at high flows becoming the three culverts under the Northwest Drive, with dimensions modelled as below. The existing spillway capacity is shown on Figure 3.5, whilst a long section along the dam crest (looking downstream) is shown in Figure 3.3.

There were some improvements to the channel upstream of West Drive bridge in early 2016, but these did not include any enlargement of the culverts under West Drive bridge.

Table 3-2: Dimensions of spillway and outlet gates

Feature	Spillway		Gates
	Lower culvert	Second culvert	
Crest level	19.29m	19.63m	
Soffit	20.28m	20.37m	
Control	1.96 m ² Orifice	1.28m ² Orifice	4 x 0.64m ² gate
Downstream control	West Drive bridge		
	Arches	Spill unit over bridge	3m diameter culvert.
Invert	19.21m (soffit 20.8)	22.78m	Water level in Turkey
Control	8.2 m ²	11m long weir	Mill Pond.

Figure 3.5: Long section down spillway channel

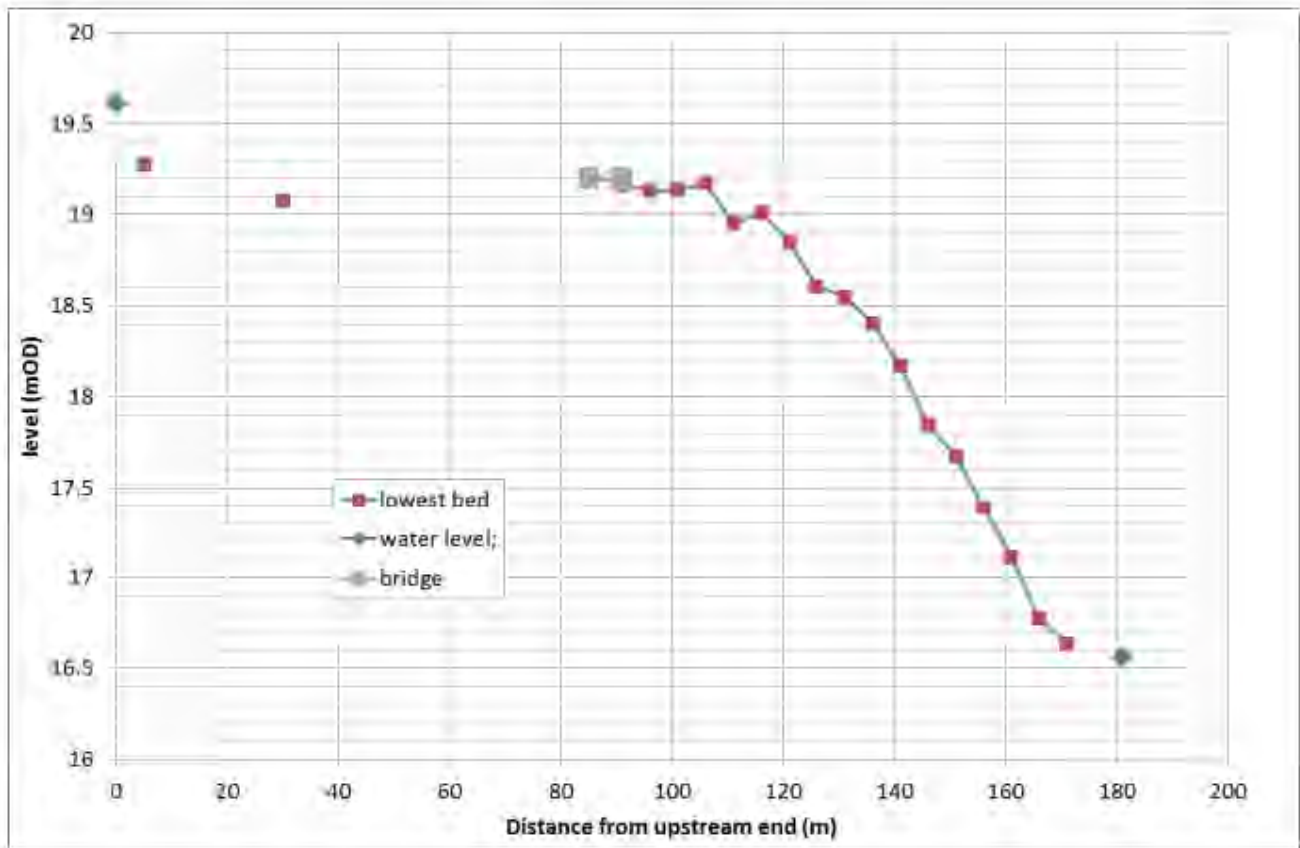


Figure 3.6: Existing spillway capacity

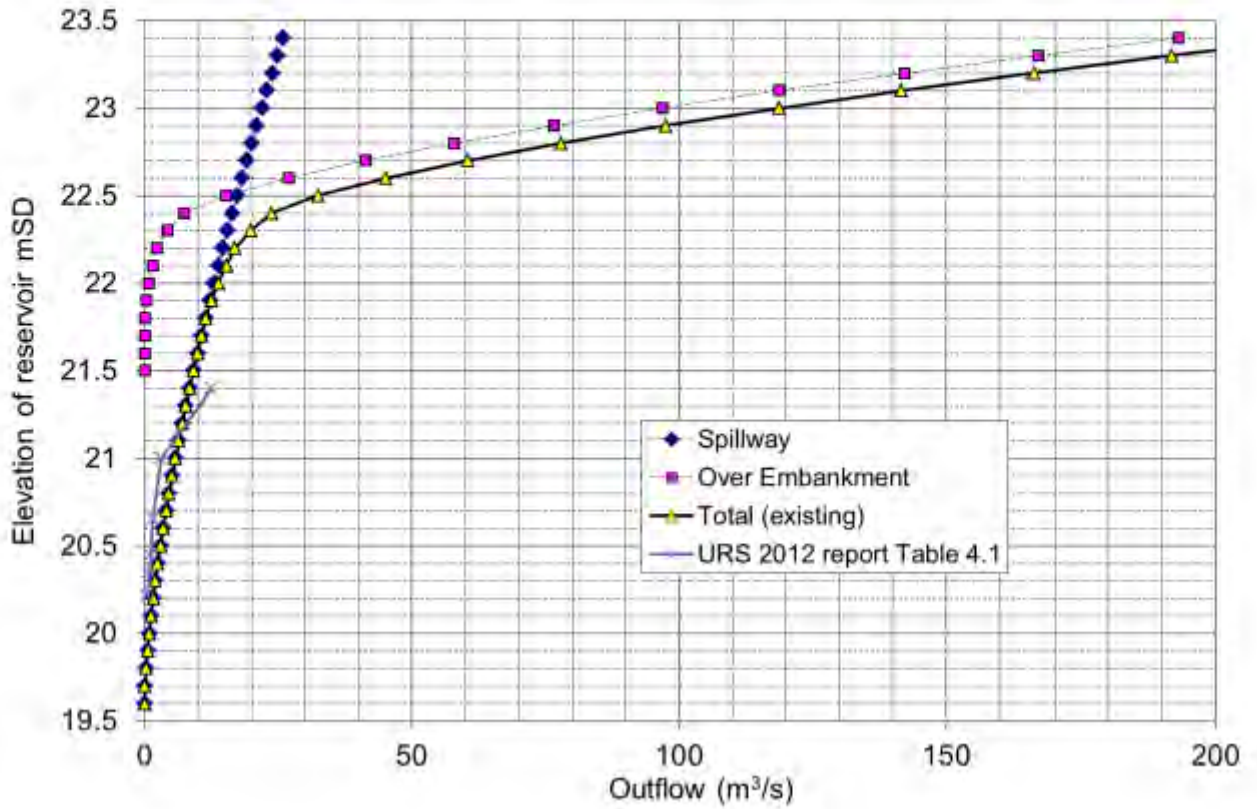
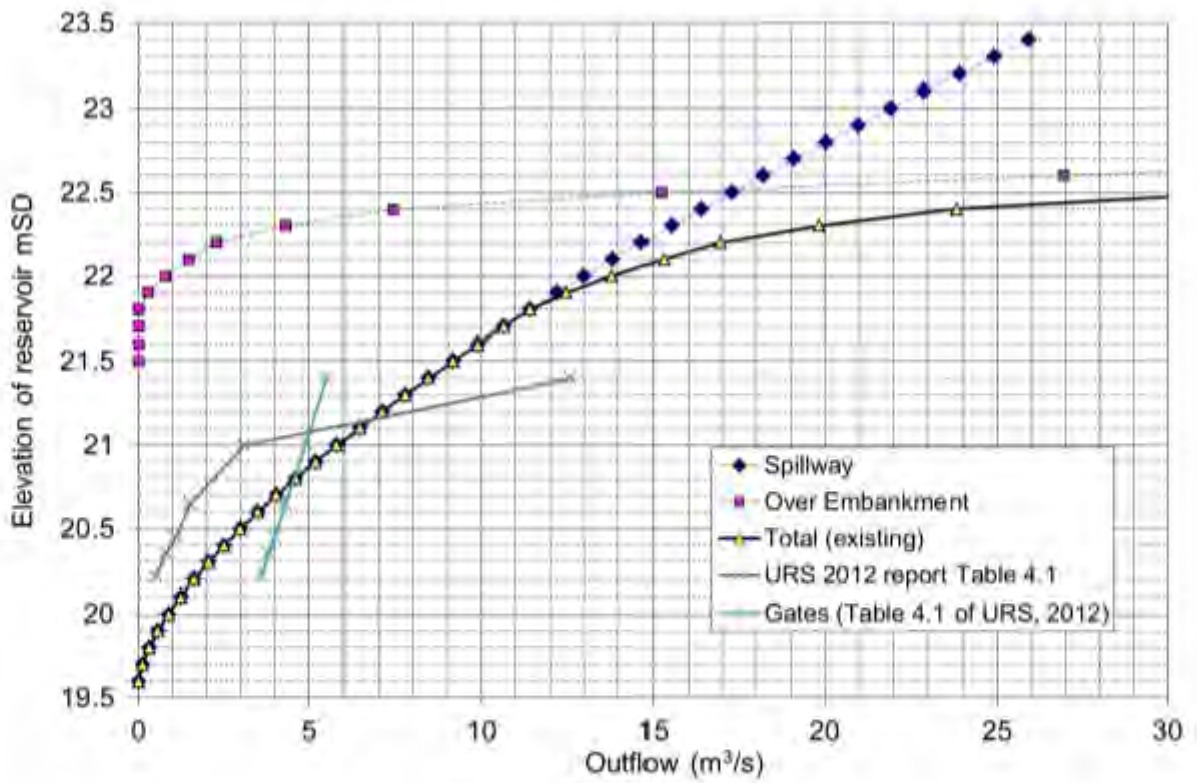


Figure 3.7: As above – detail at low flows



3.6 Reservoir operation

3.6.1 Sluice gate operation

There are no operating rules for the one functioning sluice gate.

There is no history of floods and peak water levels, other than given in Section 2.4 of the 2014 S10. The existing arrangements are therefore ad-hoc and it is understood are to be reviewed and made more systematic.

3.6.2 Emergency plans

An on-site plan has been provided, which has been prepared following the Defra template. It addresses dam failure but does not cover the risk of fluvial flooding to houses downstream.

4 Current probability of damage and failure due to floods

4.1 Introduction

This section defines the current annual chance of damage and failure. This is combined with the consequences of failure in the next section, to give the annual risk, which is a combination of probability and consequences.

The standards recommended by the ICE (2015) are summarised below.

