Hodder Water Treatment Works

Flood Risk Assessment

21 August 2019

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Contents

Executive summary

Mott MacDonald has been appointed by Mott MacDonald Bentley (MMB) to undertake a flood risk assessment (FRA) on behalf of United Utilities at Hodder Water Treatment Works (WTW) (the site), located to the north east of Slaidburn in the Forest of Bowland.

The site is bounded to the north by Stocks Reservoir, which supplies water for treatment at Hodder WTW. The River Hodder flows adjacent to the site on the south eastern boundary.

The proposed development comprises installation of a new building containing rapid gravity filters (RGFs) which are required to ensure a consistent supply of quality drinking water. In addition, works to raise the crest level of the reservoir spillway weir will be undertaken to increase the capacity of the reservoir, helping to safeguard water supply to the north west of England in times of drought.

The proposed development is not considered to be at risk of flooding from fluvial, surface water, groundwater or sewer flooding. The treatment works themselves pose a risk should infrastructure become blocked or malfunction, however, this is considered to be managed through ongoing operation and maintenance of the treatment works by trained operatives. Due to the location of the site immediately downstream of Stocks Reservoir, the site will be inundated should the reservoir embankment breach or fail. However, this is not considered to be a significant source of flood risk owing to regulations under the Reservoirs Act 1975 for inspection and maintenance of Category A reservoirs and the requirement to safely pass flows up to the PMF (approximately equivalent to a 1:10,000-year flood) over the spillway.

Similarly, although works to increase the capacity of the reservoir by raising the spillway level may result in a greater extent of flooding downstream should the embankment breach, the likelihood of this occurring is very low. Raising the spillway crest level will not result in flooding of any additional receptors. Therefore, no significant change to reservoir flood risk is anticipated as a result of this scheme. In addition, the proposed development will not increase flood risk elsewhere from fluvial, surface water, groundwater or sewer flooding. The increased capacity of the reservoir will contribute to greater attenuation of fluvial flood flows downstream.

The proposed development meets the Exception Test to allow the development of essential infrastructure in Flood Zone 3. Therefore, it is considered that the proposed development is compliant with the National Planning Policy Framework (NPPF).

1 Introduction

1.1 Background

Mott MacDonald has been appointed by Mott MacDonald Bentley (MMB) to undertake a flood risk assessment (FRA) on behalf of United Utilities at Hodder Water Treatment Works (WTW), located in the Hodder Valley to the north east of Slaidburn, Forest of Bowland, Lancashire. The National Grid Reference for the Site is SD718545.

United Utilities are undertaking works at Hodder WTW (hereafter referred to as the site) to upgrade the treatment process to improve water quality and increase reservoir capacity.

1.2 Scope of assessment

The site is partially situated within Flood Zones 2 and 3. Therefore, a FRA must accompany any planning application to ensure the renovations do not lead to an increase in risk of flooding either at the site or downstream.

This FRA has been carried out in accordance with the National Planning Policy Framework (NPPF)[1] and associated Planning Practice Guidance (PPG)[2] to assess the risk of flooding to the site from all sources, and the possible impact of the development on flood risk elsewhere. The scale and nature of the FRA is considered appropriate for the development.

Information presented within this report is dependent upon the accuracy and reliability of the supplied information, correspondence, and data available to Mott MacDonald, at the time of the assessment. Any party developing detailed design should not rely on assumptions made in this report but should satisfy themselves in that regard.

Mott MacDonald has followed accepted procedure in providing the services but, given the residual risk associated with any prediction and the variability that can be experienced in flood conditions, Mott MacDonald takes no liability for and gives no warranty against actual flooding of any property or the consequences of flooding in relation to the performance of the service. This report has been prepared for the purposes of supporting a planning application only. Mott MacDonald accepts no responsibility or liability for this document to any party other than by whom it was commissioned.

1.3 Proposed development

The proposed development comprises installation of a new building containing rapid gravity filters (RGFs) which are required to ensure a consistent supply of quality drinking water. Water treatment works which need to remain operational in times of flood are classed as “essential infrastructure” in the PPG. Therefore, the proposed development is also considered to be essential infrastructure.

In addition, permitted development rights have been granted for works to raise the crest level of the reservoir spillway weir. This will increase the capacity of the reservoir, helping to safeguard water supply to the north west of England in times of drought. While these works are permitted

[1] Ministry of Housing, Communities & Local Government (2019) National Planning Policy Framework. Accessed 16/08/2019. Available from: https://www.gov.uk/government/publications/national-planning-policy-framework--2

[2] Ministry of Housing, Communities & Local Government (2019) Planning Practice Guidance. Accessed 16/08/2019. Available from: https://www.gov.uk/government/collections/planning-practice-guidance

development, Ribble Valley Borough Council (RVBC) have requested their inclusion in the FRA. These works are considered to be “water compatible” under the PPG.

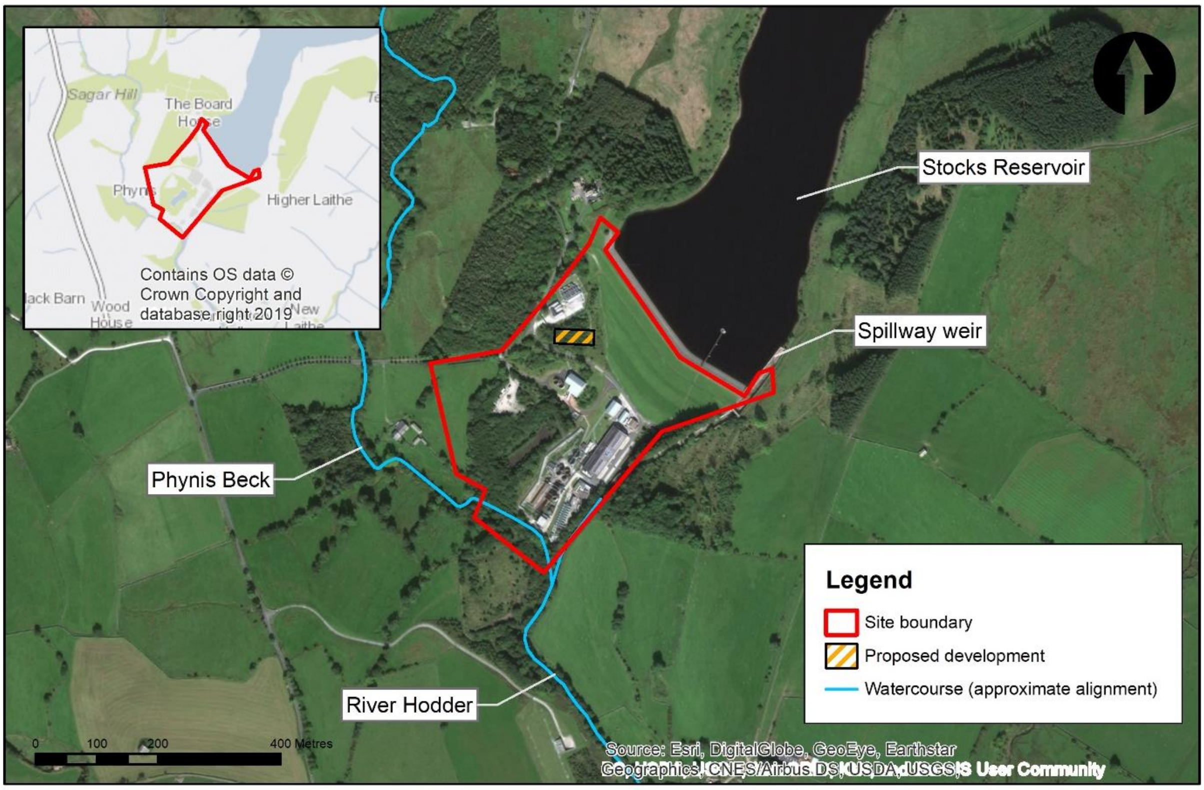
The design life of the combined works is taken to be 100 years for the purpose of this assessment. Individual elements may have shorter lifespans and require replacement over the lifespan of the overall works.

2 Site information

2.1 Site map

The layout and location of the site is shown in Figure 2.1.

Figure 2.1: Site location



Source: Mott MacDonald, 2019

2.2 Existing infrastructure

2.2.1 Existing watercourses

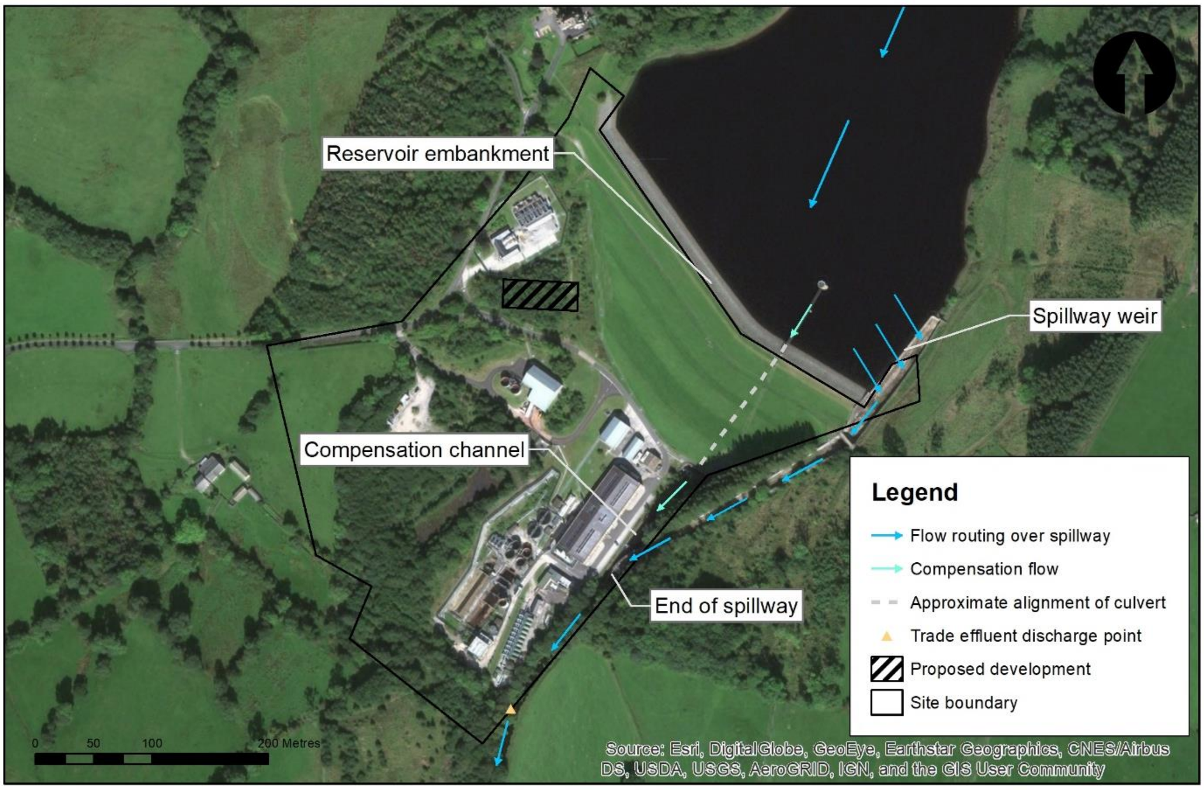
The site is bounded to the south east by the River Hodder, which flows within an engineered channel for approximately 200m before returning to a natural channel profile. To the west, Phynis Beck flows south east and joins the River Hodder at the southern corner of the site.

2.2.2 Stocks Reservoir

Stocks Reservoir is a Category A reservoir located immediately upstream of the site. The spillway weir is located at the south east corner of the reservoir. The spillway discharges to the River Hodder approximately 240m downstream of the weir, adjacent to the site. A compensation flow is maintained to the River Hodder via a culvert through the reservoir embankment. There is a daily flow gauge downstream of the spillway which indicates a mean flow of 0.583m^3/s[3]. It is assumed that the compensation flow is roughly equivalent to this.

Figure 2.2 shows how flows are routed through the reservoir to the River Hodder.

Figure 2.2: Flow routing through Stocks Reservoir



Source: Mott MacDonald, 2019

2.2.3 Existing water mains and drainage infrastructure

Utilities records for the site indicate the presence of several surface water sewers as well as land drains. Flows are either captured by the treatment process on site or discharged to the River Hodder.

In addition, United Utilities hold a consent to discharge up to 50l/s trade effluent into the River Hodder, consisting of settled filter backwash effluent during planned downtime and in an emergency. The location of the discharge point at SD 71675 54226 is shown in Figure 2.2.

2.3 Evidence of historical flooding

The reach of the River Hodder extending between the site and Slaidburn (2.3km downstream) is not a designated Main River and therefore the Environment Agency do not hold a record of past flood events on this reach of the river.

Anecdotal evidence from the site operatives indicates that no flooding has occurred at the site within the last 10 years.

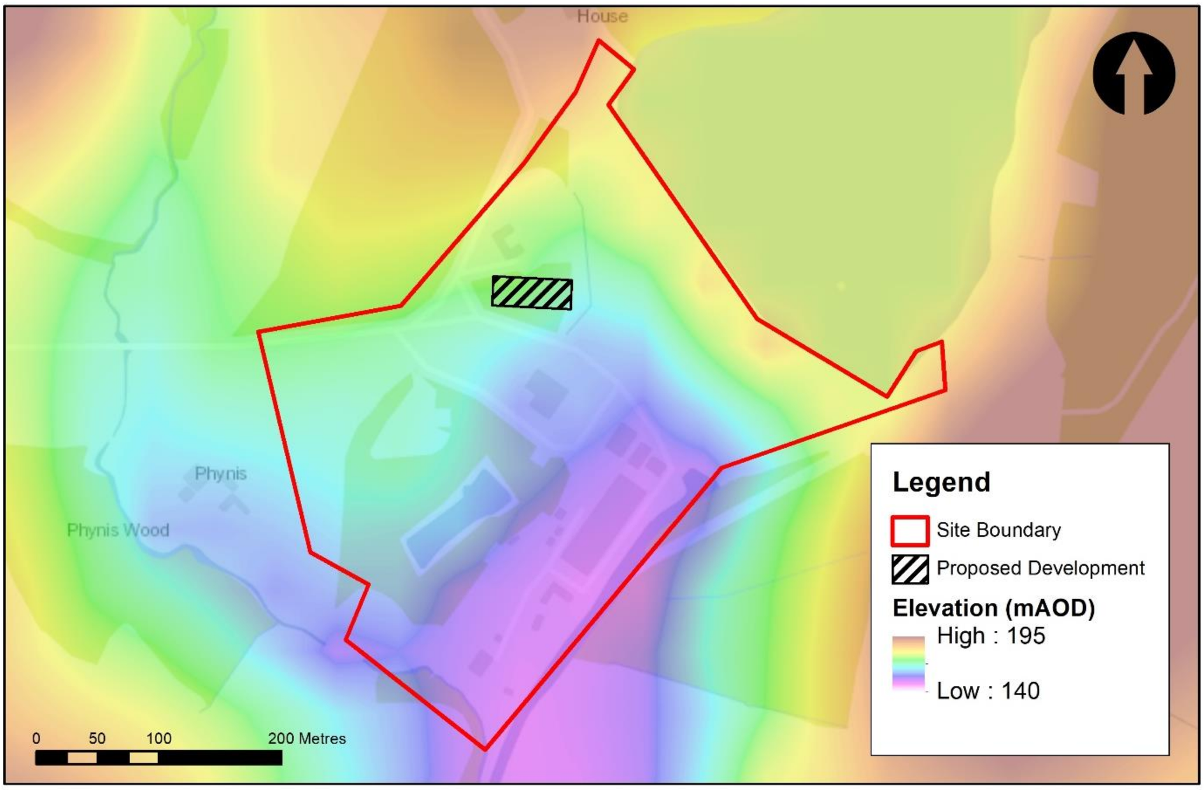
[3] National River Flow Archive. 71002 – Hodder at Stocks Reservoir. Accessed 16/08/2019. Available from: https://nrfa.ceh.ac.uk/data/station/meanflow/71002

2.4 Site topography

The site is slopes downwards from the northern corner to the south east, where the site is bordered by the River Hodder. Elevations on site range from approximately 195mAOD to approximately 140mAOD, as indicated in Figure 2.3.

The proposed development is located at an approximate elevation of 168mAOD.

Figure 2.3: Site topography



Source: Mott MacDonald, 2019. Elevation data derived from Esri World Elevation Terrain data, source: Airbus, USGS, NGA, NASA, CGIAR, NLS, OS, NMA, Geodatastyrelsen, GSA, GSI and the GIS User Community

2.5 Existing ground conditions

According to the British Geological Survey (BGS) mapping[4], the site is underlain by a combination of limestone and mudstone overlain by superficial till deposits. Records indicate that much of the site has been artificially raised with made ground of variable composition.

