Report on aspects of SESRO dam design

January 2024

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Contents

Executive Summary

Report scope and availability of design information

This report gives my comments as a former All Reservoirs Panel Engineer on the engineering design aspects of the proposed 150 Mm^3 SESRO reservoir, as described in the reports made available to me by GARD. My report also provides commentary on the adequacy of the available information for determining the safety of the design and estimating the cost of reservoir construction.

The most recent source of information on the dam design is the 2022 Gate 2 Concept Design Report. This contains little detail of the engineering design of the embankment and its associated inlet/outlet works. There are no engineering drawings. Thames Water have told GARD that the design of the embankment has not progressed since the draft Jacobs Preliminary Design Report in 2007, which contains some engineering detail, but only at the 'preliminary' level implied by the report title. I find the absence of design development over the past 16 years surprising, especially in view of the considerable expenditure allocated by Ofwat for the Gate 2 investigations of SESRO reservoir.

Trial embankments

One of the recommendations of the Preliminary Jacobs report in 2007 was for a large scale trial embankment with a height of at least 20m. This would establish the soil parameters for analysing embankment stability and leakage, as well as providing information on constructability and cost of the embankment. Thames Water now proposes to delay the large scale trial embankment until after the construction of the reservoir has been approved through the Development Consent Order. Instead, Thames Water has proposed construction this year of a much smaller trial embankment, only 3m height and over a far smaller area than the large scale trial planned to be conducted later during construction.

In my opinion, the initial 3m trial embankment will not provide sufficient information to reliably determine the embankment slopes and cost of the reservoir for the purpose of justifying the reservoir in the WRMP or for approval of the DCO. The large scale embankment, as recommended by Jacobs, is needed to determine the embankment slopes and inform reliable estimates of construction costs. Thus, I believe that the large scale trial should be completed before the scheme design, and costs, are submitted for DCO and regulatory approval. I think it would be unsafe for the Government to make a decision on whether SESRO reservoir should be the next major source for the South East before the main trial bank findings are established, the design known, and the dam break assessment completed.

Adequacy of current designs

My comments on the adequacy of the current SESRO design are severely limited by the very limited engineering design detail. However, from the available information my views are:

1. The factors of safety assumed by Jacobs in 2007, although generally reasonable for a fully designed dam, may need to be made more conservative to reflect the 10 km length of the embankment, the limited experience of designing and constructing such a large new dam in UK in the last 40 years, the known variability of the soils within the 6 km^2 borrow pit, and the potential human and physical impact of dam failure.

2. If the embankment slopes have to be as flat as the "worst" case considered by Jacobs, which are similar to the slopes eventually used at Abberton reservoir after a slip failure during construction, the reservoir costs would be appreciably more than current estimates.
3. The assumed freeboard of 1m seems reasonable for the estimated wind speeds, fetch, rip rap protection, and wave wall, provided the settlement allowance is appropriate.
4. An explanation is needed for reduction of riprap quantity of over 60% since the Mott MacDonald feasibility report in 2018, along with appropriate supply sources.
5. The apparent absence of any detailed design of the reservoir inlet/outlet works needs to be rectified. This applies particularly to safe design of the emergency drawdown facility which would have to release about 76 m^3/s, whilst safely dissipating about 25m of energy head. Mitigation of its physical and environmental impact on the downstream water course and the River Thames needs to be properly detailed.
6. The risk of embankment failure and the resulting loss of life and economic damage need to be taken into account in the safety factors assumed in the embankment stability analysis which will determine the embankment slopes. Therefore, in my opinion, the dam break analysis should be undertaken before the design is finalised for the regulatory and DCO approval.

Cost implications

The current Thames Water proposal for SESRO is to seek WRMP and DCO approval prior to carrying out the dam break analysis, prior to the main field soil trial embankment, prior to the main stability analysis, prior to identifying sources of imported riprap and drainage material and thus prior to finalising the design of the embankment and outlet works. These all carry high potential for generating cost over-runs.

Dams are notorious for cost over-runs. Aside from Carsington dam which failed during construction, with consequent large cost over-run, in the past 40 years there has been no UK experience of the design and costs of new large dam construction.

However, overseas there has been a lot of experience of cost over-runs typified by a recent Australian paper based on experience of 40 dams whose findings suggest a median cost overrun of all types of dam of 49% and for embankment dams 106%, based on costs estimated immediately prior to construction and excluding inflation increases. The Australian paper also states "*some authors take this last point a step further and implicate manipulated forecasts as a probable cause for many cost overruns.*" In my opinion, Thames Water's combined allowance of 51% for costed risk and optimism bias is far too low in view of the immature state of the of the reservoir design and the apparent failure to consider a lot of the cost risks that I have identified.

This all points to the need for much more engineering investigation, design and cost assessment work before a decision can be safely taken on the choice of the next major water source for the South East.

1. Introduction

Scope of this report

I was for about 30 years, a Panel Engineer under the Reservoirs Act 1975 including being Construction Engineer for two new small embankment dams as well as having responsibility for the design or construction of two other dams over 50m high overseas. I was also, for several years, a member of the ICE Reservoirs Committee who interviewed applications to become Panel Engineers under the Reservoirs Act. My post graduate study was in Soil Mechanics for embankment dams at Imperial College. My CV is attached as Appendix 1.

I have been asked by GARD to comment on engineering design aspects of the South East Strategic Reservoir Option (SESRO) as contained in various reports and presentations. My scope of work includes commentary on the adequacy of currently available information for determining the safety of the design and estimating the cost of reservoir construction. The Terms of Reference of this assignment are attached as Appendix 2.

Information availability

The primary sources of information on the proposed SESRO design are:

1. Factual reports on geotechnical investigations carried out in 1992 and 2006
2. Excerpts from the 2007 Jacobs draft Preliminary Design Report, included as Appendix 3 to this report.
3. The 2022 Gate 2 Concept design report
4. The 2018 Mott MacDonald Reservoir Feasibility Report for Thames Water's WRMP19

The Jacobs report excerpts, and the factual site investigation reports were provided by Thames Water in response to an information request by GARD in September 2023, as shown in Appendix 3.

The 2007 Jacobs draft report contains a fair amount of engineering detail of the type that I would expect in a "preliminary" design report. However, neither the 2018 feasibility report nor the 2022 Gate 2 report contain drawings showing engineering details of the reservoir or sufficient technical justification for the envisaged design of the works.

GARD's information request in September 2023, as in Appendix 4, asked for details of the embankment cross-section and "*Provisional stability analyses of the SESRO embankment, including reasons for the choice of parameters and what further information would be collected to support them and reasons for the factor of safety chosen.*"

Thames Water's response to this request, as in Appendix 5, was "*At the time of writing (Q4 2023) the design of the dam has not been developed further than the 'Preliminary Design' described in the 2007 Preliminary Design Report.*" I find this response most surprising. It has been 16 years since the Preliminary Design Report. The Gate 2 main report shows that Ofwat allowances for the costs of Gate 1 and Gate 2 investigations were greatly under-spent[1]:

Table 11.1 Gate 2 forecast total cost for each partner company

Company	Forecast Total Cost to RAPID Gate 2 (£M, 2017/18 prices)	Ofwat FD Allowance for Gate 2 (£M, 2017/18 prices)	Previous underspend on Gate 1 (£M, 2017/18 prices)	Saving (£M)
Thames Water	4.84	12.23	7.13	14.52
Affinity Water	2.39	6.02	3.51	7.15
TOTAL	**7.23**	**18.26**	**10.65**	**21.67**

It appears that the available allowance to the end of Gate 2 was under-spent by about £22 million. In my opinion, the full allowance should have been used to improve the availability of design details for the reservoir, enabling more comprehensive and reliable cost estimates and providing a better basis for comparison with other strategic options in WRMP24. In this report, I will comment further on the adequacy of design detail for various aspects of the reservoir.

2. Jacobs draft Preliminary Design Report, 2007

Following a request for information by GARD, Thames Water sent a copy of part of the Preliminary Design Report of what was then called the 'Upper Thames Reservoir' (since retitled as SESRO)[2].The draft, work in progress Preliminary Design report v4 by Jacobs dated 6th January 2007, comprised 43 pages in chapters 2, 3 &5. By Thames Water's own email of 19.10.23, this is the latest work on the Preliminary Reservoir Design, but there is no explanation of why the whole report was not provided.

Jacobs' draft 2007 report goes into the available soil materials, their (Jacobs) laboratory test results, and the design of the reservoir embankments in appreciable detail. My understanding is that this report has only recently been released into the public domain, 16 years later. If so, as the reservoir has been in all the intervening Water Resource Management Plans, it would have been helpful from its original date. As Jacobs' report has not previously been in the public domain (it is not on the web-sites for Thames Water's WRMP24 or for the Gate 2 reports), it is attached as Appendix 3.

I cannot find the size of the reservoir quoted but it would appear to be the 150Mm^3 volume which was part of the 2009 Thames Water dWRMP proposal.

Historic experience

In reviewing the Jacobs report, I have borne in mind that there have been a number of failures of embankment dams in the UK, generally near the end of construction. In this instance I will just highlight a few to illustrate the situation and what can occur. For example Defra's report

[1] SESRO main Gate 2 report, page 56
[2] 'Upper Thames Major Resource Development;: Draft Preliminary Design Report, v4.0' 2.5.1 , Jacobs 2007

on historic dam failures states "*During 1937 major slips occurred at three embankment dams under construction*." [3]These were William Girling, Hollowell, and Abberton dams.

The report on Abberton states "*The original slope was 1:4… major deep-seated slip of the upstream slope took place during construction on 20th July 1937 with the embankment within two metres of the planned height. The dam crest dropped by 3.5m and the upstream toe moved outward by 15m…The original upstream slope of 1:4 was changed to between 1:7 and 1:11…Back analysis of the construction slip indicates that failure was caused by high pore pressures in the foundation…where rapid construction did not allow sufficient time for pore pressures to dissipate"*[4]. "*Extensive remedial works were undertaken to remove the slip and rebuild the embankment on an upstream slope which progressively slackens from 1 on 7 to 1 on 11 "*[5]

The rebuild would have both delayed the project completion appreciably and increased capital cost substantially. This illustrates the importance of allowing sufficient contingency in the design parameters and embankment slope, especially bearing in mind that the 10 km length of the SESRO embankment makes the risk of failure much higher than for an impounding dam across a valley.

Design data

The following are taken from the Jacobs report: *"For current design purposes, tests have been carried out on samples re-compacted in the laboratory. Whilst these would not be truly representative of field behaviour, they are used conventionally and are adequate for preliminary design purposes*." pages 2-8.

Jacobs Table 2-2 shows the variation in the soil index properties. Most of the fill will be Kimmeridge clay. Its moisture content varies from 7% to 45% with a mean of 26%, whilst the Plasticity Index varies from 13% to 47% with a mean of 32%. This indicates that the Kimmeridge clay is not a uniform material. Also "*The plot of undrained shear strength versus depth shows considerable scatter." "Both the Gault and Kimmeridge are plastic clays which can be expected to exhibit strain softening. Strain softening is a loss in strength with increasing strain after peak strength has been mobilised."*[6] Figure 2.3 shows 6 test results to be below the Lower Bound of undrained shear strength. These factors would indicate that care would be needed in selecting appropriate index properties and factors of safety.

"*The slope angles as currently defined are based on the 10 percentile strength profile for the foundations. " 3.4.3. "There is a possibility that the trial embankment would show this to have been an overly conservative estimate of the foundation strength."* 3.4.3

However, those cross sections are based on there being 10 % of soil test results being weaker, see Figure 2.3 page 2-13. Is it possible that this proves to be too risky and flatter slopes are required?

[3] Defra EA Lessons from historical dam incidents SC080046/R1 page18
[4] Defra EA Lesons from historical dam failures SC080046/R1 page 133.
[5] Hird et al Monitoring embankment performance during the raising of Abberton Reservoir Thomas Telford 2012 page 3
[6] Jacobs Draft Preliminary Design Report 2007

Jacobs' Draft Preliminary Design Report quotes target factors of safety and compliance from their preliminary modelling as in their Table 3.5 below:

Table 3.5 Summary of limit equilibrium analysis for 25m high embankment

Case	Description	Embankment Shoulder	Critical Failure Mode	Factor of Safety	Target FOS
1	End of construction Design parameters	Outside	Non circular	1.51	1.30
		Inside	Non circular	1.34	1.30
2	End of construction Lower bound parameters	Outside	Non circular	1.27	1.10
		Inside	Non circular	1.18	1.10
3	First filling Design parameters	Outside	Non circular	1.50	1.50
4	First filling Lower bound parameters	Outside	Non circular	1.33	1.30
5	Steady seepage	Outside	Non Circular	3.02	1.50
6	Steady seepage	Inside	Non circular	3.11	1.50
7	Rapid drawdown	Inside	Non circular	1.48	1.30
8	Seismic (OBE)	Outside	Circular	2.03	1.15
		Inside	Circular	1.86	1.15

Note. The cases with the smallest margin above target are highlighted.

These basic Factors of Safety are generally along conventional /theoretical lines for the end of the design process, ie after a trial bank and dam break analysis have been completed.

Note that the rapid drawdown factor of safety of 1.48 was assessed in 2007. Since then the rapid drawdown rate has generally been increased and for SESRO is now 1m/day. The assessment should be checked against this requirement.

I am concerned that the end of construction lower bound parameter is set at a target Factor of Safety of 1.10. This leaves little for unforeseen factors such as at Carsington where, with a Factor of Safety of 1.2, shears occurred in the foundation and then progressive failure occurred. Also the index properties of the Kimmeridge and Gault Clay vary appreciably with significant number of samples below the lower bound envelop and the clay fill is expected to exhibit strain softening, peak strength reducing towards residual strength

Carsington dam failed in 1984 when close to bank completion. "*The factor of safety based on peak strengths was about 1.4.*" [7]"*The slip propagated along the embankment in both directions extending to a length of nearly 500m, with the embankment crest dropping 11m. The initial slip sheared through the core which contained shear surfaces due to rutting and along a layer of yellow clay in the foundation which contained solifluction shears. …Both materials were brittle with low residual strength…safety factor of about 1.2 and progressive failure reduced it to 1.0…failure of the original dam prior to impounding added another*

[7] DEFRA and Environment Agency Lessons from historical dam incidents 2011 page 82.

seven years to the original programme. "[8] [9]as well as appreciable costs because of the time overrun. My understanding is that the rutting was largely due to the plant used to place the fill. The trial bank will use "a typical earth moving roller."[10] Thus the SESRO main dam may well exhibit similar rutting features.

When I worked at Binnie & Partners during the era of UK-wide dam building in the 1960s/70s, following several "near misses" it became a requirement to include a further factor to take account of ignorance/uncertainty of the data on which the safety assessments had been made, the general risks of the particular dam design, such as needing to have recent experiences of the type of dam design, and of the potential impact of any dam failure. These near misses included cracking of the cores at Shek Pik dam in Hong Kong in 1963 and also at Balderhead dam in UK in 1966. I was site staff on Grafham Water dam when its design was adjusted to increase its safety accordingly.

Jacobs' draft Preliminary Design Report 2007 goes on to say:

"*the cross section cannot be finalised until after completion of a trial embankment and associated investigations described in section 5.11 below., thus at this preliminary stage the following have been defined.*

- *A "best estimate" embankment cross section based on 10 percentile foundation strengths and most probable fill strengths.*
- *A "maximum" (flattest side slopes, maximum volume) embankment cross section based on 10 percentile foundation strengths combined with increased factors of safety.*
- *A "minimum" (steepest side slopes, minimum volume) embankment cross section based on the 30 percentile foundation strengths*"

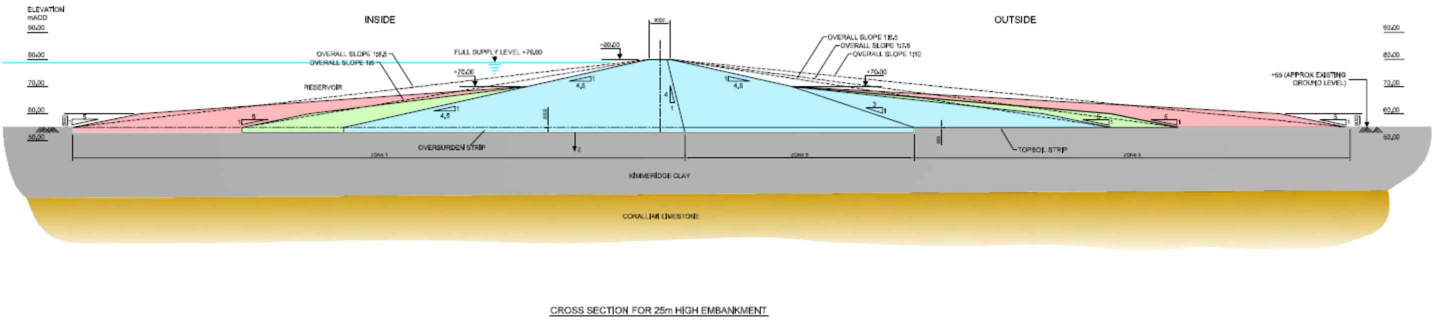
The basic design of the reservoir is based on the best estimate profile…" 3.1.1

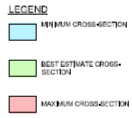
Table 3.8 of Jacobs' report shows potentially wide variations in the upstream and downstream slopes, dependent on the outcome of the trial embankment and the final stability analyses, as shown in their Table 3.8 and Figure 3.3, both copied below:

Table 3.8 Best estimate, maximum and minimum slope profiles

Slope Profile	Slope angle	
	Inner Face	Outer Face
Best Estimate	1:6	1:7.5
Maximum	1:8.5	1:10
Minimum	1:4.5	1:6.5

[8] DEFRA and Environment Agency Lessons from historical dam incidents 2011 page 81.
[9] Geotechnique vol. 43, The failure of Carsington Dam, AW Skempton and PR Vaughan 1993.
[10] Thames Water response to GARD question 8





From Figure 3.1 in Jacobs report

I note that the difference between the best estimate and maximum slope values considered by Jacobs are similar to the flattening of the slopes of Abberton reservoir, following a slip failure during construction, as I referred to earlier.

In the case of SESRO, the ignorance/uncertainty factors include general variability of soils in the very large borrow pit, the known wide variation in the soil index properties, allowance for rutting causing shear surfaces as at Carsington, soils exhibiting strain softening, lack of an appropriate size trial bank, lack of recent experience of such large embankment dams in the UK, lack of a confirmed design of the embankment and associated hydraulic structures, and lack of a dam break assessment Thus, in my opinion, the target factors of safety in Table 3.5 should be reconsidered with such criteria in mind. In my opinion the minimum factor of safety for SESRO, with the limited design development that has been done so far, should be 1.3. This would mean that the "best estimate" slopes would fail to meet the required factor of safety for the end of construction with lower bound parameters.

Resolution of these uncertainties could have an appreciable effect on the cost and economic viability of SESRO. For instance, the Wash Water Resources scheme was cancelled in the 1970s, primarily because the large outer trial bank showed that the scheme cost was uneconomic – see later comments in the section on trial embankments.

In my opinion the alternatives do not take adequate account of the embankment fill variability, the experience from the Carsington dam failure, and the lack of any trial bank.

Looking at the differences in slopes of the cross-sections, there would be large differences in embankment fill and rip-rap volumes, and hence reservoir cost, between the best estimate and maximum slope angles. As Jacobs' report says on page 3-14:

"Adoption of the maximum slopes would have the following implications:

- *An approximate 30% increase of/Zone 1 and 2 material*
- *The inner face would encroach into the reservoir area resulting in a loss in storage equal to the increase in the volume of inner face fill. Additional fill would be required from the borrow pit excavation and additional slope protection material would be required. The edge of the borrow pit would be moved to maintain the 100m buffer*

- *The outer face would need to be flattened in areas where the current profile is too steep. It is likely that this could be achieved by re-re-profiling the existing volume of landscape fill such that the extra fill would be taken from the existing areas where the slopes are significantly flatter than needed."*

In turn the change to the maximum (flatter) slopes could also lead to an increase in riprap and drainage material. It would be reasonable to assume a similar percentage increase, ie about 30%. That would result in about a 30% increase in embankment cost.

The choice of embankment slopes will not be resolved until after the construction of the full trial embankment, so, until that time, there will be substantial uncertainty in the reservoir costs.

I also note that the 150 Mm^3 reservoir appears to use all the available land in the project area. If the embankment slopes have to be flattened, it seems probable that the full size reservoir volume could not be fitted into the currently available area without the top water level having to be raised further. If so this would also potentially increase the impact of any dam failure.

3. SESRO RAPID Gate 2 submission Concept Design Report

I have examined the RAPID Gate 2 submission Document A-1 Concept Design Report[11].

At the time of the Gate 2 submission, the preferred version of SESRO had dropped to the 100 Mm^3 size, but the sizes up to 150Mm^3 are discussed. "*The Draft WRSE regional resilience plan and the draft WRMP24 for both Affinity Water and Thames Water include the 100Mm3 SESRO scheme within the preferred plan,* " (Section 1.9).

From this report:

"*The reservoir embankment would be constructed primarily using Kimmeridge and Gault clay*" (Section 2.14).

"*The dam would also include sand and gravel filter and drainage zones which are typical for embankment dams and help manage seepage through the embankment*." (Section 2.15).

"*The embankment includes internal drainage layers formed with sand and gravel*." (Section 2.25).

"*Works have been identified to meet modern requirements to drawdown reservoirs by 1m/day in an emergency*." (Section 2.77).

However, in the Jacobs 2007 report, there were three different slopes. In the Concept Design Report I cannot find textual consideration of the fill slopes to be adopted, or on what basis, or of their factors of safety, all of which were considered extensively in the Jacobs 2007 report, as I refer to earlier. It seems that, in the 16 years since the Jacobs draft preliminary report in 2007, there has been minimal additional geotechnical investigation or significant

[11] https://RAPID_Gate_2_SESRO_A-1_-_SESRO_Concept_Design_Report.pdf

development of the scheme's design. I find this surprising, bearing in mind that Abingdon reservoir has been central to Thames Water's WRMPs in 2009, 2014, 2019 and 2023. There has been a missed opportunity to firm up the reservoir's design, and hence its cost estimate to allow a proper comparison with the costs of competing schemes.

As there does not appear to be further geotechnical investigation since the 2007 Jacobs report, there has been no trial bank, the soils have wide variation in soil index properties, a significant number below the minimum line, the soils are expected to suffer strain softening, and the best estimate fails to meet an appropriate minimum Factor of Safety, it is my opinion that, at this stage of the project, it would be sensible to assume the dam slopes would be the "maximum" slope. That would increase the volume and hence cost of the embankment by about 30%.

4. GARD questions to Thames Water (22nd September 2023) and Response

GARD submitted a list of questions on SESRO Reservoir Design and Issues[12]. The email contained 8 requests – listed in Appendix 4. Thames Water replied by email on 19th October 2023[13]. The email gave links to the Jacobs 2007 report and supplied links to the factual geology survey reports from 1992 and 2005. It also provided a copy of a presentation to Oxfordshire County Council on the Reservoir (15th September 2023) and a document (pdf) of answers to the questions from GARD. This document is attached as Appendix 5.

The answers to GARD's Q1 and Q2 confirm that no geological and geotechnical reports/samples, field or laboratory tests have been carried out since 2005-6.

The answer to Q3, requesting latest details of the embankment cross-section, was: *"At the time of writing (Q4 2023) the design of the dam has not been developed further than the 2007 Preliminary Design Report"*

This may not be strictly true. The Jacobs 2007 report rejected internal drainage as too expensive and I cannot find mention of slope protection both of which are included in the Gate 2 report, see above. However, it is certainly true that there has been no significant further development of the design of the foundation and embankment slopes that will be the primary drivers of the reservoir's costs and future safety.

The very limited design development since 2007 seems surprising, bearing in mind the extensive geotechnical data that became available from the site investigations in 2005 and 2006. My scope of work for this assignment (as Appendix 2) did not include a review of these data, but a brief inspection shows that:

a) The written report shown in Appendix 1E, is the site investigation contractor's brief factual report on the investigations, dated June 2006, and contains no interpretation of the data collected.

[12] Email: Derek Stork (GARD) to Tony Owen (Thames Water): "Information Requests - SESRO Trial Embankment and related issues of Reservoir Safety" – 22nd September 2023.
[13] Email: Mark Matthews (Thames Water) to Derek Stork (GARD) – 19th October 2023

b) There is a large amount of geotechnical data available – for example, Appendix 1F contains 4,658 pages of borehole logs and laboratory test results

Noting that Jacobs 2007 report is the most recent available engineering design report and is only a draft of a "Preliminary Design Report", it seems to me that this is not a sufficiently sound basis for Thames Water's proposal to proceed with the construction of the reservoir, as proposed in their WRMP 24.

5. Trial embankments

5.1 Preliminary clay compaction trial embankment

Thames Water had applied to the Vale of the White Horse District Council for a small initial trial embankment with reference P23/V1948/LDP – STV. I understand that the planning application was withdrawn on 20th October 2023, on a planning technicality, but has since been re-submitted as P23/V2559/FUL, describing the purposes of the trial as below:

"2. The Purposes of the Clay Compaction Trial

2.1. The overarching purpose of the Clay Compaction Trial is to obtain high quality samples of the as-compacted clay material, to help inform the safe design of the embankments for the wider SESRO project.

2.2. Results from the Clay Compaction Trial will help to provide information on the following:

- *The suction and effective stresses of the compacted clay.*
- *The stress-strain of the compacted clay.*
- *The stiffness and strength of compacted fill.*
- *The variation of stiffness and strength with compaction effort.*

2.3. Furthermore, the Clay Compaction Trial will also provide greater confidence in the wider dam design for SESRO, helping to reduce the risks needing to adapt the design at a later stage. As such, the Clay Compaction Trial will help to provide other information which will be useful in planning of the SESRO construction, including:

- *The extent of water management required and the condition (traffic-ability) of the top of the bedrock clay during exposure in a summer earthworks season.*

- *The structure of the bedrock clay (exposed over a working face during pit excavation, which would be large relative to GI trial pits).*

- *The variability of the clay properties, the extent of selection material required, and the proportion of material deemed unsuitable for structural fill.*

- *The ease/difficulty of excavation, transport and compaction of the bedrock clay excavated from all elevations within the full depth of the proposed SESRO pit.*

- *The relationships between layer thickness, compaction effort and obtained density, for clay obtained from various depths within the pit when compacted in an embankment.*

- *The bulk permeability of the as-compacted clay from various depths within the pit.*

2.4. It is however important to note that this Clay Compaction Trial is a standalone temporary planning application, solely for the preliminary trial works. This application does not seek permission for any of the wider SESRO works.