Table 4-1: Recommended standards for flood safety at Category A dam

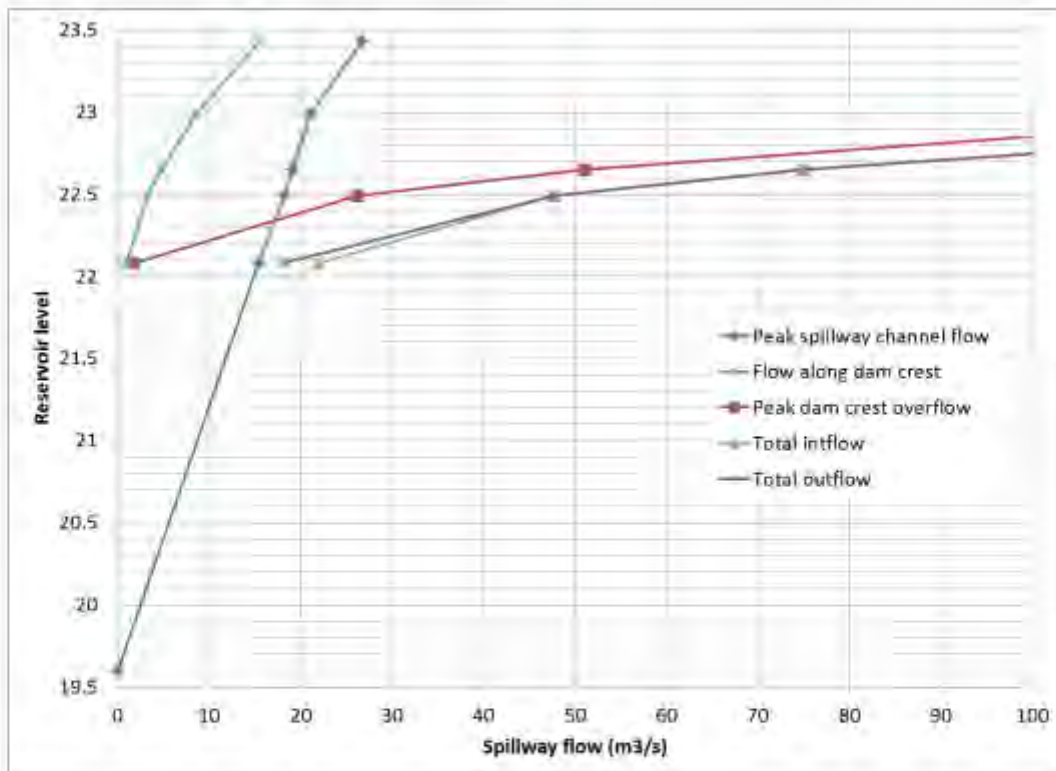
	Design Flood	Safety Check Flood
Requirements	No damage (safety margin provided by agreed freeboard)	Safety of dam cannot be assured for flood greater than this
Annual chance of flood	1 in 10,000	Probable maximum flood (1 in 400,000)
Flood freeboard	Greater of freeboard to reduce wave overtopping to zero (i.e. < 0.001l/s/m) with mean annual wind or specified minimum freeboard of 0.5m	Quantity of wave overtopping does not exceed that for “marginally safe performance” (Tables 6.2, 6.3 of FRS or such higher value as assessed by panel engineer).

Notes. Standards defined in Process diagram in Appendix 3 of ICE 2015

4.2 Annual chance of damage and of failure due to crest overflow

The routed flood outflows and peak reservoir level are shown in Table 2-2, and shown graphically here in Figure 4.1.

Figure 4.1: Estimated flow over the spillway and dam



The overtopping flow that is likely to just cause failure is termed the Dam Critical Flood in the Interim Guide for Quantitative Risk Assessment for UK Reservoirs (2004). The amount of overtopping that would trigger damage sufficient to release the reservoir will depend on the velocity needed to cut through the grass cover and erode the material below, and the stability of the downstream face with elevated pore pressures due to infiltration.

Floods and Reservoir Safety 4th Ed. (ICE 2015) provides a methodology to estimate the maximum velocity on grass for no damage. For a 1V:2H slope with poor grass cover and storm duration of say 15 hours the limiting (no damage) velocity is 1.4 m/s from Figure 12 of Floods and Reservoir Safety (reproduced in Appendix B), which is reached at an overtopping depth of 0.04m. Assuming the head over the crest necessary to cause failure is 0.14m (see Table 6.3) this corresponds to a velocity of 2.8 m/s with a unit discharge of 0.06m³/s/m. The assessed total flow at failure, and associated annual chance of failure are given in Table 6-5, and amount to 1 in 100 chance per year (with no release from the gates).

4.3 Probability of failure due to wave overtopping of crest

Table 4-1 includes the recommended standard for the risk of wave overtopping. The wide crest means that waves are unlikely to lead directly to dam failure on their own.

4.4 Other modes of failure

Elevated reservoir levels can cause structural problems through other modes of failure, such as internal erosion. There is insufficient information to reliably assess this at present.

5 Consequences and existing risk of failure

5.1 General

This section summarises a review and update of the inundation mapping and impact assessment.

5.2 National reservoir flood mapping (RFM)

In 2009 the Environment Agency carried out national flood mapping of all large raised reservoirs in England and Wales, this mapping being available on <https://flood-warning-information.service.gov.uk/long-term-flood-risk/map?easting=576957.88&northing=155389.42&address=200003725175>

The extents of inundation on the website is reproduced in Figures 5.1 to 5.3, and shows flooding downstream of the dam for the 1.5km down to the confluence with the River Medway, and then on the Medway itself.

It is noted that the EU Floods directive requires member countries to review, and if appropriate, update their flood maps every six years. The Environment Agency have just started this process, and are amending the specification of reservoir flood mapping (RFM) in England, aiming to update many of the maps by the EU 2019 deadline. It is currently envisaged that the Environment Agency will include the effect of an extreme fluvial flood (1 in 1000 chance per year) with and without dam failure, in order to separate the impacts of the fluvial flood from the incremental impacts of dam failure.

Figure 5.1: Existing fluvial flood risk (as shown on EA website)



Figure 5.2: Existing risk of surface water flooding (as shown on EA website)

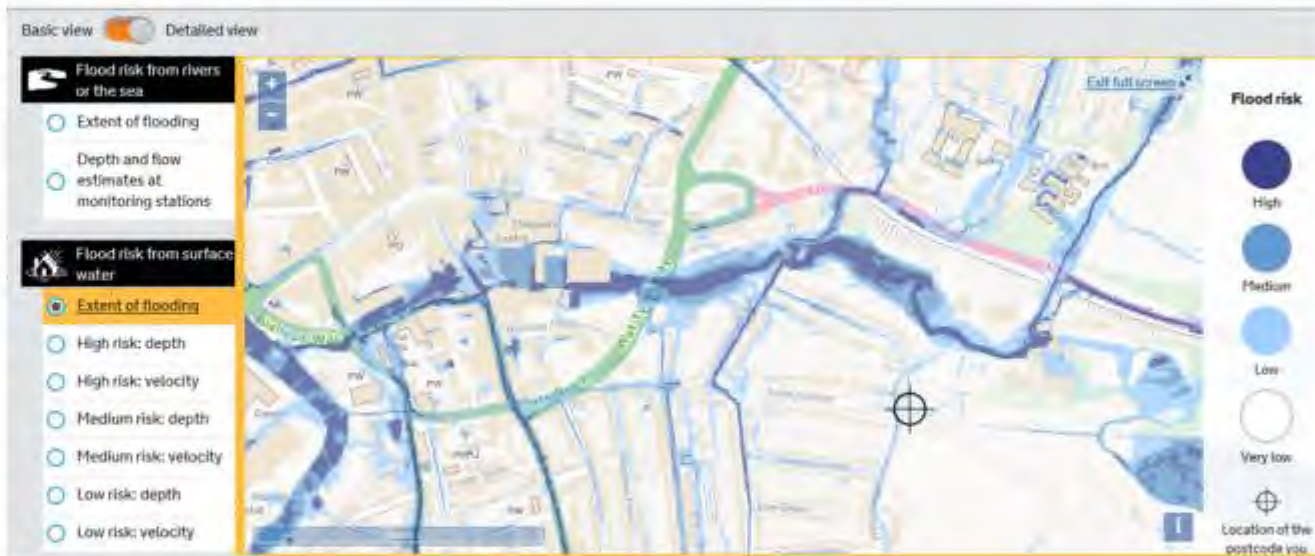


Figure 5.3: Existing risk of flooding from dam failure (as shown on EA website)



5.3 Rapid dambreak (wet (rainy) day only)

5.3.1 Introduction

A rapid impact analysis was carried out in the 2014 Inspection. This has been reviewed and updated as discussed below. This assessment is limited to wet (rainy) day failure only.

5.3.2 Breach hydrograph

The peak flow in the event of dam failure will depend on reservoir level at the time of failure and the elevation of the base of the breach. In reality any breach would occur in two phases, the part above the promenade failing first, then the fill below the promenade cutting back before eventually releasing the water stored below the promenade. For the purposes of a risk assessment it is considered reasonable to limit the analysis to the first part of the breach, the embankment above promenade level. For the purposes of assessing consequences the total flow is taken as the arithmetic sum of the breach flows from Table 5-1, and the peak 1 in 1000 flood, thus assuming failure is concurrent with the peak fluvial flow.

Table 5-1: Peak breach flow (rainy day)

Scenario	Units	Dam crest to WL in turkey Mill Pond	Upper part of dam only (above promenade)	Comment
Reservoir level	mOD	21.8	21.8	
Base of breach	mOD	16.56	19.76	
Breach height	m	5.2	2.0	
Volume of breach hydrograph	m ³	462,000	238,800	
Breach flow	m ³ /s	222	58	Froehlich

5.4 Flood routing and mapping

This can only really be done with 2D computer modelling and accurate ground data and channel sections. For this ALARP assessment the extent of flooding will be assumed to be as shown by the Environment Agency flood maps on the following website (with selected screen shots reproduced in Figures 5.1 to 5.3.). For the no dam failure the worst of fluvial or surface water flooding is used:

<https://flood-warning-information.service.gov.uk/long-term-flood-risk/#x=357683&y=355134&scale=2>

5.5 Impact of dam failure on people

This has been updated from the assessment in the 2014 Section 10 report in the following key areas:

- Number of people at risk due to dam failure is the difference between number at risk with dam failure, and number at risk in 1 in 1000 fluvial flood with no dam failure (as updated RFM specification)
- Review and update number of people at risk in Turkey Mill Business Park, based on extent of flooding on Environment Agency maps.

The output is summarised in Table 5-2. A 1000 year flood is chosen as the fluvial flood with no dam failure for a variety of reasons, including that the fluvial flood mapping is available for the T1000 event and that it could be the flood that might trigger dam failure.

One of the important considerations in estimating the likely consequence of failure is the extent to which warning and evacuation may be effective. Although there is an on-site

plan, which should reduce the chance of failure, if the dam did fail the number of people at risk and lack of a site specific off-site plan mean that evacuation cannot be relied upon. As is normal in UK, which is to follow a precautionary approach, for this ALARP analysis it is assumed that warning may not be effective.