[4] British Geological Society (2019) GeoIndex Onshore. Accessed 16/08/2019. Available from: http://mapapps2.bgs.ac.uk/geoindex/home.html

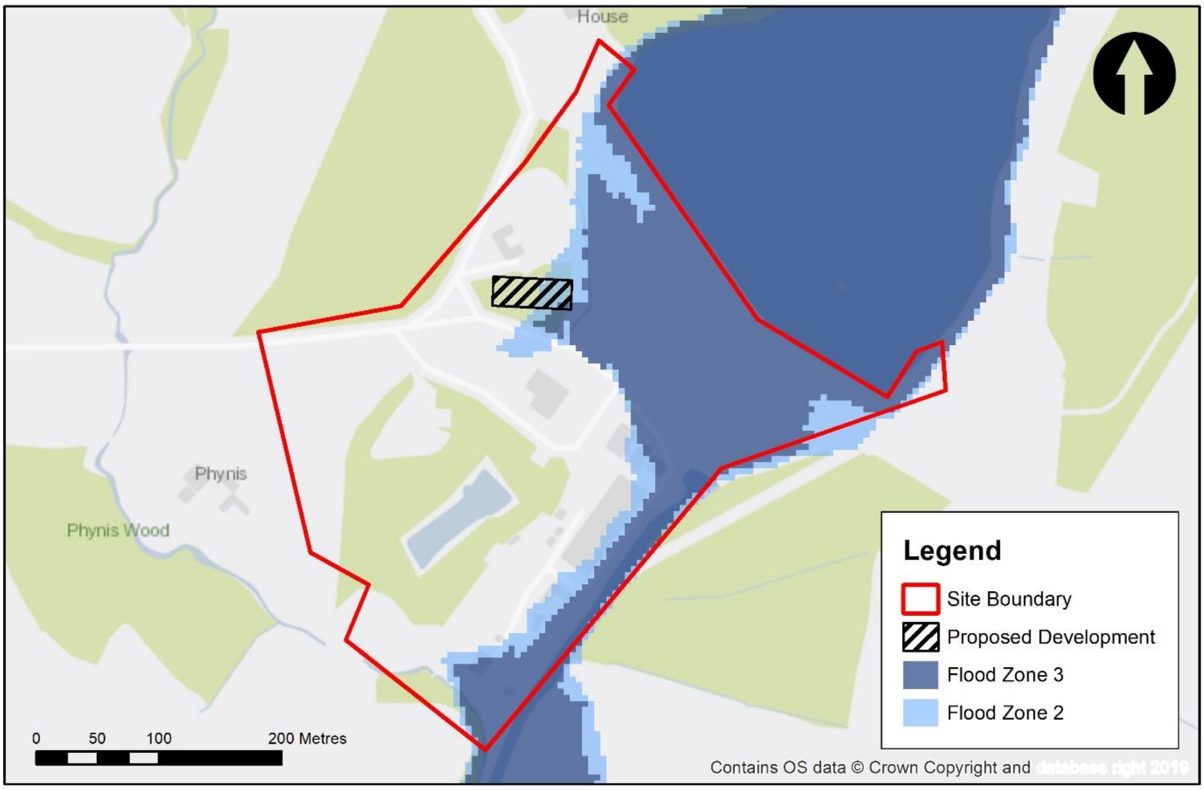
3 Assessment of flood risk to the development

3.1 Fluvial flooding

3.1.1 Fluvial Flood Map

The Environment Agency publishes floodplain extents for all significant watercourses throughout England. These extents, displayed on the Risk of Flooding from Rivers and Sea flood map[5], are available to the public via the internet and are the primary source of publicly available flood risk information. The Environment Agency also provides the Flood Map for Planning which displays the Flood Zones (Figure 3.1).

Figure 3.1: Fluvial Flood Map for Planning



Source: Mott MacDonald, 2019. Contains Environment Agency data © Crown copyright and database rights 2018 OS 100024198. Contains public sector information licensed under the Open Government License v3.0.

[5] Environment Agency. 2017 Risk of Flooding from Rivers and Sea dataset. Available under Open Government License v003.

Table 3.1 provides definitions of the Flood Zones as stated in the PPG [6]. It should be noted that the flood extents given on the Flood Map are only indicative and do not necessarily account for any man-made structures such as railway embankments, roads, or flood defences.

Table 3.1: Flood zones

Flood Zone	Definition
Zone 1 Low Probability	Land having a less than 1 in 1,000 annual probability of river or sea flooding. (Shown as 'clear' on the Flood Map – all land outside Zones 2 and 3)
Zone 2 Medium Probability	Land having between a 1 in 100 and 1 in 1,000 annual probability of river flooding; or land having between a 1 in 200 and 1 in 1,000 annual probability of sea flooding. (Land shown in light blue on the Flood Map)
Zone 3a High Probability	Land having a 1 in 100 or greater annual probability of river flooding; or Land having a 1 in 200 or greater annual probability of sea flooding.
Zone 3b The Functional Floodplain	This zone comprises land where water has to flow or be stored in times of flood. (Not separately distinguished from Zone 3a on the Flood Map)

Source: Ministry of Housing, Communities and Local Government (2014) Planning Practice Guidance

The reservoir embankment and land at the eastern boundary of the site are within Flood Zone 2 and Flood Zone 3, indicating the site is in areas of 'medium' (1% - 0.1% AEP) and 'high' (>1% AEP) flood risk respectively.

Consultation with the Environment Agency has identified that the Flood Zones in this location are based on the 2004 National Generalised Modelling (NGM). The NGM is intended to be indicative, providing basic flood risk information for areas which had not been extensively modelled. The NGM does not represent flow routed through reservoirs or any representation spillways / overflows, and so the Flood Zones do not represent the true flood risk at the site.

3.1.2 Site Specific Fluvial Flood Risk

3.1.2.1 Influence of climate change

The Environment Agency[7] advise the higher central and upper end allowances for essential infrastructure in Flood Zone 2 and upper end allowance in Flood Zone 3a.

Table 3.2: Peak river flow allowances for the North West River Basin District

Allowance Category	2020s (2015-2039)	2050s (2040-2069)	2080s (2070-2115)
Upper end	20%	35%	70%
Higher central	20%	30%	35%
Central	15%	25%	30%

Source: Environment Agency. 2019. https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

[6] Department for Communities and Local Government. 2012. Technical Guidance to the National Planning Policy Framework. Paragraph 5, Table 1: Flood Zones. [Online] Available at: https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/6000/2115548.pdf [Accessed 17/06/2019]

[7] Environment Agency (2019) Flood risk assessments: climate change allowances. Accessed 16/08/2019. Available from: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

Therefore, all considerations of fluvial flood risk in this report will consider the 1% AEP event, the 1% AEP event with an allowance of 35% for climate change, the 1% AEP event with an allowance of 70% for climate change, and the 0.1% AEP event.

3.1.2.2 Flow routing through the reservoir

The NGM does not represent flow routed through reservoirs. Category A Reservoirs are designed to safely pass the probable maximum flood (PMF) flow (approximately equivalent to 1:10,000-year flow) without overtopping the reservoir embankment. Therefore, in contrast to the flood extents produced by the NGM, in 1% AEP and 0.1% AEP events, flood waters would flow down the spillway rather than over the embankment and into an engineered channel 240m downstream of the spillway weir (Figure 2.2).

Flood Estimation Handbook (FEH) catchment descriptors[8] for the reservoir catchment were obtained and used to generate peak flows for the relevant flood events, using the ReFH method. The ReFH method is less accurate that the FEH Statistical method but, as it does not take reservoir attenuation into account, it is considered to be more conservative in this case and therefore appropriate for this assessment.

The peak flows are documented in Table 3.3. The 1% AEP and 0.1% AEP flood events correspond to Flood Zone 3 and 2, respectively.

Table 3.3: Peak flows for Stocks reservoir

Flood event	Peak flow (m^3/s)
1% AEP	117.17
1% AEP +35%CC	158.18
1% AEP +70%CC	199.19
0.1% AEP	226.49

A flood study for the reservoir was undertaken by Jacobs in 2015[9] and is included in Appendix A. The spill-weir rating curve indicates that in a flood event, when the reservoir is already full, flows are contained in behind the embankment and discharged via the spillway up to approximately $300m^3/s$. When flow exceeds this, levels in the reservoir become high enough to overtop the embankment.

As part of the works, the spillway weir will be raised by 300mm. This means that the total storage between the spillway weir crest will be reduced, and will fill more quickly than in the current situation. Therefore, the embankment will overtop at a lower flow. The rating curve for the reservoir, amended for the raised spillway, indicates that overtopping of the embankment will occur at a flow of approximately $275m^3/s$ following the works.

When compared to the peak flows in Table 3.3, no flow is anticipated to overtop the reservoir embankment in the 1% and 0.1% AEP flood events. This indicates that the extent of flooding indicated on the Flood Map for Planning (Figure 3.1) is not representative of the flood mechanism at the site.

[8] Wallingford HydroSolutions. 2019. FEH web map. Catchment at 371950 454500. [Accessed 13/08/2019] Available from: https://fehweb.ceh.ac.uk/GB/map

[9] Stocks Reservoir Flood Study and Review of Wave Analysis, JACOBS (April 2015)

3.1.2.3 Risk of flooding from compensation channel

Flow which is routed down the spillway discharges into the River Hodder, which is an engineered channel for approximately 240m downstream of the spillway outlet, downstream of which it becomes a natural channel.

If the capacity of the channel downstream of the spillway is exceeded during a flood event, it may result in spill onto the site. In addition to flows from the spillway, this channel must also carry the compensation flow from the reservoir, flow discharged under consent from the treatment works, and downstream, flow discharging from Phynis Beck.

There is a daily flow gauge downstream of the spillway which indicates a mean flow of $0.583m^3/s$[10]. It is assumed that the compensation flow is roughly equivalent to this.

Similar to Stocks Reservoir, flood flows for Phynis Beck have been estimated based on FEH catchment descriptors using the ReFH method. For the purposed of this analysis, it is assumed that the flood peaks occur simultaneously, although in reality the Phynis Beck catchment is much smaller and likely to peak earlier than the Stocks catchment.

The total estimated flow within the River Hodder channel downstream of the spillway outfall for the relevant flood events is presented in Table 3.4.

Table 3.4: River Hodder maximum flood flows downstream of spillway

Flood event	Stocks Reservoir Spillway	Compensation Flow	Discharge Consent	Phynis Beck	Total Flow
1% AEP	117.17	0.58	0.05	6.08	123.88
1% AEP +35%CC	158.18	0.58	0.05	8.21	167.02
1% AEP +70%CC	199.19	0.58	0.05	10.34	210.16
0.1% AEP	226.49	0.58	0.05	11.81	238.93

The depth of flooding in the engineered channel has been estimated using Manning's formula for open channel flow based on the dimensions of the engineered channel as indicated in Table 3.5. For the purpose of this calculation it is assumed that the channel sides are "glass-walled", i.e. no flow is allowed to spill onto the adjacent land. This can be used to generate a conservative flood level at any point along the channel, assuming that the channel is relatively uniform. The maximum bed elevation of the channel is 149.18mAOD, located at the upstream end.

The maximum depth in the channel, and maximum elevation based on the maximum bed elevation is presented in Table 3.5.

Table 3.5: Parameters for Manning's formula

Parameter	Value	Source
Slope	0.006	Historic drawings
Channel width	9m	Historic drawings – narrowest width selected
Manning's n	0.02	Chow, 1959. Value for a channel with a concrete base and stone rubble sides

[10] National River Flow Archive. 71002 – Hodder at Stocks Reservoir. Accessed 16/08/2019. Available from: https://nrfa.ceh.ac.uk/data/station/meanflow/71002

The calculated maximum flood depths and elevations are presented in Table 3.6.

Table 3.6: Estimated maximum flood elevation on site

Flood event	Maximum depth (m)	Maximum elevation (mAOD)
1% AEP	1.64	150.82
1% AEP +35%CC	1.91	151.09
1% AEP +70%CC	2.17	151.35
0.1% AEP	2.33	151.51

The minimum elevation at the location of the proposed RGFs is 166.5mAOD. The estimated maximum flood level at the site during the 0.1% AEP flood event is 151.51mAOD. Therefore, it has been calculated that the proposed development is not at risk of flooding up to and including the 0.1% AEP flood event. Similarly, the proposed development is not at risk during the 1% AEP flood event including an allowance of 70% for climate change.

3.2 Surface water flooding

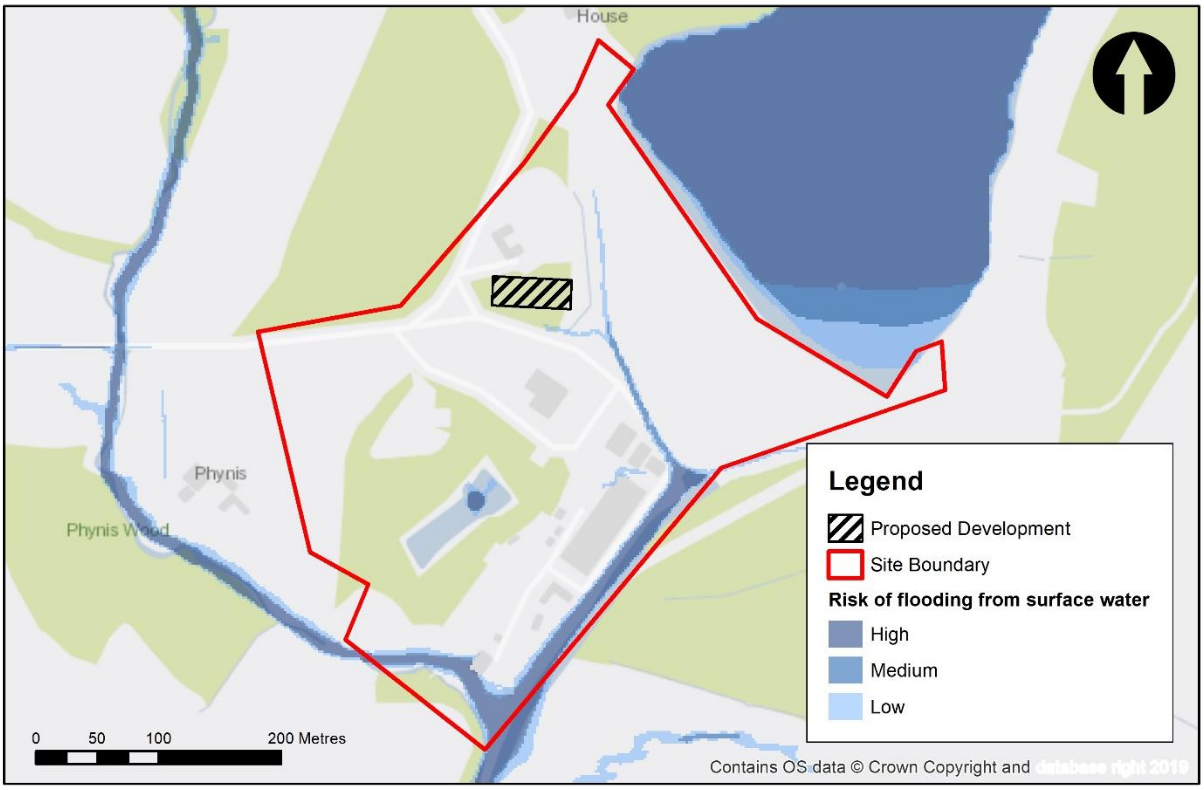
3.2.1 Surface water flood map

The 'Long term flood risk' map (Figure 3.2) includes information regarding the risk of flooding from surface water. The flood risk categories are defined in Table 3.7.

The majority of the site is at "very low risk" of flooding. This means that the chance of surface water flooding is less than 0.1% each year. There are localised areas which are at higher risk, but these occur in specified land drains or historic infrastructure (an old lagoon) which is no longer in use. There is one area of ponding which is at "low risk" of flooding from surface water. This means that the chance of flooding is between 0.1 and 1% each year.

The proposed development is located in an area of "very low risk".

Figure 3.2: Surface water flood map



Source: Mott MacDonald, 2019. Contains Environment Agency Open Government License Data. 2019

Table 3.7: Surface water flood risk

Flood Risk	Description	Annual Exceedance Probability
Very low risk	Each year the area has a chance of surface water flooding of less than 0.1%	<0.1% (1 in 1000 year) surface water flooding
Low risk	Each year the area has a chance of surface water flooding of between 0.1 and 1%.	1% - 0.1% (1 in 100 – 1 in 1000 year) surface water flooding
Medium risk	Each year the area has a chance of surface water flooding of between 1 and 3.3%	3.3 – 1% (1 in 75 – 1 in 100 year) surface water flooding
High risk	Each year the area has a chance of surface water flooding of greater than 3.3%.	>3.3% (up to 1 in 75 year) surface water flooding

Source: Environment Agency (2018) Flood Warning Information Service: Long term flood risk information.

3.2.2 Influence of climate change

Surface water is managed on site through land drains and surface water sewers. Should the capacity of the drainage network be exceeded, localised flooding may occur on site. Whilst no issues of flooding have been experienced on site in the last 10 years, it is likely that the influence of climate change will result in higher intensity rainfall events and increased surface water runoff. Rainfall intensity allowances are shown in Table 3.8. It is estimated that rainfall intensity at the site may increase by 20-40% by 2115[11].

Table 3.8: Peak rainfall intensity allowances in England

Allowance Category	2020s (2015-2039)	2050s (2040-2069)	2080s (2070-2115)
Upper end	10%	20%	40%
Central	5%	10%	20%

Source: Environment Agency. 2019. https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

3.3 Groundwater flooding

The site is underlain by a combination of limestone and mudstone overlain by superficial till deposits and made ground. The majority of the site is permeable, so if the groundwater table rises above local ground levels, groundwater flooding may occur.

The Ribble Valley Strategic Flood Risk Assessment[12] identifies that groundwater flooding is not considered to be a significant flood risk factor in the area.

3.4 Flooding from sewers and drains

Sewer flooding generally occurs in urban areas when the sewer network becomes surcharged, resulting in backing up in the upstream network. The site lies in a rural area where inputs to the sewer network are anticipated to be relatively low. Therefore, sewer flooding is not considered to be a significant source of risk to the site.

Utilities records for the site indicate several surface water sewers fed by land drains. Some flow is captured by the treatment process on site, while the rest discharges to the River Hodder. Should the capacity of the drainage network be exceeded, localised ponding in low spots may occur on site. It is assumed that this will not interfere with the proposed development which is situated on a slope. Site operatives have reported no instances of flooding to the site in the past 10 years.