2.5. At the end of the 12 month temporary planning permission, the site would be reinstated to its existing baseline state (i.e. an agricultural field), to the existing tenant farmer. Agricultural production would then resume on the site."

In my opinion, some of the objectives written above would probably not be achieved by trial banks at this scale because:

- The one year programme will need to include site mobilisation and set up. (It is not clear if site demobilisation is also to be within the year that has been applied for.) The borrow pit will then need to be opened and the trial base prepared. How long will that leave for the pore pressure dissipation within the remainder of the 12 month period?
- The plan area of the three proposed trial banks is only up to 49 m long by 23 m wide, ref MM Plan J696-DN-A01A-ZZZZ-DR-GE-100001in Geo-engineering report, but these are the sizes at the base. Assuming 1 in 3 side slopes and a trial bank 3m high then each trial bank would effectively be about 30m long by 10m wide. This is too small to accommodate properly the pattern of movements of large earth placing and compaction plant that would be used to build the full-scale embankment.
- The 3m height of the placed fill will be far less than the up to 25m height in the actual embankment, so is unlikely to replicate pore pressure build up in the clay sufficiently accurately.
- A key factor in the stability of the 25m embankment will be the rate of pore water pressure dissipation. Will a 3m bank be high enough to measure pore pressure build up and dissipation sufficiently accurately? In these clay soils pore water pressure dissipation is likely to be slow and the measurement time of a few months may well not demonstrate dissipation rate sufficiently accurately. For comparison, for the critical end of construction stability, the critical pore water will have dissipated over several years.
- This trial is unlikely to provide sufficiently reliable results to justify assumed embankment slopes and fill volumes, and hence costs, submitted for the DCO.
- The extent of the borrow pit, 30m by 5m at its base, will be too small to assess the potential variability of the fill across the whole borrow pit site of about 500 hectares.

In my opinion, this trial, although no doubt providing some useful information, would not replace the need for the large-scale trial embankment proposed by Jacobs as an essential precursor to the final design. It would not provide sufficient information to reliably determine the embankment slopes and cost of the reservoir for the purpose of justifying the reservoir in the WRMP or for approval of the DCO.

The Thames Water response to GARD, received by me on 30th October 2023, states "*The clay compaction trial proposed next year will be focussed on testing the clay….and are different from, and in addition to, a full-scale Trial Embankment which is proposed to be carried out after the DCO submission."*

5.2 Main trial embankment

Jacobs' 2007 preliminary design report says *"The cross section cannot be finalised until after completion of a trial embankment…"* 3.1.1 page 3/1. This would imply that the costs of the dam could also not be assessed sufficiently accurately until after the full scale trial. As there are alternative water resources schemes to SESRO, the trial should be done before the confirmation of the financial investment and before any DCO submission. Jacobs go on to say in paragraph 5.11.3:

"*The objectives of the trial embankment are as follows:*

- *Determine geotechnical properties for clay fill compacted under field conditions*
- *Monitor pore pressure and deformation in the embankment and foundation during construction*
- *Confirm soil parameters being used in the finite element model through back analysis of embankment behaviour*
- *Confirm the suitability of compaction plant."*

Jacobs also specify the trial embankment height:" *It is considered the trial should be built to at least 20m*." 5.11.4

I agree that, to monitor pore pressure and dissipation sufficiently accurately and confirm compaction plant suitability, a trial bank of appreciable height, at least 20m, would be appropriate.

Section 5.11 of Jacobs' report, pages 5.10 to 5.12, goes into detail of their trial embankment proposals. However, their report does not mention the duration of testing of the trial embankment to determine the rate of pore pressure dissipation and the acceptable moisture content of the clay fill.

In my opinion, embankment failure during construction due to excess pore pressure is a major risk, as happened at Carsington and Abberton dams, especially as there could be pressure for the contractor to keep placing such large volumes of fill in the wet conditions which are likely to prevail at times during the lengthy embankment construction period.

The maximum allowable moisture content of the clay fill will be a key outcome from the trial embankment, which will affect the construction time needed for fill placement and hence costs. To be sufficiently accurate, the rate of pore pressure dissipation will need to be measured over a protracted period. I note that the initial trial embankment proposal referred to 3 years of testing, reduced to 12 months in the revised application. 3 years could also be a suitable duration of testing for the main trial embankment. In which case, if the main trial embankment is delayed until after the DCO, there would be an equivalent delay in the construction programme.

Now that the design volume to be stored has increased back to 150Mm^3, the design and cost verification would be even more dependent on the large scale trial bank results, which, as I understand it, are not planned to be completed until after the Development Consent Order has been issued. This in turn will not happen until after Thames Water's WRMP24 has been approved, including, it is supposed, a decision on whether SESRO has been selected in preference to other options (eg the Severn to Thames transfer) as the next major new source in the South East.

Pore-water dissipation/drainage

The Gate 2 submission A-1 proposes the use of drainage layers to increase pore water dissipation. This was used at several mid/late 20th century dams. "*Installing drainage in the foundation, though an option, would be extremely expensive.*" 3.1.2 Jacobs 2007 report Reference 2. Such a technical change has now been made but I cannot find the "*extremely expensive*" cost in the cost schedule.

The volume of drainage material required is quoted as 315,000m^3 and it is proposed to come from dredging in the Bristol Channel.

Since this drainage material is effectively a natural material that would need to be dredged, its grading could well vary significantly from one place to another, so its grading limits should be established and checked against the filter rules. Costs in screening the "as dug" drainage material to meet the required filter grading could be significant.

There would also be the costs of transporting that volume of drainage material to site, presumably by rail, storing it, and then, as the size of the drainage layer would be relatively small, placing it carefully on site.

Recent legislation has established a requirement for Marine Net Gain for such an excavation site in the sub-tidal Bristol Channel. This could result in environmental mitigation/compensation costs, which could be significant for this large amount of drainage material.

I cannot find a cost provided for this work. In my view it would be important to establish the material sources and full costs, including dredging, screening and mitigation/compensation measures, before seeking approval through the DCO process.

Risk from delayed construction of main trial bank

In my opinion, until the uncertainty over the final embankment slopes has been resolved, ie until after construction and testing of the full trial embankment, selection of the appropriate slopes and costing of all the elements, it would be unsafe for the Government to make a decision on whether SESRO should be the next major source for the South East.

An example of the danger of delaying the trial bank is provided by the major reservoir in the Wash proposed in the 1970s as part of the Water Resources Board's National Strategy for England and Wales. Because the land was exceptionally fertile, an offshore reservoir was proposed in the Wash. An initial trial embankment was constructed and found to be sufficiently favourable that a full size circular trial bank was authorised and built.

> "*1972 a feasibility study commissioned by the Government to build a barrage across half of the Wash to capture the freshwater from the four main rivers…was undertaken. This*

led to the circular trial bank being built. The purpose of which was to act as reservoirs but the report concluded it would be too costly" [14]

This brings out the importance, before scheme authorisation, of constructing trial banks of sufficient size so that technical aspects and full capital costs can be established prior to evaluation of the alternatives and decision to implement.

Thus, I now believe that the large scale trial should be completed before the scheme design and costs are submitted for DCO and regulatory approval.

DCO proceedings

The design of the dam, ie the dam slopes, is said to be dependent on the trial bank.:

"*When examining an application for development consent, the Examining Authority must examine the environmental information. This means the Environmental Statement, any further information and any other information and representations made by other parties about the environmental effects of the development. They must reach a reasoned conclusion on the significant effects of the proposed development."*[15]

The Examining Authority may also have other evidence before it on such effects and potential interactions." For example, evidence from a community group.

"*Where some details are still to be finalised, the Environmental Statement should, to the best of the applicant's knowledge, assess the likely worst case environmental, social and economic effects of the proposed development to ensure that the impacts of the project, as it may be constructed, have been properly assessed."*[16]

In my opinion the Environmental Statement has to include such information as the slope of the dam, the trials which were carried out to decide on the slopes, the rip rap of which it is covered, the capital cost to assess the economic effect, the safety of the embankment as shown by the trial bank tests, the dam breach assessment to assess the social effect, and the impact on the environment.

Thus these tests/trials would seem need to be completed before the DCO hearings.

6. Leakage

All reservoirs leak and there is potential for significant leakage from the 10 km long embankment up to 25 m high. "*Leakage can also lead to an increase in porewater pressure in the downstream fill and foundations which could cause instability*"[17] I cannot find mention of leakage quantities in the SESRO concept design report – a significant omission in my opinion.

[14] Historic land and seascapes, Reclamations and Realignments The Wash Strategy Group. Undated but c2003
[15] Defra National Policy Statement for Water Resources Infrastructure April 2023 3.2.5
[16] Defra National Policy Statement for Water Resources Infrastructure April 2023 3.2.9
[17] An engineering guide to the safety of embankment dams in the United Kingdom 1999 page 14

Jacobs' Preliminary Design Report, section 5.2.3, considers leakage through the embankment foundations:

"Lateral seepage beneath the embankment would be governed by the Lower Greensand outcrop as this is several orders of magnitude more permeable than the clay strata. The adopted permeability of the Lower Greensand is $5x10^{-5}$m/s. The seepage path beneath the embankment is of the order of 100m with a mean head difference of about 20m. The average thickness of the Lower Greensand is about 3m. Considering seepage through a 100m width of Lower Greensand on either side of the reservoir, the predicted seepage is 0.6 1/s which is trivial. Notwithstanding this assessment, provision would be made to blanket the exposed face of the Lower Greensand with clay in case there happen to be bands of higher permeability material, and also to minimise the possibility of piping failure.

Sealing the Lower Greensand in this manner would also be beneficial in preventing the development of uplift pressures in the Lower Greensand where it is present beneath the outer toe of the embankment along the south side of the reservoir."

It is not clear to me why the foundation leakage has been calculated only for 100 m widths on either side of the reservoir, rather than a greater extent of the Lower Greensand outcrop. Neither the Jacobs report nor the SESRO concept design report appears to have considered leakage through the 10 km of embankment, which might be significant in water resources terms as well as a potential piping threat. In my opinion, leakage should be properly addressed, including costs of control measures and water resources impacts, prior to any decision on the reservoir.

7. Freeboard and wave protection

The reservoir is to be filled by pumping. Thus excess pumping above reservoir top water level should be low.

The reservoir level would be augmented by storm rainfall, but this should be accommodated by the available freeboard and there would no doubt be measures in place to avoid over-pumping and to release excess storm water.

Even without inflow from an impounded catchment, overtopping by waves can be a threat to dam safety. For example, at Blithfield in 1962, the Defra report on lessons from dam incidents says "*with the reservoir full, a severe six-hour storm caused the dam to be overtopped by waves and spray which led to saturation of the downstream slope that triggered a slip.*[18]"

A similar situation of overtopping and erosion of the downstream shoulder occurred at Maich reservoir in 2008[19].

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[18] Defra and EA Lessons from historical dam incidents 2011, page 96
https://assets.publishing.service.gov.uk/media/603369e7e90e07660cc43890/_Lessons_from_Historical_Dam_Incidents_Technical_Report.pdf
[19] Ibid, page 41

Consolidation of the underlying clay will occur including during the dissipation of pore water pressure in the clay fill and the foundations. *"Where an embankment dam is built on a deep deposit of clay, consolidation of the foundation soil may continue over a long period. "* [20] In the clay fills to be used at SESRO this is likely to take a number of years. *"These movements might be significant in reducing freeboard."* [21] That would mean the dam crest would have to be built somewhat higher than the nominal level to allow for long term consolidation of the clay fill and foundation. This would result in some increase in costs.

It is normal practice to protect the upstream face of any clay embankment from wave erosion either by concrete slabs or rip rap.

I cannot find any consideration of any upstream slope protection in the Jacobs documents, but section 2.15 of the Gate 2 report says "*The inner face of the embankment would be protected with riprap*".

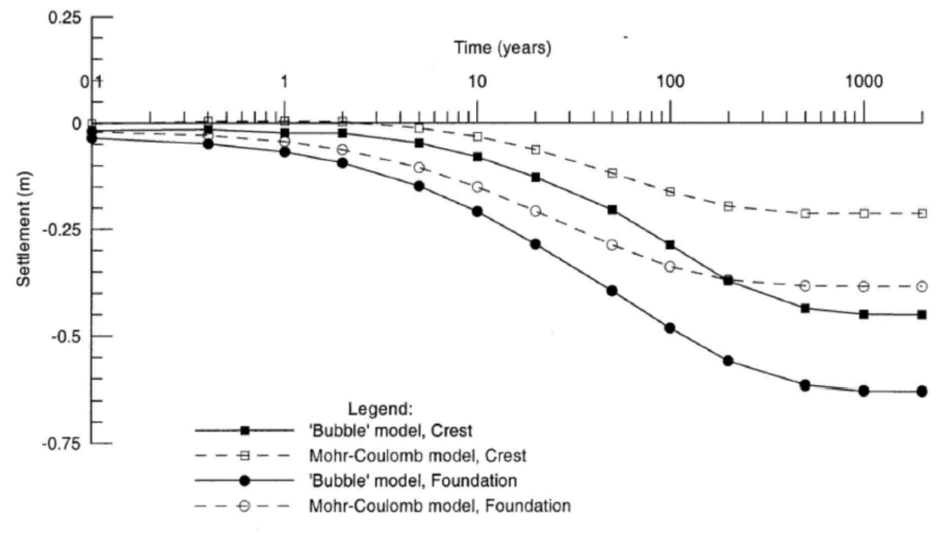
"*The riprap would consist of large, angular blocks of natural rock, which would interlock and dissipate the wave energy. The riprap would be laid on a sand filter layer and a gravel bedding layer to prevent washout of the embankment clay from between the riprap stones*." (section 2.22 of the Gate 2 report). This is a standard process but thicknesses are shown.

Embankment crest settlement

High clay embankments will settle/consolidate over the life of the dam. With the clay found at this site this consolidation/settlement may take some time. The long term amount would need to be predicted and the dam crest built to incorporate this amount.

The Jacobs 2007 Draft Preliminary Design Report figure 5.1 see below shows a plot of the foundation and crest settlement over a period of 1,000 years. The text says "*After 100 years the predicted crest settlement of the 25m high embankment is approximately 0.25m.*"[22]

Figure 5.1 Embankment settlement



[20] An engineering guide to the safety of embankment dams in the United Kingdom 1999DETR BRE page 14
[21] An engineering guide to the safety of embankment dams in the United Kingdom 1999 DETR BRE page14
[22] Preliminary Design Report 2007section 5.3 Figure 5.1.

However there are two curves, the crest and the foundations, with legends very similar. The 0.25m after 100 years is for the smaller settlement graph. However, this would imply that the crest would not settle as much as the foundation. This is not credible. Thus the graphs are transposed. Thus the crest would settle about 0.5m after 100 years. "*Given the uncertainties in the prediction of settlement, it is considered pragmatic to adopt the 100 year settlement allowance*." That would be 0.5m. Therefore, the Jacobs recommendation of 0.25m settlement allowance does appear to be inadequate and provision of the proper settlement allowance would increase the capital cost of the SESRO somewhat.

The nominal life of the embankment works of the reservoir is said to be 250 years[23], and the length of the crest is about 10km. The reservoir would operate for much of its life in a full or near full condition, thus access to further fill from within the reservoir basin would be very difficult. Thus, it might be prudent to adopt the 250 year life, in which case the settlement allowance would be about 0.7m where the embankment would be 25m high.

Assessment of Freeboard needed

"Figure 2.1 of the Conceptual Design Report shows reservoir cross sections and indicates that the crest of the reservoir will have the following characteristics:

Crest 8m wide with cycle/footpath, low wave wall available for seating. Crest level 1m higher than maximum water level."[24]

The fetch over the reservoir could be up to about 4.3 km, which is considerable.

ICE Floods and reservoir safety guide, FRS, edition 4[25] shows the 50 year wind speed for the site as about 20m/sec, and a critical wind direction about 240°. With embankment all around the 150Mm^3 reservoir, this would be an exposed site. For reservoirs with minimal natural inflow, as for SESRO, the ICE Floods and Reservoir Safety Guide, edition 4 page 11, recommends using the 200-year wind speed. Factor 1.06.

"*Where water resources infrastructure includes safety critical elements, the applicant should apply high emissions scenarios at different probability levels so as to include high impact, low likelihood scenarios to those elements critical to the safe operation of the infrastructure*."[26] Thus, for a long life feature such as a reservoir, the 200 year wind speed would need reconsidering.

"Both also project a tendency towards more wet and unsettled conditions over the UK in winter under a high emissions scenario, which would imply a trend towards stormier conditions on average."[27] Thus it would be necessary to project the 200 year return period wind over the assumed life of the reservoir, possibly 250 years.

The open water factor would be 1.23 and the duration factor 1.05. Based on the current climate, this would result in a wind speed of 29.6 m/s. This would result in a significant wave

[23] SESRO A-2 cot report Table 7-1
[24] GARD response to Thames Water Consultation on WRMP 2024 21.3.2023 page 96.
[25] https://www.icevirtuallibrary.com/doi/book/10.1680/frs.60067
[26] Defra National Policy statement for Water Resources Infrastructure April 2023 page 35
[27] Met Office UKCP18 Factsheet: storms 2023 page 6.

height of about 1m, Figure 5.3 of the Guide. The SESRO Concept Design report Figure 2. shows that the proposed reservoir freeboard is 1 m (one metre). According to the Guide, Edition 3 page 24, this would result in about 14% of the waves exceeding the crest of the dam. However Edition 4 Table 5.5 shows that "*rock armour in one layer on a low permeable base has a run-up factor of 0.6"*. Edition 3 approaches this somewhat differently but, with the inner slope proposed at 1:6, reaches a similar conclusion. Thus, with the riprap, the wave run up would be appreciably reduced.

The crest would have a "*low wave wall available for seating*". Such wave walls are normally on the upstream side of the dam crest and are generally about 1.5m high. In this case it would be *"low*" and *"suitable for seating"* so is likely to be about 0.7m high.

The combination of the riprap, the wave wall and the long term settlement allowance, would need to be sufficient to limit wave overtopping to an acceptable amount.

Quantity and cost of wave protection

The riprap protection against waves eroding the embankment will add appreciably to the capital cost of the reservoir. The quantity of riprap quoted in the Gate 2 concept design report is 545,000m^3 for the 150 Mm^3 reservoir[28]. Thames Water's reservoir feasibility report for WRMP19 quoted 1,558,000 m^3 of riprap for the 150 Mm^3 reservoir[29] – a vastly larger amount.

The large difference in riprap quantity between the WRMP19 feasibility report and the Gate 2 report – 1,558,000 m^3 and 545,000 m^3 – has not been explained in the Gate 2 Concept Design Report. The Gate 2 report contains no properly drawn embankment cross-section to show depths of riprap protection or bedding layers.

A rough calculation of the area of riprap required is 10,000 m embankment length x 120m slope length (average 20m high at 1:6 slope) = 1,200,000 m^2 of rip rap. On that basis, the depth of riprap from 545,000m^3 would be only about 450 mm.

The USBR has analysed a large number of earth fill dams and the protection that has been satisfactory. For dams with a fetch of 2.5 miles (4.0 kms), and a slope of 3:1, then a nominal thickness of "*30 inches"* (760mm) was found to be satisfactory. [30]Whilst parts of the reservoir have a somewhat shorter fetch and the bank slope is likely to be somewhat flatter, this would indicate that a nominal thickness of about 450mm would probably be too thin.

"The individual pieces must be of sufficient weight to resist displacement by wave action"[31] The table also shows that 40% to 50% of the rip rap should have a weight of 1250 lbs (about half a ton) on a 3:1 slope. Since the SESRO embankment slope would probably be somewhat flatter it would be expected that the rip rap could somewhat lighter. However this illustrates the problem of finding an appropriate source.

As the reservoir embankment would be expected to have a very long life, 250 years, the rip rap protection rock would need to be particularly durable. *"Laboratory tests should be*

[28] SESRO concept design report, paragraph 2.23
[29] Thames Water WRMP19 Reservoir Feasibility Report, Mott MacDonald, July 2017, PDF page 249
[30] USBR Design of small dams 1987 Table 20 page 263
[31] USBR Design of small dams 1972 page 277.

undertaken to determine the resistance to weathering and abrasion"[32] There are limited places in southern UK where such durable rock in these quantities would be available. That for Empingham Reservoir (Rutland Water) came from Derbyshire, an appreciable distance. For some current coastal protection schemes the sources of riprap/rock armour have been Scotland or Norway.

The SESRO delivery mechanism is quoted as being by rail, which is sensible and less polluting. However it would mean the quarry would also need to be connected to the rail network.

With such a large volume required, it might be that the riprap quarry would need planning permission to be expanded. There would anyway need to be a Biodiversity Net Gain for the impact on the quarry.

I have been unable to find in the Concept Design Report detail of the dimensions of the chimney drain or other drainage layers.

Thames Water has stated that "*The source of sand to be used to be used in the filter drainage zones has not yet been confirmed, but various potential sources have been identified (such as the Bristol Channel). Investigations into the suitability of aggregates from various sources will be carried out and the sources confirmed post DCO consent*"[33]

Sand and aggregates are expensive, especially when transported a considerable distance. In view of the large quantities required in 10 km of embankment, there are likely to be appreciable environmental impacts on their source.

Ought the requirement for Biodiversity Net Gain or Marine Net Gain at all the sources of material imported into the scheme to be taken into account? I understand that this should be a requirement at a DCO inquiry.

In my view, the potential sources, their effect on costs and environmental impact of the riprap and drainage materials and the provision of Biodiversity Net Gain at the sources could have a significant effect on the overall costs and should be established prior to selection of the next major source for the South East and seeking scheme DCO approval.

8. Dam break analysis

The threat of dam failure is emphasised in Defra's report on lessons from historic dam incidents:

"*With most structural failures damage is limited to an area in the immediate vicinity of the structure, but the breaching of a dam and the consequent uncontrolled release of the impounded reservoir water can cause destruction over a large area downstream of the dam. The structural stability and security of such dams, therefore, is of major importance for*

[32] USBR Design of small dams 1972 page 277
[33] Response to each question raised by GARD on 22nd September 2023.item 6, as in Appendix 5

public safety, particularly in Great Britain where many reservoirs are located in river valleys upstream of densely populated and industrial areas,"[34]

Breach could occur from embankment slips, internal erosion, erosion of the dam crest due to overtopping flow, an event such as an airliner crashing on the reservoir wall or terrorist activity.

Historic incidents

The Dale Dyke dam failed on first filling in 1864 resulting in 244 deaths.

About 10,000 people were drowned in Dera in Libya as a result of a recent 2023 dam break due to flood overtopping and cascade failure.

On June 18th 1972, a Trident aircraft that was mal-operated on take-off from Heathrow, stalled and crashed near Staines killing all those on board. For the last several seconds of its flight it had been over the reservoir embankment of the King George VI. BEA Captain Eric Pritchard reported that "*No2 engine had dug a considerable crater.*"[35]Had the uncontrollable aircraft landed only a short distance away on the reservoir embankment it is possible the King George V1 embankment might have been breached leading to a significant release of water onto the surrounding built up area of Staines and potentially significant loss of life.

In 22nd December 1999 a 747 cargo plane taking off from Stanstead crashed into the Beggars Hall Lake creating a crater on the embankment[36]. I have visited this site. The Defra report says "*Although unlikely, such an event could produce catastrophic consequences on a larger dam.*"[37]

I have obtained a plan from BAA of recent aircraft movements from Heathrow. 9% of all flight take offs followed 27L/R CPT flying WNW above Reading. Thus it would appear that SESRO is in an area of significant aircraft movement albeit by then the aircraft would normally be at significant height. In addition there would be aircraft movements from Brize Norton airfield, about 20km to the north-west.

In 2019, following a large flood damaging the auxiliary spillway of Toddbrook dam, the 1,500 population of Whaley Bridge in Derbyshire had to be evacuated for 6 days. I was one of the panel of about 6 dam engineers called in to advise the Authorities on when it would be safe enough for the residents to return.

Those events have heightened community appreciation of the risks that failure of a large dam can result in.

Terrorism

On one of the high dams I had designed I was asked by the relevant authority to write on non-photocopy-able paper how the dam could be attacked, (easily) and how best to defend it.

The Jacobs 2007 Stage 2 Preferred Scheme Design Options Report considers the recreational options. These vary from just fishing to space for 500 boats to be parked on site[38]. The

[34] Defra and EA, Evidence Report-Lessons from historical dam incidents, 2011 page 8
[35] Wikipedia interrogated 26th December 2023
[36] Lessons from historical dam incidents, Defra and EA, 2011 page 5 and page 147
[37] Defra EA Lessons from historical dam incidents page 147
[38] Stage 2 Preferred Scheme and Design Options Report Jacobs 2007 page 100.

Concept Design Report talks of *"a sailing club including internal/external boat storage, a clubhouse and access to the reservoir for controlled water-based activity…An extensive network of walking, cycling, and riding routes around the site"*[39]

Whilst these would be positive features for the community, they would mean greater risk of opportunity and vulnerability to terrorist attack on the crest with a bomb, having the potential to cause a breach, which could lead to successive crest erosion and serious flooding downstream. For instance, with the height of dam now envisaged, it is likely that vehicles would need to be used to launch and recover the larger boats on the reservoir. That in turn would mean greater risk that a significantly sized bomb could be launched within a boat and thus might be able to be moved to the dam crest elsewhere on the reservoir and exploded to cause a breach in the embankment.

GARD took specialist advice in its Response to Thames Water's Consultation on Draft Water Resources Management Plan 2024 21st March 2023 and concluded on page 95 "*a medium sized VBIED could easily cause a breach at the dam crest with subsequent rapid erosion of a section of the downstream earth fill and total embankment breach, with resulting loss of life and publicity…."*

Thus a terrorist attack, leading to dam breach and a flood wave downstream, would need to be considered as a dam break risk.

Long life of dams

A dam can be a very long-life structure. Many UK dams are already over 100 years old. The Carew Mill dam in Pembrokeshire dates from about 1600 and was still in use when I last visited it.[40]

Most of the fill for the SESRO dam would come from within the reservoir basin itself and, once operational, that would be largely underwater, and thus no longer accessible. Thus it would be appropriate to consider the life of the dam to be 250 years and to design accordingly.

Dams and reservoirs can be attractive features, thus they may attract development around them such as new housing for the community to enjoy the view across the water and the provision of support services to the activities on and around the reservoir.

All structures can deteriorate with age and have to deal with changing operating conditions as well as a changing climate such as probable increased wind and rain conditions at some time in the future.