Table 5-2: Summary of impact assessment

Failure scenario	Units	1 in 1,000 fluvial flood		Rainy day dam failure		Incremental effect of dam failure
		Value	Comment	Value	Comment	
Hydraulic analysis						
Peak flow	m ³ /s	75	Table 2.2	58 additional	Table 5.1. Increment on fluvial	
Volume of breach hydrograph	m ³	2.3Mm ³		0.24Mm ³		
Flow/width (Q/W) at following points						
Turkey Mill	m ³ /s/m	1.8		2.6		
People at risk						
Number of houses at risk		16		20		
Population at risk Max PAR		257	Table 5.3 plus houses	324		
Population at risk (time averaged)		96		119		
Impact of failure						
Likely loss of life	Num lives	1.2		2.6		1.4
Cost of 3 rd party property damage	£M	4.2		5.1		0.9

Table 5-3: Flood risk to properties in Turkey Mill Pond

Building name	area of footprint		Number of floors	Total area	Number of floors flooded in	
	m2	comment			Fluvial/ surface	dam break
Day nursey	350		1	350		1.0
Tolherst Court	860		1	860		
The Orangery	470		1	470		0.5
Northern annex	340		2	680	0.5	1.0
Turkey Court	1000		1	1000	1.0	1.0
southern annex	920		2	1840	1.0	1.0
Paul Sandby Court	650	new building	2	1300		
James Whitman court	650		1	650		0.3
Hollingworth court	690		1	690	1.0	1.0
The Beater House	350		1	350	1.0	1.0
Various extension	520		1	520	1.0	1.0
	6,800			8,710	3,650	4,600
Number of people				600	Internet	
average area/ person				14.5	m2/ person	
Inferred number of people					251	317

5.6 Tolerability of existing risk to those downstream

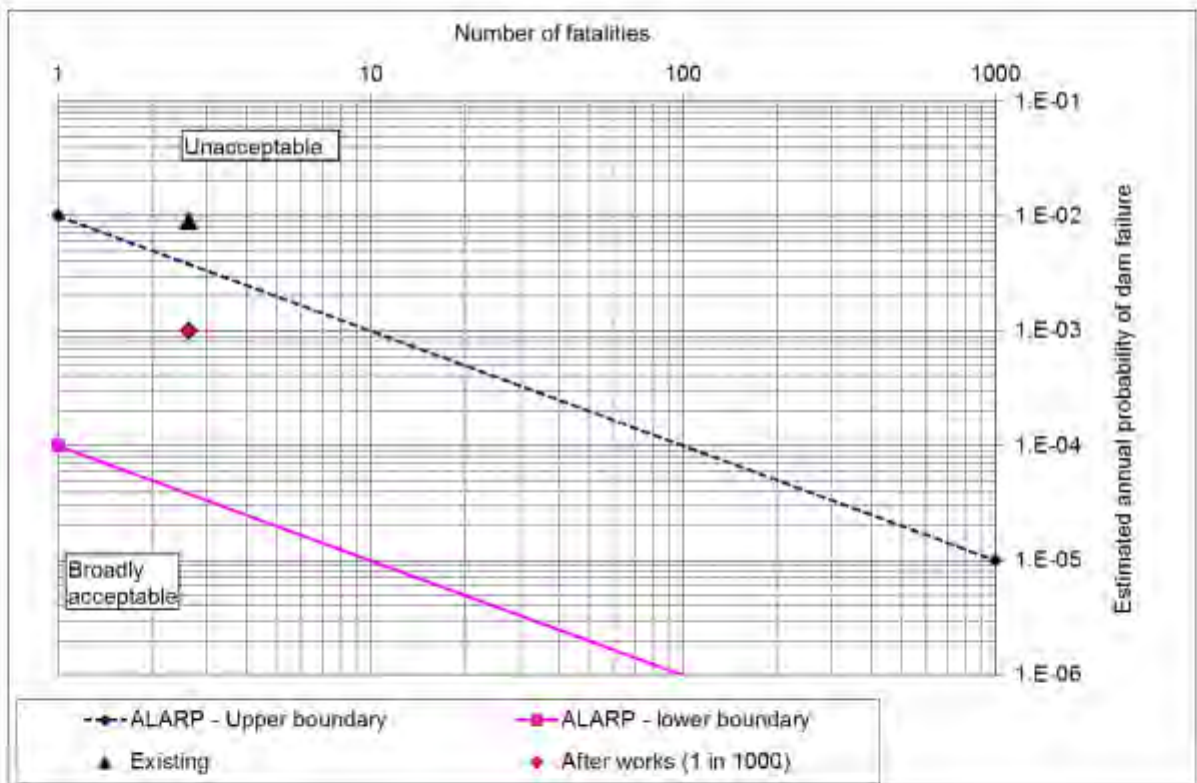
5.6.1 Risk to individuals

The risk of loss of life of those living in the house nearest the dam due to the dam failing in floods, is the product of the probability of failure (1 in 110 annual chance per year) and the probability of death given that the dam had failed (0.7%), namely 1 in 16,000 per year chance per year. This is between 1 in 10,000 and 1 in one million chance per year and is therefore in the ALARP zone (using limits recommended by HSE as tolerable for risk to the public from hazardous installations (HSE, 2001).

5.6.2 Cumulative (Societal) risk

The current risk is shown on the figure below as in the ALARP zone, where the risk should be reduced as low as reasonably practicable (see Appendix A). If the probability of failure could be reduced to 1 in 10,000 per year, the dam would then be in the broadly acceptable zone.

Figure 5.4: FN chart showing tolerability of risk of dam failure



6 Options for reducing risks from floods

6.1 Introduction

This section sets out potential physical options for upgrading the spillway capacity, prior to assessing whether the cost is proportionate to the benefits. This section builds on the preliminary list given in Table 5.6 of the Section 10 report. For those options which are considered viable, budget costs are given in Appendix E, summarised below, with a sketch plans in this section, and photographs of locations affected in Appendix D.

6.2 Screening options to reduce likelihood of failure due to floods

6.2.1 Constraints and Long list

The major constraints are shown in Table 6-1, with a screening of all options in Table 6-2

Table 6-1: Constraints on options to reduce likelihood of failure due to floods

	Constraint	Practical implications
1	Not increasing flood risk downstream, for events up to the 1 in 100 chance per year flood	<p>a) The existing freeboard is in effect a flood elevation reservoir, so flood flows are stored (attenuated) in the reservoir.</p> <p>b) The current lowest embankment crest level is 21.76mOD; located on the line of the permissive footpath at the right abutment (Section 4.8 of 2012 Scott Wilson report)</p> <p>c) A minimum crest level of 21.8mOD will be adopted as the minimum crest level to avoid increasing the frequency of overtopping/ pass forward flows for floods up to the 1 in 100 chance per year.</p> <p>d) At this reservoir level the pass forward flow at the 1 in 100 chance per year event is estimated at around 11 m³/s by the Stillwater flood study, and 7.5m³/s in the URS 2012 report.</p>
2	Downstream face not owned by council	<p>The council would need agreement from the downstream landowner to carry out any works on the downstream face.</p> <p>Note that Turkey Mill Pond is used as a backdrop for weddings, which is significant part of the Turkey Mill Business.</p>
3	West Drive	<p>This is a historic route, so any changes in ground level should not impact onto the line of the drive. However, the parapet walls could be replaced by open parapets to give a small increase in flow and this has been included in all options.</p>
4	Maximum crest level of non-overflow section	<p>a) The crest level where a path runs near the crest is around 22.5mOD. However it varies from this with a 35m long section at the right hand side which is only around 22mOD, and it rises to around 24m (approx., no survey) near the spillway channel..</p> <p>b) It will be assumed that the maximum level of the non-overflow section could be raised to 23.5mOD (see Figure 6.2), which with a normal minimum crest width of say 3m, will require an upstream wall (similar to the existing, but higher), and/or elimination of the footpath</p>
5	Grade II park	<p>Preference (but not absolute – see precedents on Butterley, Hampstead Heath) to minimise visual impact e.g. grass rather than concrete appearance to spillway.</p>
6	Maximum width of auxiliary spillway	<p>The maximum width is constrained by the bends in plan alignment of the embankment dam, presence of downstream boathouse and width of the valley. Practicable maximum widths</p>

		are shown on Figure 6.3.
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Table 6-2: Screening of long list of approaches to reduce risk of failure due to floods

	Option	Description	Application at Mote Park
1	Discontinuance	<i>If the weir crest level was lowered sufficiently to reduce the stored volume to less than 25,000m³, then the reservoir could be discontinued under the Reservoirs Act and the spillway upgrade would no longer be mandatory. However, this threshold may be lowered to 10,000m³ (as it has in Wales) and the reservoir would once again be subject requirements of the Act and the safety recommendations would apply. The safest discontinuance option would be to lower the stored volume to below 10,000m³</i>	<i>In view of the importance of the lake to the public park, and the large decrease in lake depth this is not considered an acceptable option</i>
2	Modify existing spillways	<i>On some dams it may be possible to modify existing spillways, by lowering the weir crest or other minor modification, to increase the flow significantly</i>	<i>This would mean enlarging (replacing) the culverts under the West Drive, but this would increase frequency of downstream flooding and is therefore rejected</i>
3	Modify crest level of dam	<i>Where the existing crest level is irregular then addition of a concrete crest marker and making up low spots may reduce the likelihood of failure significantly, if flow can be spread out along the whole length of the dam.</i>	<i>This is viable, and carried forward to the short list as applying to all option</i>
4	Raise crest	<i>On some dams with a modest shortfall in the required discharge capacity, the crest can be raised by the addition of a flood wall, which increases the driving head in the reservoir and thus the flow that would pass down the spillway. This assumes that there are no roads, or other features, around the reservoir rim that would become vulnerable to flooding if the flood level was higher.</i>	<i>As spillway flow is governed by the culverts, this would not significantly increase flow</i>
5	Auxiliary Spillway over embankment crest	<i>A description and details of grass reinforced spillways are given in Appendix B.</i>	<i>This is viable, and carried forward to the short list. Option A1 and Option C1</i>
6	New concrete spillway	<i>This would entail a new concrete weir with chute and bridge/culvert to provide vehicular access over the chute. It is recommended that concrete spillways are located on the dam abutments, so that they are founded on rock, rather than the weak alluvium in the valley bottom. To prevent scour the downstream end of the chute would need a formal stilling basin and/or other energy dissipation measures. A new river channel would have to be formed to return flows back to the existing channel.</i>	<i>This is viable, and carried forward to the short list. To meet the recommendations of the best practice guidance given in ICE 2015 in full it would be necessary to provide a discharge capacity of around 300m³/s or 20 times the capacity of the existing spillway, so this would be expensive Option A2 and Option C2</i>
7	Rehabilitate sluice gates	<i>In principle the gates can increase the outflow capacity by up to 6m³/s, but guidance to panel engineers suggests that reliability in a flood cannot be guaranteed, unless there are at least two levels of backup for every aspect (power, hydraulics, electrical systems, mechanical parts etc)</i>	<i>There is insufficient backup to be able to rely on the gates, particularly as access to operate the gates is cut off as soon as the promenade floods.</i>

	Option	Description	Application at Mote Park
8	<i>Automate sluice gates</i>	<i>In principle an automatic control system could be designed and installed so that the gates operate automatically, with no need for manual control.</i>	<i>At Mote Park this would only be a small proportion of the required increase in spillway capacity, and so cannot be justified in terms of dam safety capacity. It might be worthwhile in terms of management of normal lake levels.</i>

6.2.2 Reinforced grass spillways

(a) General

Grass reinforced spillways can be used for auxiliary spillways which only operate less than say once every ten years in extreme storms, and provide advantages of reduced cost and improved visual appearance. CIRIA Repot 116 notes that *“From the aesthetic viewpoint, geotextile-reinforced grass is indistinguishable from plain unreinforced grass. From a distance, concrete-reinforced grass appears similar to plain grass, although from close up some concrete is generally visible - particularly if the grass sward is short or thin”*.

The grass within these type of systems provides structural protection to the underlying subsoil by:

- a) Roots binding the soil together (enhanced where matting/concrete blocks are used)
- b) Blades of grass (recommended length 50 to 150mm long) lie flat under the flow of water and provide an additional layer of scour protection.

Design criteria are given in CIRIA (1987), and summarised in Appendix B. Technical papers on some of the practical issues in design and construction are given in Freer (1992) and Gosden (2014). Photographs of some examples of grass reinforced spillways are given in the same Appendix

However, grass spillways do impose important maintenance obligations, including:

- The system will not provide design protection until the grass has grown, so there is a period of increased risk whilst the grass grows
- Vigilant prevention of bare patches is necessary. These may form when the public take informal footpaths across the grass (termed desire lines)
- The grass must be cut regularly and maintained within 50 to 150mm length
- The auxiliary spillway crest should not be vulnerable to blockage, so the existing balustrade fencing would need to be removed (or the gaps widened significantly).