3.5 Failure of infrastructure

3.5.1 Stocks Reservoir

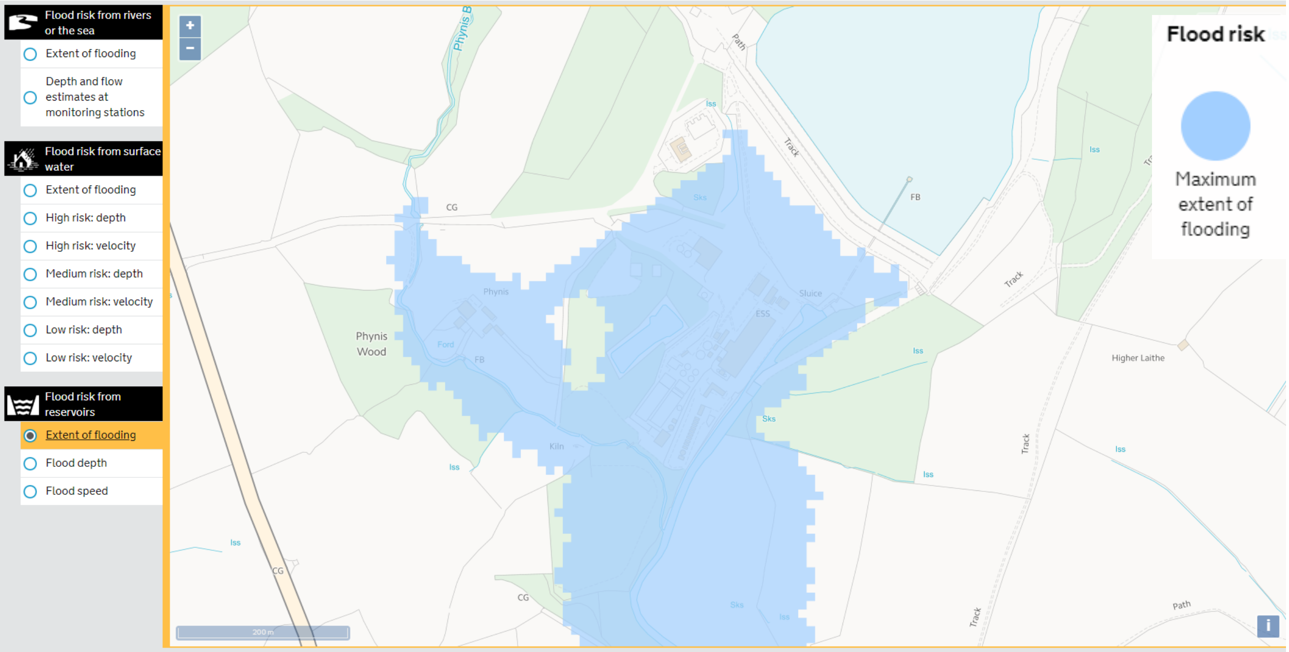
The 'Long term flood risk map'[13] shows the potential maximum extent of flooding if an uncontrolled release of water from a reservoir were to occur. In the event of an uncontrolled breach the site would be at risk of flooding, as shown in Figure 3.3**Error! Reference source not found.**. Under the Reservoirs Act 1975, the Environment Agency ensures that reservoirs are inspected regularly by reservoir safety panel engineers and that essential safety works are carried out where required. Therefore, the risk of flooding from reservoir failure is considered to be low.

[11] Environment Agency (2019) Flood risk assessments: climate change allowances. Accessed 16/08/2019. Available from: https://www.gov.uk/guidance/flood-risk-assessments-climate-change-allowances

[12] Ribble Valley Borough Council (2017) Strategic Flood Risk Assessment. Accessed 16/08/2019. Available from: https://www.ribblevalley.gov.uk/download/downloads/id/11030/strategic_flood_risk_assessment_level_1_revised_2017.pdf

[13] UK Government. 2019. Long term flood risk information. Accessed 16/08/2019. Available from: https://flood-warning-information.service.gov.uk/longterm-flood-risk/map

Figure 3.3: Risk of flooding from reservoirs (extract from long term flood risk map)



Source: UK Government, 2019. © Crown copyright and database rights 2018 OS 100024198. Contains public sector information licensed under the Open Government License v3.0.

3.6 Water treatment works infrastructure

Should treatment infrastructure at the site become blocked or malfunction, flooding may occur by this mechanism. However, appropriate operation and maintenance of assets however should mitigate this risk.

3.7 Access and egress

Access to the site is from the west, which lies outside of the Flood Zones. The road crosses Phynis Beck via a small bridge. If the bridge became blocked, localised flooding of the road may occur. It is likely that this would pass quickly, as the catchment area contributing to the beck is small.

4 Assessment of flood risk as a result of the development

4.1 Fluvial flooding

4.1.1 Pass forward flows from reservoir

Raising the weir will slightly increase attenuation in reservoir, as the total surface area will increase. The efficiency of the weir will not change, as the cross section and materials are to remain as current. With greater attenuation in the reservoir, pass forward flows to the river could reduce slightly. A compensation flow will be maintained to the river which will be unaffected by the works. Therefore, no increase to fluvial flood risk elsewhere is anticipated as a result of raising the spillway weir crest level.

4.1.2 Loss of floodplain storage

Analysis of flood flows in Section 3.1.2 indicated that the proposed development will be located on ground elevated above the maximum flood level up to and including the 0.1% AEP event. Therefore, there will be no loss of flood plain storage as a result of the proposed development. Therefore, no increase to fluvial flood risk elsewhere is anticipated as a result of raising the construction of the proposed development.

4.2 Surface water flooding

The proposed development will result in an increase in impermeable area of approximately 1,600m^2. This represents approximately 1% of the total site area. The runoff generated as a result of the increase in impermeable area will be captured by the existing drainage network which discharges to the River Hodder. No increase to surface water flood risk elsewhere as a result of the proposed development is anticipated.

4.2.1 Management of surface water flood risk during construction

It is recommended care is taken to ensure materials are not washed into the drainage system causing blockages which could lead to localised flooding. Existing drains around the works should be investigated to ensure there has been no damage during construction.

4.3 Groundwater flooding

The proposed development will not alter ground levels significantly or involve significant below ground works. No increase to ground water flooding elsewhere is anticipated as a result of the proposed development.

4.4 Flooding from sewers and drains

No additional flows to sewers or surface water drains leaving the site are anticipated as a result of the proposed development. Therefore, there will be no increase to flood risk from sewers elsewhere as a result of these works.

4.5 Reservoir flood risk

4.5.1 Upstream flood risk

Works to raise the overflow weir of Stocks Reservoir by 300mm will result in an increase of the top water level of the reservoir by an equivalent amount to a level of 180.87mAOD. Similarly, during flood events, water levels will increase by up to 300mm compared to the baseline, with the potential to flood upstream receptors. Reservoir levels for the key flood events are presented in Table 4.1.

Table 4.1: Reservoir water levels during flooding

Flood event/scenario	Water level pre-development (mAOD)	Estimated water level post-development (mAOD)
Reservoir at top water level (TWL)	180.57	180.87
1% AEP	181.39	181.69
1% AEP +30%CC	181.66	181.96
1% AEP +70%CC	182.21	182.51
0.1% AEP	182.56	182.86

An analysis of the terrain was undertaken to identify any receptors lying below the new 0.1% AEP flood level. No receptors will be affected by the increase in water level up to and including the 0.1% AEP flood event. The lowest lying potential receptor upstream of the reservoir was identified as the Hole House Lane bridge at SD737560 with a deck level of approximately 184mAOD[14]. The rate of water level rise in the reservoir is anticipated to be sufficiently low so as to not cause differential loading on the bridge.

Therefore, the flood risk to upstream receptors from the reservoir will not significantly change as a result of the works.

4.5.2 Downstream flood risk

Increasing the top water level will also result in an increase in capacity within the reservoir. Should a breach of the reservoir embankment occur, the extent of reservoir flooding downstream may be increased. Under the Reservoirs Act 1975, the Environment Agency ensures that reservoirs are inspected regularly by reservoir safety panel engineers and that essential safety works are carried out where required. The probability of failure is therefore considered to be low, with limited impact to the overall change in reservoir flood risk downstream.

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5 Application of the National Planning Policy Framework

5.1 National Planning Policy

This section provides an overview of the flood risk specific planning context. Further details on the wider planning context are provided in the accompanying Planning Statement.

5.1.1 National Planning Policy Framework

The NPPF sets out the Government's planning policies for England and defines how these are to be applied. The associated PPG on flood risk provides additional guidance to local planning authorities to ensure the effective implementation of the planning policy set out in the NPPF, on development in areas at risk of flooding.

As set out in the NPPF, inappropriate development in areas at risk of flooding should be avoided by directing development away from areas at highest risk, but where development is necessary, making it safe without increasing flood risk elsewhere. For purposes of applying the NPPF: 'areas at risk of flooding' means land within Flood Zones 2 and 3: or land within Flood Zone 1, which the Environment Agency has notified the local planning authority as having critical drainage problems; and 'flood risk' means a combination of the probability and the potential consequences of flooding from all sources- including from rivers and the sea, and directly from rainfall on the ground surface and rising groundwater, overwhelmed sewers and drainage systems, and from reservoirs, canals and lakes and other artificial sources.

The stated overall aim of the NPPF is to steer new development to Flood Zone 1. If following the application of the Sequential Test, it is not possible for the development to be located in zones with a lower probability of flooding, the Exception Test can be applied if deemed appropriate.

5.1.2 The Sequential Test

The aim of the sequential test is to steer the new developments to locations in Flood Zone 1, where the flood risk is lowest. Due to the nature of the Site, its existing assets and water treatment works facilities, no alterative location is suitable for the development.

The proposed development is categorised as 'essential utility infrastructure' which is classed as essential infrastructure in the PPG[15], and therefore deemed an appropriate use of land located in Flood Zone 2 but requires the Exception Test to be applied in Flood Zone 3.

The works to increase level of the spillway weir fall under Permitted Development. Nevertheless, as water compatible infrastructure, these works would be allowable within any of the Flood Zones.

[15] Department for Communities and Local Government. 2012. Technical Guidance to the National Planning Policy Framework. Paragraph 5, Table 2: Flood risk vulnerability classification. [Online] Available at: https://assets.publishing.service.gov.uk/government/uploads/system/uploads/attachment_data/file/6000/2115548.pdf. [Accessed 17/06/2019]

Table 5.1: Flood risk vulnerability classification

Flood Zones	Flood risk vulnerability classification				
	Essential infrastructure	**Highly vulnerable**	**More vulnerable**	**Less vulnerable**	**Water compatible**
Zone 1	✓	✓	✓	✓	✓
Zone 2	✓	Exception test required	✓	✓	✓
Zone 3a	Exception test required	x	Exception test required	✓	✓
Zone 3b	Exception test required	x	Exception test required		✓

Key:

✓ Development is appropriate

x Development should not be permitted

Source: Planning Practice Guidance (2019)

5.1.3 Exception Test

Essential infrastructure which is located in Flood Zone 3 as shown on the Flood Map for Planning must pass the Exception Test under the NPPF[16] and PPG[17].

The test is formed of two parts:

- Demonstrate wider sustainability benefits to the community that outweigh flood risk
- Demonstrate that the development will be safe for its lifetime.

The proposed development is required as part of United Utilities strategy to safeguard potable water supply to the north west region against the increasing risk of drought. This flood risk assessment has identified that the proposed development will not be at risk up to and including the 0.1% AEP flood event. The proposed development will not be at risk in the 1% AEP event, including an allowance of 70% for climate change. Furthermore, the proposed development will not increase flood risk elsewhere.

It is therefore considered that the proposed development meets the exception test and should be allowed in Flood Zone 3.

[16] Ministry of Housing, Communities & Local Government (2019) National Planning Policy Framework. Accessed 16/08/2019. Available from: https://www.gov.uk/government/publications/national-planning-policy-framework--2

[17] Ministry of Housing, Communities & Local Government (2019) Planning Practice Guidance. Accessed 16/08/2019. Available from: https://www.gov.uk/government/collections/planning-practice-guidance

6 Conclusions

6.1 Flood risk to the proposed development

The site is partially situated within Flood Zones 2 and 3 as defined by the Flood Map for Planning. However, in this area, the Flood Zones are based on the NGM undertaken in 2004 which does not represent flow routing through reservoirs. Therefore, the Flood Zones do not represent real fluvial flood risk at the site.

Stocks Reservoir is a Category A reservoir, which means it is designed to safely pass the PMF flow by the spillway, before the embankment is overtopped. Therefore, in 1% AEP and 0.1% AEP events, flood waters would flow down the spillway rather than over the embankment and into an engineered channel adjacent to the site. An estimate of the resulting maximum water level in the channel indicated that the proposed development would not be at risk of flooding up to and including the 0.1% AEP flood events. Similarly, the proposed development would not be at risk during the 1% AEP flood event including an allowance of 70% for climate change.

The proposed development is not considered to be at risk of surface water, groundwater flooding or sewer flooding. The treatment works themselves pose a risk should infrastructure become blocked or malfunction, however, this is considered to be managed through ongoing operation and maintenance of the treatment works by trained operatives.

Should a breach of the Stocks Reservoir embankment occur, the site will be inundated. Under the Reservoirs Act 1975, the Environment Agency ensures that reservoirs are inspected regularly by reservoir safety panel engineers and that essential safety works are carried out where required. Therefore, reservoir flooding is not considered to be a significant risk to the proposed development.

Access to and from the site may be affected in the case of blockage of a road bridge over Phynis Beck causing localised flooding to the access road. However, it is likely that any flooding would pass quickly given the small size of the Phynis Beck catchment.

6.2 Flood risk resulting from the proposed development

The proposed development will not increase fluvial flood risk elsewhere, and flood flows are likely to be slightly attenuated downstream by increasing the capacity of the reservoir.

The proposed will not result in an increase to surface water flooding, sewer flooding or groundwater flooding elsewhere.

Increasing the water level of the reservoir will not result in flooding to any additional upstream receptors. The reservoir water level during flood events will increase by up to 300mm. This may increase loading on the Hole House Lane road bridge during flood events. No new receptors were identified below the maximum elevation anticipated during the 0.1% AEP flood event.

Increasing the capacity of the reservoir means that should a breach of the embankment occur, the resulting extent of flooding downstream may increase. However, under the Reservoirs Act reservoirs must be inspected and maintained regularly, and so the risk of failure remains low. Therefore, the overall risk elsewhere as a result of the works will not be significantly changed.

6.3 Compliance with National Planning Policy

Due to the nature of the site, its existing assets and water treatment works facilities, no alterative location is suitable for the proposed development.

The proposed development is categorised as essential infrastructure, deemed an appropriate use of land located in Flood Zone 2, but requires the Exception Test to be passed within Flood Zone 3.

The proposed development is required as part of United Utilities strategy to safeguard potable water supply to the north west region against the increasing risk of drought. This flood risk assessment has identified that the proposed development will not be at risk up to and including the 0.1% AEP flood event and that it will not be at risk in the 1% AEP event, including an allowance of 70% for climate change.

It is therefore considered that the proposed development meets conditions of the exception test and should be allowed in Flood Zone 3.

Appendices

A. Stocks Reservoir Flood Study

Stocks Reservoir

Flood Study and Review of Wave Analysis

April 2015

JACOBS

Document Control Sheet

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Client: United Utilities   Project No: B16000EP
Document title: Stocks Reservoir Flood Study and Review of Wave Analysis
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	Originated by	Checked by	Reviewed by	
ORIGINAL	NAME **Ross Holman**	NAME **David Price/Paul Roberts**	NAME **John Gosden** **David Price** **Mathieu Valois**	
Approved by	NAME **Ed McGurin**	As Project Manager I confirm that the above document(s) have been subjected to Jacobs' Check and Review procedure and that I **approve them for issue**		INITIALS
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DATE		**Document status**		

Contents

1 Introduction

1.1 Background

Stocks Reservoir is located approximately 25km east of Lancaster and 2km north of Slaidburn, a small village in the Forest of Bowland (an Area of Outstanding Natural Beauty), Lancashire. The reservoir is located in the Hodder Valley with the outflow from the reservoir discharging into the River Hodder which flows through Slaidburn. Stocks Reservoir has two watercourses feeding into it: River Hodder located to the north of the reservoir and Bottoms Beck located to the east of the reservoir.

In 2012[1], United Utilities undertook a study to revisit analysis for Stocks Reservoir to determine whether there is scope to increase the TWL of the reservoir to provide additional storage for supply to the Hodder WTW. The study concluded that the amount of freeboard was potentially conservative and there was an estimated 570mm of additional freeboard available. This additional freeboard was calculated to be equivalent to an additional eight days of supply if the reservoir top water level was increased by this amount.

However, the peak still water level associated with the PMF event was last calculated in 1969[2] by the then Fylde Water Board. There is therefore a need to update the PMF estimate in accordance with the latest guidance and standards.

1.2 Objectives

United Utilities have commissioned Jacobs UK Ltd to undertake a flood study for the Stocks Reservoir in order to:

i. Update the PMF Study.
ii. Review the wave analysis.
iii. Based on the outcome of i and ii, re-assess the available freeboard.
iv. Recalculate the available storage should the TWL be raised to reduce freeboard to zero.
v. Assess the effect of dam overtopping and consider if this is permissible.

[1] United Utilities (2012): Technical Report Stocks IR Wave Surcharge
[2] Fylde Water Board (1969): Report on Stocks Reservoir

2 Study Site

Stocks Reservoir is located near the village of Slaidburn in the Ribble Valley district of Lancashire (Figure 2-1).

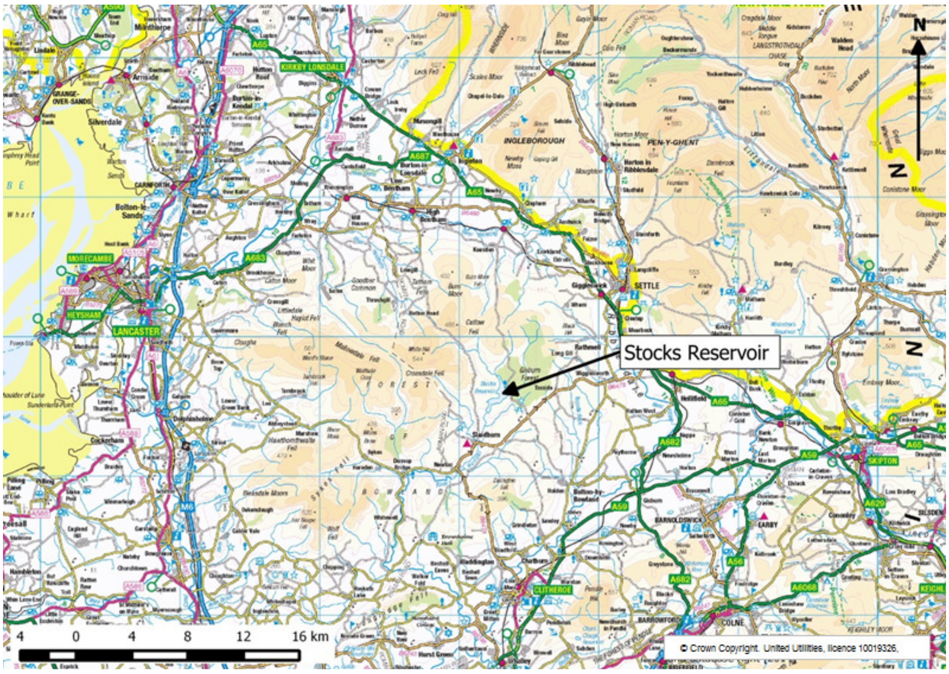

Figure 2-1 Location of Stocks Reservoir

Stocks Reservoir receives direct inflow from two primary watercourses: River Hodder and Bottoms Beck. Downstream of the reservoir the River Hodder runs in an open channel for approximately 30km until it reaches its confluence with the River Ribble. Further reservoir details are shown in Figure 2-2 below. Characteristics of the reservoir are detailed in Table 2-A.