"If asked to cite failures of British dams, most engineers in the reservoir industry would be able to quote Dale Dyke, Bilberry and Dolgarrog, together with recent serious incidents such as Ulley, but many would struggle to name more of the several hundred incidents that have occurred. The lack of knowledge of dams can give rise to misplaced optimism with respect to the long-term deterioration of dams."[41]

[39] SESRO Concept Design Report section 2.103
[40] Wikipedia Carew Tidal Mill interogated5.1.2024
[41] DEFRA EA Evidence Report-Lessons from historical dam incidents 2011 page 8

Thus the design of a dam must also include consideration of how its operation, surroundings, and climate conditions might change over its long life. I have seen no evidence of how this has been considered in the development of the SESRO scheme.

Dam break assessment

The DEFRA and Environment Agency joint guidance report FD2658[42] defines high risk dams as those with >5m elevation or 10,000m^3volume. Clearly SESRO meets both of these criteria.

The FD2658 Guide sets out certain processes for dams including dam break analysis. This would identify which properties would be affected should the dam fail.

Community impact

The Defra/Environment Agency Guide on reservoirs states[43] "*Site reservoirs in rural areas away from houses*" SESRO is in a largely rural area but the communities of Steventon, West Hanney and the South parts of Abingdon are nearby and potentially vulnerable It continues "*Site reservoirs in locations where a breach flood would not impact critical infrastructure such as busy roads or railways.*" To the east is the busy A34 which connects the South Coast ports to the Midlands industrial area. To the south is the main railway line between London, Bristol and South Wales.

In the case of SESRO the crest length is about 10 km with varying heights of embankment and varying numbers of people at risk of a breach. Thus, certain lengths of the dam wall will provide more risk than others. The ICE Floods and Reservoir Safety 3rd Edition, page 7 states:

"It is considered that public opinion will not accept conscious design for a specific threat to a community, even though it tolerates to an extent both random and accidental loss of life. Consequently, no dam above a village or town should be designed knowingly with a finite chance of a disastrous breach."

A community in this context is considered to be more than 10 persons, (ref [42] table 2.2). There are several communities that could be affected around the perimeter of the reservoir. Thus, in those sections of the long embankment, it may be appropriate for the Construction Engineer to provide an increased factor of safety in certain critical locations.

The people living in the communities that could be affected by a dam breach may well wish to know the risk to which they could be subjected.

Would there be likely to be a community at risk? GARD has provided me with a short report, *Appendix B DEFRA Reservoir flood assessment-Simplified Method*, which, I am told, was included in their submission to the Thames Water and RAPID Gate 2 consultations. The

[42] ttps://assets.publishing.service.gov.uk/media/603390fc8fa8f54334a5a673/small_reservoirs_simplified_risk_assessment_methodology_guidance.pdf

[43] Defra/Environment Agency Small Reservoirs Simplified Risk Assessment Methodology Guidance Report FD2658 page 6

report was done for the 100 Mm^3 reservoir, on which those consultations were based[44]. It uses parameters calculated from the formulae in [45] to identify cases of High Risk communities. The GARD report identifies these as the east part of East Hanney, the west part of Steventon, and the south part of Drayton. It calculates the arrival time of a breach flood as between 3 minutes and 8 minutes. "*These perimeter communities each consist of dozens of houses, every community having a population of the order of 60, so that the likely loss of life in a single breach (un-warned) would be about 11*"[46]

I understand that GARD has raised this issue in its representations to Thames Water, Ref 4 but without success. I have been told that "*Thames Water refuse to comment on GARD's analysis which is based on DEFRA guidance, or to undertake their own analysis*."[47]

GARD also posed several questions to Thames Water on 22nd September 2023. The answer to Q4 on Dam break analysis "*A dam break analysis will be undertaken after the design of the reservoir is finalised, as required by emergency planning regulations… . We expect to undertake the dam break post DCO consent*."

GARD has stated "*…so there is clearly no intention to share at public examination the very relevant details of flooding, property damage and potential loss of life*."[48]

This seems surprising since the Concept Design Report states "*In Gate 3 further SESRO specific consultation would be undertaken to ensure that a wide spectrum of local views are considered as the scheme develops.*"[49]

"Community and stakeholder engagement is crucial to the development of the SESRO…Much more detailed community engagement and formal consultation is required…Before applying for permission Thames Water and Affinity Water will need to demonstrate that they have presented information about the proposals to the community, gathered feedback, and considered the views of stakeholders. We will have regard to that feedback and, where possible, make changes to the design as a result."[50]

GARD represents a sector of the local community. Thus, to refuse to undertake the requested dam break studies prior to seeking permission, does appear to be contrary to what Thames Water say they need to do.

The size of the reservoir has now changed, from 100 Mm^3 to 150 Mm^3 in the revised Thames Water draft plan[51]. Considering the large volume of water stored, 150Mm^3, it would also be important to assess the potential impact of a dam break on the population living downstream,

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[44] https://www.gard-oxon.org.uk/downloads/21.3.23%20GARD%20response%20to%20TW%20WRMP%2021.3.23.pdf, Appendix B.
[45] HR Wallingford Small reservoirs simplified risk assessment methodology Guidance Report (2014)
[46] https://www.gardoxon.org.uk/downloads/21.3.23%20GARD%20response%20to%20TW%20WRMP%2021.3.23.pdf, page 144
[47] Derek Stork, Chair of GARD, private communication.
[48] Derek Stork, Chair of GARD, private communication.
[49] SESRO report 2.4
[50] SESRO Cost Report A-2 unnumbered page
[51] https://thames-wrmp.co.uk/sesro/

both now and in the future. The increase in storage, 50%, and the higher top water level, I understand 2m, is an appreciable change and would increase the potential damage from any dam failure. For instance it is now possible that the main railway line from London to Bristol and South Wales, a greater proportion of south Abingdon, and a greater proportion of the population living by the River Thames, would be affected.

The risk of embankment failure and the potential resulting loss of life and economic damage, needs to be taken into account in appraising the safety factors assumed in the embankment stability analysis which will determine the embankment slopes. Therefore, in my opinion, the dam break analysis should be undertaken before the design is submitted for regulatory and DCO approval.

Consideration would also need to consider those people and properties who could be affected by an emergency drawdown.

I also believe that those living in properties that could be affected would wish to know before a Development Consent Order application has been submitted so they could, if necessary, make representation at the hearings. In my view a dam break assessment before DCO submission should now be important to the proposers, regulators and relevant communities.

9. Emergency drawdown

As Anglian Water have stated in a paper on emergency drawdown of their reservoirs, *"The immediate and fast drawdown of a reservoir is often one of the initial steps to be taken in an emergency."* [52]As an emergency would have been declared there would be limited scope for ramping up the discharge rate, so the impact on downstream communities and river users needs to be properly considered as part of the promotion of the reservoir. I have found no evidence of this in the reports I have seen.

There have been several emergency drawdowns of reservoirs to stop the reservoir from failing and causing serious loss of life. The most recent was the evacuation in 2019 of Whaley Bridge in case the adjacent Toddbrook dam failed. I myself have had to order two such events, albeit on small reservoirs.

Following Government guidance[53] it is now general practice to provide drawdown facilities such that a category A reservoir, as would be SESRO, the water level could be reduced initially by 1m/day. The SESRO Concept Design Report recognises that "*Provision of pipework to enable an emergency drawdown at an initial rate of 1m/day -this is the maximum recommended installed rate within current UK guidance for reservoirs and matches that adopted at all other major Thames Water reservoirs*,"[54]so a 1m/day drawdown must be achievable and quotes a 76 m^3/sec capacity for the auxiliary outlet works.

[52] Improving Anglian Water's emergency response for reservoir safety in Dams: Engineering in a social & Environmental Context Thomas Telford 2012, page1.

[53] https://www.gov.uk/flood-and-coastal-erosion-risk-management-research-reports/guide-to-drawdown-capacity-for-reservoir-safety-and-emergency-planning, volume 1, page 38

[54] SESRO Concept Design Report 2.12 on page 2-5.

The normal "*release rate of 321 Ml/d* "[55]would be about 3.7m^3/sec, too small to be of significant help.

The Concept Design Report states that the emergency capacity would be provided through a combination of 30 m^3/s through valved outlets in the draw-off tunnel and 46 m^3/s through "*Four auxiliary drawdown siphons (metal pipes), which are buried under the surface of the reservoir embankment, that discharge via valves into a buried concrete chamber at the outer toe of the embankment.*"[56] There are few such siphons on UK dams and their design and operation is not yet well known.

As stated in the Government guide to reservoir drawdown[57], "*If a dam is in serious danger of failing, the priority must be to lower water levels in the reservoir as quickly as possible in order to prevent an uncontrolled release of water, which could cause widespread flooding and potentially loss of life.*"

Whilst it would be possible to construct a 76 m^3/s outlet facility to give the required drawdown rate of 1m per day, it would entail very substantial hydraulic engineering works and no details are provided in the Concept Design Report. The transmission of 76m^3/s through an earth embankment, including the dissipation of about 25m of energy head, is an inherently risky engineering operation, which requires careful design and, probably, physical model testing. The Gate 2 reports contain no reference to the requirements of model testing of these works.

As stated on page 7 of the Government guide[58], "*The disadvantage of siphons is that they often need to be primed using an external pump to start the water flowing….Another limitation of siphons is that there is a maximum depth to which they will work of around 5m.*" Presumably with a reservoir depth of around 25m this could mean several sequential siphons, all with the attendant seepage/piping risk of passing a large pipe through an earth embankment. However, that would mean the control of the lower siphons such that they did not activate unless required, which would be a further design complication.

The Guide also says[59] "*It would be reasonable for temporary and emergency drawdown capacity to not make up more than 50% of the total capacity deemed necessary.*" As currently proposed the siphons would be classified as emergency works and would have limited range, possibly only the top 5m of the reservoir. Thus the large and complex emergency drawdown facilities require much more design work before they can be considered adequate and reliably costed.

In addition to the difficulty of designing the safe passage of 76 m^3/s through the embankment, the impact on downstream water courses and the River Thames could be substantial. The Gate 2 submission Concept Design Report describes the discharge through 4.9 km of open channel connecting to the River Thames. The lower 4 km of this channel is expected to use the reinstated Wilts & Berks canal. However, I am told by GARD that the conveyance

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[55] SESRO Concept Design Report 2.46.
[56] Concept Design Report 2.81
[57] Defra EA Guide to drawdown capacity for reservoir safety and emergency planning SC130001 Vol 1 2017 page 15
[58] Ibid page 7
[59] Ibid page 38

capacity of the canal itself will only be about 3 m^3/s, so most of the flow will need to be contained by levees. The full 76 m^3/s will have to be passed under the A34, requiring substantial engineering works. This is briefly described in the Concept Design Report, but no engineering detail is provided, giving the impression that no proper engineering design has yet been undertaken for these extensive and costly works. I cannot find a specific item in the capital cost breakdown for these works.

The river flow gauge closest to the emergency drawdown point is Sutton Courtenay. A discharge of 76 m^3/s would be more than the flow at Sutton Courtenay on the River Thames, for about 92% of the year. Thus the emergency drawdown flow would change the river Thames at Sutton Courtenay from its normal flow to a flood flow in a short period of time. That could be a significant issue for river users such as swimmers, rowers, sailors and could be an issue with fishermen. At times of high river flow the increased discharge might well result in increased flooding.

For example at Reading, the discharge flow by itself would only be exceeded about 20% of the time, so there would still be significant change in flow. Thus the impact of the emergency discharge on the stretch between Sutton Courtenay and Reading would need studying.

Emergency plans

The need to plan for emergency drawdown releases is recognised by Government:

"The Environment Agency…working to establish flood depth and velocity information to meet the needs of the Flood Risk Regulations… Local authorities should collate and map the main flood risk management …assets."[60]

"*Off-site plans which are developed by local authorities to ensure communities are well prepared. In particular, they set out what the emergency services will do to warn and protect people and property."*

"The method of rapidly lowering the reservoir at short notice should be a key aspect of any on-site plan."[61]

At lower river flows the rapid change in flow due to emergency drawdown flow might also result in environmental impact.

Whilst it may never be necessary to implement the Emergency Drawdown, the measures that would be taken must be identified and the facilities set in place. That would include being able to inform those who would be affected should it be, and when it were, implemented.

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[60] The Governments Response to Sir Michael Pitt's Review of the summer2007 Floods Final ProgressReport27th January 2012 pages 12 and 19.

[61] Defra EA Guide to drawdown capacity for reservoir safety and emergency planning SC130001 Vol 1 2017 page 14

Operation of emergency drawdown

The first aspect to consider is what would be discharged. The flows would be substantial so boats and swimmers should be kept clear of the outlet works. Secondly what fish would be in the reservoir and could any invasive species be discharged to the River Thames?

There is then the question as to what extent occasional trials should be undertaken to test whether the system would work, for instance, at each 10 yearly statuary inspection.

The importance of this was brought home to me when, carrying out a reservoir statutory inspection under the Reservoirs Act, I asked for the emergency drawdown system to be operated. With the relevant valves fully open the flow rate was no more than a trickle, clearly inadequate even for that smallish reservoir. The reason turned out to be that the local Environment Agency had restricted such tests to limit sediment being discharged to the downstream river so, at each previous operation, bottom sediment had been drawn into the pipe but had not been flushed through, so the scour pipe had become blocked. In that case the reservoir did not get its certificate until the emergency drawdown system had been rectified.

Conclusion on the emergency drawdown proposals

As the ability to action an emergency drawdown is effectively a mandatory requirement, in my opinion, the design, probably including hydraulic modelling, communication system to those who could be affected, and environmental impacts of these emergency outlet works, needs to be properly established and costed before any decision is taken on the reservoir viability.

10. Cost implications of the reservoir design review

10.1 Cost issues arising from the review

As previously mentioned, Thames Water have stated that the design of the reservoir has not been progressed since the Jacobs Preliminary Design Report in 2007[62]:

> *"At the time of writing (Q4 2023) the design of the dam has not been developed further than the 'Preliminary Design' described in the 2007 Preliminary Design Report."*

This review has raised a lot of issues where there could be increases in estimated capital costs when the present preliminary design, based on Jacobs 2007 work, is developed into the detailed final design, ready for construction.

The cost escalation risks include:

1. **Embankment slopes**: My review has identified several issues which could lead to the need to adopt flatter slopes than those assumed in the Gate 2 Concept Design Report:
    - The trial embankment proposed by Jacobs has not yet been built and is not intended to be built until after DCO approval.

[62] Thames Water response to GARD's EIR request, 22.9.2023, page 2

- Consequently, soil strength parameters may be less favourable than those assumed in Jacobs' preliminary stability analysis.
- The end of construction factor of safety of 1.1 assumed by Jacobs is less than the 1.2 safety factor assumed in designing Carsington dam, which failed during construction.
- Even after the trial embankment has been built, some of the 6 km^2 borrow pit, of which only a small part will be sampled for the trial embankment, may have lower soil strengths, requiring design adjustment.
- The dam break analysis has not yet been done and this may show that higher factors of safety may be needed in some parts of the embankment, because of the proximity of villages and other high risk features like the main line railway and A34 trunk road.

In my opinion, it would be prudent for cost estimates at this stage to assume the "maximum slope" values shown in Jacobs' report. According to Jacobs' analysis, maximum slopes would increase the embankment fill quantities by about 30%, probably with an equivalent increase in earthworks cost.

2. **Crest settlement:** the preliminary design makes inadequate allowance for crest settlement of the 25 m high clay embankment, placed on a compressible clay foundation. This could be of the order of 0.5m so would need to be allowed for in the design and would add significantly to earthfill quantities and hence cost.
3. **Rip-rap quantities and sourcing:** The estimated volume of rip-rap has been reduced to less than half of the volume estimated in 2018, with no explanation. The implied rip-rap thickness of 450 mm looks low. There is a large volume of rip-rap required and the source of rock of suitable quality has not been identified. This carries a high risk of cost over-run.
4. **Drainage materials sourcing:** There is a large volume of drainage material required and its source has not been identified – likely to be marine dredging. This is another high risk item that has not been adequately investigated.
5. **BNG requirements for rip-rap and drainage materials:** the rip-rap quarry and the drainage material source are likely to have appreciable BNG implications which have not been investigated (and possibly not allowed for?)
6. **Design of emergency draw-down facility:** The requirement to release nearly 80 cumecs in an emergency draw-down requires heavily engineered outlet works, both in the inlet/outlet tunnel and in the separate emergency outlet siphons. Releasing this volume of water into the Thames will greatly exceed the capacity of the local natural channel as well as creating a flood risk in the River Thames. It appears that none of these works have been designed, even to a preliminary stage. It is another significant cost escalation risk.
7. **Construction delays due to wet weather and slow pore pressure dissipation:** In the absence of the trial embankment, the maximum allowable moisture content of the clay fill has not been determined. There has been no assessment of the practicality of achieving acceptable moisture contents in the range of weather conditions which

could exist over the multi-year period (expected to be four years) of fill placement. There has been no assessment of the required rates of pore pressure dissipation which could determine the speed of fill placement and the length of the construction period. This carries a high risk of cost over-run.

A further risk factor is the limited recent experience in the design and construction of large embankment dams in the UK. Although a number of relatively small dams have been built in UK in recent years, and a few dams raised, the last major embankment dam to be built was Carsington Dam and even that failed as it neared completion in 1984 due to a deep-seated slip in the upstream shoulder and it took about 7 years to rebuild. "*Unfortunately, a 30 year lull in reservoir construction has resulted in a dwindling number of engineers with the necessary skills"*[63], as written by Binnies, one of the previously foremost firms of dam designers.

Some UK dam engineers have worked on overseas dams but the soils and foundation conditions are almost invariably different to those in the UK, and anyway the dams are often of different design such as concrete dams, concrete arch dams, rockfill dams, or deck dams.

Thus it is nearly 40 years since there has been experience of the design and construction of a large new embankment dam in UK. That is nearly a full working life, so there can be few, if any, practising dam engineers with sufficient relevant UK experience. In my opinion, this is reflected in the failure to progress the embankment design significantly since Jacobs' preliminary work in 2007 and in the lack of action since then in dealing with the other cost risk factors that I have mentioned above.

Overall, the list of issues summarised above leads to a high risk of cost over-runs, which needs to be properly reflected in the allowances for costed risk and optimism bias.

10.2 Cost overruns of dams, experience from Australia

Large embankment dams are notorious for cost over-runs. Although there has been no recent experience of building large dams in UK, there has been a lot of overseas experience of cost over-runs typified in a 2019 Australian paper[64] which summarises its findings as:

> *"The results of this study of Australian dams are in keeping with international studies that have found the estimated costs of large infrastructure projects are systematically biased downwards. In this study the median and mean cost overruns (40 dams), expressed as a percentage of the dam cost estimated immediately prior to construction, are 49% and 120% respectively with the smallest and largest cost overrun values being -48% and 825% respectively. Based on the available data dam cost overruns appear to be more prevalent in sedimentary rock"* [ie as for SESRO]*" than hard rock (e.g. igneous and metamorphic rocks) settings. The strong likelihood of dam cost overruns occurring has implications to forecasted benefit-cost ratios and supports assertions that large dam cost and contingency*

[63] Matt Coombs Binnies reservoir delivery director podcast Navigating reservoir expansion challenges in the United Kingdom4.1.2024.

[64] Dams, dam costs and damnable cost overruns, Petheram and McMahon, Journal of Hydrology, page 1 April 2019 https://www.sciencedirect.com/science/article/pii/S2589915519300100#:~:text=In%20this%20paper%20the%20term,cost%20overrun%20would%20be%2050%25

estimates should be checked at pre-feasibility and feasibility stages by an independent organisation and by persons highly experienced in dam design, construction and costing."

It should be noted that the median increase of 49% is against the "*publicly stated or contracted cost immediately prior to construction"* [65] The current state of advancement of the SESRO design is a long way short of '*immediately prior'* to construction. Many of the cost risk factors that I have summarised in Section 10.1 apply to the design and planning of the reservoir, before the start of construction. Therefore, the potential for cost over-runs is a lot higher than the median 49% quoted in the Australian paper.

The report emphasises the particularly high risk of cost over-runs with earth embankment dams:

> *"Sampled by dam type the largest median percentage cost overruns are recorded for earthfill embankment dams."*[66]

> *The median cost overrun for embankment dams was 106%.*[67]

The quoted cost over-runs were after adjustment for inflation: "*The dam costs were adjusted to 2016 costs in Australian $."*[68] .

The report showed that the length of the construction period adds to the risk of cost over-runs – a particular risk for SESRO because the 10 km length of the embankment necessitates a multi-year construction period, a total of 8 years shown in programme in the Concept Design Report. This increases the risk of encountering exceptionally wet weather which will delay the earthworks operations:

> *"The most important independent variable is years of construction which accounts for 42% of the variance*." [69]

This effect is not because of special conditions in Australia: "*in a summary of cost overruns in large infrastructure assets from non-Australian countries Ansar et al 2014 reported the mean cost overruns for construction projects…mega dams 96%*" [70]

The report emphasises the risks of over-optimism from scheme promoters:

> "*Based on their analysis some authors take* "over optimistic assumptions" *a step further and implicate manipulated forecasts as a probable cause for many cost overruns… taking evidence from transportation infrastructure projects…issued a warning to legislators, administrators, investors…that reported cost estimates were often highly and systematically misleading and that they should not trust cost estimates and benefit cost analysis produced by project promoters."* [71]

> *"The strong likelihood of dam cost overruns occurring has implications to forecasted benefit cost ratios and supports assertions that large dam cost and contingency estimates*

[65] Ibid page 2
[66] Ibid page 2
[67] Ibid page 3 Table 5
[68] Ibid page 2
[69] Ibid page 9
[70] Ibid page 1
[71] Ibid page 11

should be checked at pre-feasibility and feasibility stages by an independent organisation and by persons highly experienced in dam design, construction and costing." [72]

These warnings against over-optimism of scheme promoters apply to SESRO and its water company promoters. Again, I would emphasise that the median cost increase of 106% quoted for embankment dams, applies to the pre-construction cost estimate, so it doesn't allow for the design cost escalation risks that I have identified above.

10.3 Allowances for costed risk and optimism bias

The likelihood of cost over-runs is provided for in the WRMP and Gate 2 costings by allowances for optimism bias and costed risk. The Gate 2 SESRO cost report states allowances of 23% for costed risk[73] and 28% for optimism bias[74] giving a total 51% allowance for cost escalation.

The 23% allowance for costed risk is said to take account of the risks identified below[75]:

Table 3-1: Examples of Key Risks from Monte-Carlo Simulation

Aspect	Description
Reservoir Embankment	Poor weather conditions inhibit placement of fill to form the reservoir embankment.
Off-site compensation / On-site improvements	Biodiversity Net Gain (BNG) requirements result in a need for further onsite habitat creation and / or offsite land purchase.

Aspect	Description
Reservoir Borrow Pit	Excessive groundwater or surface water is encountered in the borrow pit excavation requiring extensive dewatering measures and works to dry out clay prior to placement.
Construction Plant	Use of low-carbon plant may cause a significant increase in rates used in the current capital cost build-up.
Material Delivery by Rail	Increased traffic on the railway line restricts ability to import construction materials by rail.
Inflation	Above-RPI inflation of key materials, particularly fuel for earthmoving plant.
Reservoir Embankment	The available clay in the borrow pit is less suitable for embankment construction than expected, requiring modification of the embankment design or processing of the clay.
Renewable Energy	More extensive renewable energy generation may be developed as part of scheme (above the currently included hydropower turbines).
Reservoir Embankment	Foundation of the perimeter embankment is weaker than expected requiring a modification to the section and increased cut and fill volumes.
Recreational Use	Recreation facilities are more costly than currently estimated.

The risks identified in the above table nominally cover three of the risks I have identified in Section 10.1 for embankment slopes, crest settlement and construction delays due to wet weather and slow pore pressure dissipation. However, noting adoption of maximum slopes

[72] Ibid page 1
[73] SESRO cost report, PDF page 13
[74] Ibid, PDF page 14
[75] Ibid, PDF page 12

alone would add 30% to the embankment fill volume, the 23% allowance seems to me to be much too low even for these three items.

There are then all the other cost risk factors that I have identified in Section 10.1 that have not been considered at all in the above table. In my opinion, the quantities and sources for the rip-rap and drainage materials, together with their unknown BNG costs, are a particularly likely source of cost-over-runs. The failure to give any consideration to the detailed design of the emergency draw-down works is also a potential source of appreciable cost overruns. These items, which have not been considered at all in the above table, are all pre-construction cost escalation risks, which would be in addition to the median 106% cost escalation quoted in the Australian paper for the construction phase of embankment dams.

The reported optimism bias of 28% compares with a quoted upper bound to the optimism bias of 66%. The reduction from 66% to 28% is said to be due to scaling back *"to account for risks that have been identified, understood and managed"*[76], referring back to the Table 3-1 that I have copied above, which has been used to justify the costed risk allowance of 23%. In my opinion, in view of the immature state of the reservoir design and the apparent failure to consider a lot of the cost risks that I have identified, the optimism bias allowance of 28% is far too low and should be something not far short of the upper bound of 66%.

10.4 Risks inherent in the SESRO approach

The current Thames Water proposal for SESRO is to seek WRMP and DCO approval prior to carrying out the dam break analysis, prior to the main field soil trial embankment, prior to the main stability analyses, prior to identifying sources /grading of imported riprap and drainage material and thus prior to finalising the design of the embankment and outlet works. As I have pointed out earlier in this report, all of the design activities that are being left until after DCO approval, carry high potential for generating appreciable cost over-runs.

This seems illogical. Surely there should be more refinement of the design and capital cost prior to selection of the next major source for the South East. This is especially when one of the outstanding features is the slope of the dam, where quite small changes can have a large effect on capital cost. Other outstanding issues are the cost of the drainage material, quoted by Jacobs as "extremely expensive", the unknown sources and cost of the riprap and drainage material, the lack of a dam break assessment, and the design of the outlet works, especially the 76 m^3/s emergency drawdown facilities and its potential impact elsewhere.

Overall, there would appear to be a large scope, and hence appreciable risk, for capital cost escalation of SESRO. The Australian study quoted above suggests a median cost overrun of 49% and, for embankment dams in excess of 100%, after final estimate at the point of construction. Clearly the current estimate has been based on a level of design that is a long way short of ready for construction. This all points to the need for much more engineering investigation, design work and scheme costing before a decision can safely be taken on the choice of the next major water source for the South East of England.

CJA Binnie MA, DIC, Hon DEng, FREng, FICE, FCIWEM
12th January 2024

[76] Ibid, PDF page 14

Appendix 1: C J A Binnie: Curriculum Vitae

Name	**Christopher J A Binnie**
Profession	**Independent water consultant**
Year of Birth	**1938**

Key qualifications
Chris Binnie is an expert in water engineering, dams, water resources development, flooding issues, tidal range power and the management of large water projects.