(b) Hydraulic design at Mote Park

The ranges of limiting velocity with varying slope protection are shown in Table 6-3 and figure in Appendix B. The maximum practicable width is around 100m, so the maximum allowable (no damage) and failure flows are shown in Table 6-3. In view of the large shortfall plain grass and open matting are not considered viable, leaving concrete blocks or wedge blocks.

6.2.3 Minimum design standard

It is considered that the absolute minimum design standard for the spillways should be a 1 in 1000 chance per year flood, namely $75\text{m}^3/\text{s}$, with no damage which is an increase of $64\text{m}^3/\text{s}$ on the existing capacity. The corresponding safety check flood (damage no failure) would be larger by the depth of overtopping of the non-overflow sections necessary to cause failure. This will vary with the options but assuming 80m length overtopped and a steep slope with no grass could be $0.01\text{m}^3/\text{s}/\text{m} \times 80 = 0.8\text{m}^3/\text{s}$ overtopping plus the additional flow over the spillways could be say 5% increase on the design flood, say 1 in 1200.

The ICE standard would recommend a no-failure flood of PMF, namely $336\text{m}^3/\text{s}$ which would be an increase of $325\text{m}^3/\text{s}$. However, the ICE guide accepts a risk-based approach may give a lower standard. The flow provided by an auxiliary spillway is shown below, and suggest that for a head of say 1.2m (crest of non-overflow section of 23mOD) the crest

would need to be about 35m wide to pass the T1000, whilst to pass the PMF the auxiliary spillway would need to be 80m wide with a head of 2m.

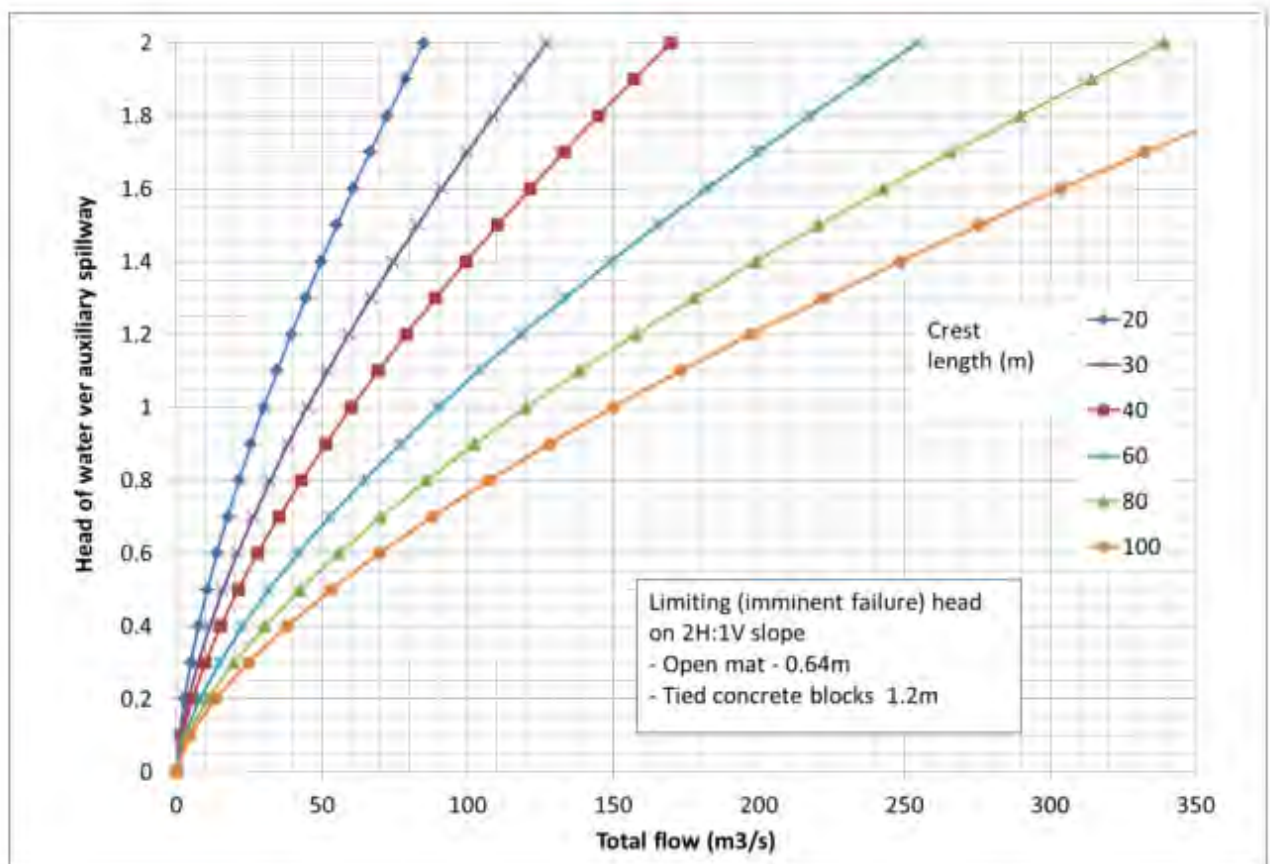
Table 6-3: Limiting velocities on slope protection

Type of slope protection	Allowable (no damage) / failure values – see Note 1 (for 15 hour overtopping)		
	Velocity (Note 1) m/s	For 2H:1V downstream slope	
		Head on crest m	Unit discharge m ³ /s/m
Plain grass – poor cover	1.4 / 2.8	0.04 / 0.14	0.01 / 0.08
20mm thick open matting to reinforce grass e.g. Enkamat (Note 2)	5 / 7	0.36 / 0.64	0.363 / 0.76
Concrete block systems to reinforce grass with good interblock restraint (note 2)	8 / 10	0.8 / 1.2	1.1 / 1.9
Wedge blocks	10 / 15 (Note 3)	1.2 / 2.3	1.9 / 5
Insitu concrete	20 / 30 (Note 3)	3.6 / 7.2	10 / 28

Notes

1. Allowable velocity as shown on the figure in Appendix B. Failure taken as 2 times allowable, capped at overstress of 2 m/s
2. The grass spillway is a structural element of the dam and will need regular cutting to keep it at 50 to 150mm length
3. No definitive limit as would depend on detailing and other site features. These values taken for illustrative purposes
4. Failure would occur first at the non-overflow section, so not relevant

Figure 6.1: Flow over auxiliary spillway



6.3 Option A – Auxiliary Spillway over embankment crest

This is shown on Figure 6.2 and would entail:

- a) Removal of the trees on the downstream face; construction of auxiliary spillway
- b) Provide access for construction, probably as an oblique track at say 10H:1V longitudinal slope on an adjacent section of the downstream face
- c) Constructing side bunds (or walls) to contain flow within the armoured section
- d) Raising the remainder of the crest remote from the auxiliary spillway to a level above the design flood level, to prevent flow outflanking the reinforced overflow section. This is likely to include a wall, depending on the alignment relative to the auxiliary spillway crest, which could be faced with masonry or otherwise subject to landscape treatment.
- e) The armouring would need to continue below water level in Turkey Mill Pond. In order to maintain the reservoir perimeter path it is suggested that this is constructed as a concrete path/steps to water level, with the armoured section anchored to the upslope side, and sheet piles on the downstream side to inhibit scour/ undermining of the path.
- f) Rather than ramps down to the path crossing of the spillway this is probably better as a small wall on the upslope side of the path for say 10m each side of the armoured section, so flood water can spill laterally. This would mean rerouting the paths along the crest
- g) No stilling basin, with energy dissipation due to mixing with water in Turkey Mill Pond.

As grass spillways allow water to infiltrate the slope, the material forming the subgrade must have a permeability of less than 10^{-5} m/s (as required in Section 5.4.1 of CIRIA Report 116), which means that a ground investigation is required to confirm ground conditions, and if the existing soils is too permeable then a one metre clay formation would need to be constructed.

In addition for grass spillways the slope must be no steeper than 3H:1V, or preferably 4H:1V to avoid slope instability in operation. This can only be achieved by trimming the slope above the path (say above 17.1mOD) and for narrower options by locating the armoured sections on shallower slopes at west or east ends. A 4H:1V slope would require a 18m plan distance which is around 4m longer than the available distance at the narrowest section, but less than that available at the right and left abutments. This distance can be reduced if a massive weir is adopted with a drop of say 1m onto the downstream face, rather than a simple slab at ground level (with a cut-off trench).

A steeper slope could be adopted for a reinforced concrete chute, which would be watertight, but additional detailing such as articulated movement joints, key trenches and possibly soil anchors would be required to ensure slope stability and avoid cracking.

Clearly there are various options for locating the spillway in plan, with constraints including that the embankment toe is curved in plan, the downstream boathouse and visual impacts. The plans at this stage can only be indicative as the available topographic survey is incomplete and the overflow width will depend on the edge detail adopted. The width required is constrained by these factors, with the widths adopted for costing and ALARP analysis shown indicatively in plan on Figure 6.3 and summarised in Table 6.4.

Table 6-4: Options for auxiliary spillway with weir at 21.8mOD

Option (protection type-width)	Slope protection against overtopping (options in bold taken forward to ALARP)	Width	Non-overtoppable bank crest level (freeboard above 21.8mOD)	Flow over auxiliary spillway (m ³ /s)
A1-40	Grass reinforced by cable tied concrete blocks	40	23.0 (1.2 – Note 1)	78
A2-40	Reinforced concrete	40	23.5 – Note 2 (1.7)	132

Notes
 1. Maximum head on auxiliary spillway – see Table 6-3
 2. Maximum raising of crest – see table 6.1