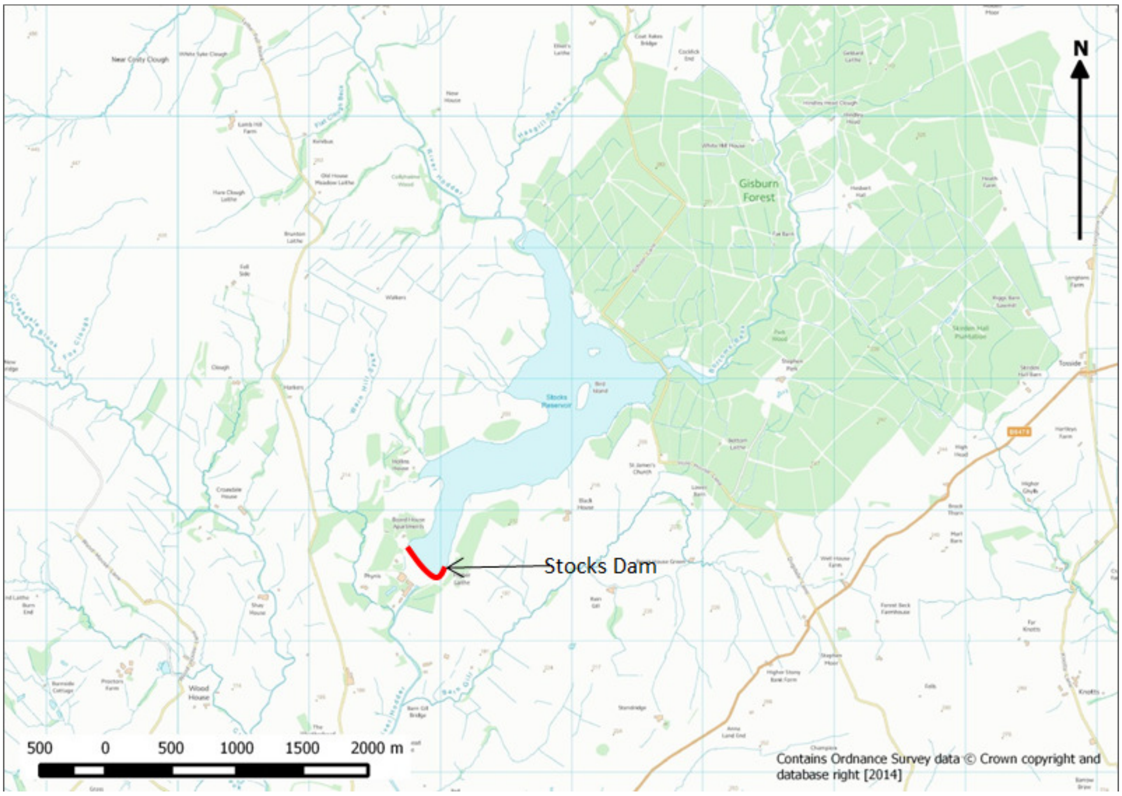

Figure 2-2 Overview of Stocks Reservoir

Parameter	Value	Source
Surface area at Full Supply Level (km^2)	1.32	OS 25k mapping
Spillway crest level (mLD*)	180.57	2014 topographic survey/2011 Section 10 report
Spillway weir width (m)	91.44	2014 topographic survey/2011 Section 10 report
Spillway weir coefficient	1.70	ISIS Default for broad crested weir. Acceptance of default coefficient based upon photographic evidence.
Minimum dam crest level (mLD)	183.71	2014 topographic survey
Dam crest length (m)	350	2014 topographic survey
Wave wall minimum level (mLD)	184.60	2011 Stocks S10 report

Table 2-A Reservoir details

*Note all levels in this report are given to the reservoir local datum in line with the prescribed form of record. OS datum = Local Datum minus 0.07m.

3 Methodology

The general approach was to develop an integrated hydrological and hydraulic model of Stocks Reservoir using the ISIS modelling package. Key tasks in this study are:

i. Construction of ISIS hydraulic routing model
   - An ISIS hydraulic routing model representing the Stocks reservoir was built using recent topographic survey data. A composite reservoir discharge rating was developed independently for the overflow using hand calculations, accounting for progression from modular to drowned flow and also considering submergence effects of the tumble bay.

ii. Derivation of inflow
   - PMF flows were derived following the methodology and guidance given in the "Floods Studies Report (FSR)"[3] in combination with the approaches as stated in the "Flood Estimation Handbook (FEH) Vol. 4"[4].

The following study specific considerations were made:

i. Refinement of the rainfall-runoff parameters (Tp(0) and SPR) using a donor catchment was undertaken (as recommended in FEH Vol 4).

ii. Whether HOST Class 4 soils are present in the Stocks Reservoir catchment. HOST Class 4 is prevalent in the north west of Britain and has been linked by some[5] to a significant underestimate of the Standard Percentage Runoff (SPR) catchment parameter. The flood volumes and peak flows simulated by the rainfall-runoff model are relatively sensitive to this parameter.

iii. Given the upland nature of the catchment the potential need for a higher snowmelt rate than 42mm/day was investigated.

iv. For PMF scenarios, the level of the reservoir at the start of the simulation is required to be set to a level that permits the long-term average catchment flow (Qmean) to pass. Qmean was estimated using the following equation:

   $$Qmean = 1.06 \times SAAR_{1961\text{-}1990} - \text{Average Annual PE}$$

   Where $SAAR_{1961\text{-}1990}$ = standard average annual rainfall for the period 1961 – 1990; 1.06 is a factor required to correct the under catch of standard Met Office rain gauges[6, 7] and PE = potential evaporation.

---

[3] Natural Environment Research Council (1975), Flood Studies Report
[4] Institute of Hydrology (1999), Flood Estimation Handbook, Volume 4.
[5] Davison, (2005), Concern over catchment run-off estimation. Dams & Reservoirs, Vol 15, Number 1.
[6] Rodda J & Smith S, 1986. The significance of the systematic error in rainfall measurement of assessing wet deposition. Atmos. Environ. 20 Pp 1059 – 1064.
[7] Price DJ, 1999, Systematic error of standard UK rain-gauges in the Central Scottish Highlands. Weather, October 1999, Vol. 54, No. 10.

4 Catchment Hydrology

Figure 4-1 shows the extent of the Stocks Reservoir catchment. The key default FEH catchment descriptors (obtained from FEH CDROM V3) are given in Table 4-A.

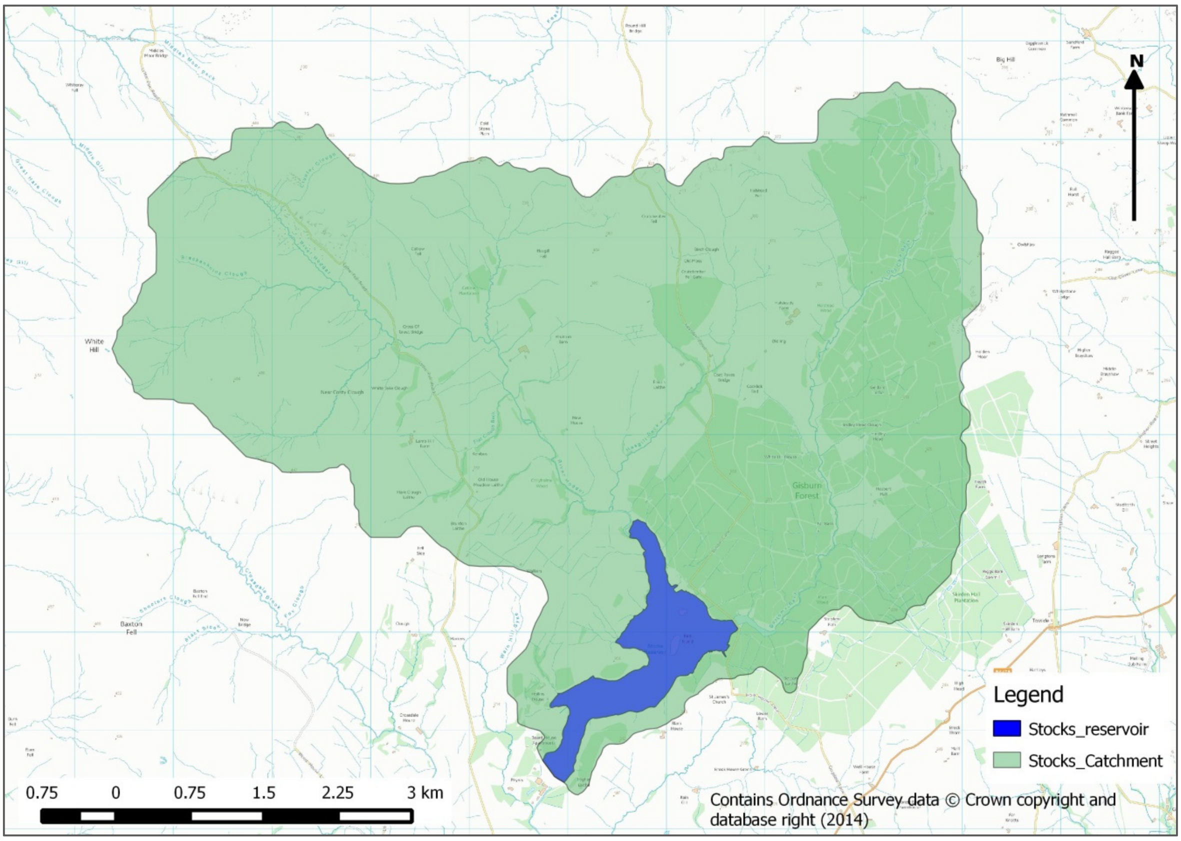

Figure 4-1 Stocks Reservoir Catchment area

Catchment Descriptor	Stocks Reservoir Catchment
Area (km^2)	37.51
DPLBAR (km)	6.6
DPSBAR (m/km)	114.2
PROPWET	0.6
SAAR (mm)	1658
SPRHOST	50.44
Tp(0) (hr)	3.39

Table 4-A Summary of the key FEH catchment descriptors for Stocks Reservoir

Table notes:

DPLBAR (Average drainage path length) – an index describing the catchment size and drainage path configuration.
DPSBAR (Average drainage path slope) – an index of catchment steepness.
SAAR (Standard Average annual rainfall) – calculated for the period 1961 – 1990.
SPRHOST (Standard Percentage Runoff Hydrology of Soil Types) – an index of how impermeable a catchment is under average climatic conditions.
Tp(0) (Instantaneous time to peak) – a derived parameter that represents the speed of response of the catchment. (Not subject to the PMF adjustment factor of 0.667)

4.1 Rainfall parameters

The precipitation parameters required for estimating PMP rainfall depths and snowmelt rates are presented in Table 4-B.

Parameter	Stocks Reservoir	Source
EM-2hr (mm)	158	FSR Volume V maps
EM-24hr (mm)	310	FSR Volume V maps
EM-25d (mm)	550	FSR Volume V maps
S100 (mm)	150	FEH Volume 4
Snowmelt (mm/d)	42	FSR

Table 4-B FSR precipitation parameters

4.2 Donor catchment refinement of rainfall-runoff model parameters

The flood event analysis archive given in Appendix A of FEH Vol 4 includes detailed analyses of 24 floods for the similarly sized and adjacent gauged catchment: Croasdale Beck at Croasdale Flume (Stn No 71003). This catchment is judged to be hydrologically similar to that of the Stocks catchment. The resulting SPR and Tp(0) estimates are considered by the FEH to offer the best means of estimating these parameters, and as such should be considered superior estimates to those derived from the FEH CDROM catchment descriptors. The relative sizes of the estimates can be used to refine FEH catchment descriptor estimates for hydrologically similar adjacent catchments. Table ***4-C*** compares the estimates obtained from both methods. The FEH catchment descriptor derived values match well with those obtained from site specific data. On this basis the FEH catchment descriptor estimates for the Stocks catchment are considered likely to be reliable estimates. It is recognised that the FEH catchment descriptors derived Tp(0) is slightly shorter and that accepting its use within the Stocks study will result in a slightly more conservative assessment. Based on experience the agreement between the two methods is remarkably good and allows the project to have greater confidence in the parameter values used than would have been the case had only the default FEH catchment descriptors been available.

Parameter	Source of estimate	
	Flood event analysis	FEH catchment descriptors
SPR (%)	54	54.51
Tp(0) [hr]	2.3	2.19

Table 4-C Comparison of flood event analysis and FEH catchment descriptor estimates of Tp(0) and SPR for adjacent donor catchment.

4.3 HOST Class 4

Investigation of the presence of HOST Class 4 soils within the Stocks catchment was undertaken (Appendix B). Slightly less than 2% of the total Stocks catchment was estimated to be covered by HOST Class 4 soil. This HOST Class is given a SPR value of 2%[8]. Had the SPR estimate been 20% (as suggested by an

[8] Boorman DB, Hollis JM & Lilly A, 1995. Hydrology of soil types: a hydrologically-based classification of soils of the United Kingdom. Institute of Hydrology Report No. 126

alternative, though at the time less favoured, Institute of Hydrology methodology) then the catchment SPRHOST would have risen by 0.36% (i.e. from 50.44% to 50.80%). This has a very small impact upon the predicted runoff, and coupled with the finding that the PMF event is formed by the winter event in which the SPR is fixed to 53% anyway, it was not judged necessary to amend the catchment SPR estimate from that provided by the FEH catchment descriptors.

The extent of HOST Class 4 in the donor catchment (Croasdale Beck at Croasdale Flume (Stn No 71003) was also calculated and estimated to be 2%. This is almost identical to that of the Stocks catchment suggesting that this issue does not complicate the interpretation of the donor catchment assessment described in Section 4.2.

4.4 Snowmelt

In the winter PMF study two values of snowmelt rate have been used:
42mm/day; the standard value suggested by FSR
65mm/day; since the map in Floods and Reservoir Safety 3rd Edition (Institution of Civil Engineers, 1996) suggested the target site was just inside the area which may be prone to experiencing higher snowmelt rates.

The empirical equations of Hough and Howlis (1997[9]) that relate climatic and location parameters to snowmelt rates were used to derive catchment values for the 100-year daily snowmelt rate (Appendix C), resulting in the 65mm/day snowmelt rate proposed above.

The subsequent analysis (section 6.2) indicated that the peak reservoir water level is sensitive to the snowmelt parameter. As a result the design case has used a snowmelt rate of 65mm/day, with a rate of 42mm/day reported for comparison.

4.5 Reservoir Lag

The RLAG iterative procedure was undertaken for PMF summer and winter events. The Winter PMF resulted in a higher peak stillwater level than the Summer PMF. Table 4-D provides the predicted RLAG and critical duration for both events.

Return Period (yrs)	RLAG (hrs)	Critical duration (hrs)
PMF Summer	2.08	11.7
PMF Winter	2.17	11.9

Table 4-D Critical duration of Stocks Reservoir for summer and Winter PMF events.

[9] Hough MN and Howlis D, 1997. Rare snowmelt estimation in the United Kingdom. Metreorol. Appl. 5, 127-138

5 Hydraulic Model

Version 3.7.0 of the ISIS river modelling software package was used for modelling the Stocks Reservoir system. The double precision engine was used to ensure model accuracy.

5.1 Model Schematisation

Initially the ISIS model fully represented the Stocks reservoir and the spillway arrangement which is composed of three 2.4m diameter pipes with a chute above the pipes. The chute operates as an overflow when the pipe capacity is exceeded. The spillway chute and pipes re-join in the stilling basin approximately 180m downstream of the inlets (Photographs showing the reservoir spillway are illustrated in Figure 5-1).

Figure 5-1 Stocks reservoir spillway photographs

i. Looking downstream from tumble bay
ii. Looking upstream to tumble bay

However, the steep nature of the spillway chute and pipes resulted in the modelled headloss at the pipe inlets being high due to the high velocity in the chute; this resulted in the estimated flows within the pipes being lower than expected (this was confirmed by hand calculations).

As such it was decided that since the primary objective of this study was to estimate the peak still water level of the PMF event, the model should be simplified by removing the spillway component and manually deriving a rating curve for the Stocks Reservoir spillweir (see Section 5.3).

The simplified representation of the Stocks reservoir routing model is illustrated in Figure 5-2 below. It is composed of the following components:

- A reservoir unit representing the available reservoir storage
- A weir unit represented by a simplified overflow arrangement
- Downstream boundary

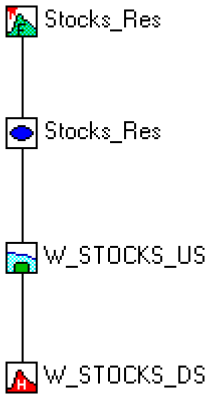

Figure 5-2 ISIS Stocks reservoir routing model schematisation

5.2 Reservoir Storage

Storage available in Stocks reservoir was represented using an ISIS reservoir unit informed with an area/elevation relationship.