For nearly twenty years he was director for water consultancy at WSAtkins, one of the largest UK engineering consultancy firms. He has been project manager or project director for many schemes in UK, Asia, and Africa and has experience of promoting, designing and constructing many large projects.

Chris was, from 1980 to 2008, a Panel AR Engineer under the Reservoirs Act.
Construction Engineer Norton Fitzwarren dam, Newdale dam,
Construction Engineer overseas equivalent, Ain Zada dam 50m, Gargar dam 70m,
Design verified Yuvecik Turkey 110m
Project Manager to AR Engineer Marchlyn Mawr dam 70m.
Resident site staff on a dam, Grafham Water, Farmoor 1, Durban Heights, Sar Chesmeh reservoir
Studies for dam development, Abberton raising, Darwell raising, Bartley raising, Longdon Marsh, English Stones barrage, Severn Barrage, Caborra Bassa 170m, .

Chris is a Visting Professor at Exeter University and lectures on dams and dam design.

Career Summary

2017-present	Director of Tidal Engineering and Environmental Services Ltd.
1998-present	Independent Consultant
1998 - 2001	Deputy Chairman, Binnie Black & Veatch,
1994 - 1997	Director, W S Atkins plc,
1983 - 1995	Managing Director, Atkins Water and Director, W S Atkins Consultants Ltd
1978 - 1983	Chief Engineer, W S Atkins & Partners,
1969 - 1978	Senior Engineer, Binnie & Partners,
1968 - 1969	Civil Engineer, Roberts Construction, South Africa
1967 - 1968	Civil Engineer, George Wimpey
1964 - 1967	Project Engineer, National Coal Board
1963 - 1964	Graduate Engineer, Binnie & Partners,
1958 - 1959	Site Engineer McAlpines, Great Ouse Flood Protection Scheme and A1(M)

Education and Professional Status
MA, Engineering and Law, University of Cambridge, 1962
Post Graduate Diploma, Geotechnical Engineering and hydro power, Imperial College, London, 1963
Honorary Doctorate of Engineering, Bradford University 1997

Fellow, Institution of Civil Engineers, 1976 to present (Member, 1967)
Fellow, Chartered Institution of Water & Environmental Management 1976, President 1995/6
Fellow, Royal Academy of Engineering, 1994 to present

Member, Association of Consulting Engineers, 1984-1997 and member of Council

Member, British Hydrological Society, 1994 to present
Member, All Reservoirs Panel of Engineer under the Reservoir Act, 1975, 1980- 2008
Member British Dams Society, 1973- present
Visiting Professor at Exeter University 2008-present, lecturing on dams and dam design.
Visiting Professor at Kingston University 1995-2003
Member of the CIWEM Water Resources Expert Panel 1992-present
Member of the ICE Water Expert Panel and water CAB 1987-present
Member ICE Energy Expert Panel 2018-present
Member of ICE Reservoirs Committee 2002- 2007 including interviewing prospective panelists.

Countries of Work Experience Algeria, South Africa, Egypt, Iran, Malaysia, Singapore, Indonesia, Hong Kong, Nigeria, Cameroon, Philippines, Portugal, Zimbabwe, Turkey, Kenya, Iraq, Zambia, China, Sri Lanka,

Experience Record

1998-present Independent Consultant

2022 Report on Avonmouth tidal range scheme
2021 Review of the Wratton tidal range scheme
2020-23 Welsh tidal range energy pathfinder project at Aberthaw
2020 Report for Anglian Water on Capital Maintenance for AMP7.
2018-19 Review Board of the Mersey tidal range power scheme
2018-2019 Advice on flooding scheme in Cardiff
2018 Review of flooding from a fish pass in Hampshire
2017 Review of flooding from Lake Windermere for WLLFG
2017-2023 Project developer for the £8.5bn, 2,500MW West Somerset Tidal Lagoon project
2016-19 Advice on value of the Aylesford Paper Company water abstraction licences
2015 Report on hydro power potential on Exmoor
2014 Review of the Wyre tidal barrage project
2013 Advice on Llanishen Dam redevelopment in Cardiff
2012 Review board of the Peel Mersey barrage
2009-2014 Review and Expert Witness of the Abingdon Reservoir for GARD
2007 Expert witness evidence to EIP on RSS on flooding east of Swindon
2007-10 Chairman of the DECC Independent Technical and Engineering Review Board for the Severn tidal power studies
2005-8 Construction Engineer for Norton Fitzwarren dam
2005-6 Expert witness on appeal by Thames Water on the Becton desalination plant
2005 Expert witness on planning application to extend Ardleigh reservoir
2003 Design report on 130 Mm3 Longdon Marsh reservoir.
2002-7 Flood Risk Assessment reports for about 25 different developments
2002-3 Expert witness for British Waterways on court action on flooding from Oxford Canal
2002 *Thames Water*, referee under the Reservoirs Act for King George v reservoir
2002 *Severn Trent*. Prefeasibility study for four reservoir sites, including raising Frankley dam
2002 *Thames Water* assessment of industry standards for dam operation and reservoir drawdown.
2000 Expert witness for Institution of Civil Engineers presentation to MPs on flooding issues
1999-2006 Chairman of Thames Tideway Strategy Steering Committee investigating improvements to Thames Tideway water quality, estimated capital cost £2bn
1999 *Water UK* Expert witness at Competition Commission hearings of appeal by two water companies.
1999 Construction Engineer for raising Cherry Orchard reservoir
1999-2005 Construction Engineer for raising of Upper Compton Verney reservoir embankment.
1998 *Water UK* Report to Ministers on AMP3 draft determination on capital maintenance

1998 - 2001 Binnie Black & Veatch
Deputy Chairman, Redhill office

Advising on water projects
Northern Ireland Water Service Technical direction of long term water resources strategy study
British Waterways. Technical direction of a water resources strategy for canal transfers.
Director BCB, now British Expertise, representing BBV

1997 - 1998 W S Atkins plc
Main Board Director and Director, South East Asia

Responsible for all work carried out in South East Asian offices of WS Atkins.plc

1995 – 1997 W S Atkins Ltd (from 1996 W S Atkins plc)
Main Board Director

Responsible for International Operations and Development including all overseas offices of WS Atkins in Europe, Middle East, South and South East Asia, and Australia.

1992 – 1995 W S Atkins Water
Chairman, Water, Environment and Tunnelling Group

Responsible to Board for all Atkins water, environmental, and geotechnical work (over 300 staff).
Member of Council of the Association of Consulting Engineers

1983 – 1995 W S Atkins Consultants Ltd
Director, Epsom office and overseas

Also Managing Director of Atkins Water.

All Reservoirs Panel Engineer licensed by British Government for inspection of dams. Inspected some 25 dams under the Reservoirs Act including existing 20 m high Bartley dam for Severn Trent involving hydrology and site investigations and 30m high Colliford dam

Newdale dam Construction Engineer for the Newdale flood protection dam

Yuvecik dam, for Izmit, Turkey: Design certification of 110 m high rockfill dam with clay core in a highly seismic area. Checked dam against enhanced seismic loading and it subsequently survived a major earthquake in 1998.

Abberton embankment dam. Study of the raising of Abberton dam, including site investigation of the existing dam and planning of the raising.

Mombassa Water Supply, Kenya: responsible for World Bank funded new 200 000 m^3/day water supply, outline and detailed design stages and environmental assessment. Project cost US$300 million.

Greater Algiers Water Supply Algeria: water demand projection and design of next phase of development. Joint venture Binnies/Atkins/COBA. Project cost about $500 M.

Ashford Great Park Expert witness on flooding issues at Planning Inquiry into 600 ha development partly in the flood plain including modelling of river and design of mitigation works.

Water Supply Master Plan, Northern Ireland: Project Director. Water demand projections to 2020, assessment of existing source yields, identification of water resources needed, and development master plan.

Thames Water London Water Ring Main Tunnel: Project Director. Engineering, environmental and planning studies for new Holland Park shaft including comparison of 6 alternative sites for the pump out shaft in congested urban London. Followed by the design and site supervision of the facilities. Tunnel 2.8 m diameter, wedge block constructed by TBM..

Severn Barrage, United Kingdom: Project Engineer and Management Board for Severn Tidal Power Group, outline design of embankments on the Cardiff/Weston line, 50 m high on alluvial foundations for 5 000 MW tidal power project. Capital cost c £5bn

Severn Barrage, English Stones, UK: Project Engineer for engineering layout of the feasibility study of tidal power generation on River Severn including 40 m high embankments and 1 000 MW hydropower, cost about £1bn.

Karkh Water Supply Scheme, Stage IIA, Iraq: Director responsible for detailed design and site management of extension of the works to ultimate capacity of 1.3 M m^3/day. The work included design of pre-settlement tanks, clarifiers, filters, reservoirs, pumphouses, and ancillary buildings and management of scheme interfaces. Cost about £400Million

1978 – 1983 W S Atkins & Partners
Chief Engineer, Epsom office and overseas

Gargar Dam Project, Algeria: Project Director for the 70 m high earthfill dam with high seismic loading in north west Algeria. Spillway capacity, 5 000 m^3/s.

Algiers Water Supply Project, Algeria: Project Manager for the IBRD funded joint venture project with Binnie & Partners. The work included water demand/supply balance; selection and investigation of dam sites including about ten to feasibility level with several dams over 50 m high; preparation of water development master plan; modelling of existing pipe network system in Algiers; master plan for water distribution; detailed design of first phase works including intakes, pumping stations, pipeline, 500 000 m^3/day treatment works, service reservoir and water distribution system. Estimated cost about £500 million. .

Ain Zada Dam, Algeria: Project Manager for the design and construction supervision of 50 m high rockfill dam.

Gubi dam. Nigeria. Report on construction of 10m high dam

Nile Delta Fish Farm Project, Egypt: Project Manager for Egyptian Ministry of Agriculture for project. 30,00 acres of fish pond. Funded by the World Bank (IBRD), the British Government (ODA) and the Egyptian Government and supervised by the FAO Cooperative Programme.

1977 - 1978 Binnie & Partners
Chief Engineer, London office and Egypt

Ismailia Canal Study, Egypt: Project Engineer for study of 125 km canal carrying 150 m^3/s including engineering works for widening and deepening the canal, bank protection, and barrage control structures. Funded by British Government (ODA).

1976 - 1977 Binnie & Partners

Resident Engineer, Iran

Sar Cheshmeh Copper Mine, Iran: re design and supervision of re-construction of 30 m high embankments to form terminal reservoir for 50 Mld water supply scheme, earth fill with plastic liner.

1971 - 1976 Binnie & Partners
Senior Engineer, London office and Indonesia

Brenig dam, Review of design of 70m high rockfill dam.

Dinorwig, North Wales hydroelectric project: Project Engineer for the feasibility investigations, Parliamentary Bill including negotiation and preparation of Parliamentary evidence, design, and supervision of construction of the 70 m high Marchlyn Mawr rockfill dam of the 1 800 MW Dinorwic pumped storage hydro-electric project in North Wales. Rockfill with asphalt liner.

Darwell dam Study for raising Darwell dam in Sussex

1969 – 1971 Binnie & Partners (Malaysia)
Engineer, Kuala Lumpur office

Palembang Water Supply Study, Indonesia: assessment of future water demand, uprating the existing treatment works (45,000 m^3/day) improving the operation of the water system and planning development of a water treatment works and distribution systems to year 2000. Funded by ODA.

Project Engineer for

- Sandakan groundwater investigation scheme, East Malaysia*:*
- Garinono dam, review of project.
- Brunei hydrological survey
- Brunei water supply development
- Selangor State hydrological survey
- Kuala Lumpur water master plan study (population 1 000 000 rising to 2 000 000 by 1985) for World Bank including demand projection, choice of next water source, outline design of dam, and optimum phasing of development.
- Report on failed dam near Penang.
- Report on dam for palm oil plantation, near Sandakan

1968 – 1969 Roberts Construction Company
Civil Engineer, South Africa
Design of cofferdams for 170m Carbora Bassa dam on Zambezi river, overtopping flow 10,000m3/sec.
Resident engineer on works for large Bloemhof reservoir
Resident Engineer on Durban Heights 250,000m3 tent shaped service reservoir, concrete lined rockfill embankment.
Tour of several recent or under construction dams, Bethlehem dam raising, Bridle Drift Dam, Pongolapoort concrete arch dam u/c.

1967 – 1968 George Wimpey
Civil Engineer
Design of storage tanks
Investigation of a failure

1964 – 1967 National Coal Board
Project Engineer

Working as client's civil engineer on multi contract project including supervision of consulting engineers.
Planning for Immingham coal shipping terminal.

1963 – 1964 **Binnie & Partners**
Graduate Engineer, London office and on site
Site staff on Grafham embankment dam and intake works.
Investigation of using Stewartby pits for new water resource storage reservoir.
Site staff on Farmoor reservoir stage 1.

Summer 1963 Tour of European dams, two made of moraine in the arctic, Venemo dam in Norway u/c, a pumped storage scheme in Austria u/c, a pumped storage scheme in Germany and La Rance tidal power in France u/c.

1962-1963 **Imperial College**
Post graduate study in soil mechanics of embankment dams and hydro power including trip to see dams hydro power dams including Cruachan under construction. and Ffestiniog. Also field researcher on geotechnical properties of Selset dam, Visit to the remains of Malpasset concrete arch dam in southern France.

1958-1959 **Sir Robert McAlpine**
Site staff on a factory development, Great Ouse Flood Protection Scheme earthworks, and A1(M) motorway bridges and earthworks.

Publications and Papers

Binnie & Finlayson, 'Nile Delta Fish Farm Project', Institution of Water and Environmental Management (IWEM), 1982.
Binnie, 'Rural Water Supply Development', WEDC Water Development Conference, Harare, 1984
Binnie & Roe, 'Civil Engineering Aspects of English Stones Barrage', Institution of Civil Engineers (ICE) Symposium, 'Tidal Power', Thomas Telford, 1986.
Binnie & Askew, 'Remedial Works at Brayton Barff reservoir', British Dams Society Biannual Conference, Reservoir restoration, 1988.
Editor for ICE, 'Water Supplies in the UK', Thomas Telford, 1990.
Binnie, 'The Effect on Consulting Engineers in the UK' – 'Water Privatisation One Year On' Conference, BICS, 1990.
Binnie, 'Securing Our Future Water Supply', Institution of Water Officers (IWO) Conference 1991.
Binnie, 'Do we need a National Plan for Water?', 'Conference Managing Water in a Dry Decade', Anglian Water, 1991.
Binnie & Herrington, 'Possible Impact of Climate Change on Water Resources and Water Demand', 'Engineering Implications of Climate Change', Conference, ICE Thomas Telford, 1992.
Binnie, ' Demand Management, Tariffs and Metering', ' Paying for Water' Conference, IWEM, 1992.
Binnie, 'The Consultants View', 'Water Market Awareness' Seminar, South West Water, 1992.
Binnie, ' Demand and Demand Management', ' The Management of Scarce Water Resources' Conference, IWEM, 1992.
Binnie & Sweeney, 'Response of a Clay Embankment to Rapid Drawdown', British Dams Society, 'Water Resources and Reservoir Engineer' Conference, Thomas Telford, 1992.
Binnie, 'Future Trends in Water Resource Development', Salmon and Trout Association Seminar, 1993.
Plester & Binnie, 'The Evolution of Northern Ireland Water Resources', 'Water Resources Development Strategies', CIWEM, 1994.
Binnie, ' Chartered Institution of Water and Environmental Management Centenary Address', 16 times UK, 3 times overseas, 1994-1995

Binnie, 'Effect of Climate Change on Water Resources in South East Asia', IWSA Regional Seminar, Hong Kong, October 1996 (Selected as one of 3 best in 96 papers), Vol. 46 No.5, 1996.

Binnie, 'Private Sector Financed Infrastructure', British Consultants Bureau Seminars in Viet Nam and Philippines, 1996.

Binnie, 'International Experience in Water Conservation' in 'Towards Efficient Water Use in Urban Areas in Asia and the Pacific', UN Economic and Social Commission for Asia and the Pacific, 1997.

Binnie, 'Commonwealth Water Sector Overview', Commonwealth Water Forum in Penang Malaysia, 'Sustainable Water Resources Management into the 21st Century - Policy and Technological Innovations', 1997.

Binnie, 'Water the New Currency - Why?' AEA/NZ bi annual conference in Cairns, Australia, 1997.

Binnie, 'Water Demand Management - Efficient Water Use in Urban Areas', UN ESCAP Conference, Singapore, 1997.

Binnie, 'Effect of Climate Change on Water', Commonwealth Engineers Conference, Penang, Malaysia, 1997.

Binnie, 'Possible Effects of Climate Change in South East Asia' in International Conference on 'Hydrology in a Changing Environment' organised by British Hydrological Society, 1998.

Binnie, 'Water Demand Management' lecture to Hong Kong Branch of Chartered Institution of Water and Environmental Management, 1998.

Binnie, 'Centenary Address of Chartered Institution of Water and Environmental Management', 1998.

Binnie, Hughes & Rowland, 'Dam Raising – The Economic Approach with Minimum Impact', CIWEM Conference , 1999.

Binnie, 'Environment and Climate Change', topic leader at consultation seminar on 'Environment Agency Sustainable Abstraction' Seminar, 2000.

Binnie History of London Water in Millennium 2000

Bridle and Binnie submission to World Commission on Dams on behalf of the British Dams Society.

Binnie "Water resources for the future "given at joint conference of ASCE and ICE on International Water Crisis, Washington USA 2000

Binnie Report of the World Commission on Dams , presentation at joint ASCE/ICE conference on International Water Crisis Washington USA 2000

Binnie Water for Life supply and demand in the 21st Century. Ingenia Magazine of Royal Academy of Engineering may 2001

Binnie" Implications of World Commission on Dams on water supply" at conference of British Dams Society on the World Commission of Dams Report Feb 2001

Binnie Aspects of the Draft Water Bill, British Dams Society 2001

Binnie Water Millennium Address on behalf of Institution of Civil Engineers given 17 times in 2000 and 2001

Binnie Lets fix the Assets First, Institution of Water Officers Annual Conference, May 2001.

Carpenter, Binnie et al Environment, Construction, and Sustainable Development, Wiley 2001

Binnie Capital Maintenance at AMP3, to Conference on Capital Maintenance in the Water Industry, Aston University Jan 2002.

Binnie, Water Privatisation, at Conference on PFI/PPP for BCCB?IFSL City of London Jan 2002

Binnie The History and Future of Water and Plumbing Development. Joint lecture for Worshipful Company or Plumbers and Worshipful Company of Water Conservators.

Binnie, Kimber, Thomas Basic Water Treatment published by Thomas Telford, now 6th Edition.

Binnie 1976 A lesson from the Past CIWEM Conference on Drought Planning 2002

Binnie and Burston Reservoirs- Maximising Environmental Gain, CIWEM Conference on New Approaches to Water Resources Development, 2002

Environment Agency internal seminar Water demand and supply spring 2006

Presented Geoffrey Binnie lecture at British Dams Society Biennial conference Durham 2006

Binnie and Nithsdale, Thames Tideway Project, Conference on Wastewater Management 2006.

Binnie, Institution of Civil Engineers Bicentenary Water lecture 2018

Lectured at Exeter University on dams and dam design 2008-2023

Appendix 2: Terms of Reference

1. To review the availability of engineering design details of the SESRO reservoir and comment on their adequacy for determining the safety of the design and the cost of reservoir construction.

2. Within the constraints of available engineering design details, to comment on the design in general such as design development, trial embankments, materials, factors of safety, internal drainage, dam break, emergency drawdown, and opportunities for cost escalation with reference to the experience of other projects.

3. To provide a professional opinion on the trial bank and whether it should perform the function intended. GARD requires a professional view on such questions as:

   - Are the three 30x18x2m trial embankments big enough to give meaningful results for a 25m high x 10 km long actual embankment?

   - Will the 2m high trial embankments provide information on the build-up of pore pressure and its rate of dissipation in the lower layers of the 25m high embankment?

   - Will the plan area of the trial embankments be big enough to represent compaction due to construction plant traffic on the actual embankment?

   - Is the c .6 month (summer only) trial long enough to cater for the variation in climate likely during multi-year actual construction or to monitor rates of pore pressure dissipation?

   - Is the single small trial borrow pit going to be representative of the variation in material coming over the c. 3km x 2km actual borrow pit?

4. In addition a survey of the relevant referenced literature on trial embankment findings and shortfalls on similar embankment dam projects would be required.

5. We would require a recommendation for the outline of a trial programme which would yield relevant information and reassurance on the safety of the SESRO design, and the timing of such trials.

6. We require a professional opinion on the issues of Freeboard design and leakage from the Reservoir.

Appendix 3 – Jacobs 2007 Preliminary Design Report

2 Geology, hydrogeology and geotechnics

2.1 Introduction

This Chapter describes the key geological, hydrogeological and geotechnical issues relating to the design of the reservoir. The understanding of the site is derived from published data and major site investigations undertaken in 1991 and 2005.

2.2 Regional setting

The reservoir site is situated in the Vale of the White Horse to the south of the River Ock, as shown in Figure 1.1. The topography of the site is generally flat with an elevation in the range 55 to 65m AOD. To the south of the site the ground rises to an escarpment and the rolling downland topography of the Berkshire Downs ranging in elevation from 150 to 200m AOD. To the north of the River Ock the ground rises to the Midvale Ridge at an elevation of 80 to 100m AOD before descending into the Thames valley.

The low-lying catchment of the Vale of the White Horse drains eastwards along its axis via the Ock to the Thames. In the area of the site, a series of small streams flow northwards from the Berkshire Downs escarpment to the Ock.

The strata encountered at the reservoir site comprise shallow dipping Cretaceous and Jurassic formations, namely Gault and Kimmeridge clay which are separated by the Lower Greensand. These strata are covered by thin Quaternary deposits and underlain by the Corallian limestone. The strata dip to the south-east with the older strata outcropping to the north. An extract of the local geological map is presented in Figure 2.1.

The proposed reservoir site is situated between the Corallian outcrop in the north and the Upper Greensand and Chalk in the south. It is immediately underlain mainly by solid strata of the Kimmeridge Clay Formation and, further south, by the Lower Greensand Group, bounded by unconformities, and then by the Gault Formation (Figure 2.1). The Kimmeridge Clay Formation is underlain successively by the Ampthill Clay Formation, the Corallian Group, the West Walton Formation and the Oxford Clay Formation, though these are not exposed at the reservoir site.

2.3 Geology

2.3.1 Stratigraphy of the Abingdon area

(a) Stratigraphic sequence

The stratigraphic sequence encountered in the reservoir area is summarised in Table 2.1 and illustrated in Figure 2.2.

Table 2.1 Stratigraphy of the Abingdon area

System	Series	Stratigraphy	
Quaternary	Holocene	Topsoil	
		Alluvium	
	Pleistocene	River Terrace Deposits	
		Head Deposits	
		Group	**Formation**
Cretaceous	Upper Cretaceous	Chalk	Upper Chalk
			Middle Chalk
			Lower Chalk
	Lower Cretaceous		Upper Greensand
			Gault
		Lower Greensand	
Jurassic	Upper Jurassic		Kimmeridge Clay
			Ampthill Clay
		Corallian	
			West Walton
			Oxford Clay

The strata which are of relevance to the project are the Corallian, Kimmeridge Clay, Lower Greensand, Gault Clay and Quaternary Deposits.

(b) Corallian Group

The Corallian Group outcrops about 1km north-west of the proposed reservoir footprint and occupies the Midvale Ridge further to the north. The Corallian succeeds the sands and clays of West Walton Formation with an abrupt change to a laterally variable interbedded sequence of calcareous sandstones, limestones and sands. The complete sequence of the Corallian, within the reservoir area, varies in thickness from about 19m to 28m. Although the limestones are mainly strong, the sandstones exhibit marked variations in strength from very weak to very strong. The sands are interpreted as uncemented in situ rather than disintegrated by the site investigation drilling process.

(c) Kimmeridge Clay Formation

The Kimmeridge Clay Formation is separated from the Corallian Group by the Ampthill Clay Formation. Although the Ampthill Clay rarely exceeds 1m in thickness, the stratum is easily distinguishable from the overlying Kimmeridge Clay being represented by sandy clays, which often contain phosphatic nodules, oyster shells and belemnites.

The Kimmeridge Clay attains a thickness of about 45m. It comprises stiff to hard, fissured, thinly laminated clays, locally silty and/or sandy, which include prominent horizons of shell debris, oyster shells and phosphatic nodules. Some parts of the sequence are slightly bituminous and isolated bands of bitumen up to 30mm thick were encountered occasionally in core samples. The sequence is locally interbedded with stiff to hard planar thinly laminated mudstones. Nodular cementstone horizons, which are underlain and overlain by about 1m of hard calcareous clays are also present. These horizons, shown in Figure 2.2 and labelled H1 to H6 in ascending stratigraphic order, tend to be impersistent due to their nodular character and rarely exceed 0.5m in thickness.

(d) Lower Greensand Group

The proposed embankment footprint straddles the narrow outcrop of the Lower Greensand Group, which is separated by unconformities from the underlying Kimmeridge Clay and the overlying Gault Clay. The Lower Greensand varies in thickness along its outcrop due to these unconformities, attaining a maximum thickness of 6m. The Lower Greensand comprises mainly firm, sandy, silty clay with only infrequent and discontinuous thin beds of weak to strong sandstone and rarely of strong conglomerate. This relatively high fines content of the Lower Greensand in the Abingdon area contrasts with the sands and gravels with very subordinate clay and silt fractions, which are more typical of the group in other parts of the UK.

(e) Gault Clay Formation

The embankment footprint in the south lies on the lower part of the Gault Clay Formation, which rests unconformably on the Lower Greensand Group. The results of the recent ground investigations indicate that the Gault Clay comprises a firm to very stiff, fissured, clay with occasional partings of shell debris, sand and silt. In contrast to the Kimmeridge Clay Formation, the Gault Clay is characterised by the absence of nodular cementstone bands, relatively few phosphatic nodules and very subordinate partings of shell debris, sand and silt.

(f) Quaternary Deposits

The Quaternary deposits occur as a more-or-less continuous cover overlying the solid strata across the site with an average depth of 2m. These deposits include surface Topsoil, which is successively underlain by Alluvium, River Terrace Deposits and Head Deposits at those locations where the complete sequence of deposits is represented.