Figure: 6.2 Cross section on Option A

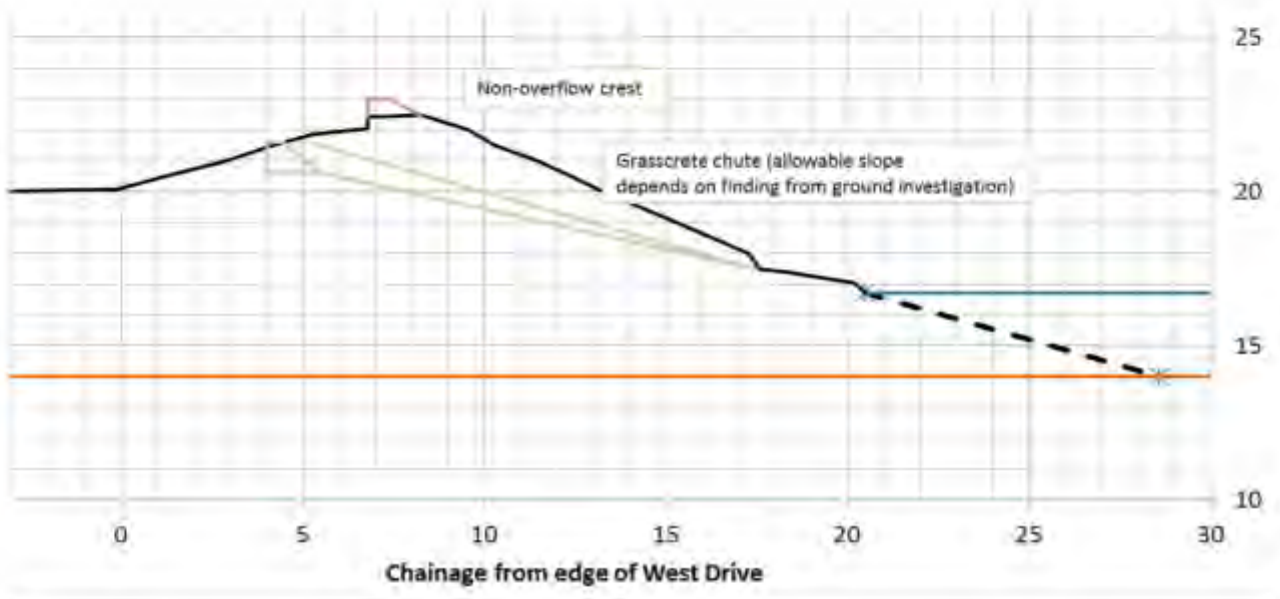
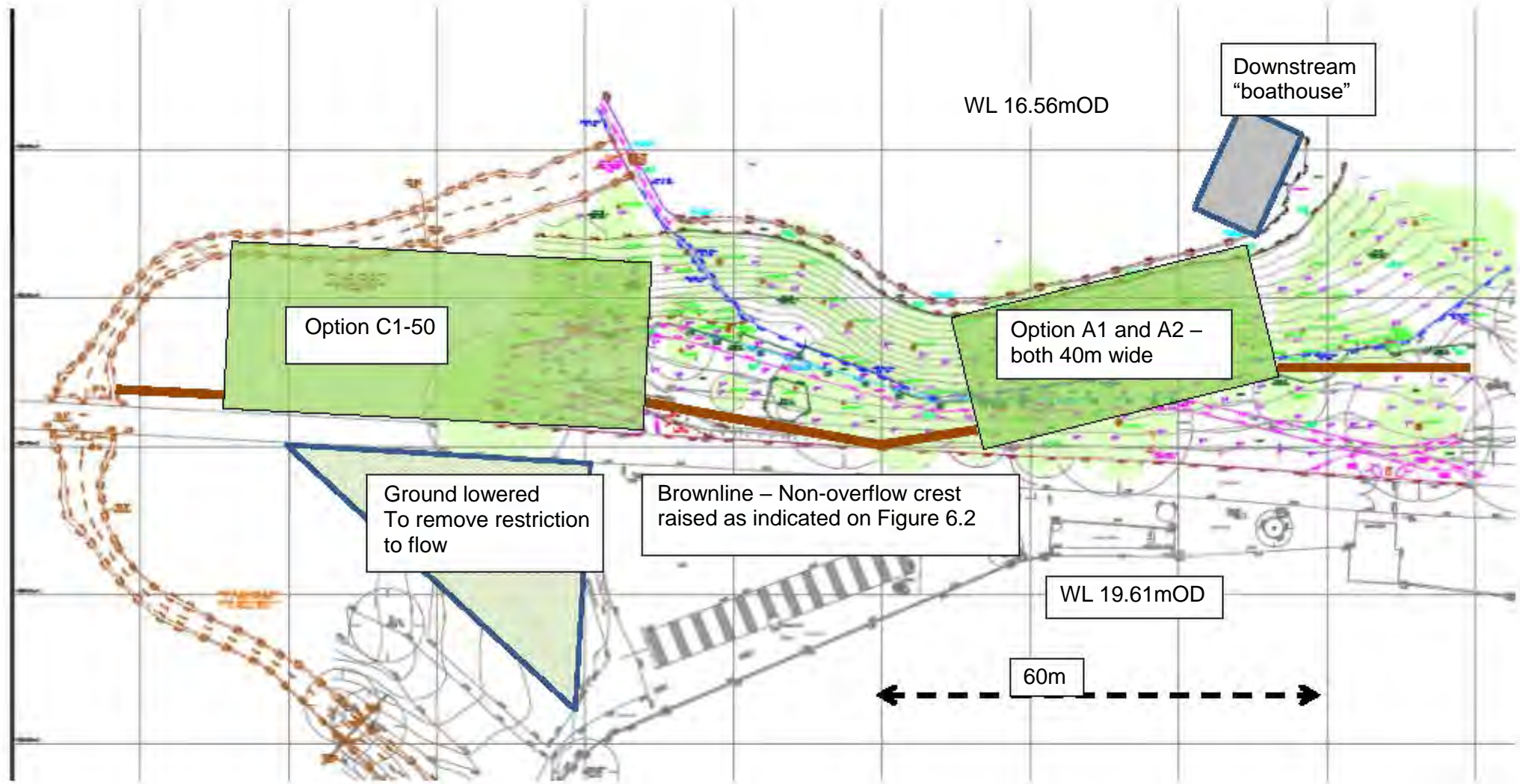


Figure 6.3: Diagrammatic plan of options to increase spillway capacity



6.4 Option B – Strengthen crest to inhibit breach

One option that would minimise the works on the downstream face, which is not owned by the undertaker, would be to strengthen the crest to increase resilience to breach. This would not change the annual chance of damage to the downstream face, and it would be prudent to have contingency plans in place for repairs, which should include constructing an auxiliary spillway over the damaged section.

This option would include the following features:

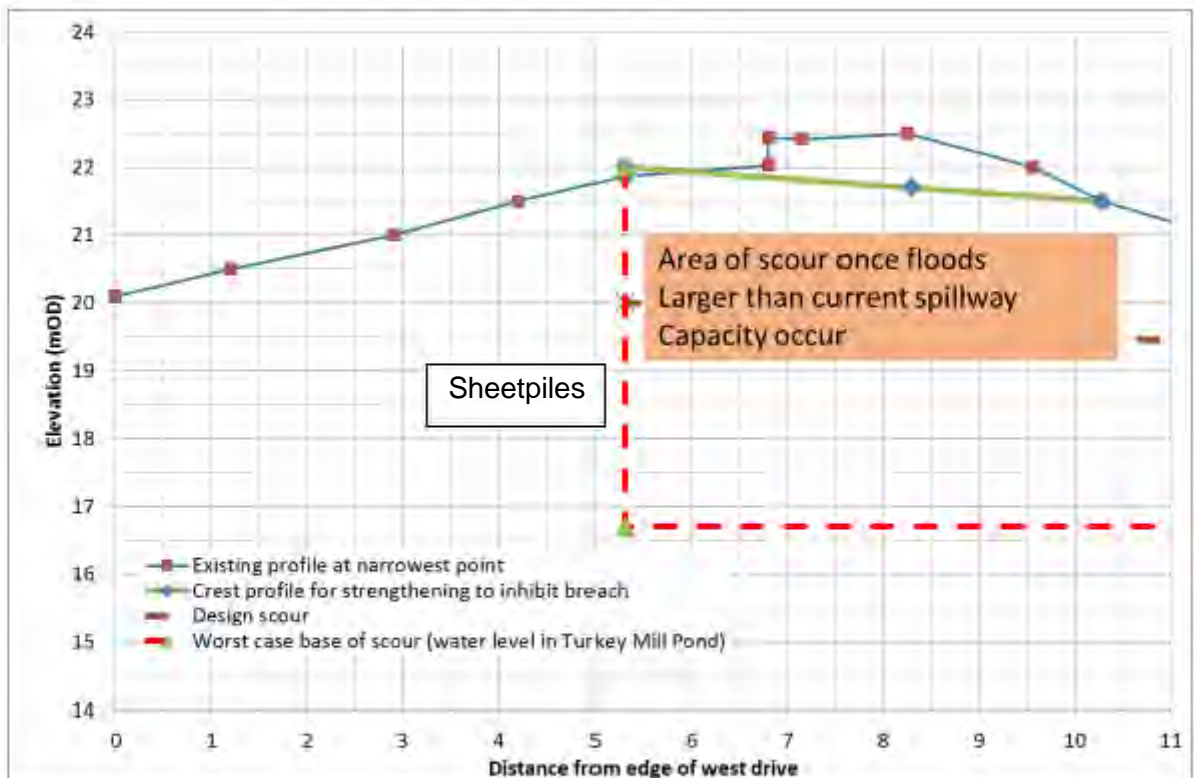
- a) Regrade the crest to a level of 22.0mOD (or similar level), to spread out overtopping flow, and increase the crest width thus prolonging time to breach (this is above minimum level of 21.8mOD to avoid increasing fluvial flood risk – see Table 6.1)
- b) Excavate top metre of graded crest and replace with clay, to provide increased scour resistance
- c) 6m deep sheetpiles on the upstream side to also prolong time to breach. These would be designed to allow up to 2m scour on the downstream side
- d) Fence marking land boundary reinstated on same line, but at revised level (generally within footprint of modified crest).

The impact on the park would include:

- Modify the existing upper path to be along modified crest
- Loss of trees on, and near, crest but trees along the lower part of the downstream face could be retained.

It is estimated that this would provide an annual probability of failure equivalent to use of a grass reinforced (open mat) auxiliary spillway. Clearly the annual chance of damage (scour to the downstream face) would remain at 1 in 100 per year.

Figure 6.4: Cross section on strengthened crest (Option B)



6.5 Option C – Auxiliary spillway on MBC land on left abutment

This would be similar to Option A (see previous description and Figures 6.2 and 6.3), except being located on MBC land on the hillside and discharging into the stream, rather than Turkey Mill Lake. Other differences are likely to include:

- a) Removing parapets so water can flow over bridge (21.8mOD approx. to be confirmed)
- b) Excavating an approach channel on the east side of the road cutting say 25m wide; upstream set at 21.2mOD, so 0.6m below hydraulic control (weir)
- c) Scour protection at downstream toe (and side), to prevent scour undermining the concrete blocks

The overflow width would be the maximum practicable whilst remaining on MBC land. This would in effect give a weir which is around 50m wide with head of 1.2m.

It would increase the risk of damage to the spillway channel in extreme floods, and if / when this occurred channel works would be required.

The impact on the park would be limited to the left abutment area, and crest of the embankment dam. It would be necessary to relocate the high level footpath and footbridge coring of the spillway stream just downstream of the West Ride bridge.

6.6 Reduction in probability of failure due to candidate options

The spillway capacity, with no damage and at imminent failure, for each option is tabulated below.

Table 6-5: Options to reduce risk of failure (release of reservoir) in floods

Option Ref. and Description		Level of		Comments	At imminent failure		Outflow (with gates closed)				Annual chance of failure (Fig 2.3)	
		auxiliary spillway	Non-overtoppable dam crest		Mode of failure	Reservoir level	Existing spillway	Auxiliary spillway	Over embankment (Note 1)	Total		
		mOD	mOD			mOD	m ³ /s	m ³ /s	m ³ /s	m ³ /s		
	Existing	n/a	21.8			overtop d/s face	21.9	11	n/a	0.1	11.1	110
A1-40	40m wide Grass reinforced (concrete blocks)	21.8	23.0			overstress grass reinforcement	23.05	21	78	aux spillway	99	2,000
A2- 40	40m wide Concrete chute	21.8	23.5	prevents infiltration of water into slope		scour from overtop adjacent embankment	23.55	23	132	aux spillway	155	8,000
B	Strengthen crest to inhibit breach (100m length)	None	22.0	reduce works on downstream face		local scour of downstream face at trees sufficient to undermine crest armouring, say equivalent to grass reinforcement	Take as 22.6	18	None	76	94	1,500
C1-50	50m wide Grass reinforced (concrete blocks) on abutment	21.8 (existing west Drive)	23.0	Large excavation, but no impact on view from turkey mill		scour from overtop adjacent embankment	23.05	21	98	8 (over bridge)	127	4,000
C2-50	50m wide concrete chute on abutment	21.8 (existing west Drive)	23.5	Large excavation, but no impact on view from turkey mill		scour from overtop adjacent embankment	23.55	21	166	8 (over bridge)	195	20,000
A2-40 and C1-50	Both options	21.8	23.5					23	230	8 (over bridge)	261	80,000

Notes

1 current crest is uneven, and so failure would occur by concentrated flow at a low spot, unless the crest is regulated to provide a contact level and spread out overtopping flows.

7 Evaluation of risk reduction options

7.1 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:**

- **A decision on the option to be adopted must be taken by 6th June 2017, with a letter from the council to confirm this**
- **The works must be completed within three years i.e. by 6th June 2020.**

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
- 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.