Available topographical survey drawing[10] was used to determine the reservoir geometry; however, the survey drawing did not include contour data around the entirety of the reservoir. As such, bank profile gradients were derived for a number of locations for which contour data was available which allowed an area/elevation relationship to be derived (see Table 5-A).

Level (mLD)	Area (m^2)	Source
180.57	1320000	2014 Topographical survey
183.57	1368000	2014 Topographical survey

Table 5-A Derived area/elevation relationship for Stocks reservoir

5.3 Overflow Modelling

Stocks Reservoir has a single primary overflow which discharges into a tumble bay where the flows turn through 90 degrees and proceeds down to the relatively complex spillway chute structure (composed of 3no. pipes and a spillway overflow chute directly above the pipes. The Spillway arrangement for Stocks Reservoir is presented in Figure 5-3 below.

It is noted that the hydraulic performance of the overflow structure is relatively complex due to the immediate 90 degree change in flow direction in the tumble bay and the hydraulic interactions between the 3no. pipes and the overflow spillway. The

[10] United Utilities (2010) 0304_NL01_A.dwg – Topographical survey at Stocks reservoir

hydraulic performance of the overflow structure has been shown to be outside the normal operating parameters of standard 1D hydraulic modelling tools such as ISIS. The representation of the overflow arrangements was therefore simplified and assessed using a range of standard 1D hydraulic calculations (see Appendix E) to develop a composite reservoir discharge rating curve (Table 5-B). The derived rating represents the progression of flow from free broad crested weir equation to downstream channel control (Figure 5-4).

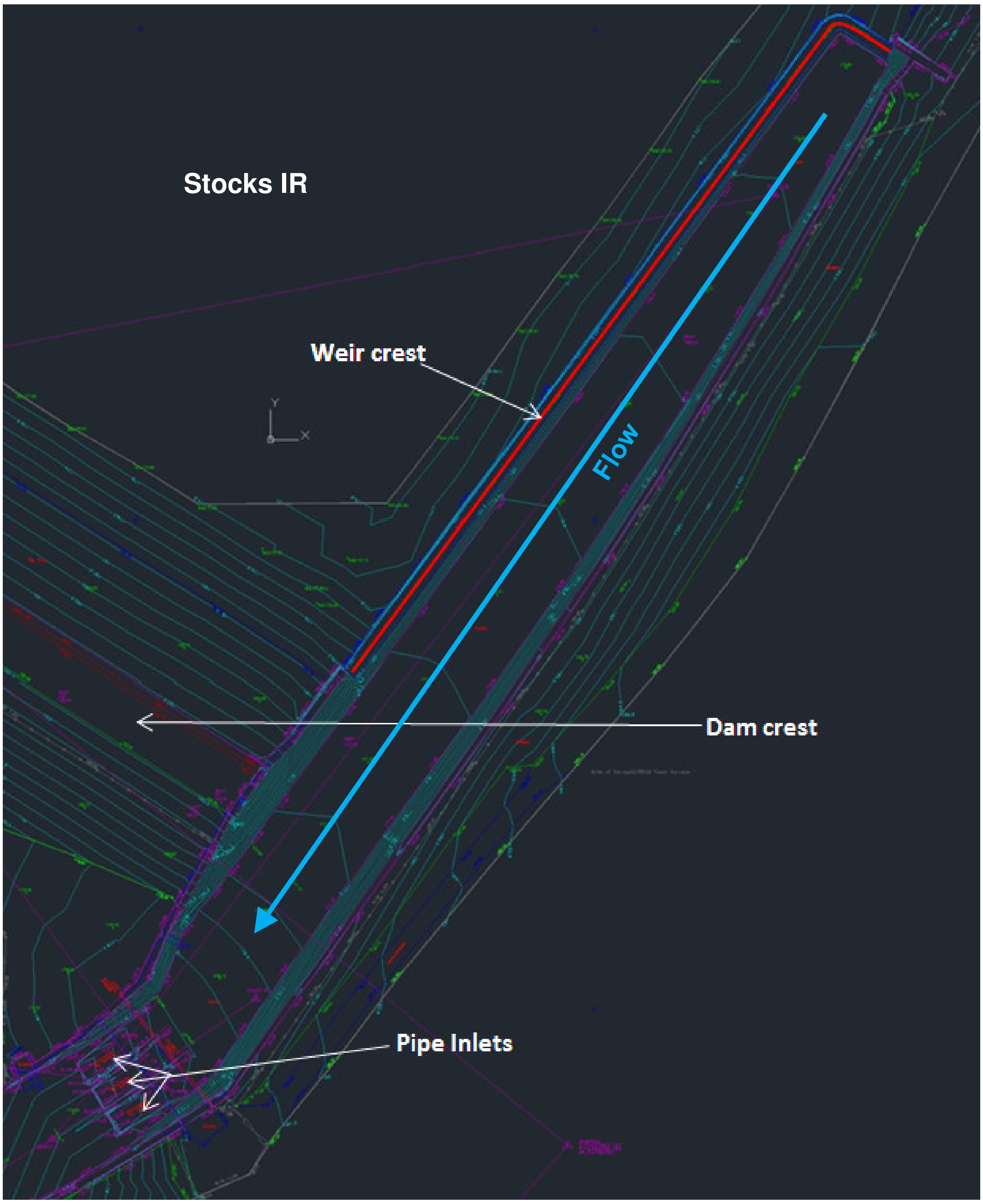

Figure 5-3 Spillway arrangements for Stocks Reservoir (Topographic survey 2014)

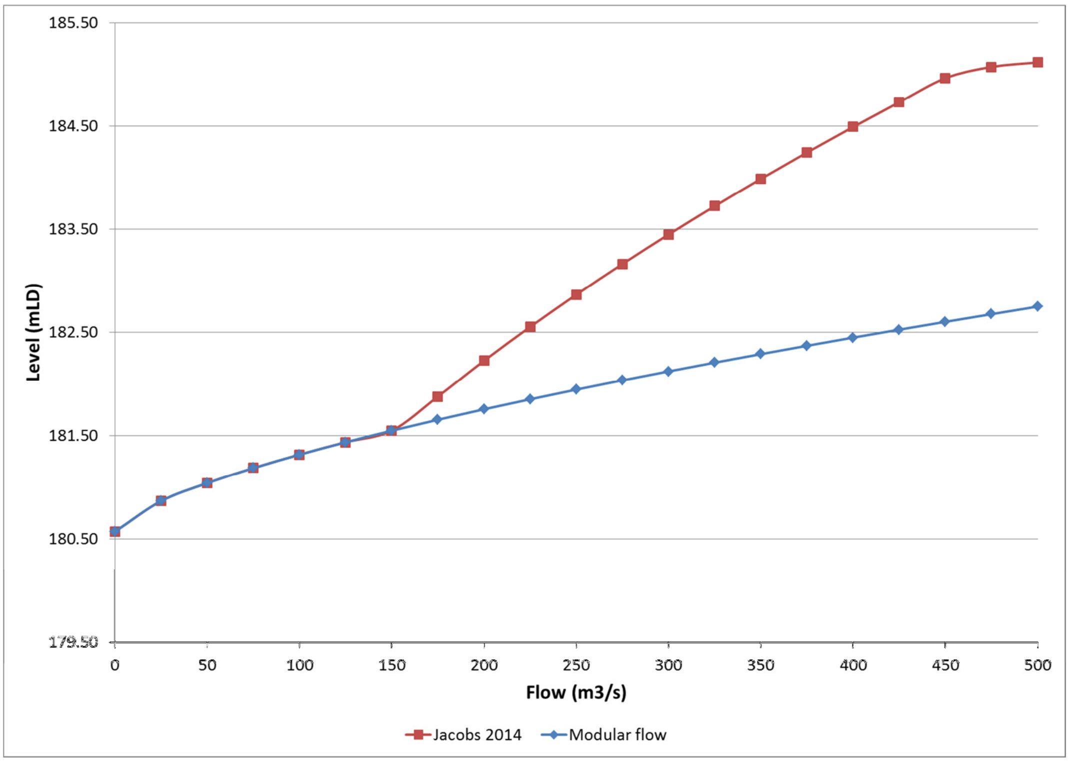

Figure 5-4 – Stocks Reservoir discharge rating curves
Jacobs 2014 = composite overflow weir rating derived for the current study
Modular Flow = Broad crested weir rating, for comparison

Flow (m^3s^{-1})	Reservoir Level (mLD)	Flow Description	Dam Overtopping?	Freeboard to wave wall crest (m)
0	180.57	weir control	flood contained	4.03
25	180.87	weir control	flood contained	3.73
50	181.04	weir control	flood contained	3.56
75	181.19	weir control	flood contained	3.41
100	181.32	weir control	flood contained	3.28
125	181.43	weir control	flood contained	3.17
150	181.55	weir control	flood contained	3.05
175	181.88	channel control	flood contained	2.72
200	182.22	channel control	flood contained	2.38
225	182.55	channel control	flood contained	2.05
250	182.86	channel control	flood contained	1.74
275	183.16	channel control	flood contained	1.44
300	183.45	channel control	flood contained	1.15
325	183.72	channel control	dam overtopped	0.88
350	183.99	channel control	dam overtopped	0.61
375	184.24	channel control	dam overtopped	0.36
400	184.49	channel control	dam overtopped	0.11
425	184.73	channel control	wave wall overtopped	-0.13
450	184.96	channel control	wave wall overtopped	-0.36
475	185.07	channel control	wave wall overtopped	-0.47
500	185.12	channel control	wave wall overtopped	-0.52

Table 5-B Stocks Reservoir spill-weir rating curve

5.4 Downstream Boundary

Flow out of the model is represented as an ISIS stage/time (H/T) boundary unit set to a constant level nominally low to provide free flow conditions.

6 Results

6.1 Design Case Modelling

The PMF event was routed through the Stocks Reservoir model with the key model results provided in Table 6-A for the critical storm event.

A winter storm event of 11.9 hours has been determined to be the critical PMF event following MRlag analysis (see Section 4.5 for further details). Water level profiles are shown with the embankment and (minimum) wave wall profile for the critical winter PMF event in Figure 6-1. Details of wind wave calculations are provided in Section 7.

	Summer PMF	Winter PMF
Top water level (mLD)	180.57	180.57
Critical storm duration (hrs)	11.7	11.9
Peak Inflow (m^3/s)	405.82	475.39
Peak Outflow (m^3/s)	231.10	282.69
Flood surcharge (m)	2.06	2.68
Peak still water flood level (mLD)	182.63	183.25
Minimum dam crest level (mLD)	183.71	183.71
Available freeboard to dam crest (m)	1.08	0.46
Wind wave surcharge (m)	1.05	1.05
Peak flood & wave surcharge level (mLD)	183.68	184.30
Minimum wave wall level (mLD)	184.60	184.60
Flood & wind-wave freeboard to wave wall (m)	0.92	0.3

Table 6-A Key model results

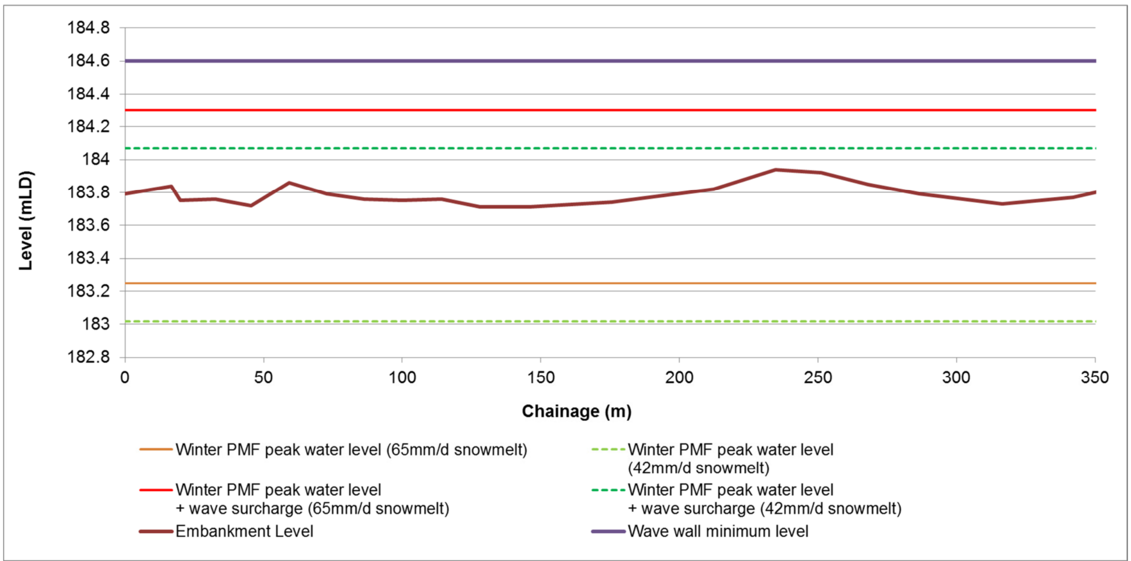

Figure 6-1 Stocks reservoir water levels, Critical Winter PMF scenario. NB: embankment profile is presented “looking upstream”.

6.2 Sensitivity Testing – Snowmelt

As discussed in Appendix B, in some upland regions of the UK there is evidence to suggest that higher rates of snowmelt may be more appropriate than the default UK rate of 42mm/day. Regression analysis undertaken for the Stocks Reservoir catchment suggested that a higher snowmelt rate of 65mm/day is justified. The results for the winter PMF simulations using snowmelt rates of 42mm/day and 65mm/day are presented in Table 6-B.

	42mm/day snowmelt	65mm/day snowmelt
Critical storm duration (hrs)	11.9	11.9
Peak Inflow (m^3/s)	442.47	475.39
Peak Outflow (m^3/s)	263.20	282.69
Peak stillwater flood level (mLD)	183.02	183.25

Table 6-B Winter PMF snowmelt sensitivity analysis

Results indicate that the peak flood level is sensitive to the value of snowmelt rate adopted and since the higher snowmelt rate of 65mm/day can be justified this will be taken as the design case. The design case results in a remaining freeboard above the wave surcharge allowance of 300mm.

With a snowmelt rate of 42mm/day the amount of runoff decreases, leading to a 7% reduction in peak inflow. As a result the peak still water flood level in the reservoir reduces by 230mm to a level of 183.02mLD. Including wind wave surcharge results in a peak flood and wave surcharge level of 184.07mLD giving a remaining freeboard of 530mm.

It is recommended that the available reservoir storage analysis uses the design case 65mm/day snowmelt rate still water level results from the winter PMF simulation. As such the available freeboard to the wave wall crest for the critical winter PMF event is 300mm (Table 6-A).

7 Wave Surcharge Analysis and wave wall overtopping

Wind wave surcharge was calculated using the standard methodology[11,12] including Eurotop. It is unlikely a fetch distance from the northern most point of the reservoir to the dam is possible due to the almost 90° change in direction of the reservoir. Additionally, for waves to be funnelled in a “banana effect” the south east shore would require very steep banks to contain and channel the wind in a south westerly direction. The south east shore is relatively flat (rising 50m over a distance of 1.7km). Therefore, the most realistic fetch length was determined to be approximately 2.4km from the North Eastern shore.

The upstream face of the dam consists of a 1v:3h slope surfaced with large block pitching with an approximately 0.8m high wave wall at the crest. Behind the wave wall the crest is approximately 4.8m wide and has a grass surface. A tarmac access road across the embankment is around one third of the way down the downstream face. The downstream face is at a slope of 1v:2.5h with two wide intermediate berms and has a good coverage of grass. Given the good grass cover the downstream face would have some resistance to wave overtopping discharge. However waves overtopping the wave wall will be deflected and drop onto the crest (and possibly the upper part of the downstream face) with some force and could result in erosion and possible undermining of the wave wall. As a result only a very low amount of overtopping can be permitted.

Two methods have been used to assess the required wave surcharge; EuroToP[12] method of calculating overtopping discharge and Floods and Reservoir Safety[11]. The estimation of the significant wave height is common to both methods. The calculation of significant wave height and FRS wave surcharge is set out in Table 7-A below. The ratio of design wave height to significant wave height is set to 1.3, which is the recommended value for an embankment with a grass crest and grassed downstream face. This represents a limited amount of wave overtopping, with 4% of waves being higher than this value. If no wave overtopping were permitted this factor would increase to 1.67, which would result in a wave surcharge allowance of 1.35m.

[11] ICE (1996), Floods and Reservoir Safety, 3rd Edition.
[12] EuroTop Wave Calculation Tool

U50=	23.5	m/s	50 year maximum hourly wind speed reduced to sea level (from Fig.3 presenting a wind speed map taken from BS6399)
fT=	0.79	-	Adjustment factor for estimating the mean annual maximum hourly wind speed - FIXED VALUE
Altitude=	183.71	mLD	Altitude of embankment crest
fA=	1.18	-	Adjustment factor for altitude
F=	2.4	km	Fetch is generally determined from the point where there is maximum potential for breaching (lowest point of the top of the dam). Longest available fetch used = conservative
fw=	1.18	-	Overwater adjustment factor lookup table 4 ICE. If fetch less than 1000m use value of 1.1
fD=	1.02	-	Duration factor (to convert the hourly wind speed to 10-20min duration for full development of waves - typical for UK reservoirs) (Source: CIRIA Special Publication No. 83/CUR Report SR 345) - FIXED VALUE
Fetch dir =	30	deg	Fetch direction (degrees from North). GIS auto calculation.
fN=	0.73	-	Wind direction adjustment factor (from Table 5 allows for the orientation of the principal axis of the reservoir with respect to 'general UK' wind direction). Regional data on wind direction could be used for each specific site. Lookup table based on guide
U=	19.25	m/s	Required wind speed
Hs=	0.54	m	Significant wave height for extreme conditions on the reservoir (mean height of the highest third of all waves) - Donelan/JONSWAP method
f=	**1.3**	-	Factor to be applied to Hs in order to estimate the design wave height H_D
HD=	**0.70**	m	design wave height
RF=	1.5	-	Run-up factor 1.5 = 1:3 slope, (assumed to be rough stone)
Wave surcharge=	**1.05**	m	Wave surcharge allowance (modified significant wave height to allow for: influence of structures and land near the dam; tolerance of dam to overtopping and wave carry over; wave run-up on the upstream face of the dam)

Table 7-A Stocks reservoir, wave surcharge calculation data.