2.3.2 Tectonic or periglacial disturbance of strata

The geology of the site is well suited to the construction of a reservoir with it being predominately underlain by significant thickness of impermeable clay deposits. However the engineering behaviour of the embankment foundations and reservoir floor could be influenced detrimentally if the solid strata have been affected by tectonic or Quaternary deformation of the solid strata. Such deformation could have resulted in significant changes to the sequence and engineering properties of the strata beneath the proposed reservoir site. Of prime concern would be:

- Reduction in the thickness of clay material overlying the Corallian in the borrow pit excavation within the reservoir
- Loss of strength in the embankment foundations due to the presence of tectonic or periglacial shear surfaces

Considerable effort was therefore made in the interpretation of the site investigations. Structure contour plots were generated for each of the boundaries and for the cementstone horizons. These plots confirm that formation boundaries and the cementstone horizons within the Kimmeridge Clay exhibit a uniform spacing and alignment. These plots confirm that the strata are planar, though tilted, with a very shallow and consistent dip across the site. The structural model for the site is illustrated as a representative geological cross-section in Figure 2.2. The absence of any abrupt offsets of contours on the structure contour plots, together with the lack of bedding/lamination displacement, periglacial brecciation or shear zones within the core samples, supports the interpretation that the solid strata are unfaulted and have not been affected by valley bulging.

A main conclusion of the investigation was that shears within the solid strata (i.e. the Gault and Kimmeridge Clays) are most probably caused by stress relief due to erosion and not by any large scale mass movement. As such, there is no evidence for extensive shear surfaces within the solid strata which could reduce the stability of the embankment.

Shear surfaces also exist within the superficial Head deposits. These tend to be of a random nature with variable dips and dip directions. It is considered that the origin of these shears is unlikely to be periglacial, but it cannot be ruled out. In this case it is considered prudent to excavate all head deposits from the foundations and so remove the risk of preferentially orientated shear surfaces occurring in the head deposits beneath the embankment.

2.4 Hydrogeology

2.4.1 Introduction

The hydrogeological conditions under the proposed reservoir site are important for the following reasons:

- Leakage from the operational reservoir into the underlying aquifers may have an impact on the existing groundwater quality and flow conditions below the site which, in turn, may affect surface water quality in watercourses such as the River Ock. Understanding the vertical permeability is, therefore, important to allow the quantitative assessment of the leakage.
- High heads or groundwater pressures in the underlying Corallian need to be understood in order that sufficient Kimmeridge Clay is left in the base of the reservoir to avoid heave or fracturing due to excessive uplift pressures during construction and operation.
- The horizontal permeability of the Kimmeridge Clay and, in particular, any laterally continuous limestone bands may have an influence on the slope stability and rate of settlement of the embankment.

2.4.2 Conceptual model

The main formations of relevance to the hydrogeology of the site area broadly comprises the Corallian aquifer dipping gently under the very low permeability Kimmeridge Clay, with the relatively more permeable Lower Greensand forming a separate very thin aquifer unit between the Kimmeridge Clay and Gault Clay. Thin permeable but discontinuous limestone bands occur within the Kimmeridge Clay. The Cretaceous Chalk aquifer outcropping on the higher ground to the south also dips gently away from the site. Groundwater within the superficial deposits is largely confined to the fluvial gravels, with the finer grained Alluvium and Head Deposits acting as barriers to vertical groundwater flow.

Recharge to the Corallian aquifer from precipitation occurs where the formation outcrops to the north of the River Ock and on the higher ground to the west of the site. The Lower Greensand has a very thin outcrop in the Main Study Area which mostly acts as a discharge zone for the formation. Most of the groundwater flow in the formation appears to be driven from slow seepage into the formation from the vertically adjacent clays. A more significant area of Lower Greensand outcrop occurs in the west of the Ock catchment. This appears to act as a recharge area for part of the confined Corallian. Precipitation falling on the northern slopes of the Chalk outcrop either runs off to enter the streams flowing northwards in the vicinity of the site or recharges the Chalk to flow southwards away from the site within the aquifer.

Where the Corallian aquifer is overlain by the Kimmeridge Clay the groundwater is confined, with heads in the formation significantly above the base of the clay. Groundwater contour plots of measured heads in the Corallian indicate that groundwater flow is in an easterly direction from the recharge area in the west but swings to the north to discharge into the River Ock in the vicinity of the site. Numerical modelling has shown that this flow pattern is broadly consistent with the understanding of the hydraulic boundaries influencing the aquifer which are as follows:

- North – higher heads to the north of the Ock due to recharge in the higher ground.
- South – anticipated lower transmissivities and groundwater velocities where the formation dips further into the confined zone.
- East – the River Thames controlling groundwater flow lines.

Groundwater quality data indicate that the salinity of the groundwater in the Corallian aquifer under the site and further to the east is brackish while that of groundwater along the flowpath between the recharge area to the west and the area discharging to the River Ock is fresh.

It is considered likely that the whole of the Corallian aquifer was brackish at some time in the past. Brackish water remains under and to the east of the site where groundwater flow is very low. However, increased recharge during the last few thousand years has led to flushing of this brackish water along the flowpath from the recharge zone to the west to the discharge zone to the River Ock near Marcham Mill.

Groundwater from the Lower Greensand issues to the immediate south of the site, feeding the north flowing streams. Since the Lower Greensand is relatively thin and of low permeability, these flows are very low in comparison with those in the Corallian.

2.4.3 Issues relating to reservoir design

The hydrogeological issues which relate to the design of the reservoir are:

- Potential for heave or fracturing of the clay base of the borrow pit due to excessive uplift pressures
- Potential for leakage from the reservoir
- Rate of consolidation of foundations under embankment loading

Scoping calculations have shown that there is no risk of excessive uplift pressures provided that the depth of excavation does not exceed 50% of the total depth to the Corallian from ground level. The design of the borrow pit is addressed in Section 5.9

The main potential seepage path from the reservoir is likely to be the Lower Greensand outcrop. Of secondary importance is the potential for downward seepage through the Kimmeridge Clay in the floor of the reservoir into the Corallian. The results of preliminary seepage analyses are reported in Section 5.2.

The rate of consolidation of the embankment foundations would be appreciably influenced by the presence or otherwise of discontinuities and permeable layers within the clay strata. In the absence of permeable layers, consolidation would occur as a combination of horizontal dissipation to the outer edges of the embankment and vertical dissipation into the Corallian. The dissipation of pore pressures has been modelled by finite element analyses as reported in Section 3.3.

The hydrogeological parameters which are required for design of the embankment are therefore:

- Horizontal and vertical permeability of Kimmeridge Clay
- Horizontal permeability of Lower Greensand
- Distribution, continuity and permeability of fissures/joints within the clay formations
- Distribution, continuity and permeability of permeable bands within the Kimmeridge Clay

2.4.4 Hydrogeological parameters for preliminary design

(a) Permeability

The adopted design permeabilities for seepage analysis are:

Kimmeridge and Gault Clay	Maximum	1×10^{-9} m/s
(horizontal)	Minimum	1×10^{-10} m/s
Kimmeridge and Gault Clay	Maximum	1×10^{-10} m/s
(vertical)	Minimum	1×10^{-11} m/s
Lower Greensand	Maximum	5×10^{-6} m/s
	Minimum	1×10^{-9} m/s

These permeabilities are based on measurements made during the 1991 and 2005 site investigations. It has been concluded that there are no significant fissures/joints or continuous permeable bands which will affect the bulk permeability of the Kimmeridge Clay.

(b) Ground water levels

Within the reservoir area, it is assumed that ground water pressures under the embankment foundations are hydrostatic from 1m below ground level, based on measurements made by the vibrating wire piezometers installed during the 2005 site investigations.

2.5 Geotechnics

2.5.1 Embankment foundations

(a) Index properties

The embankment would be founded primarily on the Gault and Kimmeridge Clay formations. These are both stiff, plastic overconsolidated clays with very low permeability. Index properties of the Gault and Kimmeridge Clay are presented in Table 2.2.

Table 2.2 Index properties of the Gault and Kimmeridge Clay

Stratum	Plasticity Index (%)				Moisture Content (%)			
	Mean	Max	Min	SD	Mean	Max	Min	SD
Gault Clay	39	50	16	8	26	34	18	4
Kimmeridge Clay	32	47	13	6	26	45	7	26

(b) Bulk density

The bulk density of both the Gault and the Kimmeridge clay is 19.5kN/m.^{2}

(c) Undrained shear strength

Due to the importance of this parameter, a significant part of the 2005 investigations was devoted to high quality sampling and testing of 100mm diameter samples. Local strain measurement sensors were installed on the samples to provide stiffness data for use in the finite element modelling.

The plot of undrained shear strength (c_u) versus depth shows considerable scatter with there being no clear difference in strength between the Gault and Kimmeridge Clays. On this basis the same design parameters have been adopted for both strata.

An initial design line for the strength of the embankment foundation was developed taking account of all data except for results which were obviously suspect due to testing errors. This design line was set with approximately 10% of the results falling below the design line, which gave a design strength profile of:

$c_u = 30 + 4z$ kN/m^2 where z is the depth below the top of the solid strata at a depth of 2m below ground level

For sensitivity studies strength-depth profiles were also developed for the lower bound line (0% of the results falling below), and a line, for the top 10m of the foundations only, where 30% of the data points were below the line. These profiles were

Lower bound $c_u = 20 + 4z$ kN/m^2
30 percentile $c_u = 41 + 5.4z$ kN/m^2

The test data and these profiles are shown in Figure 2.3.

(d) Effective stress strength parameters

Effective stress parameters are required for the assessment of long-term, drained behaviour of the foundations. They are also used as input to those finite element analyses where undrained strengths are derived from effective stress parameters.

The results of the unconsolidated undrained triaxial can be analysed to determine effective stress strength parameters by plotting the strength at failure in terms of effective stress. This analysis gives the following parameters for the Gault and Kimmeridge Clay

$c' = 2kN/m2, \quad \phi' = 23°$

Both the Gault and the Kimmeridge are plastic clays which can be expected to exhibit strain softening. Strain softening is a loss in strength with increasing strain after peak strength has been mobilised, resulting from particle reorientation on the failure plane. Residual strength parameters were measured in ring shear tests. The results are:

$c' = 0kN/m^2, \quad \phi'_r = 10°$

As stated in Section 2.3.2, no evidence has been found for pre-existing shear planes in the foundation which could adversely affect the stability of the embankment. This means that the embankment does not need to be designed for residual strength parameters. However, the possibility of strain-softening due to overstress of the foundations does need to be considered, and thus the residual strength is a necessary input parameter for the finite element analyses. This is covered in detail in Section 3.3.

2.5.2 Fill

It is assumed that material excavated from the embankment foundations and borrow pit would be used in two main zones within the embankment as follows:

- High quality or structural fill (zones 1and 2) in the inner shoulder and central portion of the embankment
- Weaker random or landscape fill (zones 3 and 4) towards the outer toe of the embankment and in landscaping.

The high quality fill would come largely from the borrow pit excavation in the Gault and Kimmeridge Clay. The random and landscape fill would generally comprise overburden and weak or unsuitable material from the borrow pit.

For current design purposes, tests have been carried out on samples recompacted in the laboratory. Whilst these would not be truly representative of field behaviour, they are used conventionally and are adequate for preliminary design purposes.
The following strengths would be adopted:

Structural fill $80kN/m^2$
Landscape fill $50kN/m^2$

These parameters are low compared with data on the strength of similar clay fill reported by Bridle, Vaughan and Jones[6] in relation to Empingham Dam, where the mean strengths are 140kN/m² for the shoulders and 160kN/m² for the central zone. The significance of this would be discussed later in Section 3.3.

It is proposed that a trial embankment would be constructed in advance of the main contract for construction of the reservoir. One of the purposes of the trial embankment would be to test compaction techniques and obtain representative samples of recompacted material for testing.

2.5.3 Seismicity

(a) Selection criteria

As recommended by ICOLD bulletin 72[7], the Building Research Establishment (BRE) Guide to Seismic Risk to Dams[8] and the application note to the UK seismic guide[9], the seismic safety evaluation of the reservoir would be carried out for two representative design earthquakes that are as follows:

OBE - Operating Basis Earthquake
SEE - Safety Evaluation Earthquake

The OBE and SEE earthquakes are defined by peak ground acceleration (PGA) which is the maximum ground acceleration for a defined earthquake level.

OBE or "no significant damage earthquake" is the earthquake which is liable to occur on average not more than once during the expected life of the structure (of not less than 100 years). Typically, the OBE is taken to have a return period of not less than 1 in 200 years. During an OBE, the dam and its ancillary works should remain functional but may need repair. Seed[10] recommends that minimum factor of safety for the OBE should be greater than 1.15 when the stability is checked using pseudo static analyses.

SEE is the earthquake which would produce the most severe level of ground motion under which the safety of the dam against catastrophic failure should be ensured. The return period of an SSE depends on the dam risk category, as defined in references 4, 5 and 6 above. Based on a high risk of potential downstream damage the reservoir would be classified as Category IV, for which a recommended return period for the SEE is 30,000 years. For this earthquake, factors of safety less than unity are acceptable provided the embankment is not overtopped; i.e some freeboard should still be maintained. The displacements of the crest would need to be assessed in this condition and compared with the available freeboard.

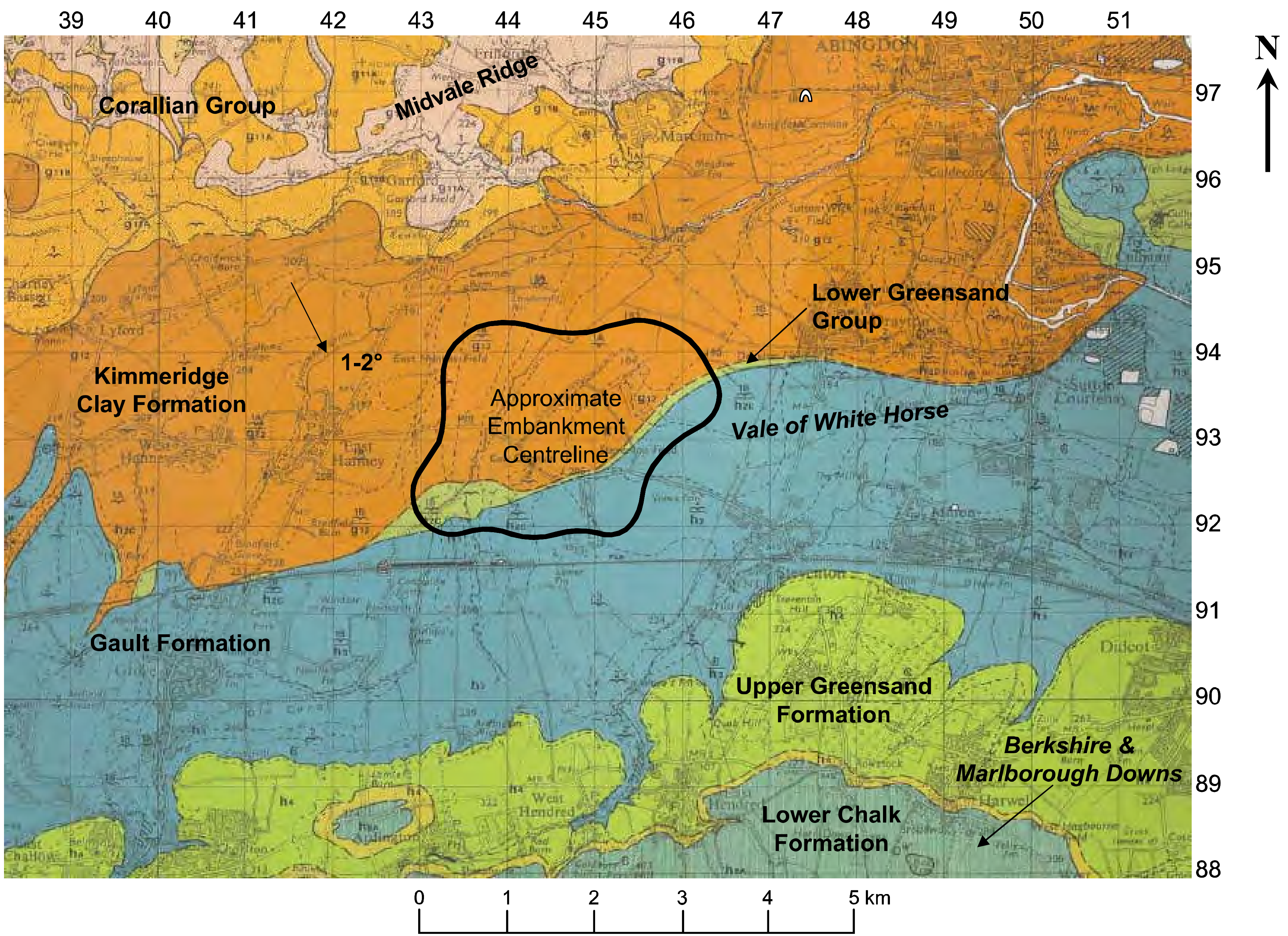
(b) Selected peak ground motion parameters

<u>OBE</u>

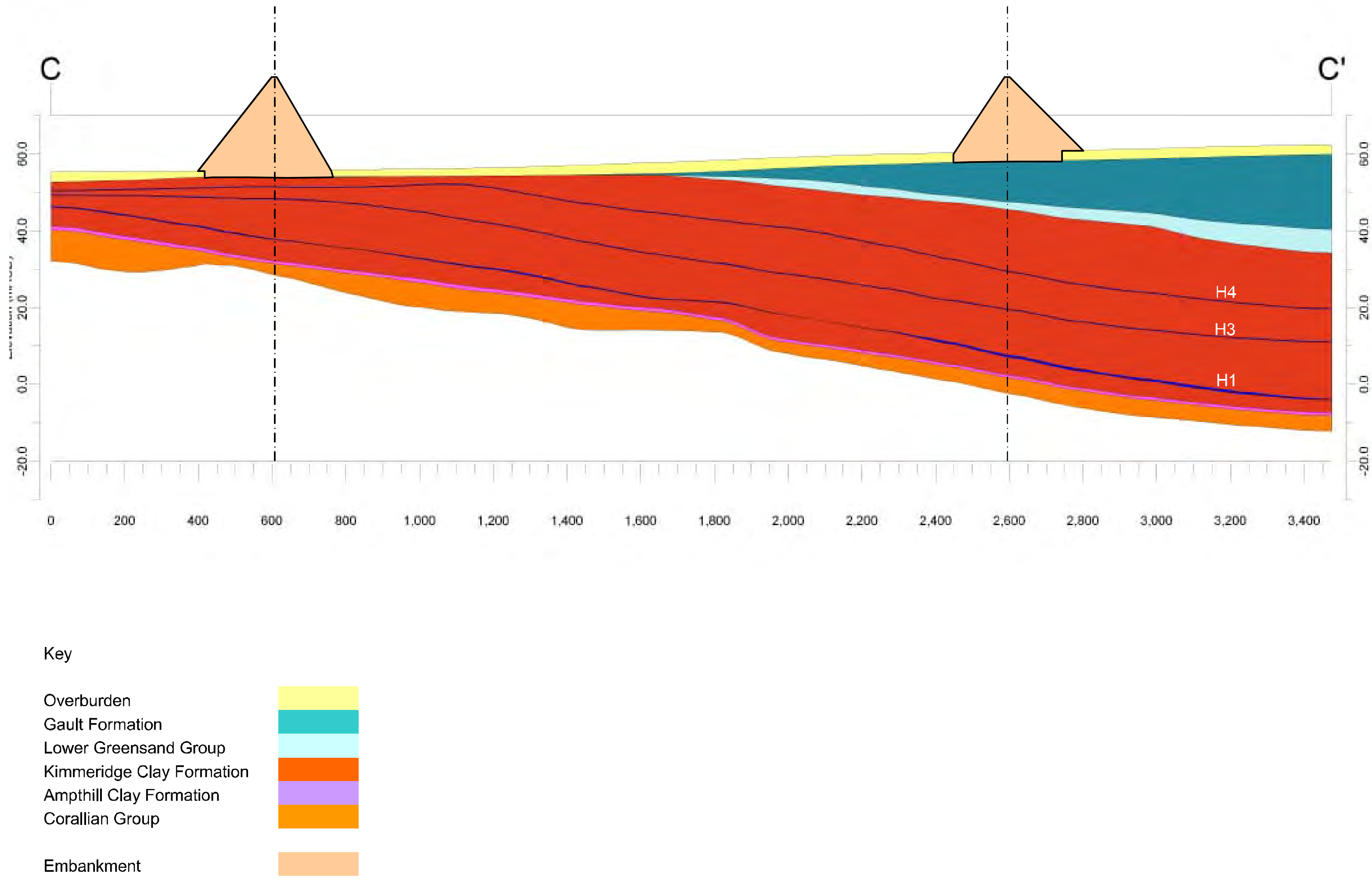
[6] Bridle, Vaughan and Jones (1985), Empingham Dam – design, construction and performance, Proceedings of the Institution of Civil Engineers, Part 1, 1985, 78, Apr., 247-289.
[7] ICOLD Bulletin 72 – Selecting Seismic Parameters for Large Dams, 1989
[8] BRE Guide to Seismic Risk to Dams, 1991
[9] An application note to an engineering guide to seismic risk to dams in the United Kingdom, DETR, 1998.
[10] Seed, H. B and F.I. Makdisi (1979) Simplified Procedure for Evaluating Embankment Response, Journal of the Geotechnical Engineering Division, Vol. 105 No GT12. December 1979

A PGA of 0.05g is recommended based on a return period of 200 years and the recommendations given in the BRE guide.

<u>SEE</u>
A PGA of 0.15g is recommended.



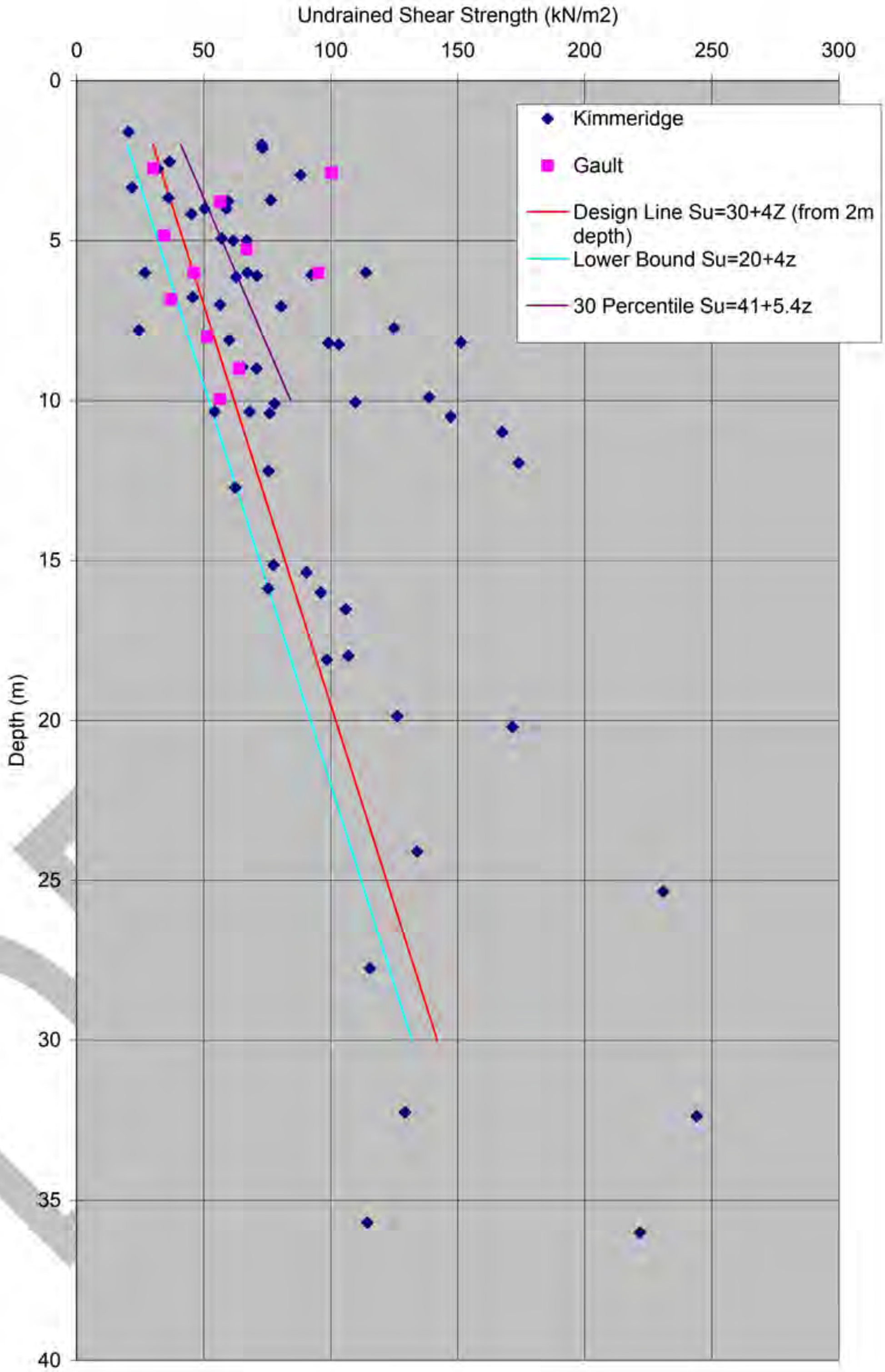
JACOBS	Upper Thames Reservoir Preliminary Design Report	**Extract from the geological map of Abingdon Sheet 253 (after IGS, 1971)**	**Figure 2.1**



Downdip geological cross-section (North West – South East)

Figure 2.2

Figure 2.3 Undrained shear strength test data and design lines



3 Embankment stability analyses

3.1 Introduction

3.1.1 Objective

The objective of this section is to define the preliminary cross section of the embankment. For the purposes of this report the cross section would be developed as far as currently appropriate, but it must be noted that the cross section cannot be finalised until after completion of a trial embankment and associated investigations, described in Section 5.11 below: thus at this preliminary stage the following have been defined:

- A "best estimate" embankment cross section based on 10 percentile foundation strengths and most probable fill strengths
- A "maximum" (flattest side slopes, maximum volume) embankment cross section based on 10 percentile foundation strengths combined with increased factors of safety
- A "minimum" (steepest side slopes, minimum volume) embankment cross section based on the 30 percentile foundation strengths

The basic design of the reservoir is based on the best estimate profile while sufficient space has been allowed to revert to the maximum profile without the need to revise the overall design.