7.2 Costs

High level, estimated project costs are given in Appendix E and summarised in Table 7-1. 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), professional fees for planning etc. to arrive at a project cost for the purposes of the ALARP analysis. There are no allowances for MBC staff costs, and it is suggested that MBC budgets allow for these and other costs not shown as included in Appendix D.

7.3 Buildability

All of the options will involve loss of trees and earthworks. These should be straightforward in terms of constructability, although construction access will disrupt park activities.

Environmental issues have not been included in this assessment.

7.4 Engineering standards

These are given in the publication *Floods and Reservoir Safety* 4th Ed. (ICE 2015), published by the Institution of Civil Engineers.

The consequence of failure would be risks to lives in 20 houses, 600 people in the Business Park downstream, such that the dam has been classed in the last ten yearly Section 10 Inspection as Flood Category A: where a dam breach could endanger lives in a community. It very unlikely that any future Section 10 Inspections will recommend a different flood category given the consequences of failure.

The Institution of Civil Engineers (ICE 2015) recommends that for a Category A dam the spillway system must:

- a) Safely pass a design flood of annual chance of 1 in 10,000 with no damage, and
- b) Not result in dam failure in a Probable Maximum Flood (PMF – annual chance of 1 in 400,000).

To meet current engineering standards the only acceptable option would be in effect both options A2 and C2, which is a concrete spillway for most of the valley width. However, ICE 2015 also recommends that where an existing dam does not meet current standards, then

a risk based approach may be adopted to assess the extent of upgrading. This risk-based approach is presented below.

7.5 Acceptability of damage to dam

A separate consideration is the risk of damage to the dam, in that currently in floods of the order of 1 in 100 chance per year, there will be overtopping and erosion of the downstream face.

Construction of an auxiliary spillway would reduce this to the annual chance of the design flood, for example 1 in 3,000 or 10,000 chance per year depending on the option selected.

Option B reduces the chance of failure (release of) the reservoir, but do not reduce the chance of damage to the downstream face respectively. The decision of the tolerability of this damage is a matter for the Council.

7.6 Risk based approach - Economics

7.6.1 Base case

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 A for further information).

The results of this assessment are given below. This shows that the cost of all the options are proportionate in cost. In terms of the practicable options to reduce the likelihood of dam failure this economic assessment shows that:

- a) Two concrete auxiliary spillways with the crest of the non-overflow section 1.7m above the auxiliary spillway crest (freeboard) would pass a 1 in 60,000 chance per year flood and be just disproportionate in cost
- b) However, the Council does not own the downstream face of the dam, and the landowner is likely to be resistant to Option A which would remove the existing tree backdrop to Turkey Mill Pond, used as a wedding venue by the downstream landowner. This leaves Option C which if constructed in concrete with a 1.7m freeboard (C2-50) would pass a 1 in 10,000 chance per year flood and be proportionate in cost
- c) It could be argued that a concrete chute would be unacceptable in a Grade II listed park, which would leave a grasscrete option (C1-50) with 1.2m freeboard (the maximum depth of overtopping grasscrete can withstand) which could pass a 1 in 3000 chance per year flood and be proportionate in cost

It is concluded that to satisfy the Reservoirs Act 1975 “matters in the interests of safety” to reduce the risk of failure due to overtopping, one of these options should be selected and implemented.

Table 7-1: Summary of costs and benefits for options shown in Table 6.5

Option	Works involved (detail in table 6.5)	Budget project cost (Appendix E)	Annual chance (1 in) of dam failure with release of reservoir	Cost to save a life £M (Note 1)
Existing			100	n/a
A1-40	40m wide grasscrete on dam	£1.2M	2000	3.3
A2-40	40m concrete on dam	£1.7M	8,000	4.5 (Note 2)
B	strengthen crest	£0.7M	1, 500	1.9
C1-50	50m grasscrete on abutment	£1.4M	4,000	3.6
C2-50	50m concrete on abutment	£1.9M	20,000	5.1
A2-40 and C2-50	Both concrete spillways	£3.6M	80,000	9.3

Notes. Annual chance of onset of damage governed by overtopping of crest of non-overflow embankment and say typically 95% of failure flow (given in Table 6.5)

7.6.2 Sensitivity

As with any analysis there is uncertainty in the various input values and assumptions, which provides uncertainty in the output value. Key uncertainties include the following

	Assumption	Effect on economic case
1	Neglect economic damages	If included then economic benefits justify scheme independent of risk to life
2	Use total ASLL, rather than incremental	Could double CSL
3	If two spillways were considered, what is the CSL for the second spillway	CSL for A2-40 as increment on scheme which reduces probability of failure to 1 in 1000 is £35M/ life i.e. disproportionate
4	Peak flood flows	Current estimates (figure 2.3) have been made using current approved methodologies, including independent estimates by separate hydrologist to check. They may increase with climate change

7.7 Other considerations

7.7.1 General

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, heritage and environment.

Overall the spillway should be updated to meet current engineering standards (1 in 10,000 design standard, PMF for safety check flood), unless there are compelling reasons why this is not reasonably practicable.

It is recognised that land issues mean that Option A is unlikely to be acceptable to the owner of the downstream face of the dam, and that only Options B or C are likely to be acceptable to him.

Option B is not recommended unless there are compelling reason why option C cannot be adopted, which have not been presented in discussions on the draft report.

This leaves Option C which will include the following impacts

- a) loss of paths along west side of West Drive (and bridge over stream)
- b) reprofile/ lower land east of West Drive to remove obstruction to flow in extreme floods
- c) creation of sections of wall as part of forming non-overflow crest
- d) loss of trees

None of these is considered disproportionate in the context of reducing the risk to life from dam failure, such that Option C is considered proportionate with the only choice being between a concrete chute or grasscrete, the choice affecting both appearance and the height of the non-overflow embankment remote from the auxiliary spillway.

7.7.2 Downstream reservoir

Although the downstream reservoir does not currently come under the Reservoirs Act, there is provision following the 2010 FWMA to lower the threshold for registration from 25,000 to 10,000m³. **It is therefore recommended that this is brought to the attention of the owner of the downstream reservoir, and that they commission a review of the spillway capacity of Turkey Mill Pond, and if practicable increase it to the same level of resilience as Mote Park Lake. The costs for the study and any works should be borne by the owner of that lake.**

7.7.3 Risks 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 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 increases in spillway capacity.

A further risk 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.

7.7.4 Downstream fluvial flood risk

The proposed works should make no change in flood risk up to the 1 in 100 chance per year flood. For larger floods water would spill in a controlled manner with the likelihood of the dam failing reduced.

The main issue is the effect of the works on the water bodies at Turkey Mill. **It is recommended that:**

- a) **The Environment Agency be asked for their surveys of the various gates and river channel sections through Turkey Mill**
- b) **A working group be set up with the owners of Turkey Mill to understand the capacity and condition of their spillways/gates, make them aware of their responsibilities to pass river flows on and agree liaison over operation of the Mote Park gates.**

7.7.5 Heritage

The park has statutory listing under UK national legislation, as a Grade II park, such that consent will be required from Historic England (see Section 2.6). It is considered that all of the options presented should be capable of detailing to achieve their consent, with early consultation recommended.

7.7.6 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 A 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.

7.8 Discussion and Conclusions

The economic assessment shows that **all options are proportionate, with a cost to save a life of the order of £5M, and thus a major upgrade along these lines is unavoidable, and will be enforced by the Environment Agency.**

8 References

- | | | |
|----------------------------|------|--|
| ICE | 2015 | Floods and Reservoir Safety 4 th Ed. |
| CIRIA | 1987 | Design of reinforced grass spillways. Report 116. 119pp |
| Environment Agency | 2013 | Guide to Risk Assessment for Reservoir Safety Management (RARS) |
| Freer | 1992 | Recent examples of reinforced grass spillways on embankment dams based on CIRIA Report 116. . Water Resources and Reservoir Engineering (eds N M Parr, J A Charles and S Walker). Proceedings of 7th British Dam Society Conference, Stirling, pp 167-174. Thomas Telford, London. |
| 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 - 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

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

where $\Delta \text{ Pf}$ 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:-

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

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 $5 \times \text{£1.7M} = \text{£8.5M}$ 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	<p>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.</p> <p>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.</p>
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	<ul style="list-style-type: none"> ○ Rules of thumb adopted by D/Ds; ○ NSD takes as its starting point the HSE submission to the 1987 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 B - Examples of reinforced grass spillways

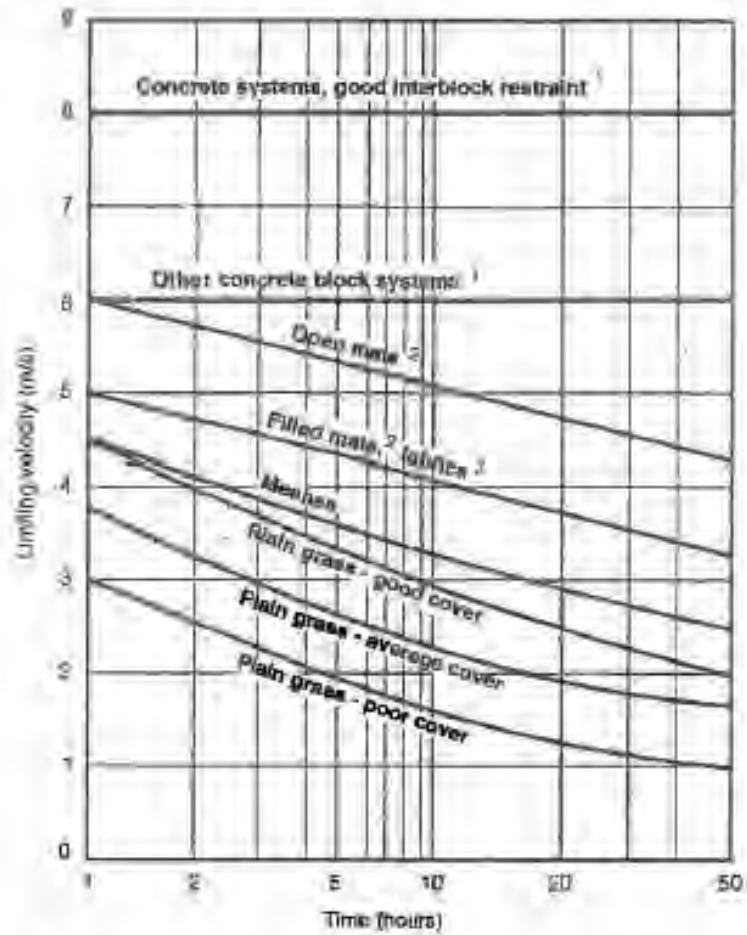
Key aspects of reinforced grass open mat systems, as defined in CIRIA Report 116

	Geotextile systems (three dimensional open)	Comment
Reinforcement	The entire surface area is grassed, and the roots bind around the geotextile to form a strong continuous geotextile/soil/root mat	
Allowable velocity	See Figure below	
Topsoil cover	In order to protect the geotextile from damage by vandalism, animals or mowing machinery, the geotextile may be covered with a thin (up to 20 mm) layer of topsoil.	
Subgrade	The foregoing section relates to subsoil of moderate to low permeability (typically less than 10^{-5} m/s) . This guide is not generally applicable to subsoil with high permeability which requires special consideration (5.4.1)	The existing dam is a heterogeneous mix of clay and sand, so the upper part needs to be replaced/infilled with low permeability fill

Key aspects of reinforced grass concrete block systems, as defined in CIRIA Report 116