The EuroTop calculation tool suggests that the wave overtopping discharge will be 0.001l/s/m for the design case freeboard of PMF stillwater 1.05m below the wave wall. This is an acceptable value for mean overtopping discharge for an embankment with a grass crest and downstream face. There is no wave discharge in the existing situation, (1.35m freeboard) scenario.

The adopted calculated wave surcharge is 1.05m which is indicated by both methods. This is above the recommended 0.6m minimum freeboard given in Table 1 of the ICE Floods and Reservoir Safety Guidance.

Table 7-B compares the results and parameters used for the wind wave surcharge calculation between the current study, the United Utilities 2012 study and the analysis undertaken by B.H. Rofe in the 1998 (RKL – Arup) Section10 Inspection report.

Parameter	Jacobs2015	UU 2012	Rofe 1998
Fetch length (km)	2.40	2.41	2.50
Fetch Direction (^oN)	30	30	45 (NE)
Significant wave height (H_s) [m]	0.54	0.53	0.79
Wave design height (H_s) [m]	0.7	0.69	1.03
Run-up factor	1.5	1.5	1.5
Wave surcharge (m)	1.05	1.04	1.55

Table 7-B Comparison between parameter values used within wind wave calculations between: Jacobs 2015 (present study), 2012 UU study and 1998 Rofe study.

The UU 2012 study used parameters which compare very closely to those within the present study. As such the resultant calculated wave surcharge levels are similar (1.05m wave surcharge calculated during the present study compared to 1.04m calculated during the 2012 study). There is a larger discrepancy when the present study is compared to that of the 1998 Rofe study. As stated in the 2012 study report, Rofe's calculation is conservative to allow for the fetch to take into account the bend caused by the island, whilst Rofe also applies a conservative direction adjustment factor of 1.0 contrary to the ICE guidance value of 0.73.

The wave surcharge value of 1.05m does not require re-calculating for the proposed future scenario (i.e. raising spillweir crest level to increase capacity of reservoir). This is because increasing the overflow weir by 300mm gives a winter PMF still water level approximately 160mm below the dam crest, (accommodating the 1.05m wave surcharge). The still water level remains on the 1v:2.5h upstream slope and not the vertical wave wall, therefore the existing situation run up factor does not require any adjustment.

8 Available Storage

Utilising the wave surcharge of 1.05m, the peak flood and wave surcharge level for winter PMF is 184.3mLD, resulting in a PMF freeboard of 300mm for Stocks Reservoir. Increasing the level of the spill weir crest to utilise this freeboard (i.e. increasing the spill weir by 300mm) would provide an increase in available storage volume. It should be noted that this would result in zero additional freeboard above the wave surcharge allowance.

The winter PMF with 42mm/day snowmelt simulation results in the freeboard to the top of the wave wall increasing to 530mm.

Utilising the available freeboard would result in revised spill weir levels:

- 180.87mLD for the 65mm/day snowmelt scenario (adopted snowmelt rate).
- 181.10mLD for the 42mm/day snowmelt scenario

The additional storage was calculated assuming a side slope of 1v:4h (which was derived from the limited survey data on Stocks Reservoir). Table 8-A shows details of the additional storage available if the top water level of the reservoir was raised.

Estimated additional storage utilising 300mm freeboard (65mm/day snowmelt)		
Level (mLD)	**Area (m^2)**	**Volume (Ml)**
180.57	1320000	0.00
180.87	1324763	396.7
Estimated additional storage utilising 530mm freeboard (42mm/day snowmelt)		
Level (mLD)	**Area (m^2)**	**Volume (Ml)**
180.57	1320000	0.00
181.10	1328421	701.8

Table 8-A Additional storage if spill-weir level is increased so that reservoir additional freeboard is zero.

Assuming a winter PMF scenario with 65mm/day snowmelt results in the freeboard is reducing to 300mm. Utilising this reduced freeboard by raising the existing weir level up to a new level of 180.87mLD would result in an additional storage capacity of approximately 397Ml.

The winter PMF scenario for 42mm/day snowmelt, results in a freeboard of 530mm up to the dam wave wall crest level. Utilising this freeboard would result in approximately 702Ml of additional storage.

The 2012 study states that the Hodder WTW has an average output of approximately 61Ml/d (for the period April – May 2012). Using this demand value and raising the Stocks reservoir overflow weir by 300mm would result in up to 6.5 days additional supply.

The maximum WTW output is 85Ml/d. Therefore during peak demand periods the increased reservoir storage would result in up to 4.7 days additional supply.

9 Spillway Chute Performance

The Stocks Reservoir spillway chute performance was investigated using a number of hand calculations due to the issues encountered when using ISIS software (see section 5). As previously mentioned, the layout of the spillway chute consists of three 2.4m diameter pipes that pass the flow under the base of the chute. The chute is there as an overflow when the pipe capacity is reached. Both the spillway chute and pipes re-join in the stilling basin approximately 180m downstream of the pipe inlets. Two primary calculations were undertaken:

1. Calculating the pressurised pipe capacity of the 3no. 2.4m diameter pipes.
2. Calculating the chute capacity.

It is estimated that each pipe can convey up to 57.8m^3/s, as such the combined capacity of the pipes is: 173.4m^3/s.

The peak outflow from the critical winter PMF event is estimated to be 263.2m^3/s. As such approximately 90m^3/s will flow down the spillway chute. To check the spillway chute capacity the Manning's formula was used:

$$Q = \frac{AR^{2/3}S^{1/2}}{n}$$

Where: A = Area
R = Hydraulic radius
S = Slope

Section	Chainage (m)	Area (m^2)	Hydraulic radius (m)	Slope (m/m)	Flow (m^3/s)
SEC_1	0.0	27.53	1.67	0.09	886
SEC_2	9.2	22.79	1.53	0.15	897
SEC_3	17.1	23.99	1.59	0.15	977
SEC_4	24.6	23.18	1.60	0.18	1027
SEC_5	30.8	26.13	1.73	0.16	1154
SEC_6	43.9	28.41	1.85	0.15	1283
SEC_7	56.4	30.45	1.92	0.15	1409
SEC_8	68.7	32.18	1.99	0.15	1519
SEC_9	81.0	34.24	2.06	0.15	1659
SEC_10	93.6	34.49	2.07	0.15	1672
SEC_11	105.8	34.13	2.05	0.15	1644
SEC_12	117.7	34.51	2.07	0.15	1676
SEC_13	129.8	34.18	2.06	0.15	1659
SEC_14	140.7	34.54	2.07	0.15	1668
SEC_15	152.2	34.32	2.06	0.15	1666
SEC_16	162.9	34.34	2.06	0.15	1653
SEC_17	173.5	34.02	2.05	0.13	1549
SEC_18	176.2	33.58	2.04	0.08	1141
SEC_19	179.0	31.64	1.97	0.04	765
SEC_20	184.5	25.83	1.74	0.02	448

Table 9-A Flow capacity of the Stocks Reservoir spillway chute calculated for cross sections taken from the available survey. Cross section parameters are included.

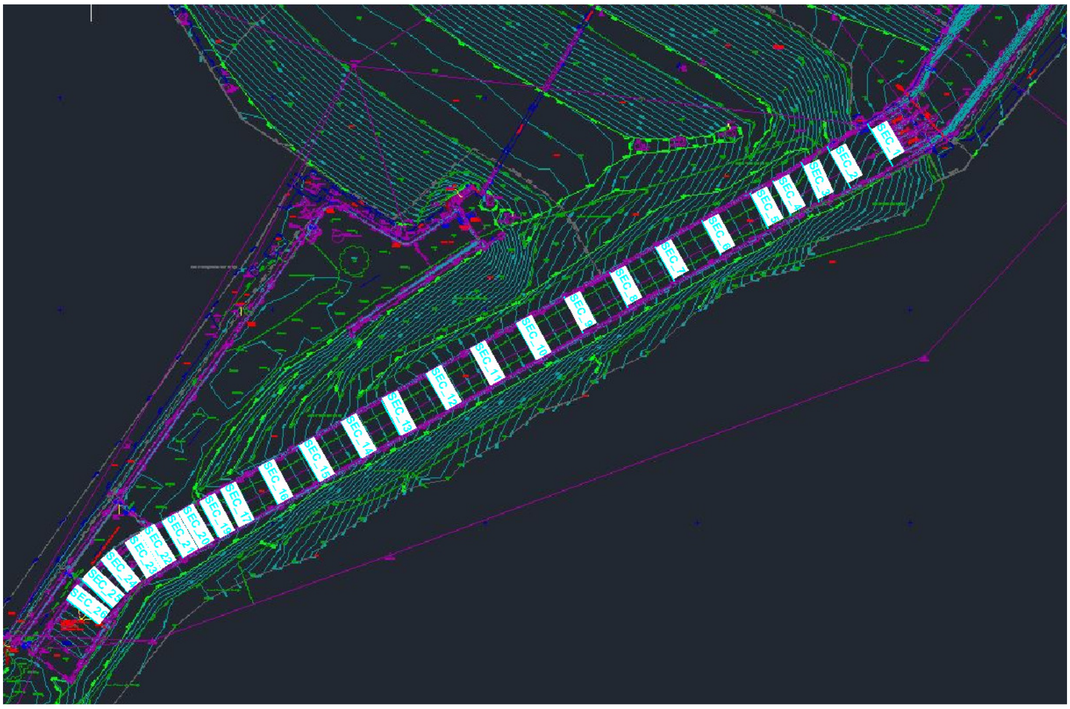

Figure 9-1 Stocks overflow chute cross section location plan.

Table 9-A above illustrates that the spillway chute has sufficient capacity to convey any flow not routed through the 2.4m diameter pipes. Sections 1, 2 and 3 in the upper section of the spillway and Sections 19 and 20 at the end of the spillway chute (as it enters the stilling basin) are those at most risk of overtopping (see Figure 9-1). However, calculations indicate that this will occur for flows above approximately 448 m^3/s. Since the critical PMF total outflow from Stocks Reservoir is only 282.7m^3/s, the risk of overtopping in the spillway chute is negligible.

10 Conclusions

A hydrodynamic model of Stocks Reservoir was produced with representation of reservoir storage and the overflow level-discharge relationship based on standalone calculations.

Catchment hydrology was developed for the PMF event using the latest industry standard methods and guidance. Model simulations were undertaken with provision for winter and summer storm event profiles and critical storm duration was established for both types of events. A snowmelt study indicated that an increase in snow melt from the standard value of 42mm/day to 65mm/day should be used given the location of the catchment and the sensitivity of the reservoir peak still water level to this factor. The higher value was taken forward in the study as the design case.

R-lag Analysis indicated that the winter PMF 11.9hr event was critical for Stocks reservoir. The still water winter flood rise was shown to be 183.25mLD which is approximately 460mm below the embankment crest level.

The embankment crest has an approximately 0.8 metre high masonry wave wall on the upstream edge with the ground surface behind covered with grass. The embankment crest will have limited resistance to wave overtopping and two methods have been used to determine the appropriate wave freeboard allowance. Using the method set out in Floods and Reservoir Safety gives a wave freeboard requirement of 1.05m allowing for 4% of waves to overtop the wall and 1.35m if no wave overtopping is accepted. The second method used the EuroTop calculation tool, indicated that a required wave freeboard allowance of 1.05m generates a very low mean wave overtopping rate of 0.001l/s/m. A minimum wave surcharge allowance of 1.05 metres was adopted.

Application of the 1.05m wave surcharge to the critical winter PMF (with 65mm/day snowmelt) still water level gives a remaining freeboard of 300mm below the minimum wave wall level.

Estimates of potential storage volume above the current top water level were made using typical values of the bank gradient where survey data was available. These calculations indicate that by utilising the estimated remaining freeboard by raising the reservoir TWL by 300mm could result in additional reservoir storage of 397Ml

The analysis suggests that there is a potential to increase the reservoir storage capacity by increasing the spillweir crest level without compromising reservoir safety. To confirm this potential the following next steps are recommended:

- Extend the survey of the reservoir area to confirm the additional storage capacity that can be realised
- Investigation of the wave wall and embankment crest to confirm that the assessed risk of damage from wave overtopping is acceptable
- Carry out a physical scale model of the spillway to confirm the spillweir rating curve.

Appendix A Flood Study Audit

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*************************************************************
ISIS
*************************************************************