3.1.2 Design approach

Founding the embankment on low permeability clay is ideal for control of seepage through the foundations. However, for construction of the embankment clay foundations can present problems due to very low rates of strength gain due to consolidation as fill is placed. There are three possible approaches to ensure the safe construction of the embankment:

- Install drainage in the foundations to accelerate consolidation and strength gain as fill is placed
- Construct the embankment slowly allowing natural consolidation to occur as the fill is placed
- Construct the embankment relying only on the existing strength of the foundations.

Installing drainage in the foundations, though an option, would be extremely expensive because of the large amount of granular drainage material which would be required and the high importing costs. Relying on natural consolidation of the foundations is also impractical as the low permeability and long seepage paths would result in many decades being needed for significant consolidation to occur. This means that the only practical option is to design the embankment on the basis of the in situ undrained shear strength of the foundation, making no allowance for any consolidation which may occur.

3.1.3 Methods of analysis

Stability analyses have been undertaken using two methods: Limit Equilibrium and Finite Element (FE) analyses. Limit Equilibrium analyses used the program SLOPE by GEOSOLVE. Finite element analysis was carried out by Geotechnical

Consulting Group (GCG) using the Imperial College Finite Element Program (ICFEP).

In modelling end of construction conditions, both the analyses are based on an undrained approach on the assumption that the foundations are of very low permeability and that no significant consolidation would take place during the construction period, as discussed in Section 3.1.2 above.

In limit equilibrium analysis the stability of earth slope is expressed as a factor of safety, which is the ratio of the maximum total strength available along a potential slip surface to the total strength that is actually mobilised. This ratio is calculated for a large number of different slip surfaces and the lowest ratio of these is taken as the critical factor of safety for the particular condition under examination. The results of the limit equilibrium analysis should be treated with caution as the actual behaviour of the embankment is highly dependent on the relative stiffness of the fill and foundations, and on the internal stresses in the fill, none of which are modelled in a limit equilibrium analysis. Notwithstanding this limitation, limit equilibrium is the conventional practice for analysing embankments and there is well accepted precedent for factors of safety. Whilst the FE analysis may provide a better representation of actual behaviour, it cannot on its own be used as a design tool, but should be used in parallel with limit equilibrium methods. This point is discussed further in Section 3.4.1.

3.1.4 Embankment cross section

(a) embankment zones

The basic concept for the external profile is to have a relatively steep sided central portion of the embankment supported by berms (zones of more gently sloping fill) on the inner and outer faces. The steeper central section improves the efficiency of the section and reduces the volume of fill required. The embankment cross section includes internal drainage in the form of a sub vertical chimney drain in the outer shoulder just beyond the crest (see Section 5.6).

Within the cross section the bulk cohesive fill is divided into four zones as shown in Table 3.1.

Table 3.1 Embankment Zones

Zone	Description
1	Inner shoulder and central zone
2	Outer structural shoulder
3	Random fill
4	Landscape zone

The Zone 1, inner shoulder and central zone, would be constructed entirely of structural fill excavated from within the borrow pit. This material has to be uniform and compacted to a high standard giving a strong, low permeability fill which would minimise both seepage through the embankment and settlement.

Zone 2 has similar strength requirements to Zone 1, but as it is downstream of the chimney drain there is no requirement for low permeability. Zone 2 can therefore accommodate more granular material from the overburden excavation or from the Lower Greensand excavation.

Zone 3 is random fill at the outer toe and makes up the minimum cross section of the embankment required for stability purposes. The random fill can be any material taken from either the overburden or the borrow pit excavation

Zone 4 is excess random fill placed outside the nominal outer face of the embankment to create a landscaped external profile. The profile of Zone 4 would vary continuously around the perimeter of the embankment. Zone 4 is not included in the stability analyses as it would have a neutral or beneficial effect on stability.

In practice there may be little difference between zones 2 and 3, in which case they could be combined. However at this stage of the design the distinction is retained to make the point that the stronger material should be used in zone 2.

(b) embankment profile

An iterative approach was used to develop an appropriate embankment cross section with slope angles and berm heights being varied to determine an optimal geometry.

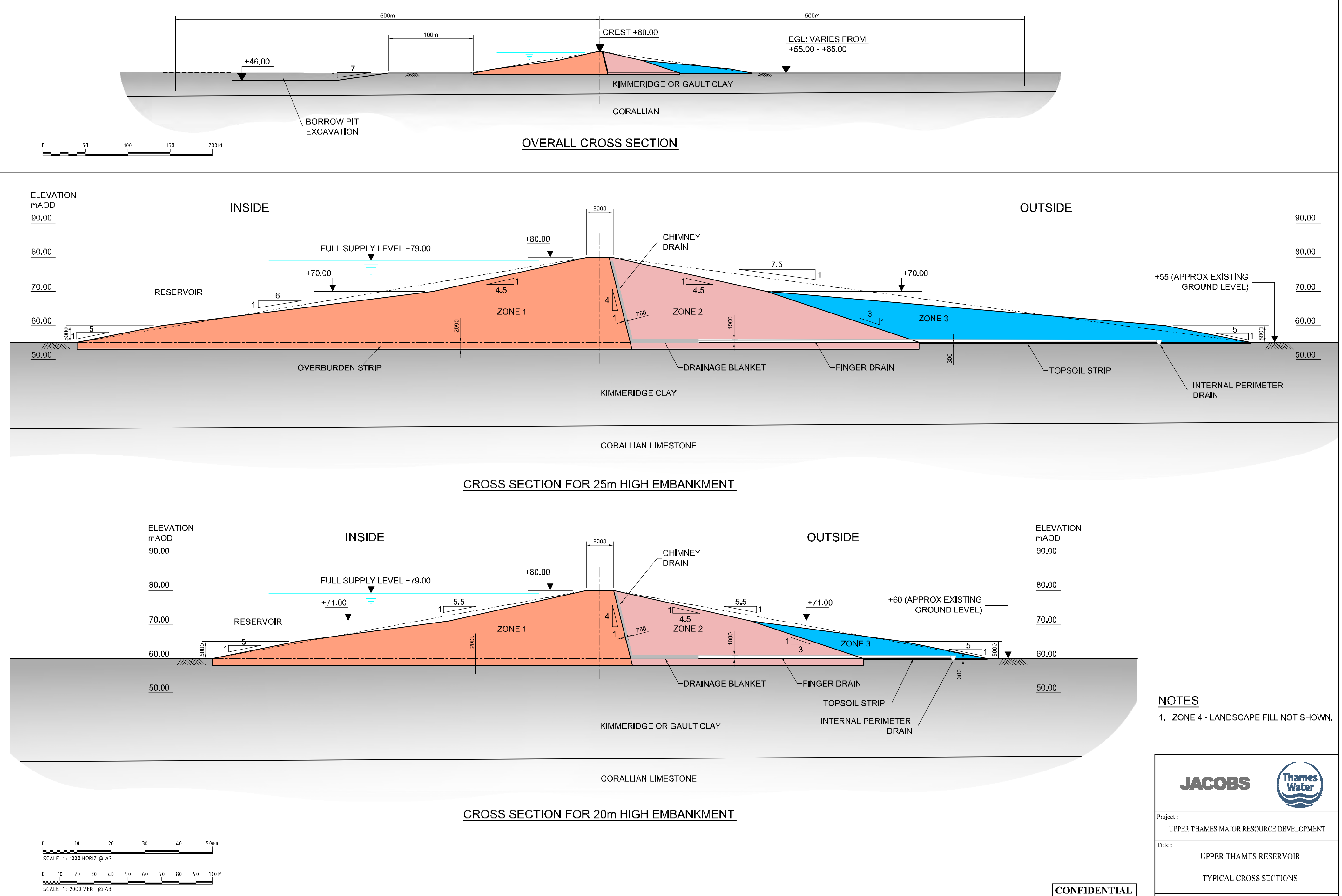
The adopted cross section is shown in Figure 3.1. The general form of the embankment is to have upper slopes at a gradient of 1:4.5 over approximately the top 40% of the embankment height, typically above 70m AOD over the northern half of the embankment and 72m AOD over the southern part. The lower slopes are gently sloping berms extending from the base of the upper slope to the embankment toe. The bottom 5m of the lower slope is steepened to 1:5 as there is no benefit in extending the gentle berm slope out to a feather edge where it would intersect ground level.

Overall slope angles for the different embankment heights are given in Table 3.2.

Table 3.2 Summary of overall slope angles (excluding landscape fill)

Embankment Height (m)	Inner Face	Outer Face
15	1:5.5	1:4.5
20	1:5.5	1:5.5
25	1:6	1:7.5

The outer face is flatter than the inner face because a higher factor of safety is applied to the outer face to safeguard stability with the reservoir full (see Section 3.2).



NOTES

1. ZONE 4 - LANDSCAPE FILL NOT SHOWN.

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Thames Water

Project : UPPER THAMES MAJOR RESOURCE DEVELOPMENT

Title : UPPER THAMES RESERVOIR

TYPICAL CROSS SECTIONS

Location :

Date : DEC. 2006

Fig. No. : FIGURE 3.1

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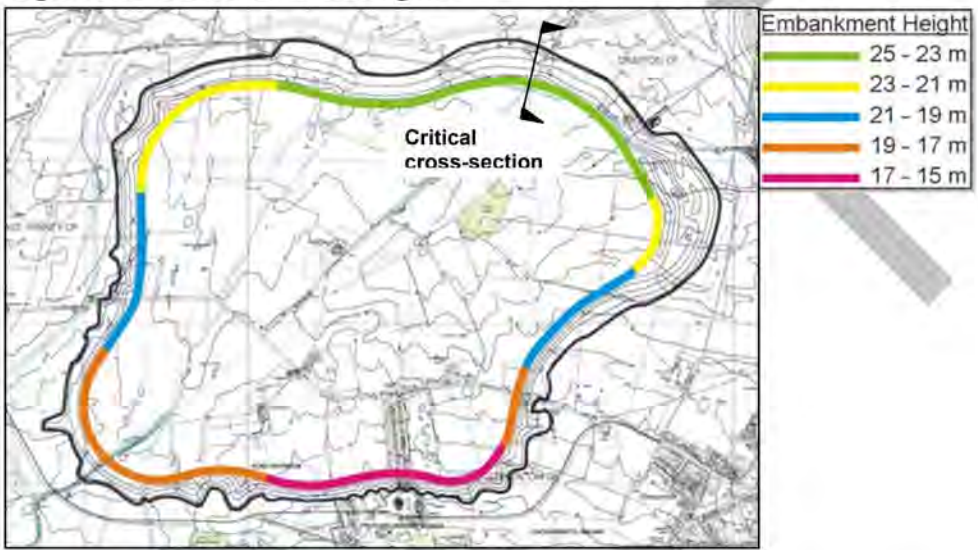
(c) embankment height

The current layout of the embankment adopts the cross section for 25m height where the actual height is between 20 and 25m, and the cross section for 20m height where the actual height is 20m or less. This would need to be refined at a later stage to optimise the cross section at intermediate heights.

Actual embankment heights are shown in Figure 3.2.

Figure 3.2 Embankment heights



The critical cross section for use in the stability analyses is the north-east corner of the reservoir where the embankment is at maximum height. The location of the critical cross section is shown on Figure 3.2.

3.1.5 Borrow pit

As shown in the overall section in Figure 3.1, the excavated slopes for the borrow pit are 1:7 or flatter and there is a 100m buffer zone between the inner toe of the embankment and the top of the borrow pit. As such the borrow pit would not affect the stability of the embankment and has not been included in this analysis.

3.1.6 Embankment foundations

Different lengths of the embankment would be founded on either the Kimmeridge Clay, the Gault Clay or the Lower Greensand as shown on Figure 2.1. Based on laboratory test data, the properties of the Gault and Kimmeridge clays are similar. The Lower Greensand need not be treated as a special case in terms of stability as the sections of foundation are limited and the Lower Greensand would be stronger than the clay strata. This means that different sections of the embankment can be modelled by varying the embankment height without need to make alteration to the foundation parameters.

The depth to the Corallian would be important in the long-term as this affects the length of the vertical drainage path beneath the embankment. In the north of the reservoir the depth to the Corallian is 25m on the embankment centreline.

3.2 Limit equilibrium analyses

3.2.1 Design cases

The design cases for the limit equilibrium analyses are set out in Table 3.4. The analyses were primarily carried out on the critical, 25m high section of the embankment but some computer runs were also carried out on 20m and 15m high embankments so as to define the external slopes around the full perimeter of the reservoir.

Table 3.4 Design cases for limit equilibrium analyses

Case	Description	Embankment Shoulder	Soil Parameters		Target FOS
			Foundation	Fill	
1	End of construction	Outer / Inner	Undrained, total stress parameters (design).	Undrained, total stress parameters.	1.30
2	End of construction	Outer / inner	Undrained, total stress parameters (lower bound).	Undrained, total stress parameters.	1.10
3	First filling (Reservoir at FSL)	Outer	Undrained, total stress parameters (design)	Undrained, total stress parameters	1.50
4	First filling (Reservoir at FSL)	Outer	Undrained, total stress parameters (lower bound)	Undrained, total stress parameters	1.30
5	Steady seepage (Reservoir at FSL)	Outer	Drained, effective stress parameters, piezometric surface from flow net.	Drained, effective stress parameters, piezometric surface from flow net.	1.50
6	Steady seepage (Reservoir at FSL)	Inner	Drained, effective stress parameters, piezometric surface from flow net.	Drained, effective stress parameters, piezometric surface from flow net.	1.50
7	Rapid drawdown	Inner	Drained, effective stress parameters, piezometric surface on inside face at rip rap bedding layer /clay interface.	Drained, effective stress parameters, piezometric surface on inside face at base of rip rap.	1.30
8	Seismic (OBE)	Outer / inner	Drained, effective stress parameters, piezometric surface from flow net as for steady seepage.	Drained, effective stress parameters, piezometric surface from flow net as for steady seepage.	1.15
9	Seismic (SSE)	ditto	Ditto	Ditto	> 1.00

3.2.2 Design parameters

Design parameters are summarised in Table 3.4 below.

Table 3.4 Design parameters used for limit equilibrium analysis

Strata	Strength			Unit Weight γ (kN/m^3)
	Effective Stress		Total Stress	
	c' (kN/m^2)	ϕ' (°)	c_u (kN/m^2)	
Structural Fill (Zones 1 and 2)	5	23	80	19.0
Landscape Fill (Zone 3)	5	23	50	18.5
Foundation <3m below foundation level (bfl)	2	23	30+4z*	19.5
Foundation > 3m bfl	5	23	30+4z*	19.5

(*where z is depth below foundation level which is at 2m below ground level)

The design total stress strength parameters are based on the 10-percentile design line, c_u = 30+4z, where z is depth below foundation level which is 2m bgl.

Lower bound total stress parameters in the foundation have been taken as c_u = 20+4z.

The profile for the 30 percentile design line is c_u = 41+5.4z.

3.2.3 Results of the analyses

Analyses were carried out for both circular and non-circular slip surfaces. The critical surfaces were generally non-circular with the slip surface running along the top of the foundations

The limit equilibrium analyses all show that the stability of the proposed cross section satisfies the design criteria.

A summary of required and achieved factors of safety is presented in Table 3.5.

Table 3.5 Summary of limit equilibrium analysis for 25m high embankment

Case	Description	Embankment Shoulder	Critical Failure Mode	Factor of Safety	Target FOS
1	End of construction	Outside	Non circular	1.51	1.30
	Design parameters	Inside	Non circular	1.34	1.30
2	End of construction	Outside	Non circular	1.27	1.10
	Lower bound parameters	Inside	Non circular	1.18	1.10
3	First filling Design parameters	Outside	Non circular	1.50	1.50
4	First filling Lower bound parameters	Outside	Non circular	1.33	1.30
5	Steady seepage	Outside	Non Circular	3.02	1.50

Case	Description	Embankment Shoulder	Critical Failure Mode	Factor of Safety	Target FOS
6	Steady seepage	Inside	Non circular	3.11	1.50
7	Rapid drawdown	Inside	Non circular	1.48	1.30
8	Seismic (OBE)	Outside	Circular	2.03	1.15
		Inside	Circular	1.86	1.15

Note. The cases with the smallest margin above target are highlighted.

3.2.4 Analysis of lower embankment heights.

Based on the results shown in Table 3.5 above for the 25m high embankment, the following four design cases used to assess the lower embankment heights:

- Case 1 End of construction – design parameters (inside shoulder only)
- Case 3 First filling – design parameters
- Case 4 First filling – lower bound parameters
- Case 7 Rapid drawdown

Embankment heights of 20m and 15m were analysed and the results are presented in Tables 3.6 and 3.7 below.

Table 3.6 Summary of limit equilibrium analysis for 20m high embankment

Case	Description	Embankment Shoulder	Critical Failure Mode	Factor of Safety	Target FOS
1	End of construction Design parameters	Inside	Circular	1.56	1.30
3	First filling Design parameters	Outside	Non circular	1.52	1.50
4	First filling Lower bound parameters	Outside	Non circular	1.32	1.30
7	Rapid drawdown	Inside	Non Circular	1.40	1.30

Table 3.7 Summary of limit equilibrium analysis for 15m high embankment

Case	Description	Embankment Shoulder	Critical Failure Mode	Factor of Safety	Target FOS
1	End of construction Design parameters	Inside	Circular	1.60	1.30
3	First filling Design parameters	Outside	Circular	1.60	1.50
4	First filling Lower bound parameters	Outside	Circular	1.43	1.30
7	Rapid drawdown	Inside	Circular	1.44	1.30

3.3 Finite Element analyses

3.3.1 Design parameters

The material parameters for the finite element modelling are much more complex than those for the limit equilibrium analyses because of the need to model the stiffness and stresses as well as the strengths of the materials. The peak strength of the foundation material assumed for FE modelling was the same as that used in the limit equilibrium modelling. However there is a difference in the strength parameters of the fill material used in the two analyses. In the limit equilibrium analyses this was a critical parameter and was taken from the results of undrained triaxial tests on samples recompacted in the laboratory. For the finite element analyses the fill strength is of secondary importance to the stiffness of the foundation because the fill strength is not fully mobilised during the construction or operation of the embankment. The modelling of the fill stiffness was based on the results of tests on samples taken from Empingham dam, which gave a higher fill strength than that used in the limit equilibrium analyses. The effect of reducing the fill strength, and stiffness, in the finite element models to make it compatible with the limit equilibrium analyses is covered later as part of the sensitivity studies.

3.3.2 Models

There are three fundamental properties which should be captured by a finite element model if it is to replicate actual soil behaviour during embankment construction, first impounding and subsequent operation:

- Non linear elastic stiffness – the elastic stiffness of soil is not constant and varies with stress state and strain level
- Strain softening – depending on the plasticity of a material, the strength decreases as it is strained beyond peak strength due to particle reorientation
- Pre-peak plasticity – actual soil behaviour is elasto-plastic, which is better replicated by a model if plasticity can be incorporated pre-peak meaning that non recoverable strains can occur before the peak strength of a material is mobilised.

Ideally, a soil model would incorporate all three of these features, but at present such a model does not exist. The procedure used here was to use two separate models which between them can capture the required behaviour. These models are:

- 'Bubble' model: kinematically hardening model of the critical state type involving pre-peak plasticity
- Mohr-Coulomb model: non-linear elastic, strain-softening plastic model

The 'bubble' model is considered to give superior prediction of deformations through its ability to model pre-peak plasticity, but it cannot model strain-softening. In situations where stress levels are such that strain softening could occur it is therefore appropriate to use the Mohr-Coulomb model.

The suitability of both models was demonstrated against the well documented behaviour observed during construction of the Empingham Dam which was also founded on a stiff plastic clay foundation.

3.3.3 Interpretation of results of FE analyses

Unlike the limit equilibrium analyses, the FE analyses do not give an overall factor of safety for the embankment stability. Instead the model gives a prediction of deformations, stresses and mobilised strength. Whilst no definitive criteria can be applied to decide if the predicted behaviour is acceptable, a key output is a plot of stress levels in the foundation. Stress level is defined as the ratio of mobilised strength to peak available strength. When the stress level reaches 1.0 strain-softening can occur with further straining which in turn could lead to progressive failure of the embankment.

3.3.4 Analysis of base case

(a) assumptions

The base case model assumes coupled consolidation of the fill and foundation materials with an effectively instantaneous construction so as to ensure undrained conditions. The fill strength (120kN/m²) and stiffness of the fill material was taken to be that of Empingham Dam.

(b) initial construction

The model analysed the initial construction of a bund at the outer toe followed by construction of the rest of the embankment in uniform layers. The model indicates large strains beneath the bund on account of it having a relatively steep inner face. This suggests that the inner face of the bund may need to be slackened to prevent the formation of shears in the foundation. As this is a temporary, internal boundary, such a change would not affect the overall design of the embankment.

(c) end of construction

The analyses showed that the embankment fill spreads during construction with the central portion settling and the outer and inner shoulders moving towards their respective toes. Stress levels in the foundations are typically less than 0.9 and there is no indication of strain softening.

(d) impounding

Filling the reservoir with water for the first time results in a redistribution of stresses within the embankment. Shear strains remain small and there is no indication of strain softening.

Following initial impounding the model ran two cycles of emptying and refilling the reservoir whilst maintaining undrained conditions. The maximum displacements after each cycle was negligible.

(e) long term

The model was also set to consolidate to long-term fully drained conditions after first impounding with the reservoir full. For this case there was a considerable disparity between the predictions made by the Bubble and Mohr-Coulomb models, with predicted settlements at foundation level of 651mm and 408mm respectively. The long-term settlements of the crest predicted by the Bubble model and Mohr-Coulomb model are approximately 400mm and 200mm respectively. The results of the Bubble model are considered more realistic. Long-term settlements of the crest are less than at foundation level because the fill is expected to swell as the suctions induced during construction are dissipated. The long term behaviour of the embankment was based on values of foundation permeability that are at the high end of the range quoted in Section 2.4.4(a) above. This will have no effect on the extent of the settlement but does tend to under estimate the length of time taken to achieve a fully drained state.

Finally, cycles of undrained drawdown and impounding were superimposed on the steady state conditions. Predicted movements were 4 to 8mm.

The above results indicate that the behaviour of the base case is satisfactory.

3.3.5 Sensitivity analyses

A number of sensitivity analyses were undertaken to check the robustness of the results of the base case analysis. Parameters which were varied were:

a) Influence of mesh fineness
b) time for construction
c) Use of fully undrained soil model for clays in the foundation
d) Variation in fill strength (and stiffness)
e) Combined undrained foundation and reduced fill strength
f) Sensitivity test (e) with higher foundation strength
g) Use of axi-symetric (rather than plane strain model)
h) Influence of initial stress conditions in the foundation
i) Variation of construction sequence

(a) Influence of mesh fineness

The use of a finer mesh in the model caused a slight reduction in the strength gain at the top of the foundation, but was found to have a negligible effect on modelled stresses and deformations during embankment construction, first impounding and subsequent operation.

(b) Time for construction

The base case modelled effectively instantaneous construction to ensure undrained conditions. In practice the construction of critical parts of the embankment could be spread over 4 years. A run was therefore carried out to assess the benefits of a longer construction period. The results suggested that the beneficial effect of the 4 year construction period compared with the base case was minimal.

(c) Use of fully undrained soil model

As stated in Section 3.1.2, the design is based on an undrained analysis. In the base case this was represented by using a coupled consolidation model with a very short time period. It was however noted that this model does generate some strength gain at the top of the foundation due to equalisation of the high pore pressures in the foundation with the suctions in the fill. To check the significance of this effect, the model was re-run assuming fully undrained conditions in the foundation with an impermeable interface between the fill and foundation. The results indicate larger shear strains and associated strain softening at the top of the foundation below the central portion of the outer shoulder and full mobilisation of shear strength of parts of the foundation.

(d) Reduction in fill strength

As stated in Section 3.3.4(a) the fill strength attributed to Empingham dam was 120kN/m^2 while in the limit equilibrium analysis of this reservoir a value of 80kN/m^2 was used. As part of the sensitivity analyses the base model was therefore rerun with the strength of the structural fill reduced to 80kN/m^2 and the stiffness also reduced by a commensurate amount. These changes result in an increase of both shear strains and stress levels.

(e) sensitivity (c) and (d) combined

The combination of the fully undrained soil model and reduced strength fill results in instability of the inner shoulder during construction. Large movements occur with the fill level 1m below crest level and strain softening occurs along the top of the foundation beneath the inner shoulder. The mechanism of progressive failure is in operation.

(f) Sensitivity test (e) with higher foundation strength

In reality it is considered that the combination of scenarios in case (e) is unduly pessimistic. The sensitivity of this arrangement to foundation strength has therefore been assessed in two ways as follows:

- Assuming a higher foundation strength; equivalent to the 30 percentile line of the undrained strength data
- Lowering the formation level by further 2.5m to give a higher undrained shear strength at the top of the (remaining) foundation.

In both cases the behaviour was satisfactory at the end of construction, but the undrained strength was fully mobilised at the top of the foundation after initial impounding. Although it was not analysed, it can be inferred that the foundation strength would not have been fully mobilised if the base case fill strength and stiffness had been used.

(g) Axi-symetric model

An axi-symetric model to check the possible effect of the curvature in plan of the embankment on the stability of the outer face was developed assuming a radius to the centre of the crest of 400m, representing the tightest curve on the embankment alignment. The effect on the base case of switching from plane strain to an axi-symetric model was found to be negligible both during construction and first impounding in the context of the embankment curvature.

Further analyses are required to examine the possible effect of the borrow pit on the embankment using the axi-symetrical model will be carried out during the detailed design stage.

(h) Influence of initial stress conditions in the foundation

A run was undertaken to study the effect of varying the ratio of horizontal and vertical stresses in the foundation, represented by the factor K_o. The effect of varying K_o appeared negligible, but this may have been a result of maintaining the same undrained shear strength profile.

(i) Staged construction

An analysis was undertaken to assess whether there could be any benefit in constructing the outer parts of the embankment shoulders in advance of the central part of the embankment. The model indicated no significant benefits, and could in fact introduce more non-uniform deformations in the foundation which could instigate the mechanism of progressive failure.

3.3.6 Conclusions on finite element analysis

The finite element analyses demonstrate satisfactory behaviour for the base case with coupled consolidation in the foundations and a fill strength derived from the back analysis of Empingham dam.

The sensitivity analyses show that stress levels and strains in the foundation are sensitive to foundation and fill shear strength and drainage assumptions but insensitive to the rate of construction and foundation stress state.