Aspect	Concrete systems, Insitu and good interblock restraint	Other concrete block systems
System requirements 4.3.3 of CIRIA)	Nominal face-to-face contact length of more than 75% of block or bay perimeter. In addition, for block systems, grass plants must be able to grow between the blocks to develop soil root wedging, which also provides additional root anchorage.	Nominal face-to-face contact length of between 40% and 75%
Interblock friction	Provided through cables, or for insitu concrete systems (grasscrete) dowels between large (nominally 10m x 10m) panels	Mechanical and grass root action. Note some suppliers recommend gravel/ sand wedging but if too thick can be vulnerable to wash out
Connection to subgrade	<p>S 2.2.3 of CIRIA <i>“The grass roots must grow through any underlayer into the subsoil in order to establish adequate rooting depth both for sustaining grass growth (Section 6.2.4) and for achieving additional root restraint (Sections 4.4.3 and 5.4.4).”</i></p> <p>S4.3.3 of CIRIA <i>“Concrete reinforcement systems used in conjunction with grass have been shown to be stable (i.e. not to move appreciably under hydrodynamic excitation - see Appendix 3) at velocities in excess of 8.0 m/s provided:</i></p> <ol style="list-style-type: none"> 1. Downslope seepage flow below the armour layer is minimised by good contact between the concrete armour and the underlayer/subsoil. 2. Surface irregularities, which would cause high localised uplift forces, are avoided. 3. Good lateral restraint exists between adjacent blocks or bays. 4. Good root (or mechanical) anchorage exists between the concrete armour and the subsoil” 	

Allowable velocity on plain and reinforced grass

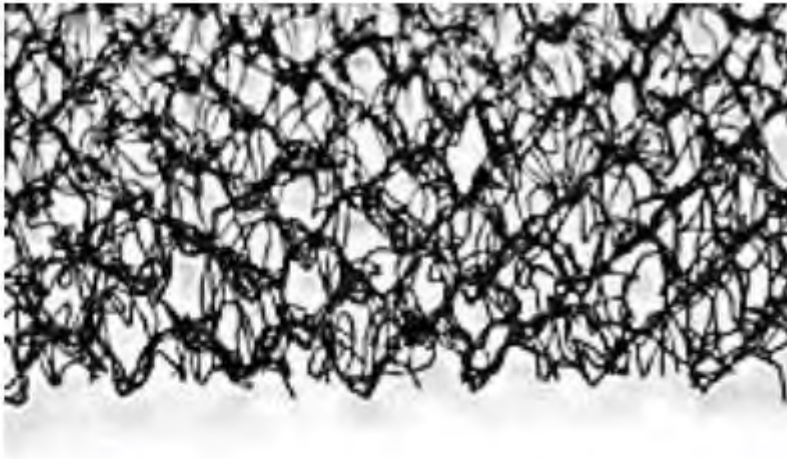


Notes

1. Minimum superficial mass 135kg/m²
2. Minimum nominal thickness 25mm
3. Installed within 20mm of soil surface, or in conjunction with a surface mesh
4. These greates should only be used for erosion resistance to unidirectional flow
5. All reinforced grass values assume well-established good grass cover

Figure 12. Recommended limiting values for erosion resistance of plain and reinforced grass (source: CIRIA Report No. 116)

Photograph 1: 20mm open mat



Photograph 2: Grass reinforced mat



Photograph 3: Grass reinforced matting in operation (Aldington FSR, Kent, 2000 floods)



Photograph 4: Cable tied concrete block system
(Armourflex by Contech)



Photograph 5: Cable tied concrete blocks (Bruton FSR, Somerset, prior to upgrade)



Photograph 6: Cable tied system (Aldermaston, Berkshire)



Photograph 7: Grasscrete block system (Wigan FSR while grass establishing)









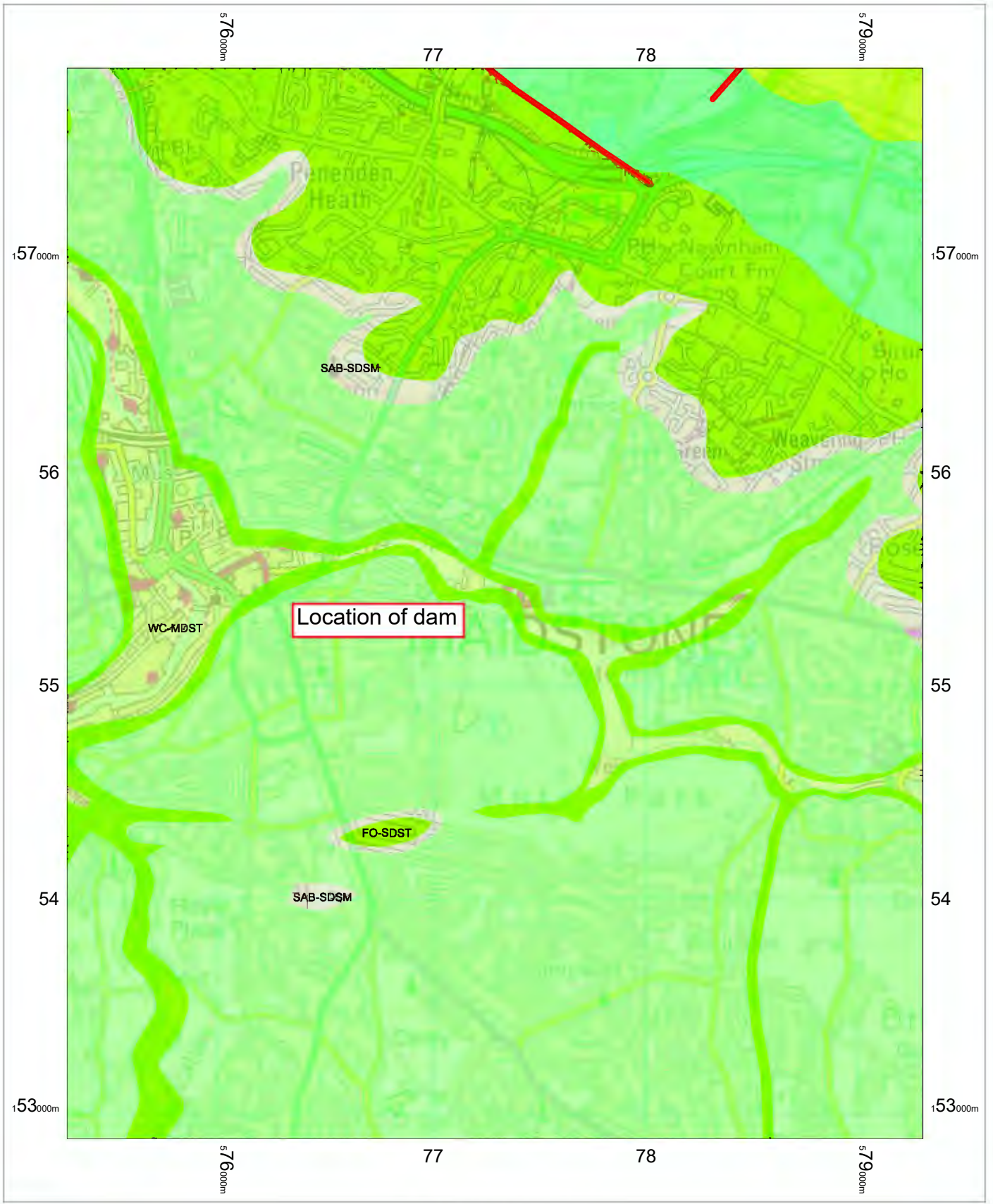
Photograph 8 slope as edge detail



Appendix C - Published Geology

KEY TO BEDROCK GEOLOGY

Map Colour	LEX Code	Rock Name	Rock Type	Max Age (Period)
	WMCH-CHLK	West Melbury Marly Chalk Formation http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=WMCH	Chalk	Cretaceous
	GLT-MDST	Gault Formation http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=GLT	Mudstone	Cretaceous
	FO-SDST	Folkestone Formation http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=FO	Sandstone	Cretaceous
	AC-STMD	Atherfield Clay Formation http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=AC	Sandstone And Mudstone	Cretaceous
	HY-SDLM	Hythe Formation http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=HY	Sandstone And [Subequal/Subordinate] Limestone, Interbedded	Cretaceous
	SAB-SDSM	Sandgate Formation http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=SAB	Sandstone, Siltstone And Mudstone	Cretaceous
	WC-MDST	Weald Clay Formation http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=WC	Mudstone	Cretaceous









BGS 50k Geology - OS 50k Mapping
 20 January 2017, ID: CM-00592689
www.centremapslive.co.uk
 1:25000 scale print at A4, Centre: 577290 E, 155385 N
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FOR A BETTER POINT OF VIEW

KEY TO SUPERFICIAL GEOLOGY

Map Colour	LEX Code	Rock Name	Rock Type	Max Age (Period)
	ALV-CZPS	Alluvium http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=ALV	Clay, Silty, Peaty, Sandy [Unlithified Deposits Coding Scheme]	Quaternary
	HEAD-XCZ	Head http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=HEAD	Clay And Silt [Unlithified Deposits Coding Scheme]	Quaternary
	HEAD-XCZSV	Head http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=HEAD	Clay, Silt, Sand And Gravel [Unlithified Deposits Coding Scheme]	Quaternary
	HEAD-XVSZC	Head http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=HEAD	Gravel, Sand, Silt And Clay [Unlithified Deposits Coding Scheme]	Quaternary
	RTD1-XSV	River Terrace Deposits, 1 http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=RTD1	Sand And Gravel [Unlithified Deposits Coding Scheme]	Quaternary
	RTD4-XSV	River Terrace Deposits, 4 http://www.bgs.ac.uk/Lexicon/lex_list.cfm?pub=RTD4	Sand And Gravel [Unlithified Deposits Coding Scheme]	Quaternary

Appendix D - Supplementary photographs relevant to spillway upgrade

Options	Photographs in	
	2014 Section 10	This report
A1 – Grasscrete auxiliary spillway	4 to 11	9
A2 – Concrete auxiliary spillway		
B – Strengthen existing crest	7, 8	
C – Grasscrete/ concrete spillway over abutment	14	12 to 16
Turkey Mill		17 to 21
River Len gauging station		22

Photograph 9: East end of high level path; showing end of crest wall



Photograph 10: Low spot providing access to Mote Park boat club storage container



Photograph 11: View west along West Ride



Photograph 12: West ride near existing stream (foreground would be lowered to road level under Option C)



Photograph 13: West Ride bridge over existing spillway channel (control on flows)



Photograph 14 View from upstream of position of Option C (holly covered stump on right is end is where fence runs up downstream face)



Photograph 15: Spillway channel from downstream end (Option C would mean removal of trees and lowering by several metres)



Photograph 16 Downstream face of abutment at location C



Photograph 17: Impact on views from Turkey Mill

Impact on views from Turkey Mill

Option A

Option C – hidden
by trees along lake



2017/04/20

2017/04/20

Photograph 18: Spillway from Turkey Mill Pond



Photograph 19: Gates at end of Mill Race



Photograph 20: Culvert under Mill



Photograph 21: View indicating floor level of Mill building well below level of Mill race



Photograph 22: River Len gauging station under car park



Photograph 23: River runs alongside (to right of) loading area to “the Mall”