HYDROLOGICAL DATA

Catchment: Stocks_Res
*************************************************************
Catchment Characteristics
*************************************************************
Easting          :     371950 Northing        :     454500
Area             :     37.510 km2
DPLBAR           :      6.600 km
DPSBAR           :    114.200 m/km
PROPWET          :      0.600
SAAR             :   1658.000 mm
Urban Extent     :      0.000
c                :      0.000
d1               :      0.000
d2               :      0.000
d3               :      0.000
e                :      0.000
f                :      0.000
SPR              :     53.000 %
*************************************************************
Summary of estimate using Flood Estimation Handbook rainfall
*************************************************************
Estimation of Probable Maximum flood
====================================
Unit hydrograph time to peak     :      2.322 hours
Instantaneous UH time to peak    :      3.391 hours
Data interval                    :      0.100 hours
Design storm duration            :     11.900 hours
Critical storm duration          :      6.171 hours
em-2h                            :    158.000
em-24h                           :    310.000
em-25d                           :    550.000
ARF                              :      0.000
Design storm depth               :    265.669 mm
CWI                              :    190.759
Standard Percentage Runoff       :     53.000 %
Percentage runoff                :     89.422 %
Snowmelt rate                    :     42.000 mm/day
Unit hydrograph peak             :      3.554 (m3/s/mm)
Quick response hydrograph peak   :    439.948 m3/s
Baseflow                         :      2.682 m3/s
Baseflow adjustment              :      0.000 m3/s
Hydrograph peak                  :    442.629 m3/s
Hydrograph adjustment factor     :      1.000
```

Table A 1 PMF Winter 42mm/day snowmelt hydrology – summary details

Time (hrs)	Inflow (m^3/s)	Outflow (m^3/s)	Water Level (mLD)
0	1.36	1.36	180.612
0.5	4.29	3.48	180.612
1	8.75	3.80	180.617
1.5	16.27	4.74	180.631
2	27.10	6.56	180.657
2.5	41.19	9.49	180.698
3	55.55	13.62	180.756
3.5	69.82	18.80	180.828
4	84.45	24.94	180.911
4.5	100.19	36.82	181.001
5	118.70	49.72	181.097
5.5	143.00	65.80	181.201

Time (hrs)	Inflow (m^3/s)	Outflow (m^3/s)	Water Level (mLD)
6	181.13	86.23	181.321
6.5	243.80	117.95	181.476
7	312.68	152.89	181.682
7.5	377.86	171.55	181.954
8	428.58	193.24	182.275
8.5	440.15	216.00	182.604
9	415.13	236.21	182.883
9.5	374.64	251.04	183.085
10	325.19	259.95	183.203
10.5	271.91	263.20	183.249
11	218.18	260.85	183.224
11.5	169.96	253.86	183.141
12	134.87	243.91	183.017
12.5	112.73	232.59	182.873
13	94.62	220.77	182.719
13.5	77.96	208.90	182.557
14	61.41	196.70	182.388
14.5	45.19	184.36	182.212
15	31.95	171.71	182.031
15.5	21.55	158.87	181.849
16	13.58	139.65	181.672
16.5	7.98	107.96	181.499
17	4.45	84.57	181.350
17.5	2.83	67.93	181.233

Table A 2 PMF winter 42mm/day snowmelt hydrology – runoff.

Appendix B HOST Class 4 assessment

Standard Percentage Runoff estimates obtained from the FEH catchment descriptors are based upon the HOST (Hydrology of Soil Types) classification developed by the Institute of Hydrology[13]. Two methods were used by the Institute of Hydrology to derive SPR estimates for the various HOST classes. Those derived via a form of multiple regression analysis were favoured over those obtained from a Baseflow Index (BFI) method. For HOST Class 4 the SPR value recommended was 2%, though the alternative BFI method provided an estimate of 20%.

Investigation of the presence of HOST Class 4 soils within the Stocks catchment was undertaken using the 1:250,000 soils maps[14] (Tables B1 & B2). Slightly less than 2% of the total Stocks catchment was estimated to be covered by HOST Class 4 soil. This HOST Class is given a SPR value of 2%. Had the SPR estimate associated with HOST Class 4 been 20% (as suggested by the alternative Institute of Hydrology methodology) then the catchment SPRHOST would have risen by 0.36% (i.e. from 50.44% to 50.80%).

Code	HOST Class	Percentage
713g	24	100.0%
651a	4	18.8%
	15	81.3%
1011b	29	100.0%
721c	10	11.1%
	26	88.9%

Table B1: HOST characteristics of the soils found within the Stocks Reservoir catchment.

Soil Code	% Subject Catchment
713g	45
651a	10
1011b	25
721c	20

Table B2: Composition of the soils in the Stocks Reservoir catchment.

HOST Class 4 proportional coverage across the catchment is given by 0.18 x 0.1 = 0.018 (i.e. 1.8%).

[13] Boorman DB, Hollis JM & Lilly A, 1995. Hydrology of soil types: a hydrologically-based classification of soils of the United Kingdom. Institute of Hydrology Report No. 126

[14] Soil Survey of England and Wales, 1983. Soils of Northern England.

Appendix C Appraisal of the winter PMF snowmelt rate for the Stocks Reservoir catchment

The design conditions for the winter PMF flood includes the addition of the 100-year snowmelt rate sustained from a 100-year snow depth water equivalent for the catchment[15] .

The default snowmelt rate recommended[16] for inclusion in UK winter PMF studies is 42mm/day. However in some upland regions of the UK there is evidence[17] to suggest that higher rates may be more appropriate. The Floods and Reservoir Safety 3rd Edition provides a UK map of these areas, though gives no prescriptive guidance as to what higher rates should be used in such circumstances. The map was adapted from a similar one for 24-hour melt rates with 5-year return period given in Hough and Howlis (1997): one they described as a "sketch map". It permits only an approximate indication of where these higher snowmelt rates may extend, and coupled with its small scale can be difficult to use.

The research study undertaken by Hough and Howlis developed several regression equations relating melt rate to either climatic or geographic variables. These permit point location melt rates for the 5-year 24 hour event to be estimated (BOX C1). The study also provides estimates of the 5, 20 and 50-year melt rates for all the climatic stations used in their analysis. From these the 100-year melt rate growth curve relative to the 5-year event can be estimated.

Table 7. *Regression equations relating the 24-hour snowmelt with 5 year return period to weather and geographical variables*

Regression type	Equation	R^2	RMS error (mm)
Single non-weather factor	5.09 + 0.085 ALT	0.72	8.85
Two non-weather factors	–3.80 + 0.083 ALT + 0.00187 NORTHING	0.82	7.28
One weather factor	71.16–9.45 MAX	0.77	8.02
Two weather factors	52.52–9.08 MAX + 1.46 WIND	0.85	6.56

ALT is height above sea level in m, NORTHING is national grid northing (four figure reference), MAX is the mean daily maximum air temperature in January and WIND is the mean January windspeed in knots at 10 m above the ground.

BOX C1 Regression equations relating the 24-hour snowmelt rate with 5-year return period to climatic and geographic variables. [Source: Hough and Hollis (1997)].

For the Stocks study the following steps have been followed to establish the appropriate melt rate to use in the PMF study.

[15] Institute of Hydrology, 1999. Flood Estimation Handbook Vol 4, Section 4.3.4. Institute of Hydrology, Wallingford, Oxon.
[16] ICE, 1996. Floods and Reservoir Safety. Thomas Telford, London.
[17] Hough MN and Hollis D, 1997. Rare snowmelt estimation in the United Kingdom. Metreorol. Appl. 5, 127-138.

Step 1:

A provisional indication of the likelihood that the Stocks Reservoir catchment falls within, or is within the vicinity of, an area of higher melt rate was obtained via reference to the map in the Floods and Reservoir Safety guide.

This suggests that rates higher than 42mm/day would not be applicable, though the catchment lies relatively close to areas that do have higher values. Given the upland nature of the catchment it was judged sensible to investigate this in more detail.

Step 2:

The four equations given in BOX C1 were used to provide melt rates. Maximum and minimum values across the catchment were calculated using the maximum and minimum of the variables within the catchment. Similarly average values were used to provide a mean catchment rate. The geographic variables are readily available from Ordnance Survey mapping. Both the climatic variables were obtained from the Met Office web site[18] where relatively detailed maps of both monthly mean wind speed and mean daily maximum air temperature are available (for example Figure C1).

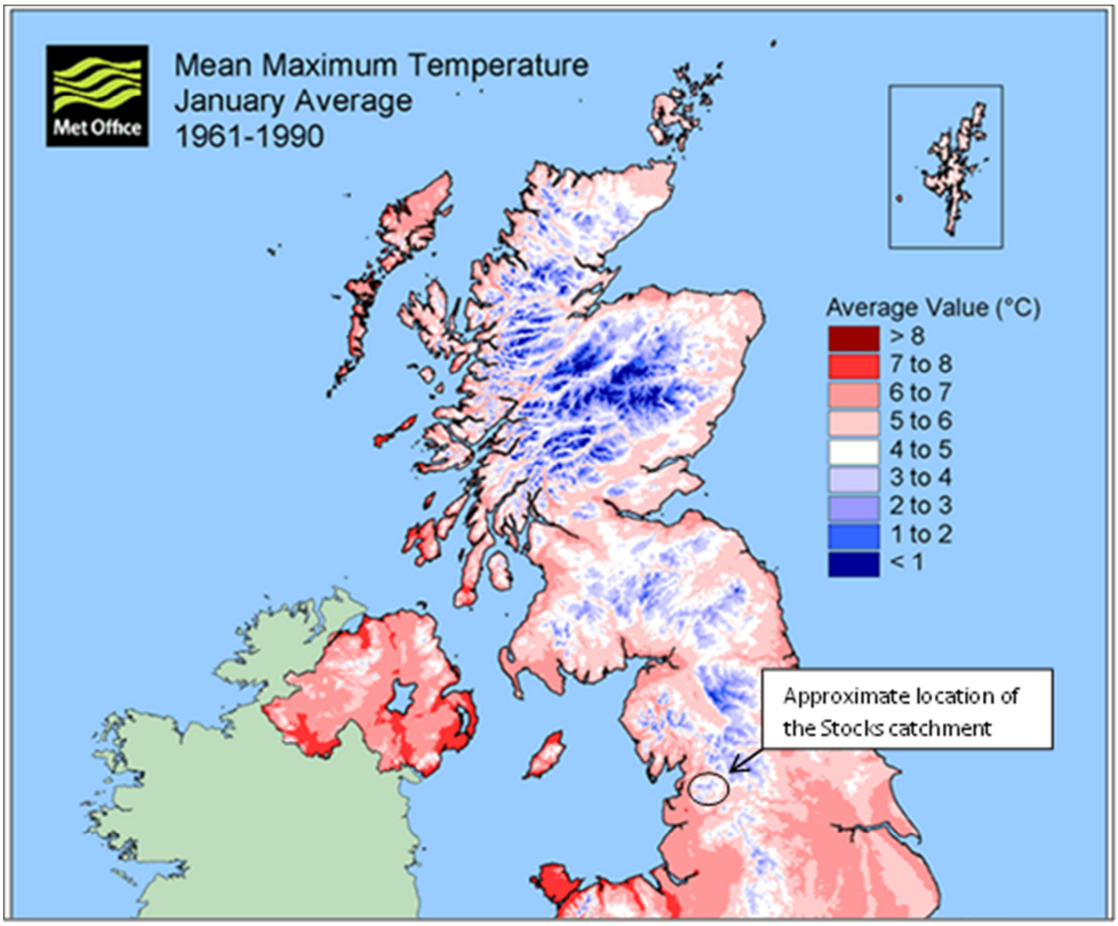

Figure C1 Reproduction of the Met Office Map showing mean maximum January temperature (1961 – 1990).

[18] http://www.metoffice.gov.uk/climate/uk/averages/ukmapavge.html

Table C1 presents the results. The mean rate is considered to be the most appropriate value to consider in the context of the PMF study. None of the equations suggest that the mean rate is above 42mm/day. Maximum rates of about 52mm/day are predicted within the catchment whereas minimum rates as low as 15mm/day are also predicted.

Table C1 Estimated 5-year 24hour snowmelt rates for Stocks reservoir catchment

Regression equation	24-hour snowmelt rate of 5-year return period (mm/day)		
	Min	Max	Mean
Single non-weather factor	20.6	51.5	31.1
Two non-weather factors	19.9	50.1	30.2
***One weather factor**	14.5	52.3	33.4
***Two weather factors**	34.5	49.0	41.8

Step 3:

The 100-year snowmelt rate growth factor was estimated from the nearest climatic stations given in Hough and Howlis (1997) that were considered to be climatically similar to the Stocks catchment.

Table C2 provides the growth factors of the two favoured stations: Wilsden and Malham Tarn. These were chosen due to both their proximity and altitudes being reasonably similar to that of the Stocks catchment whose mean altitude is 306mLD. Table C3 presents the estimated 100-year snowmelt rates for the Stocks catchment based on each of the regression equations.

Table C2 Snowmelt growth factors from nearby climatic stations considered to be climatically similar

Climatic station	Altitude (mAOD)	Growth factors for varying return periods			
		5	20	50	100
Wilsden	262	1	1.38	1.62	1.81
Malham Tarn	395	1	1.48	1.78	2.02
Average	329	1	1.43	1.70	1.92

Table C3 Estimated 100-year 24hr snowmelt rates for the Stocks Reservoir catchment

Regression equation	100-year 24-hour snowmelt rate (mm/day)		
	Min	Max	Mean
Single non-weather factor	39.5	98.8	59.7
Two non-weather factors	38.2	96.1	57.9
***One weather factor**	27.8	100	64.1
***Two weather factors**	66.3	94.0	80.2

Mean melt rates derived from all four regression equations (Table C3) exceed the 42mm/day value with an average snowmelt rate of 65.5mm/day (some 56% greater). Based upon this evidence it is considered that the case for using a higher snow melt rate than 42mm/day in the Stocks PMF study is justified. It is therefore recommended that the PMF study include analysis for a snowmelt rate of 65mm/day.

Appendix D Wave Surcharge Analysis

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CALCULATION REFERENCE SHEET

Calculation Number:	Stocks Reservoir-Cal-001	**Calculation Title:**	Stocks Reservoir: Check on Wave Height/Surcharge

Purpose/Work Stage	**RELATED DOCUMENTS**
	Stocks Reservoir - Cal - 002
	Stocks Reservoir - Cal - 003
Method of Checking	**Results used in:**
	Stocks Flood Study

Document History Record

Rev	Date	Description/Reason for Issue	Orig	Chkr	Reviewer
0A	07/10/2014	For use in Stocks Flood Study	DS	DA	DA

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CALCULATION REFERENCE SHEET

Client: United Utilities		Job No:	B16000EP
		Calc No:	Calc - 001
Project: B16000EP - Stocks Reservoir		By: D. Steven	Date: 15/09/2014
Subject: Stocks Reservoir - Check on Wave Height/Surcharge		Checked: D. Armour	Date: 07/10/2014
		Reviewed: D. Armour	Date: 07/10/2014

Objectives

To find appropriate Hs and Rc for reservoir and compare against 1998 and 2012 reports.

Reference, Documents, Standards

Floods and Reservoir Safety 3rd Edition
2011 S10 Report
Eurotop Wave Calculation Tool

Design Criteria
N/A

Conclusions & Recommendations

Most realistic Fetch Length determined to be ~2.4km from North Eastern shore

Wave Surcharge calculated as 0.54m

Freeboard required for Hs = 0.54m is 1.1m for no overtopping
Freeboard required for Hs = 0.54m is 0.46m for ~.1.0 l/s/m overtopping

Stocks – waves

Current wave surcharge allowance is 1.55m which seems to come from calculations by Rofe 1998

Last Inspecting Engineer commented that "the wave surcharge allowance is considered to be particularly prudent, as it allows for the refraction of the wave along the reservoir". *TMH question – is the use of the term "refraction" referring to the use of an increased bent fetch, or did it mean wave diffraction"*

(diffraction – bending of waves around obstacles)

(refraction – change in direction of wave due to change in wave length/speed due to depth)

Rofe calculation has been reproduced by UU, who have suggested that the calculated wave height is unduly conservative because it does not use a wind direction factor.

(prudent = wise, careful).

Note UU back calculation is based on Hs = 0.79m and Hd=1.3Hs, ie some wave overtopping

UU quote Rofe's reasons for not using the reduction factor is quoted by UU as "I feel the wind would follow the contours of this upland valley and that the design should not allow for this reduction further".

Fetch length is not straight length – hence

Two key issues:

1) Is calculation of significant wave height reasonable or overly conservative
    a. Does valley shape lend itself to funnelling ? Would that funnelling come from the west ?

    b. Should there be a separate allowance for wave refraction ? SR345/Allsop suggest 20% allowance.

2) What effect would change in calculation of the freeboard required to contain waves from run-up to tolerable overtopping flow have ?

The reservoir is set towards the head of the Hodder valley on the western side of the Pennines.

Note following assessment is based on an examination of OS Maps.

The head of the valley beyond the upstream end of the reservoir runs in a generally south-easterly direction, with the sides of the valley rising quite steeply to about 200m above the valley floor.

At the head of the reservoir the valley is aligned in a north-south direction and is flanked by reasonably steeply sided lower hills rising to about 100m above the reservoir, but is less confined that the head of the valley beyond the reservoir. The topography It is not inconceivable that winds from a north/north-westerly direction could be funnelled into the head of the reservoir.

The shape of the reservoir is such that wind from the north-south aligned head of the reservoir would need to be turned through 45 degrees to the south-west by further funnelling effects. However , this seems highly unlikely as the land along the south-western perimeter only rises to about 30m above the level of the reservoir. Thus any winds funnelled from the head of the valley would tend to continue due south, with waves impacting along the south-western perimeter of the reservoir, rather than being turned towards the dam.

Dam is sheltered from the prevailing and strongest west/south-westerly wind direction because it is situated at the south-western end of the reservoir, which is orientated in a south-west to north-west/north direction.

Waves will only reach the dam from winds blowing from a north-easterly direction.

Fetch direction F1, F2 & F4 all appear appropriate. F3 not for the reasons discussed above.

F4 (or something close to it) appears reasonable. It is therefore considered appropriate to apply a wind direction factor relative to that direction i.e. ~0.73

(Check BS 6399/BS EN to see if it qualifies use of direction factors). ✓

Refraction/defraction – already accounted for by use of bent fetch length.

Valve tower could lead to localised wave effects (diffraction) CIRIA SR345 suggest 10-20% allowance (based on observed effects at breakwaters). Check sensitivity

Check calculation indicates Hs=0.54m for mean annual maximum hourly wind speed, F=2400m fn=0.73