On balance it is considered that the base case analysis is based on reasonably conservative/realistic assumptions as follows:

- Foundation strengths are taken to be on the 10 percentile line, as discussed in Section 2.5.1(b) above, which is a conservative interpretation of the test data
- Fill strength is taken at 120kN/m^2, which is based on Empingham data and is considered to be realistic.
- The use of coupled consolidation of the foundation is realistic, but the time period for construction is conservative

It is therefore felt that the best estimate embankment section, which was used in the base case analysis, is justified for the present stage of the project. If the trial embankment shows that the strength of compacted fill is significantly less than 120kN/m^2, then the slopes of the best estimate section may have to be flattened towards the "maximum" embankment section, unless the trials indicate that greater strengths can be confidently ascribed to the foundation. If the trial indicates that the designers can be confident of both a fill strength of 120kN/m^2 <u>and</u> foundation strengths represented by the 30 percentile line, then it might be possible to steepen the slopes towards the "minimum" cross section.

3.4 Development of maximum and minimum embankment sections

3.4.1 Comparison of Limit Equilibrium and Finite Element Analyses

The preliminary embankment profile has been developed using limit equilibrium analyses based on the 10 percentile design line for foundation strengths and the target factors of safety summarised in Table 3.4 above. The use of the same input parameters in the finite element model indicates overstress in the foundations. Thus there appears to be a basic incongruity between the results of the limit equilibrium and finite element models. This is resolved by "calibrating" the limit equilibrium factor of safety against acceptable stress levels derived from the finite element analyses.

3.4.2 Maximum embankment section

The finite element analyses, with fully undrained conditions and weak fill, have shown that the behaviour is satisfactory if the foundation strength is increased to the 30 percentile line. To explore the implications of this in terms of limit equilibrium analysis, the limit equilibrium model was rerun for the higher foundation strength. This increased the factors of safety are as follows:

Inner face	1.59
Outer face	1.79

Based on this result it can then be postulated that the target factors of safety of the inner face at the end of construction and the outer face after initial impounding should be increased to 1.6 and 1.8 respectively.

Rerunning the limit equilibrium model using the original 10 percentile design line for these factors of safety gives overall inner and outer slopes for the 25m high embankment of 1:8.5 and 1:10 respectively. These can be seen as the "maximum" embankment cross section.

Adoption of the maximum slopes would have the following implications:

- An approximate 30% increase of Zone 1 and 2 material
- The inner face would encroach into the reservoir area resulting in a loss in storage equal to the increase in the volume of inner face fill. Additional fill would be required from the borrow pit excavation and additional slope protection material would be required. The edge of the borrow pit would be moved to maintain the 100m buffer
- The outer face would need to be flattened in areas where the current profile is too steep. It is likely that this could be achieved by re-re-profiling the existing volume of landscape fill such that the extra fill would be taken from the existing areas where the slopes are significantly flatter than needed.

As an alternative to adopting the maximum profile, the formation level could be deepened by 2m. This would result in a greater quantity of additional earthworks, but would allow the slope profiles to remain unchanged.

3.4.3 Minimum embankment section

The slope angles as currently defined are based on the 10 percentile strength profile for the foundations. There is a possibility that the trial embankment would show this to have been an overly conservative estimate of the foundation strength.

An assessment of minimum slope angles can be based on increasing the strengths to the 30 percentile foundation strength line with a factor of safety of 1.5. Adopting this profile in the limit equilibrium analysis gives inner and outer slopes for the 25m high embankment of 1:4.5 and 1:6 respectively. This could result in a 20% saving of Zone 1 and 2 material.

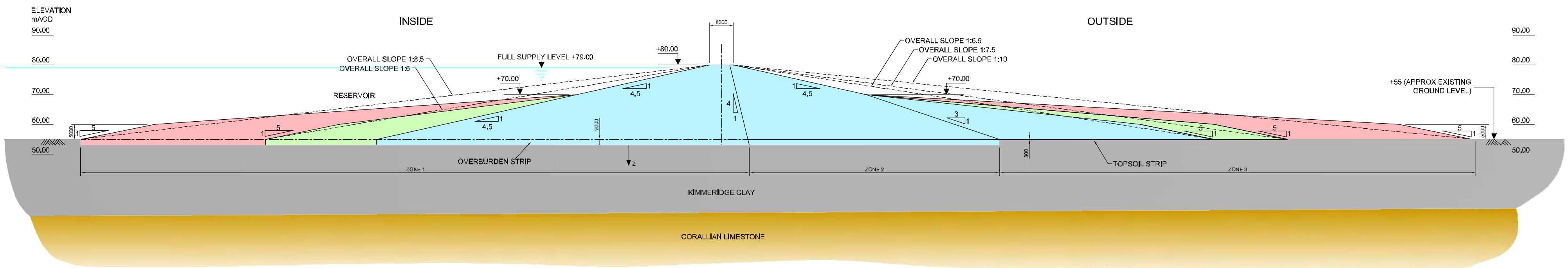
3.5 Conclusion

The stability analyses, both limit equilibrium and finite element, have demonstrated that the embankment cross section, as defined, is acceptable. However there remains an element of uncertainty over the input parameters. This cannot be resolved until after construction of the trial embankment. Best estimate, maximum and minimum slope profiles have been derived as summarised in Table 3.8

Table 3.8 Best estimate, maximum and minimum slope profiles

Slope Profile	Slope angle	
	Inner Face	Outer Face
Best Estimate	1:6	1:7.5
Maximum	1:8.5	1:10
Minimum	1:4.5	1:6.5

These slopes are shown schematically in Figure 3.3.



CROSS SECTION FOR 25m HIGH EMBANKMENT

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SCALE 1:1000 HORIZ @ A3

SCALE 1:2000 VERT @ A3

JACOBS

Thames Water

Project : UPPER THAMES MAJOR RESOURCE DEVELOPMENT

Title : UPPER THAMES RESERVOIR
BEST ESTIMATE, MINIMUM AND MAXIMUM SLOPE PROFILES

Location :

Date : DEC. 2006

Fig. No. : FIGURE 3.3

5 Embankment design

5.1 Introduction

This chapter builds upon the results of the embankment stability analyses (Section 3) and the inner face design (Section 4) to consider all the other elements of design and to incorporate these into a coherent whole.

5.2 Seepage analysis

5.2.1 Introduction

The modelling of seepage is primarily part of the hydrogeological studies being carried out independently by ESI under the supervision of Cascade. However, for completeness a simplified calculation of seepage is included in this report.

Seepage from the reservoir may express itself in two ways as follows:

- Vertical seepage from the floor of the reservoir to the Corallian
- Lateral seepage beneath the embankment

5.2.2 Vertical seepage to the Corallian

Vertical seepage to the Corallian would be governed by

- the head of water in the reservoir
- the surface area of the floor of the reservoir
- the thickness of clay above the Corallian and
- the permeability of the clay strata.

Given that permeability is a logarithmic function (i.e. it can vary over many orders of magnitude), fairly broad assumptions may be made to assess the likely order of magnitude of seepage. The following assumptions can therefore be made:

- 20m head of water in the reservoir
- 6km² floor area
- 20m of clay above the Corallian
- Permeability of the clay in the range 10^{-12} to 10^{-9}m/s

On this basis the predicted seepage is in the range 0.006 to 6 l/s (500l/d – 0.5Ml/d). This is trivial in the context of the operation of the reservoir, but at the upper limit there may be a significant effect on the regional hydrogeology. This is being assessed by ESI.

5.2.3 Lateral seepage beneath the embankment

Lateral seepage beneath the embankment would be governed by the Lower Greensand outcrop as this is several orders of magnitude more permeable than the clay strata. The adopted permeability of the Lower Greensand is 5×10^{-6}m/s. The seepage path beneath the embankment is of the order of 100m with a mean head difference of about 20m. The average thickness of the Lower Greensand is about 3m. Considering seepage through a 100m "width" of Lower Greensand on either side of the reservoir, the predicted seepage is 0.6l/s which is trivial. Notwithstanding this assessment, provision would be made to blanket the exposed face of the Lower

Greensand with clay in case there happen to be bands of higher permeability material, and also to minimise the possibility of piping failure.

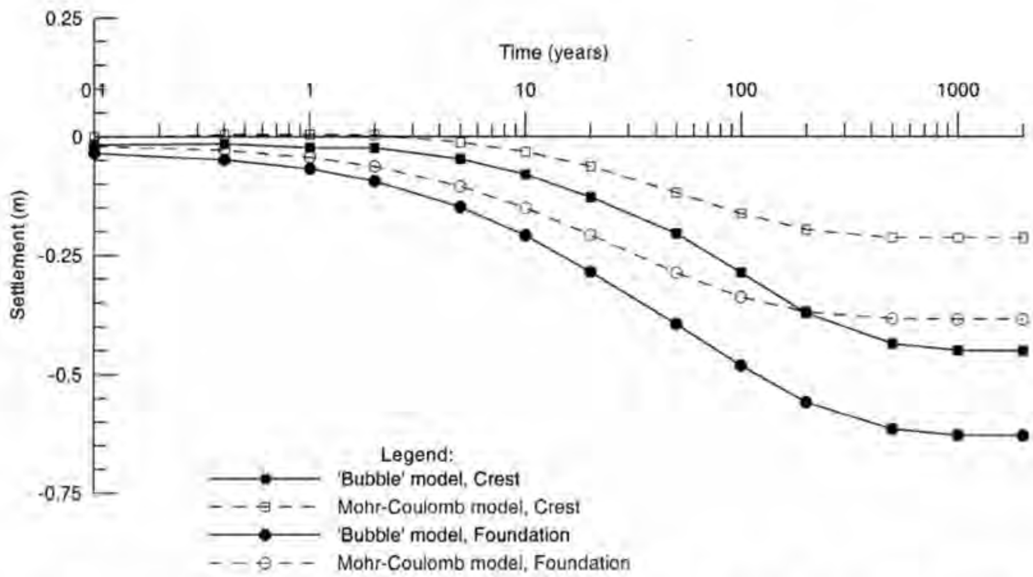
Sealing the Lower Greensand in this manner would also be beneficial in preventing the development of uplift pressures in the Lower Greensand where it is present beneath the outer toe of the embankment along the south side of the reservoir.

5.3 Settlement analysis

An analysis of the settlement of the embankment has been undertaken as part of the finite element modelling (see section 3.3.4(d) above).

Plots of settlement at the crest and foundation level against time for both the Bubble and Mohr Coulomb models are shown in Figure 5.1.

Figure 5.1 Embankment settlement



After 100 years the predicted crest settlement of the 25m high embankment is approximately 0.25m. Given the uncertainties in the prediction of settlement, it is considered pragmatic to adopt the 100 year settlement allowance for preliminary design. A crest settlement allowance of 0.25m should therefore be incorporated in the preliminary design of the crest, as discussed in Section 4.5.3 above.

5.4 Outer face

5.4.1 Profile

As described in Section 3 a basic profile for the outer face has been developed from the stability analyses. In reality the cross section is continuously varied around the perimeter of the embankment to take account of landscaping requirements. Contours on the outer face are shown in Figure 1.2. These contours respect the requirements for the cross section and in many parts of the embankment represent a considerable flattening of the outer face compared with the design requirements.

As such the stability requirements of the embankment are satisfied at all points around the reservoir perimeter. Typical sections through the embankment are shown in Figure 5.2

5.4.2 Surface drainage

The outer face would have a surface area of approximately 238ha and a system of surface drains would be required to convey surface run-off to the perimeter drains, and thence to the stream diversions, in order to prevent surface erosion. The surface drainage provision has not yet been designed as it would seem beneficial to align, where possible and appropriate, surface drains with access tracks and hedgerows, which have not yet been detailed.

5.5 Foundation treatment

5.5.1 Gault Clay and Kimmeridge Clay

No special treatment is required for the sections of the embankment founded on the Gault Clay or Kimmeridge Clay. The overburden would be stripped and the embankment founded directly on the solid strata.

The key requirements for the founding strata are that they should have adequate strength to support the embankment and be sufficiently impermeable to limit seepage from the reservoir. The foundation would be tested as construction proceeds and any localised soft areas that do not meet the adopted criterion would be excavated out as necessary. It is envisaged that the foundation strength would be proved by cone penetration testing or equivalent calibrated against the existing triaxial data.

5.5.2 Lower Greensand

The Lower Greensand would form the founding strata for the embankment where it crosses the outcrop in the south-west and east.

The Lower Greensand is relatively more permeable than the Kimmeridge Clay and Gault Clay and would rapidly consolidate and gain strength as it is loaded during construction of the embankment. The Lower Greensand would therefore be stronger than the adjacent Kimmeridge Clay or Gault Clay and would not necessitate any modification to the embankment cross-sections adopted for the clay foundations.

The Lower Greensand would, however, provide a potential seepage path from the reservoir to the outer toe. To limit seepage it would be necessary to place a clay seal over the exposed face of the Lower Greensand.

The most appropriate solution is to excavate the Lower Greensand in these areas and backfill the excavation with compacted clay fill. The arrangement would be complicated by the dip on the Lower Greensand which means that the strata becomes deeper to the south-east. This means that beyond a certain point there is no option but to leave the Lower Greensand in the foundation accepting that it would be overlain by relatively impermeable Gault Clay. An appropriate cut-off point is to say that the Lower Greensand should be excavated out where it comes within 2m of formation level i.e. excavated to a maximum depth of 4m. Laterally the Lower

Greensand should be excavated from the edge of the borrow pit excavation to the inner edge of the random fill zone beneath the outer shoulder

5.5.3 Lower Greensand outcrop in reservoir

As described above, the Lower Greensand would be excavated and backfilled with clay up to the edge of the borrow pit. Within the borrow pit the Lower Greensand would outcrop in the exposed face. In this area the Lower Greensand cannot form a potential seepage path normal to the side of the borrow pit as the strata dip to the south-east and does not therefore outcrop to the south of the reservoir. There is however a potential lateral seepage and for the water pressure in the reservoir to be transmitted beneath the embankment causing uplift at the outer toe. For this reason the Lower Greensand must be sealed off from the reservoir within the borrow pit. This can most easily be achieved by excavating the exposed face of the Lower Greensand and plugging it with compacted clay.

5.6 Internal filters and drainage

5.6.1 Chimney drain

In long term drained conditions the chimney drain within the embankment would intercept seepage from the reservoir through the inner shoulder and convey it to the outer toe in a controlled manner, so as to ensure that the downstream shoulder is drained. The chimney drain provides protection against internal erosion by preventing migration of clay particles through cracks or fissures which might develop in the long term, for instance through differential settlement, earthquake or desiccation of upper layers of the fill, if the reservoir is emptied for a prolonged period. It would also provide a filter separating the compacted clay in zone 1 from the more heterogeneous and possibly coarser grained zone 2 material which would provide more flexibility in the embankment construction methodology.

5.6.2 Horizontal drain

Provided that the capacity of the horizontal drains are equal to that of the chimney drain there is no drainage-related requirement for a continuous blanket drain linking the base of the chimney drain to the downstream toe. However a secondary function of the blanket drain is to act as a filter for seepage exiting the foundations downstream of the chimney drain and it is therefore considered prudent to place a blanket drain over a width of around 20m immediately downstream of the base of the chimney drain. The drainage blanket should be constructed at or above existing ground level such that water can flow by gravity to the outer toe. This would mean that the blanket is paced on top of compacted fill.

The grading of the chimney drain and blanket has been defined in accordance with current practice[17]. The proposed grading is shown in Figure 5.3.

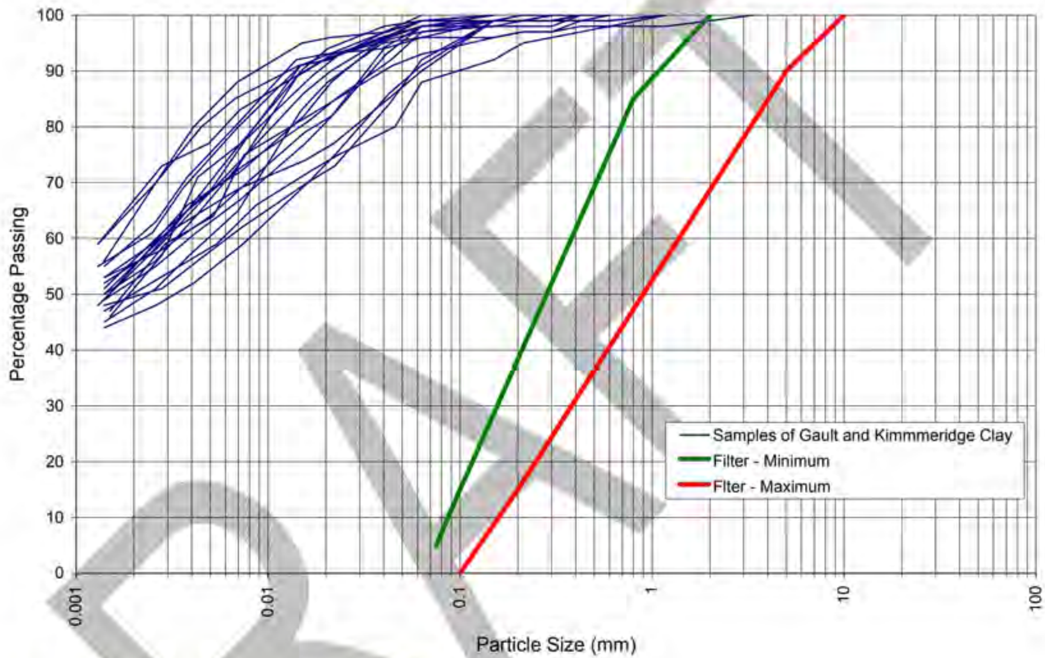
Beyond the downstream edge of the blanket drain discrete finger drains can be provided to convey the seepage to the embankment toe. These can comprise trenches cut in the fill or foundation, lined with geotextile, and backfilled with clean gravel. Typically they should be spaced at about 10m centres. At their downstream end the finger drains should be joined by a perimeter drain within the embankment which can then discharge into an external toe drain by way of a 'V' notch weir at discrete points. The internal perimeter drain should be constructed as for the finger

[17] Soil Mechanics Note 1, Guide for Determining the Gradation of Sand and Gravel Filters, Water Resources Publications, National Engineering Publications, US Soil Conservation Service, 210-v1-smn-1, January 1986.

drains, but incorporating a perforated pipe to enhance its carrying capacity. Exits to the toe drain should be at intervals of around 500m giving two runs of 250m in each section of the internal perimeter drain.

The total volume of granular material required for the internal drainage is approximately 380,000m^3 of which 315,000m^3 is filter sand and 65,000m^3 is drainage gravel.

Figure 5.3 Grading curves for internal drains



5.7 Access tracks and paths

An access track would be provided along the full length of the outer toe of the embankment. This would be connected to the embankment crest by four ramps located one at each corner of the reservoir. In addition a number of footpaths would be provided which would be used for access to the outer face of the embankment for inspection purposes. The disposition of these footpaths has yet to be detailed.

5.8 Monitoring and instrumentation

The primary purposes of instrumentation are as follows:

- Monitoring of deformations, pore pressures and seepages during construction, first filling and the initial years of operation to compare actual and predicted behaviour
- Long term monitoring of seepage and settlement

Generally the instrumentation required for both phases is similar although the intensity of monitoring would be decreased for the long term monitoring.

5.8.1 Monitoring during construction

During construction of the embankment the key observations which are required are:

- Measurement of pore pressures in the embankment and its foundations
- Horizontal deformations within the fill
- Measurements of settlements at formation level and within the fill

The instruments would be installed at selected cross sections around the perimeter of the reservoir. Given that the total length of the embankment is 10km it is considered that 8 instrumented cross sections should be provided. These need to be positioned at regular intervals around perimeter such that four cross sections are installed in the first year of construction, to provide as much data as possible at the start of the project, with the others following on in later years.

5.8.2 Monitoring during first filling and initial operation

During first filling and initial impounding the construction phase instruments described above would continue to be monitored. Where inclinometers are installed on the inside face they would be abandoned as they become submerged. Seepage and movements would also be monitored.

(a) Seepage monitoring

As described in Section 5.6.2 the internal drainage in the embankment would be compartmentalised and discharged to an external toe drain at approximately 500m intervals via 'V' notch weirs. These 'V' notches enable any seepage from the embankment to be monitored both manually and automatically by ultra-sonic level monitoring equipment.

Although the system of compartmentalised drains and 'V' notch weirs should allow all seepage through the embankment to be monitored it would be rather imprecise in locating the actual source of any seepage. This is because it is impractical to consider compartmentalising the drainage into sections shorter than 500m. This limitation can however be overcome by the further use of fibre optic sensors, in which the fibre optics are used for very accurate temperature measurement. It has been established that concentrations of seepage cause a variation in the normal, cyclic temperature fluctuation within an embankment. A fibre optic sensor can therefore locate a zone of seepage to within about 1m. The most appropriate location for a fibre optic cable to pick up embankment seepage is in the base of the chimney drain. A second loop could also be installed at mid height in the chimney drain to provide information on the elevation where any seepage is entering the drain. To assess seepage through the foundation it is also proposed to install a third loop in the base of the internal perimeter drain.

(b) Measurement of surface movements

Once the embankment has been completed survey monuments can be installed on the crest and on the inner and outer faces. These can be positioned using conventional surveying techniques to record the external deformation of the embankment.

5.8.3 Long-term monitoring

Once the satisfactory operation of the dam has been confirmed the level of monitoring may be reduced.

The most important monitoring would be that of seepage as it is seepage which is the most obvious manifestation of internal erosion within the embankment. Thus the 'V' notch weirs should be read weekly to identify any changes in seepage flows. For the fibre optic loops a lower frequency of probably once per month would be appropriate to avoid data overload.

5.9 Borrow pit shape

The shape of the borrow pit is governed by the following factors:

- Provision of sufficient fill to construct the embankment
- Maintaining the stability of the embankment
- Maintaining the stability of the borrow pit excavation
- Provision of a minimum depth of water at minimum drawdown level to satisfy water quality requirements
- Maintaining sufficient thickness of clay in the base of the excavation to prevent heave in the base of the pit and to limit seepage into the underlying aquifer

In practice there is more than sufficient material available in the borrow pit to provide the required volume of fill so there is no difficulty in leaving sufficient clay on the floor of the borrow pit to prevent heave and limit seepage.

The borrow pit is currently shaped to satisfy the requirements for water quality. This necessitates the borrow pit to be a 'v' shape with its axis running in a north-east / south-west direction, aligned with the strike of the underlying strata. The base of the borrow pit is at an elevation of 46m AOD, which provides a maximum water depth at the bottom operating level (BOL) of 5m. A nominal 100m wide buffer is left between the inner toe of the embankment and the top of the excavated slope. Excavated slopes are set at 1:7 or flatter which would be stable under normal and drawdown conditions. Borrow pit slopes above BOL (51m AOD) which may occasionally be exposed to wave attack could be protected as discussed in Section 4.4.5.

The borrow pit floor would be stable against heave provided the depth of excavation in less than half the total depth to the Corallian. At the deepest point in the borrow pit, the depth of excavation below ground level is around 15m while the depth to the Corallian is approximately 40m. The borrow pit is therefore 5m shallower than its theoretical maximum depth and would be stable against heave.

The layout and a cross section of the borrow pit are shown in Figure 5.4.

5.10 Material balance

5.10.1 Introduction

An assessment of the material balance must take account of the different types of material which would be produced in the excavations and their suitability for inclusion in the different fill zones within the embankment and external works. The materials may be summarised as Table 5.1.

Table 5.1 Materials produced by excavations and their suitability as fill

Excavated Material	Suitable fill zones
Topsoil strip	Topsoil on outer face of embankment and landscape bunds, Flood compensation areas, excavated slopes
Overburden strip	Possibly outer structural shoulder (Zone 2) General landscape fill (Zones 3 & 4)
Bulk excavation in borrow pit	Inner structural shoulder (Zone1) Outer structural shoulder (Zone 2) General landscape fill (Zones 3 & 4)
Excavation of Lower Greensand in borrow pit	Outer structural shoulder (Zone 2) General landscape fill (Zones 3 & 4)
Unsuitable material from borrow pit	General landscape fill (Zones 3 & 4)

Overburden material would be stripped beneath Zones 1 & 2 of the embankment and from the surface of the borrow pit. Based on the results of the site investigation, an average strip of 2.0m has been assumed.

No allowance would be made for bulking of excavated material as it is assumed that fill would be recompacted to close to its insitu density. This affords a conservative estimate of fill requirements as it is likely that some bulking would occur in reality.

5.10.2 Excavation

The quantity of various material types that would be excavated are presented in Table 5.2.

Table 5.2 Excavation quantities (Mm^3)

Material Source	Material Type				
	Topsoil	Overburden	Greensand	Clay	Total
Borrow pit	1.4	7.9	2.0	26.6	37.9
Embankment foundations	0.6	3.4	0		4.0
Flood compensation	0.1	0.4	0		0.5
Auxiliary drawdown	0.1	0.2		0.1	0.4
Total	2.2	11.9	2.0	26.7	42.8

Note. Quantities of excavation from the tunnel, SWOX pipeline etc represent less than 0.5% of the total and have not been included in the above table.

5.10.3 Fill

The quantity of various fill types required for the permanent works, not including imported material, are summarised in Table 5.3.

Table 5.3 Fill quantities (Mm^3)

Material source	Material Type				
	Topsoil	Overburden	Greensand	Clay	Total
Embankment zones 1 & 2	0	1.4[1]	2.0[1]	20.4	23.8
Embankment zones 3 & 4	0.5	10.3		4.0 [2]	14.8
Flood compensation	0.1				0.1
Roads	TBA				
Auxiliary drawdown				0 [3]	0.1
Total	0.6	11.7	2.0	24.4	38.8

1 Overburden and Greensand used in Zone 2 only
2 4M.m^3 of unsuitable (i.e. non structural fill) clay excavated from the borrow pit, estimated as 15% of excavated volume of clay.
3 The volume of clay required for bunds alongside the canal for auxiliary drawdown is negligible (approx 0.01Mm^3)

5.10.4 Balance

(a) Topsoil

There would be an excess of 1.6Mm³ topsoil which can either be exported from site if there is a demand at the time of excavation or it can be absorbed by increasing topsoil thickness on the flatter embankment slopes and in the flood compensation areas.

(b) Overburden

The overburden is largely used as Zone 3/4, with the balance, 1.4Mm^3 being used in Zone 2. The material used in Zone 2 is to be granular terrace deposits.

(c) Greensand

It is expected that all the greensand would be used as Zone 2 fill in the embankment.

(d) Clay

With the borrow pit geometry currently envisaged, the volume of clay that would be excavated is estimated a 26.7M.m^3. It is assumed that of this 15% or 4M.m^3 could be rejected as unsuitable primarily because of moisture content outside specified limits.

Of the 22.7Mm^3 of suitable material, 20.4Mm^3 would be placed as zones 1 & 2.