Appendix E - Cost estimate

Costing of options for ALARP analysis													
Mote park													
Items	Assumptions	Rate	Units	Quantities					Cost (£)				
				A1 - 40m wide Grass reinforced (concrete blocks)	A2-40m wide concrete spillway	B Strengthen crest to inhibit breach	C1-50 Grasscret e on abutment	C2-50 Concrete on abutment	A1 - 40m wide Grass reinforced (concrete blocks)	A2-40m wide concrete spillway	B Strengthen crest to inhibit breach	C1-50 Grasscrete on abutment	C2-50 Concrete on abutment
		1.05											
Enabling works													
E1	Services - provisional items	£20,000		1	1	1	2	2	£20,000	£20,000	£20,000	£40,000	£40,000
E2	Tree clearance	£10,000		3	3	2	2	2	£30,000	£30,000	£20,000	£20,000	£20,000
Auxiliary spillway													
S1	Excavation of soil	£16	m3	1800	1800		3,300	3,300	£28,350	£28,350		£51,975	£51,975
S2	E/O Rock excavation	£47	m3	0			330	330	£0			£15,593	£15,593
S3	Dispose of unsuitable	£26	m3	1800	1800		3,300	3,300	£47,250	£47,250		£86,625	£86,625
S4	Imported fill (clay)	£42	m3	80	80		100	100	£3,360	£3,360		£4,200	£4,200
S5	Enkamat	£9	m2	0	0		0	0	£0	£0		£0	£0
S6	Grasscrete/ armourflex	£63	m2	900	0		1,125	0	£56,700	£0		£70,875	£0
S7	concrete slab	£300	m3	0	450		0	550	£0	£135,000		£0	£165,000
S8	Reinforcement (75 kg/ m3)	£1,575	t	0	34		0	41	£0	£53,156		£0	£64,969
S9	soil dowels to resist uplift	£300	No	0	80		100	100	£0	£24,000			£30,000
S10	Mass concrete (toe/ edge beam)	£184	m3	25	25		31	31	£4,594	£4,594		£5,742	£5,742
S11	Reinforced concrete (crest weir, side walls, toe, seepage cut-off)	£231	m3	225	225		281	281	£51,975	£51,975		£64,969	£64,969
S12	Reinforcement (125 kg/ m3)	£1,575	t	28	28		35	35	£44,297	£44,297		£55,371	£55,371
S13	Formwork	£95	m2	105	105		131	131	£9,923	£9,923		£12,403	£12,403
S14	Masonry cladding	£147	m2	11	11		13	13	£1,544	£1,544		£1,929	£1,929
S15	Sheetpiling up to 6m deep	£184	m	135	135		165	165	£24,806	£24,806		£30,319	£30,319
Non-overflow spillway													
C1	Excavation of soil	£16	m3	180	180	600	150	150	£2,835	£2,835	£9,450	£2,363	£2,363
C2	E/O Rock excavation	£47	m3	0					£0				
C3	Dispose of unsuitable	£26	m3	180	180	600	150	150	£4,725	£4,725	£15,750	£3,938	£3,938
C4	Imported fill (clay)	£42	m3	360	540	300	300	450	£15,120	£22,680	£12,600	£12,600	£18,900
C5	Footpath/ road surfacing incl subbase	£95	m2	0	0	200	0	0	£0	£0	£19,000	£0	£0
C6	Mass concrete (toe/ edge beam)	£184	m3	0	0	0	0	0	£0	£0	£0	£0	£0
C7	Reinforced concrete (crest wall)	£231	m3	180	264	100	150	220	£41,580	£60,984	£23,100	£34,650	£50,820
C8	Reinforcement (125 kg/ m3)	£1,575	t	23	33	13	19	41	£35,438	£51,975	£19,688	£29,531	£64,969
C9	Formwork	£95	m2	360	528	200	450	660	£34,020	£49,896	£18,900	£42,525	£62,370
C10	Masonry cladding	£147	m2	72	106		90	132	£10,584	£15,523	£0	£13,230	£19,404
C11	Sheetpiling up to 6m deep	£184	m			600			£0	£0	£110,250	£0	£0
Bridge works													
C8	remove existing parapets / make good	£5,000	Sum	1	1	1	1	1	£5,000	£5,000	£5,000	£5,000	£5,000
C9	Concrete bridge restraint (parapets)	£315	m	20	20	20	20	20	£6,300	£6,300	£6,300	£6,300	£6,300
C10	Replacement footbridge												£0
Temporary works													
T2	Cofferdams in reservoir	£120	m2	120	120				£14,400	£14,400	£0	£0	£0
T3	Groundwater control in excavations	£1,680	week	8	8		8	8	£13,440	£13,440	£0	£13,440	£13,440
T4	Pumping to lower/ control RWL	£2,783	week	8	8				£22,260	£22,260	£0	£0	£0
T5	Temporary access road, incl site Heras fencing	£236	m	200	200	120	120	120	£47,250	£47,250	£28,350	£28,350	£28,350
									£575,749	£795,522	£308,388	£651,927	£924,947
Minor items													
	Contractors preliminaries	30%							£115,150	£159,104	£61,678	£130,385	£184,989
									£172,725	£238,657	£92,516	£195,578	£277,484
	Construction contract value								£863,624	£1,193,283	£462,581	£977,891	£1,387,421
Other works													
	Relocate container for Mote park Boat club								£20,000	£20,000	£20,000	£20,000	£20,000
Professional fees, surveys etc.													
	Engineering (QCE, design and supervision)	10%							£86,362	£119,328	£46,258	£97,789	£138,742
	Physical model test												
	Ground investigation for detailed design								£15,000	£15,000	£15,000	£15,000	£15,000
	Ecological surveys/ inputs								£20,000	£20,000	£20,000	£20,000	£20,000
	Other fees (land agents etc.)								£20,000	£20,000	£20,000	£20,000	£20,000
									£141,362	£174,328	£101,258	£152,789	£193,742
	Subtotal Project cost								£1,024,986	£1,387,612	£583,839	£1,150,680	£1,601,163
	Contingency	20%							£204,997	£277,522	£116,768	£230,136	£320,233
	TOTAL								£1,229,984	£1,665,134	£700,607	£1,380,816	£1,921,396

Appendix F - Outline scope for next stage

It is noted that the finalised layout must be approved by an All Reservoirs Panel Engineer who should oversee the design and construction in the role of Qualified Civil Engineer (QCE) as required under Section 10 of the Reservoirs Act. This oversight would then allow him to issue the Section 10(6) certificate under the Reservoirs Act 1975 confirming that the recommendations have been implemented.

The following additional surveys and site information is required:

- a) Extend topographic survey to cover area of options
- b) Ground investigation
- c) Full services search
- d) Plan with land boundaries

Planning design shall be carried out by an engineering firm familiar with design of hydraulic structures on dams. The output should be suitable for planning submission, and include

- a) Consultation with both statutory consultees and affected parties.
- b) General arrangement with plans, sections etc showing design
- c) Suitable details to confirm the technical viability and dam safety in operation, e.g. in terms of under seepage, scour etc
- d) Design report with hydraulic, geotechnical and structural calculations to substantiate design
- e) Buildability report giving designer's anticipated method of construction, including dealing with risk of floods during construction
- f) Acceptance by the QCE

Detailed design shall be shall be carried out by an engineering firm familiar with design of hydraulic structures on dams. The output should be suitable for both tender submission and construction, and provide sufficient information for acceptance by the QCE, and to form a permanent record of the dam construction suitable for use in future ten yearly dam safety reviews.

Attachment- Role of the QCE

Date April 2017

Subject Inputs by QCE into reservoir safety works

Author(s) Alan Brown, David Littlemore

This note sets out a high level summary of the inputs by a Qualified Civil Engineer (QCE) under Section 10(6) of the Reservoirs Act 1975 in relation to construction works to a reservoir, which do not affect top water level (which would require a Construction Engineer).

Stage	Indication of inputs by QCE (or his designated representative)
Design	<ol style="list-style-type: none"> 1. Site visits as appropriate, as a minimum on inception and as part of review of tender drawings 2. Review and acceptance of design by others to include <ol style="list-style-type: none"> a) Earthworks including internal filters/ drains b) Fill material source/ treatment/ compaction c) Spillway capacity and design d) Outlets – provision/ capacity/ detailing e) Foundation treatment (embankment and structures) and seepage cut-off arrangements f) Instrumentation g) Supporting documents as set out below 3. Supporting documents and drawings to support design of permanent works and suitable as record of design for use in future ten yearly dam safety reviews under the Reservoirs act, (by others), to include Design Report setting out design criteria, assumptions and output (safety factors, seepage flows, index, compaction and shear strength properties of anticipated fill materials, flood flows and levels). 4. Supporting Buildability report by designer, setting out constraints on construction and temporary works and including are least one physically possible scheme for temporary works and construction methodology. This should be made available to the Contractor (a common approach is to include it as an appendix to the CDM pre-construction information for the Construction Contract.) 5. Design is normally carried out most economically in stages <ol style="list-style-type: none"> a) Outline (planning) design, equivalent to approval in principle (AIP) for Highways (Highways Agency Design manual for roads and bridges BD 2/05); b) Detailed design/ tender documents/ specification. 6. Assistance with developing the above and detailing, which may include one or more depending on the specific project requirements: <ol style="list-style-type: none"> a) periodic design development meetings; b) telephone progress reviews; c) risk management workshops; d) value engineering workshops.

Stage	Indication of inputs by QCE (or his designated representative)
Construction	
Method statements	Review and accept all that could impact reservoir safety including <ol style="list-style-type: none"> 1. River diversion / flow control arrangements and contingency plans for high flow (including alarm levels/ emergency plan if flood flows greater than temporary works capacity occur) 2. Dam earthworks including selecting excavated materials for re-use as fill and imported fills 3. Grass reinforced structures 4. Reinforced grass structures (spillways, embankment slopes); 5. Foundation excavation and treatment and cut-off arrangements – all areas under footprint of dam, including structures (latter includes springs, soft spots if/ when encountered in the foundation); 6. Instrumentation monitoring during and post construction 7. Concrete inlet/ outlet/ spillway structures 8. CDM Construction Phase Plan.
Technical queries	Any technical queries that arise during construction, including on drawings, quality of construction, unexpected ground conditions etc. should include QCE acceptance of resolution
Site visits and records	Visits and record photographs / sketches of all stages which could affect reservoir safety, by the Construction Engineer or his designated representative; typically comprising: <ol style="list-style-type: none"> 1. On commencement to ensure reservoir safety issues understood by contractor, site supervisor etc 2. Prior to any river diversion/ removal of cofferdams etc. 3. Installation of cut-off arrangements; 4. Foundation for embankment; 5. Foundation for each structure, prior to placing blinding; 6. Formation/ first placing of any sand/ drainage blanket; 7. Early stages of placing bulk earthfill; 8. Structural concrete, to inspect formwork etc. before placing and then during concrete placing; 9. Final check on completion: as-built details including surveys of crest levels and critical structures (e.g. spillways and outlets).
End of construction	Issue 10(6) certificate with Annex (as overleaf)

Typical contents list of Annex to
• Certificate under Section 10(6)

	Title	Content includes	Completed by
1	General		QCE
2	Parties involved in design and construction		Client
3	Geology & groundwater conditions	List exploratory holes, summarizes published geology, construction stage mapping	Designer
4	Design standard	Consequences of failure, Consequence Category, Modes of failure	QCE
5	Hydrology and reservoir operation		Designer
6	Hydraulic design	Outlet, spillway	Designer
7	Embankment design	Internal zoning, volumes, compaction spec	Designer
8	Foundation design	Embankment and structures; cut-off arrangements	Designer
9	Construction	Dates of key activities, details of problems encountered, summarise construction testing	Contractor
10	Instrumentation	Schedule of instruments and purpose; monitoring regime	Designer
11	Operation and Maintenance	Operation and testing of active equipment (controls, valves); anticipated maintenance etc	Designer
12	Other Information	Any other information e.g. Contractor designed items	Designer/ Contractor
Appendices		Most as attachments (existing notes)	
A	Site plans	Extent of surface water and fluvial flooding with no dam failure	Designer
B	Design Information	PMF flood study, flood modelling report, GIR, Design Report	Designer
C	Photographs of construction features	Photographs should be date/time stamped	Designer/ Contractor
D	Construction stage compliance testing	Tests of Concrete, earthworks	Contractor
E	Anticipated operation	O&M manual, Onsite plan	Designer/ Client
F	As constructed drawings	All drawings including as-built survey	Designer/ Contractor