(e) Summary

Tables 5.2 and 5.3 indicate that with the current arrangement there is a very close balance in the overburden /unsuitable material, but that there is an excess of 2.3Mm^3 in excavation of structural fill from the borrow pit. This is due to the borrow pit dimensions being determined to suit the water quality requirements.

Table 5.4 Summary of excavation and fill quantities

Totals	Material Type				
	Topsoil	Overburden	Greensand	Clay	Total
Excavation	2.2	11.9	2.0	26.7	42.8
Fill	0.6	11.7	2.0	24.4	38.7
Excess / Deficit	1.6	0.2	0	2.3	4.1

For preliminary deign purposes this surplus in fill is not considered excessive and may possibly be mitigated by refining the shape of the borrow pit as the design progresses. However the extent of any such refinement may be limited by its possible adverse effect on water quality.

5.11 Trial embankment

5.11.1 Introduction

A trial embankment is considered necessary to validate the geotechnical design parameters in advance of construction of the main embankment. It is anticipated that the trial embankment would be constructed as part of the enabling works. The trial would be undertaken in the north of the site in the general area where the main embankment would be at its highest and where the foundations of the main embankment would not be comprised. The most appropriate location would therefore be within the area of the borrow pit at the north of the site.

It is likely that a supplementary site investigation would also be required at this time.

5.11.2 Design philosophy

The trial embankment would be designed at a scale that would reasonably replicate the behaviour of the main embankment. Side slopes would be asymmetric and designed such that the steeper side should fail towards the end of construction. The length of the embankment would be such that plane strain conditions would prevail and the stability would not be influenced by three dimensional effects.

The construction of the embankment would require a borrow pit to be dug in the immediate vicinity to provide fill. The borrow pit could be placed immediately adjacent to the embankment to effectively increase its height and increase the load on the foundations. While this would be considered at the design stage it is currently thought that this would not be desirable as it may invoke a different failure mechanism from that predicted for the main embankment. The borrow pit would therefore be kept remote from the trial embankment such that it does not compromise the stability.

The behaviour of the fill would be a key part of the trial. It would be important that the construction of the embankment is homogeneous and it would therefore be inadvisable to undertake compaction trials on the embankment itself. It is therefore proposed that compaction trials should be undertaken prior to construction of the embankment. These would establish:

- Type of compaction plant to be used
- Layer thickness
- Techniques for conditioning the fill

- Acceptable moisture content for the fill

5.11.3 Objectives

The objectives of the trial embankment are as follows:

- Determine geotechnical properties for clay fill compacted under field conditions
- Monitor pore pressures and deformations in the embankment and foundation during construction
- Confirm soil parameters being used in the finite element model through back analysis of the embankment behaviour.
- confirm the suitability of compaction plant

5.11.4 Preliminary design

Preliminary design of the trial embankment has been undertaken using limit equilibrium analysis. Given that the maximum height of the main embankment is 25m it is considered that the trial should be built to at least 20m, representing 80% of the completed height. The steep side of the embankment should have a factor of safety of 1.0; this requires a slope of 1:2.5. On the flat side of the embankment it is considered appropriate to slacken the slope to 1:4 giving a factor of safety of about 1.25. The crest width should be 5m.

The length of the full height section of the embankment should be at about 5 times the distance of the crest to the toe on the steep side to create plane strain conditions. This necessitates the embankment to be about 250m long. Ramps at a gradient of 1:5 would be provided at each end to give access for construction plant.

The plan area of the embankment is 50,000m² and the volume of material incorporated would be approximately 400,000m³. This volume is less than 10% of the difference between the structural fill volume associated with the "maximum" and "most likely" assumptions in Sections 3.4.2 and 3.4.3 above which justifies, in risk terms, the construction of the trial embankment.

5.11.5 Instrumentation

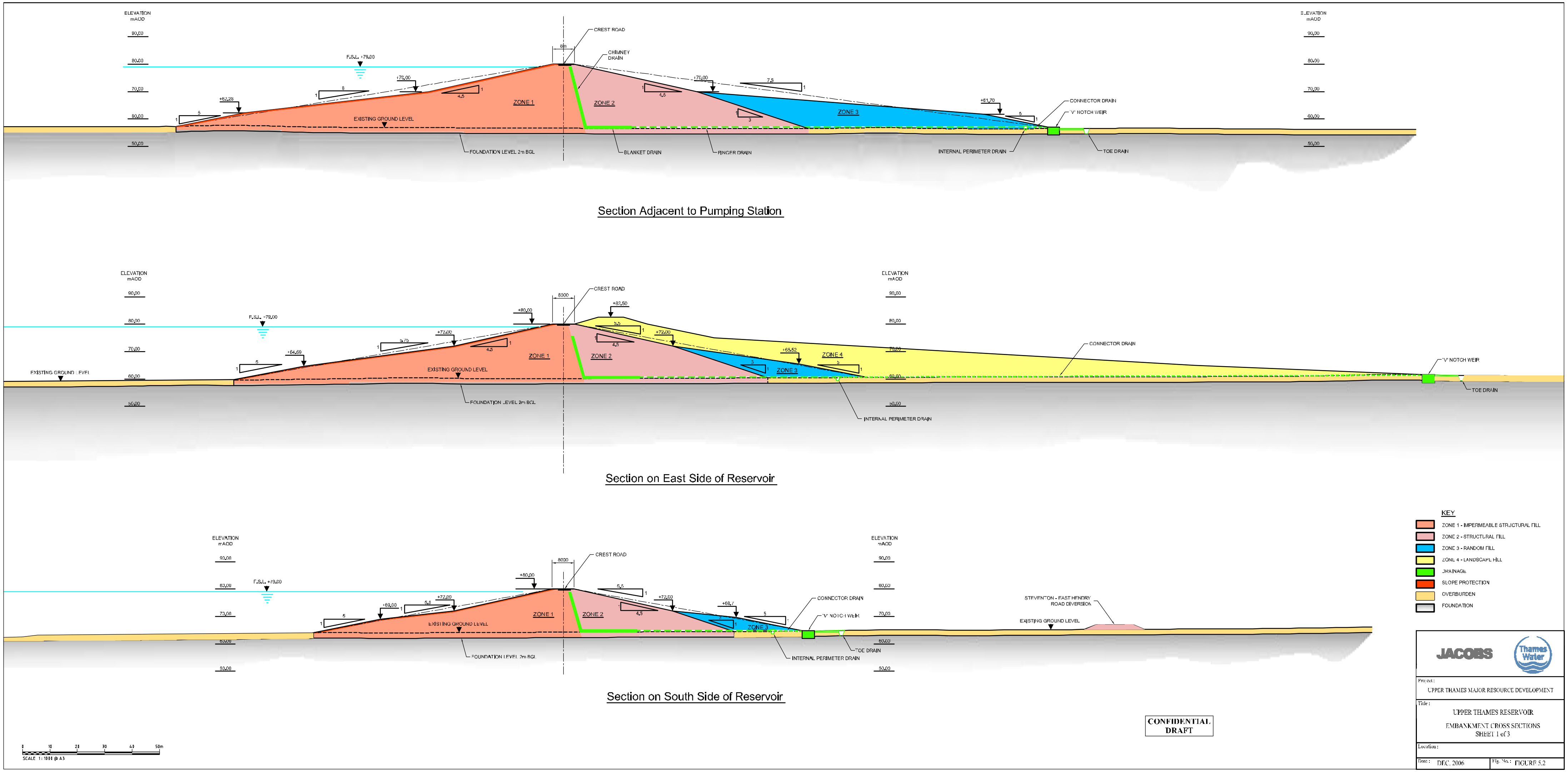
The trial embankment would be instrumented as for the construction stage of the main embankment. It is envisaged that two instrumented cross sections would be provided. These would be located 50m either side of the centre of the embankment, i.e. 75m from each end of the embankment.

5.11.6 Field and laboratory testing

The trial embankment would involve a programme of field and laboratory testing of fill and foundation material which would be developed in the next design stage.

5.11.7 Finite element analysis

A finite element analysis of the trial embankment is required before construction proceeds. This would confirm the trial embankment profile and give predictions of deformations during construction. The finite element analysis should be based on the 30 percentile foundation strength to ensure that the slope is taken close to failure even if foundation strengths are greater than anticipated. The same model would then be used for back analysis of the trial once it is completed, to derive design parameters for the final design of the main embankment.


Section Adjacent to Pumping Station
Section on East Side of Reservoir
Section on South Side of Reservoir
ELEVATION mAOD
CREST ROAD
CHIMNEY DRAIN
ZONE 1
ZONE 2
ZONE 3
ZONE 4
EXISTING GROUND LEVEL
FOUNDATION LEVEL 2m BGL
BLANKET DRAIN
FINGER DRAIN
INTERNAL PERIMETER DRAIN
CONNECTOR DRAIN
'V' NOTCH WEIR
TOE DRAIN
F.S.L. +79.00
STEVENTON - EAST HENDRY ROAD DIVERSION
KEY
ZONE 1 - IMPERMEABLE STRUCTURAL FILL
ZONE 2 - STRUCTURAL FILL
ZONE 3 - RANDOM FILL
ZONE 4 - LANDSCAPE FILL
DRAINAGE
SLOPE PROTECTION
OVERBURDEN
FOUNDATION
JACOBS
Thames Water
Project :
UPPER THAMES MAJOR RESOURCE DEVELOPMENT
Title :
UPPER THAMES RESERVOIR
EMBANKMENT CROSS SECTIONS
SHEET 1 of 3
Location :
Date : DEC. 2006
Fig. No. : FIGURE 5.2
CONFIDENTIAL DRAFT
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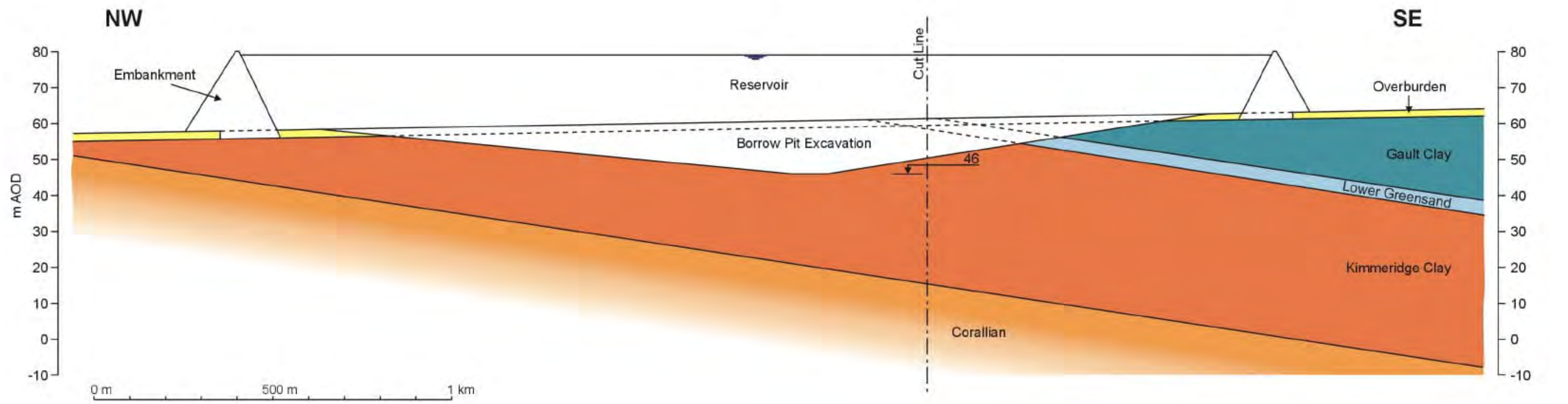

Borrow Pit Excavation

Figure 5.4

Appendix 4: Questions to Thames Water in email from GARD of 22nd September 2023

I would be grateful if you could provide us with the following information, …

Please supply:

1. Geological and geotechnical reports on the reservoir site. This site has been under consideration for several decades so these might be historical documents.
2. Reports on the site investigations, field testing, and laboratory testing that have been carried out.
3. Cross sections of the dam and including the critical section.
4. Dam break analysis, to feed into the choice of minimum factor of safety.
5. Provisional stability analyses of the SESRO embankment, including reasons for the choice of parameters and what further information would be collected to support them and reasons for the factor of safety chosen.
6. The dam is to be made from materials, some of which are naturally occurring (such as sand from the Bristol Channel) but whose grading may vary. That could result in one zone not being sufficient of a filter to an adjacent zone. Please could the limits of the grading for each zone be provided, along with demonstration that they would meet the grading criteria for filtering against the adjacent zone.
7. There have previously been reports about some of the formations being “highly fissured” and the Lower Greensand is a secondary aquifer. What measures would be adopted to cope with this and ensure the reservoir would be sufficiently water-tight? Since the bank would be up to 25m high what leakage would be expected when the reservoir is standing full and a copy of the calculation?
8. Details of the trial bank, its instrumentation , particularly the pore pressure gauges, the location of the instrumentation, the accuracy of the instrumentation, what range of response is expected, and what would be done if the response from the trial bank was different to that envisaged. This should include why it is considered that 3 very small banks are needed when previous trial banks have been much higher and why they would be removed at the end of the summer rather than monitoring such features as the pore pressure dissipation over an appropriate time.

Appendix 5 – Thames Water responses to GARD questions

Response to each question raised by GARD on the 22 September

1. Geological and geotechnical reports on the reservoir site. This site has been under consideration for several decades so these might be historical documents.

Response

Please see factual reports of ground investigations completed for the SESRO (previously called the 'Upper Thames Reservoir' or 'UTR'), enclosed as Appendix 1 to this response. These are as follows:

- 1A. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 1 1992.01.24
- 1B. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 2 1992.01.24
- 1C. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 3 1992.01.24
- 1D. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 8 1992.01.24
- 1E F1-Rp-Report on Ground Investigation at Upper Thames Reservoir, 2006.07.26
- 1F F1-Rp- Report on 2006 Ground Investigation at Upper Thames Reservoir, 2006,10,20

These reports cover the ground investigations completed in 1991, 2005 and 2006. Those completed in 2006 were the last ground investigations undertaken for the proposed reservoir. Further ground investigations will be carried out in 2023/2024. These will be used in conjunction with the historical investigations (the reports listed above) to inform the reservoir design which will be shared as part of our consultations leading up to a Development Consent Order (DCO) submission in 2026.

2. Reports on the site investigations, field testing, and laboratory testing that have been carried out.

Response

Please see factual reports of ground investigations completed for the SESRO (previously called the 'Upper Thames Reservoir' or 'UTR'), enclosed as Appendix 1 to this response. These are the same as the reports in response to Question 1, and are as follows:

- 1A. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 1 1992.01.24
- 1B. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 2 1992.01.24
- 1C. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 3 1992.01.24
- 1D. F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 8 1992.01.24
- 1E F1-Rp-Report on Ground Investigation at Upper Thames Reservoir, 2006.07.26
- 1F F1-Rp- Report on 2006 Ground Investigation at Upper Thames Reservoir, 2006,10,20.

These reports cover the ground investigations completed in 1991, 2005 and 2006. They include details of the site investigations, field testing and laboratory testing which have been carried out to date. As set out in our answer to Question 1, those completed in 2006 were the last ground investigations undertaken for the proposed reservoir. Further ground investigations will be carried out in 2023/2024. These will be used in conjunction with the historical investigations (the reports listed above) to inform the reservoir design which will be shared as part of our consultations leading up to a Development Consent Order (DCO) submission in 2026.

3. Cross sections of the dam and including the critical section

Response

At the time of writing (Q4 2023) the design of the dam has not been developed further than the 'Preliminary Design' described in the 2007 Preliminary Design Report.

The sections of the Preliminary Design Report which relate to the embankment design are enclosed as Appendix 2 (doc. Ref: F1-Rp-Preliminary Design Summary v4 Section 2, 3 and 5.pdf). These are:

- Section 2 – Geology, hydrogeology, and Geotechnics
- Section 3 – Embankment Stability Analysis
- Section 5 – Embankment Design

The cross sections of the embankment are shown within Section 5.

Ground investigation and a Compaction Trial are to be carried out in 2023-24 – this will provide more information which will be used to re-analyse the embankment design, and which may lead to the design being updated. Details of the design of the reservoir will be shared as part of our consultations leading up to a Development Consent Order (DCO) submission in 2026.

The slides presented at the briefing at Oxfordshire County Council on 15 September 2023 include an indicative dam cross section – this was taken from the 2007 Preliminary Design Report and is shown on page 10 of the slide pack which is also enclosed as Appendix 3.

4. Dam break analysis, to feed into the choice of minimum factor of safety.

Response

The factors of safety utilised to design the preliminary design embankment section are listed in Table 3.4 within Section 3.2 of the 2007 Preliminary Design Report – this is enclosed as Appendix 2. The selection of these factors of safety were not informed by any dam break analysis.

A dam break analysis will be undertaken after the design of the reservoir is finalised, as required by emergency planning regulations and in full compliance with the requirements of the Reservoirs Act 1975. We expect to undertake the dam break analysis post DCO consent. The dam break analysis does not inform the selection of factors of safety for the embankment design.

5. **Provisional stability analyses of the SESRO embankment, including reasons for the choice of parameters and what further information would be collected to support them and reasons for the factor of safety chosen.**

Response

As noted in our responses above, the design of the dam has not been developed further than the 'Preliminary Design' described in the 2007 Preliminary Design Report. The applicable sections of the Preliminary Design Report which cover the embankment design, including the cross sections are within section 5, as can be found in Appendix 2.

We expect to undertake further embankment analyses in 2024 and 2025, informed by proposed ground investigations and compaction trial in 2023-2024. These will inform the reservoir design which will be shared as part of our consultations leading up to a Development Consent Order (DCO) submission in 2026.

6. **The dam is to be made from materials, some of which are naturally occurring (such as sand from the Bristol Channel) but whose grading may vary. That could result in one zone not being sufficient of a filter to an adjacent zone. Please could the limits of the grading for each zone be provided, along with demonstration that they would meet the grading criteria for filtering against the adjacent zone.**

Response

The dam will be formed of natural materials, mainly clay from the site, but will include imported aggregates. We also recognise the critical importance of filter compliance between adjacent zones within the embankment.   Grading curves for internal drains were previously identified as part of the Preliminary Design in 2007 and are shown in Figure 5.3 of the 2007 Preliminary Design Report 2007 (enclosed within Appendix 2).

The source of sand to be used to be used in the filter drainage zones has not yet been confirmed, but various potential sources have been identified (such as the Bristol Channel). Investigations into the suitability of aggregates from various sources will be carried out and the sources for the aggregates confirmed post DCO consent.

7. There have previously been reports about some of the formations being “highly fissured” and the Lower Greensand is a secondary aquifer. What measures would be adopted to cope with this and ensure the reservoir would be sufficiently water-tight? Since the bank would be up to 25m high what leakage would be expected when the reservoir is standing full and a copy of the calculation?

<u>Response</u>

As noted in our responses above, the design of the proposed dam and reservoir has not been developed further than the ‘Preliminary Design’ described in the 2007 Preliminary Design Report. Section 5 of the Preliminary Design Report, which includes details of reservoir seepage analyses, is enclosed at Appendix 2.

In response to your first two queries in Question 7, we have included factual reports and logs of all ground investigation completed to date at the SESRO site. Note that as part of the design it is proposed that the superficial deposits are to be removed under part of the embankment, so are not relied on as part of securing the reservoir’s watertightness.

The watertightness of the reservoir will be dependent on the low permeability of the Kimmeridge and Gault clay strata, and we do not expect any significant seepage through these strata as noted in the enclosed excerpt from the Preliminary Design Report.

The Lower Greensand stratum has a higher permeability and is a potential path for significant seepage from the reservoir. The 2007 Preliminary Design included for a ‘plug’ to be formed, of reworked Kimmeridge or Gault Clay, to be placed over the greensand outcrop within the reservoir. This continues to be the basis of the solution to this issue, with the design of the plug to be finalised once the potential water pressures within the Greensand are fully understood. Proposed ground investigations for 2023/24 will include monitoring of groundwater pressures within this stratum which will inform the proposed clay plug design. Details of the design of the reservoir (which will incorporate the clay plug design) will be shared as part of our consultations leading up to a Development Consent Order (DCO) submission in 2026.

8. Details of the trial bank, its instrumentation , particularly the pore pressure gauges, the location of the instrumentation, the accuracy of the instrumentation, what range of response is expected, and what would be done if the response from the trial bank was different to that envisaged. This should include why it is considered that 3 very small banks are needed when previous trial banks have been much higher and why they would be removed at the end of the summer rather than monitoring such features as the pore pressure dissipation over an appropriate time.

Response

As discussed during the presentation at Oxfordshire County Council offices on 15 September 2023, the Compaction Trials which are proposed next year are different from and in addition to a full-scale Trial Embankment which is proposed to be carried out after the DCO submission.

The Clay Compaction Trial proposed next year will be focused on testing the clay, from various depths below ground level, after it has been compacted (as it would be within the permanent SESRO embankment). We do not intend to measure the foundation porewater response to the construction of the trials. We agree that such response measurement would need a bigger embankment to be built for longer – this will be the 'trial embankment', akin to 'previous trial banks' referred to in your query.

Obtaining the requisite parameters of the as-compacted clay in the Compaction Trial proposed next year does not require large embankments. It does require the material to be subject to comparable compactive effort as will occur during construction of the main embankment – this will be achieved by using a typical earthmoving roller during the trial. High-quality samples will be taken from the trials for laboratory tests. In addition, in-situ measurements will be undertaken. The sampling and testing schedule for the trial is still work in progress.

9) Finally, I would be grateful if you could supply the CV of your Panel Engineer, Mr Martin Deane (including hands-on experience of design and construction of similar clay embankment dams). I would also be grateful if you could clarify the background and expertise of your other Reservoir Engineer (Mr James Cameron) who was introduced as having 14 years on the All Reservoirs Panel. We cannot find his name on the current (August 2023) Panel List published on the gov-uk website. Is he therefore, like our consultant, a retired Reservoir Engineer? We would appreciate details of his CV along the same lines as requested for Mr Deane.

Response

Both Martin Deane and James Penman are All Reservoir Panel Engineers. Martin has 18 years' experience and James has 38 years' experience. As set out in previous enquiries regarding the release of CVs, their release is considered personal detail and we are not in a position to share them.

Martin and James lead a large team of specialists which has critically reviewed the 2007 Preliminary Design, identified the planned further investigations / trials, and are currently planning in detail the next phase of design. This team includes geologists, hydrogeologists and geotechnical engineers.

As set out in table 4.3 of our SESRO Gate 2 Main Report – link here: (SESRO Gate 2 Main Report), for large new dam projects, international best practice and UK guidance advocates the establishment of a panel of specialists to review key elements. A Reservoir Advisory Panel (or RAP) was set up and operational during the 2007 design phase and was recommenced in 2022 to review recent work, including proposals for further ground investigations and the compaction trial. The Chair of the original Panel has been reappointed, namely Andy Hughes. Andy is also an All Reservoirs Panel Engineer with over 45 years of experience worldwide.

The Reservoirs Act 1975 requires the appointment of a Construction Engineer to supervise the design and construction of new dams in England and Wales. For a large non-impounding reservoir like SESRO, the Construction Engineer must already be appointed either to the All Reservoirs Panel or the Non-impounding Panel. The development of the Preliminary Design was undertaken under the supervision of a Construction Engineer, but this appointment ended when work on the design stopped. Recent work to review the design, propose further investigations and plan the next design phase has not changed the dam design as yet, so has not required a Construction Engineer appointment; however, all work relating to the reservoir design has been supervised by James Penman (Martin Deane having only recently been appointed to that panel). A Construction Engineer will be appointed imminently, to supervise the design development and construction, as required by the Reservoirs Act 1975.

10) In notes taken at the meeting (copies of slides are not yet available - will participants receive them?), it is stated that a larger 'full-size height' trial is planned 'later'. Is there an indicative time-planning for this?

Response

A copy of the slide deck was sent to Oxfordshire County Council and the Vale of White Horse District Council after the briefing. We attach the slide deck for your information as Appendix 3.

The Trial Embankment is to be undertaken after DCO consent, if granted, to validate the models used in the design of the dam, particularly the response of the foundation to loading. Preliminary planning to date includes for this to be done as part of the main construction contract but in advance of the main dam earthworks, as is common in embankment dam construction. The sequencing of pre-construction and construction activities will be subject to detailed review in conjunction with design development activities in 2024. This may lead to changes to the overall programme, which is work in progress. We will be in a position to share further details on its timings as part of future engagement and consultation in 2024 and 2025.

11) Are we to take the planning indicated in figure 6.2 of the 2007 Design Report as a 'best guess' for now?

Response

We presume you refer to Figure 6.2 of the "Stage 2 Preferred Scheme and Design Options Report – Volume 1" published in 2007, which shows an Indicative Construction Programme. This construction programme has been superseded. An indicative construction programme is included in our SESRO Gate 2 Concept Design Report in Appendix B. The report is available on our website. A link to the report is here: https://www.thameswater.co.uk/media-library/home/about-us/regulation/regional-water-resources/south-east-strategic-reservoir/gate-2-reports/A-1---SESRO-Concept-Design-Report.pdf.

We expect the embankment trial to take place in years 1 to 3 of the construction programme. This will be confirmed in due course, once we have a construction partner on board.

12) We note that this proposal was to construct the large-scale trial in the 3rd year AFTER approval, and that quite a lot of groundwork excavation (Of flood compensation areas and stream diversion channels) would be undertaken before the trial results were available - this seems problematic. Is it still the preferred plan?

Response

The Trial Embankment is to be undertaken after DCO consent, if granted, to validate the models used in the design of the dam, particularly the response of the foundation to loading. Preliminary planning to date includes for this to be undertaken as part of the main construction contract but in advance of the main dam earthworks, as is common in embankment dam construction. The sequencing of pre-construction and construction activities will be subject to detailed review in conjunction with design development activities in 2024. This may lead to changes to the overall programme, which is work in progress. We will be in a position to share further details on its timings as part of future engagement and consultation in 2024 and 2025.

We would however note that watercourse diversions and flood compensation works will always be some of the first earthworks activities undertaken at the site, and their design would not be informed by the Trial Embankment.

APPENDICES

1A	F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 1 1992.01.24
1B	F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 2 1992.01.24
1C	F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 3 1992.01.24
1D	F1-Rp Factual Report on 1992 Exploration Associates SI - VOLUME 4 1992.01.24
1E	F1-Rp-Report on Ground Investigation at Upper Thames Reservoir, 2006.07.26
1F	F1-Rp- Report on 2006 Ground Investigation at Upper Thames Reservoir,2006,10,20
2	F1-Rp-Preliminary Design Summary v4 Section 2, 3 and 5.pdf.
3	Presentation slides on understanding the building of a bunded reservoir. 15 September 2023