Thus it is concluded that significant wave height of ~0.54m for mean annual hourly max wind speed (As FRS3 3rd Ed) is appropriate.

Use EurOtop to calculate the mean overtopping rate of discharge for Hs=0.54m

PMF stillwater level currently unknown. Therefore calculate freeboard (Rc) required for no overtopping (0.001 l/s/m) and a tolerable overtopping rate of 1 l/s/m

Wave wall is ~0.8m above top of 1 in 3 slope. As PMF stillwater floodrise not known calculate Rc using Empirical method for i) Composite slope with (small vertical) wall and ii) vertical wall

Slope and Wall Rc no overtopping = 1m (1.1) Rc 1 l/s/m overtopping = 0.42m (0.46) (NB these need to be checked)

Considerable reduction on previous freeboard requirement of 1.55m (which was based on design wave height of 1.3Hs so included some wave overtopping)

Check what Rc could be used if Hs = 0.79m used (to be done)

JACOBS CALCULATION SHEET

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Job No. & Title	B16000EP STOCKS RESERVOIR FLOOD STUDY	Originator	DS	Date	15/09/14
Section	WAVE FREEBOARD / OVERTOPPING	Checker	[illegible]	Date	7/10/14

SUMMARY

- FETCH OF 2.4KM FROM EASTERN SHORE

  $H_s = 0.54M$ ✓

  $R_c = 0.46M$ (FOR $q = 1$ L/s/m) ✓

  $= 1.1M$ (FOR $q = 0.001$ L/s/m) ✓

- FETCH OF 3.175KM FROM NORTH SHORE

  $H_s = 0.67M$ ✓

  $R_c = 0.62M$ (FOR $q = 1$ L/s/m) ✓

  $= 1.4M$ (FOR $q = 0.001$ L/s/m) ✓

  Most likely representative of situation at Stocks (worst case.)

- FETCH OF 2.5KM BASED ON Rofe 1998 REPORT

  $H_s = 0.79M$ ✓

  $R_c = 0.72M$ (FOR $q = 1$ L/s/m) ✓

  $= 1.6M$ (For $q = 0.001$ L/s/m.)

  check on Rofe 1998 calcs

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Section: WAVE FREEBOARD		Checker	[signature]	Date	7/10/14

From FRS 3rd Edition

$$H_s = UF^{0.5}/1760$$

where U = required wind speed
F = Fetch

$$U = U_{50} \cdot f_t \cdot f_A \cdot f_w \cdot f_D \cdot f_N$$

$U_{50} = 23.5\,m/s$ (Ref ①)

$f_t = 0.79$ (Mean Annual - pg 21 FRS).

$f_A = 1.0 + (0.001 \times Alt)$ - Reservoir TWL = 180.57m (2011 S10 - Ref ②)

$= 1.0 + (0.001 \times 180.57)$

$= 1.18$

f_w = Iterate between 1.16 & 1.23 from table 4 pg 23 FRS — Fetch = 2.4km - (Ref ③ + ④)

$= 1.18$ (round up)

$f_N = 0.73$ - Table 5 pg 23 FRS
- 30° direction (Ref ③)

$f_D = 1.02$ - iterate between values for CIRIA guidance due to fetch > 2km.

$\therefore U = 23.5 \times 0.79 \times 1.18 \times 1.18 \times 0.73 \times 1.02$

$= 19.25\,m/s$

$\therefore H_s = 19.25 \times 2400^{0.5}/1760$

$= 0.54\,m$

JACOBS CALCULATION SHEET

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Section	WAVE OVERTOPPING	Checker	[illegible]	Date	7/10/14

With $H_s = 0.54m$ calculate min freeboard before overtopping occurs

min freeboard for ~ 1 l/s/m overtopping

$$T_p = 0.0712\ F^{0.3}\ U^{0.4}$$

$$= 0.0712 \times 2400^{0.3} \times 19.25^{0.4}$$

$$= 2.4s$$

From Eurotop it can be seen that a minimum of 1.1m freeboard is when overtopping would occur (Ref ⑤) (i.e $v = 0.001\ l/s$) (slope + wall)

For ~ 1 l/s/m to overtop the wave wall freeboard between 0.48m – 1.17m would be required – (Ref ⑥ + ⑦)
(slope + wall) (Vert wall only)

If fetch was taken from the most northerly point i.e. F3 what would Hs be?

Fetch = 3.175 km – Ref ⑧

U = 20.9 m/s

assume $f_N = 0.78$ (from North)
$f_N = 1.20$ (iterate)

$$\therefore H_s = 20.9 \times 3175^{0.5} / 1760$$

$$= 0.67m$$

$$\therefore T_p = 0.0712 \times 3175^{0.3} \times 20.9^{0.4}$$

$$= 2.7s$$

- From Eurotop it can be seen that at 1.4m Freeboard overtopping will occur - Ref (9)

- For ~ 1 l/s/m to overtop the wave wall freeboard somewhere between 0.62m - 1.57m - Ref (10) + (11)

- If $H_s = 0.79m$ as calculated by Rofe 1998 what freeboard would allow for ~ 1 l/s/m

Rofe - $U = 27.92 m/s$

$F = 2500m$

$\therefore T_p = 0.0712 \times 2500^{0.3} \times 27.92^{0.4}$

$= 2.8s$

$\therefore$ R_c between 0.72 m - 1.96m - Ref (12) + (13)

(slope + wall) (vert wall)

<u>Check on EuroTop Calc Tool</u>

- For Fetch of 2.4km & $H_s = 0.54m$ - pg (4) + (5)

- For Fetch of 3.175 km & $H_s = 0.67m$ - pg (6) + (7)
  (F3 in 2012 UU Report)

- For Fetch of 2.5km & $H_s = 0.79m$ - pg (8) + (9)
  (Rofe 1998)

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TITLE	Stocks Reservoir - Wave Analysis	CHECKER		DATE	7/10/14
SECTION	Wave Overtopping for 2.4km fetch	REVIEWER		DATE	

Stocks Reservoir
Scenario: Fetch length of 2.4km

Calculation of $s_{m-1,0}$

Method: EurOtop, 2007 - Wave Overtopping of Sea Defences and Related Structures: Assessment Method

Glossary:

s	= wave steepness = H/L	[-]
s_{0p}	= wave steepness with L_0, based on $T_p = H_{m0}/L_{0p} = 2\pi H_{m0}/(gT^2_p)$	[-]
s_{0m}	= wave steepness with L_0, based on $T_m = H_{m0}/L_{0m} = 2\pi H_{m0}/(gT^2_m)$	[-]
s_0	= wave steepness with L_0, based on $T_{m-1,0} = H_{m0}/L_0 = 2\pi H_{m0}/(gT^2_{m-1,0})$	[-]

Re-arranged: $$L_0 = \frac{gT^2_{m-1,0}}{2\pi}$$

$$T_p = 1.1\,T_{m-1,0}$$ 5.2

Substituting Tp: $$L_0 = \frac{g(T_p/1.1)^2}{2\pi}$$

T_p	2.4	s	pg 2
L_0	7.432352	m	

SJK: Take Hmo = Hs

H_{m0}	0.54	m	pg 1

sm-1,0=H/L 0.072655

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Stocks Reservoir
Scenario: Fetch length of 2.4km

Calculation of mean ovetopping discharge, q ($m^3/s/m$)

Method: EurOtop, 2007 - Wave Overtopping of Sea Defences and Related Structures: Assessment Method Equation 5.9

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \quad 5.9$$

with a maximum of $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$

Data

Parameter	Value	Units	
g	9.81	m/s^2	
H_{m0}	0.54	m	take as Hs
α	18.43	deg	1:3 from historical mapping
tanα	0.33		
γ_b	1	-	no berm
$s_{m-1,0}$	0.0726553	-	from sm-1,0 sheet
$\xi_{m-1,0}$	1.2366448	-	p.69 $\xi_{m-1,0} = \tan\alpha / (s_{m-1,0})^{0.5}$
γ_f	0.9	-	1/4 of stone setting 10cm or higher
γ_β	1	-	Eqn. 5.24 wave overtopping $\gamma_\beta = 1 - 0.0033 \lvert\beta\rvert$ for $0° \le \beta \le 80°$ Let β=0°
γ_v	0.65	-	p.99
Minimum crest level	183.82	m AOD	(From 2011 S10)
Stillwater level	182.3	m AOD	(from Hydrology calc)
R_c	0.47	m	vary Rc to find when q=1l/s/m

ok. self check on the online calc. tool.

Calculation of q

$\exp\left(-4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)$ 0.0056656

$\frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0}$ 0.1435094

$\sqrt{g \cdot H_{m0}^3}$ 1.2428684

q 0.0010105 m3/m/s
1.011 l/s/m

Calculation of qmax

$0.2 \cdot \exp\left(-2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$ 0.0216291

$\sqrt{g \cdot H_{m0}^3}$ 1.2428684

q max 0.0268821 m3/m/s
26.882068 l/s/m

Limiting values for q

q < 0.1 l/s per m:	Insignificant with respect to strength of crest and rear of structure.
q = 1 l/s per m:	On crest and inner slopes grass and/or clay may start to erode.
q = 10 l/s per m:	Significant overtopping for dikes and embankments. Some overtopping for rubble mound breakwaters.
q = 100 l/s per m:	Crest and inner slopes of dikes have to be protected by asphalt or concrete; for rubble mound breakwaters transmitted waves may be generated.

Table 3.5 Limits for overtopping for damage to the defence crest or rear slope

Hazard type and reason	Mean discharge q (l/s/m)
Embankment seawalls / sea dikes	
No damage if crest and rear slope are well protected	50-200
No damage to crest and rear face of grass covered embankment of clay	1-10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

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SECTION	Wave Overtopping for 3.175km fetch (F3 from 2012 UU Report)	REVIEWER	DK	DATE	

7/10/14

Stocks Reservoir
Scenario: Fetch length of 3.175km

Calculation of $s_{m-1,0}$

Method: EurOtop, 2007 - Wave Overtopping of Sea Defences and Related Structures: Assessment Method

Glossary:

s	= wave steepness = H/L	[-]
s_{0p}	= wave steepness with L_0, based on $T_p = H_{m0}/L_{0p} = 2\pi H_{m0}/(gT^2_p)$	[-]
s_{0m}	= wave steepness with L_0, based on $T_m = H_{m0}/L_{0m} = 2\pi H_{m0}/(gT^2_m)$	[-]
s_0	= wave steepness with L_0, based on $T_{m-1,0} = H_{m0}/L_0 = 2\pi H_{m0}/(gT^2_{m-1,0})$	[-]

Re-arranged: $$L_0 = \frac{gT^2_{m-1,0}}{2\pi}$$

$$T_p = 1.1\,T_{m-1,0} \qquad 5.2$$

Substituting Tp: $$L_0 = \frac{g(T_p/1.1)^2}{2\pi}$$

T_p	2.7	s	pg 2
L_0	9.40657	m	

SJK: Take Hmo = Hs

H_{m0}	0.67	m	pg 2

sm-1,0=H/L 0.071227

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SECTION	Wave Overtopping for 3.175km fetch (F3 from 2012 UU Report)	REVIEWER		DATE	

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Stocks Reservoir
Scenario: Fetch length of 3.175km

Calculation of mean ovetopping discharge, q ($m^3/s/m$)

Method: EurOtop, 2007 - Wave Overtopping of Sea Defences and Related Structures: Assessment Method
Equation 5.9

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \quad 5.9$$

with a maximum of: $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$

Data

Parameter	Value	Units	
g	9.81	m/s^2	
H_{m0}	0.67	m	take as Hs
α	18.43	deg	1:3 from historical mapping
tanα	0.33		
γ_b	1	-	no berm
$s_{m-1,0}$	0.0712268	-	from sm-1,0 sheet
$\xi_{m-1,0}$	1.2489843	-	p.69 $\xi_{m-1,0} = \tan\alpha / (s_{m-1,0})^{0.5}$
γ_f	0.9	-	1/4 of stone setting 10cm or higher
γ_β	1	-	Eqn. 5.24 wave overtopping $\gamma_\beta = 1 - 0.0033 \lvert\beta\rvert$ for $0° \le \beta \le 80°$ Let β=0°
γ_v	0.65	-	p.99
Minimum crest level	183.82	m AOD	(From 2011 S10)
Stillwater level	182.3	m AOD	(from Hydrology calc)
R_c	0.62	m	vary Rc to find when q=1l/s/m

Calculation of q

$\exp\left(-4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)$	0.0043138
$\frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0}$	0.1449414
$\sqrt{g \cdot H_{m0}^3}$	1.7176976
q	0.001074 m3/m/s 1.074 l/s/m

ok.

Calculation of qmax

$0.2 \cdot \exp\left(-2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$	0.0187928
$\sqrt{g \cdot H_{m0}^3}$	1.7176976
q max	0.0322804 m3/m/s 32.280383 l/s/m

Limiting values for q

q < 0.1 l/s per m:	Insignificant with respect to strength of crest and rear of structure.
q = 1 l/s per m:	On crest and inner slopes grass and/or clay may start to erode.
q = 10 l/s per m:	Significant overtopping for dikes and embankments. Some overtopping for rubble mound breakwaters.
q = 100 l/s per m:	Crest and inner slopes of dikes have to be protected by asphalt or concrete; for rubble mound breakwaters transmitted waves may be generated.

Table 3.5 Limits for overtopping for damage to the defence crest or rear slope

Hazard type and reason	Mean discharge q (l/s/m)
Embankment seawalls / sea dikes	
No damage if crest and rear slope are well protected	50-200
No damage to crest and rear face of grass covered embankment of clay	1-10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

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TITLE	Stocks Reservoir - Wave Analysis	CHECKER		DATE	
SECTION	Wave Overtopping for 2.5km fetch (Rofe 1998)	REVIEWER		DATE	

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Stocks Reservoir
Scenario: Rofe (1998) check

Calculation of $s_{m-1,0}$

Method: EurOtop, 2007 - Wave Overtopping of Sea Defences and Related Structures: Assessment Method

Glossary:

s	= wave steepness = H/L	[-]
s_{0p}	= wave steepness with L_0, based on $T_p = H_{m0}/L_{0p} = 2\pi H_{m0}/(gT^2_p)$	[-]
s_{0m}	= wave steepness with L_0, based on $T_m = H_{m0}/L_{0m} = 2\pi H_{m0}/(gT^2_m)$	[-]
s_0	= wave steepness with L_0, based on $T_{m-1,0} = H_{m0}/L_0 = 2\pi H_{m0}/(gT^2_{m-1,0})$	[-]

Re-arranged: $$L_0 = \frac{gT^2_{m-1,0}}{2\pi}$$

$$T_p = 1.1\, T_{m-1,0} \quad 5.2$$

Substituting Tp: $$L_0 = \frac{g(T_p/1.1)^2}{2\pi}$$

T_p	2.8	s	pg 3
L_0	10.11626	m	

SJK: Take Hmo = Hs

H_{m0}	0.79	m	pg 3
sm-1,0=H/L	0.078092		

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Stocks Reservoir
Scenario: Rofe (1998) check

Calculation of mean ovetopping discharge, q ($m^3/s/m$)

Method: EurOtop, 2007 - Wave Overtopping of Sea Defences and Related Structures: Assessment Method
Equation 5.9

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp\left(-4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \qquad 5.9$$

with a maximum of $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$

Data

Parameter	Value	Units	
g	9.81	m/s^2	
H_{m0}	0.79	m	take as Hs
α	18.43	deg	1:3 from historical mapping
$\tan\alpha$	0.33		
γ_b	1	-	no berm
$s_{m-1,0}$	0.0780921	-	from sm-1,0 sheet
$\xi_{m-1,0}$	1.1928206	-	p.69 $\xi_{m-1,0} = \tan\alpha/(s_{m-1,0})^{0.5}$
γ_f	0.9	-	1/4 of stone setting 10cm or higher
γ_β	1	-	Eqn. 5.24 wave overtopping $\gamma_\beta = 1 - 0.0033 \lvert\beta\rvert$ for $0° \le \beta \le 80°$ Let $\beta=0°$
γ_v	0.65	-	p.99
Minimum crest level	183.82	m AOD	(From 2011 S10)
Stillwater level	182.3	m AOD	(from Hydrology calc)
R_c	0.73	m	vary Rc to find when q=1l/s/m

Calculation of q

$\exp\left(-4.3 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)$ 0.0033654

$\frac{0.067}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0}$ 0.1384237

$\sqrt{g \cdot H_{m0}^3}$ 2.1992527

q 0.0010245 m3/m/s
1.025 l/s/m

Calculation of qmax

$0.2 \cdot \exp\left(-2.3 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right)$ 0.0188564

$\sqrt{g \cdot H_{m0}^3}$ 2.1992527

q max 0.0414701 m3/m/s
41.470083 l/s/m

Limiting values for q

q < 0.1 l/s per m:	Insignificant with respect to strength of crest and rear of structure.
q = 1 l/s per m:	On crest and inner slopes grass and/or clay may start to erode.
q = 10 l/s per m:	Significant overtopping for dikes and embankments. Some overtopping for rubble mound breakwaters.
q = 100 l/s per m:	Crest and inner slopes of dikes have to be protected by asphalt or concrete; for rubble mound breakwaters transmitted waves may be generated.

Table 3.5 Limits for overtopping for damage to the defence crest or rear slope

Hazard type and reason	Mean discharge q (l/s/m)
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No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

Ref ①

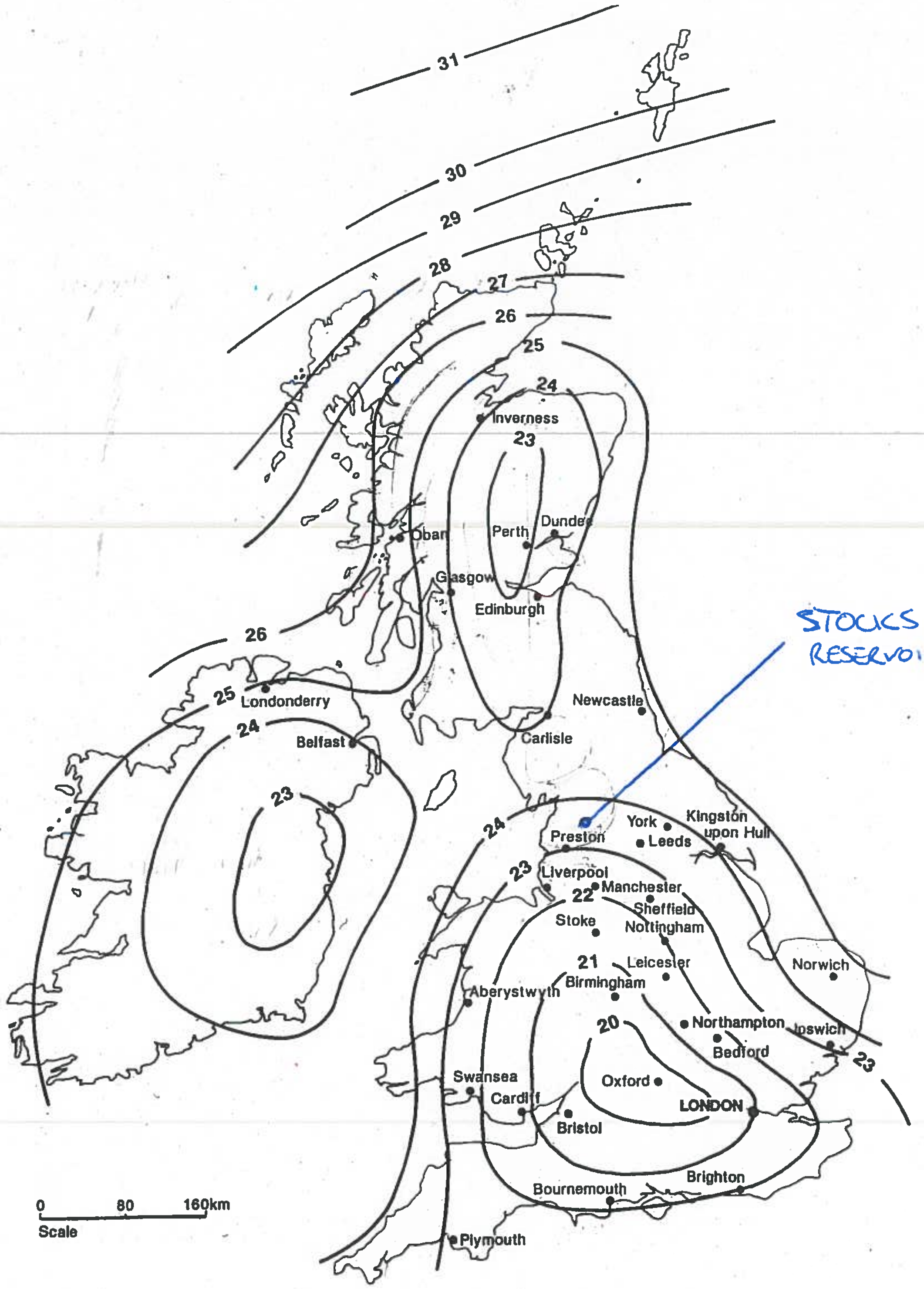


Figure 3 Fifty-year maximum hourly windspeed (m/s) reduced to sea level (source: BS6399 Part II)

2011 S10

Ref (2)

9 General Description

9.1 *Description of the Reservoir.*

The construction of Stocks reservoir was finished in 1932 and was formed by construction of an earth embankment with clay core on the River Hodder. The reservoir has a surface area of 139ha and a storage volume of 12 million m^3 at top water level of 180.57mAOD and is used for water supply. A water treatment works is located immediately downstream of the embankment.

The reservoir was originally constructed by The Flyde Water Board and is now owned and operated by United Utilities. The reservoir is fed from a large direct catchment comprising farmland, forestry and moorland.

9.2 *Geology of Dam*

Geological mapping (published by the Institute of Geological Services) shows the reservoir to be underlain by the Carboniferous Worston Shales and Limestone Beds. The Stocks embankment is constructed across a natural valley feature. Boreholes drilled prior to construction showed rock at shallow depths overlain by a variable thickness of drift deposits. These superficial deposit typically comprise Glacial Clay (Boulder Clay) overlain by a "yellow clay and gravel" which proved to be most extensive on the hills above the valley floor.

9.3 *Catchment*

The reservoir receives water from a direct catchment of 37.47 km^2 comprising undulating farmland, forestry and moorland as shown in Figure 1.

9.4 *Dam Details*

The reservoir is impounded by an earth embankment dam approximately 350 metres long with a maximum height of 31 metres. The dam has a central puddle clay core taken down to a central clay core trench with concrete infill at the deepest points.

The upstream slope of the embankment is at a gradient of 1 vertical to 3 horizontal and is covered with large block pitching. The joints between these blocks are not mortared but have been filled with gravel to help prevent movement. The downstream slope is generally at 1 vertical to 2.5 horizontal with wide intervening berms giving an overall slope similar to the upstream slop at 1 vertical to 3 horizontal. There is a masonry wave wall along the crest at its upstream side of

Ref ③

Measure a Distance

8+1 122 Tweet 104

Like 1,830 people like this.

Free Map Tools

The measure distance tool is a simple way of find the distance between two points on a map. Start Clicking!

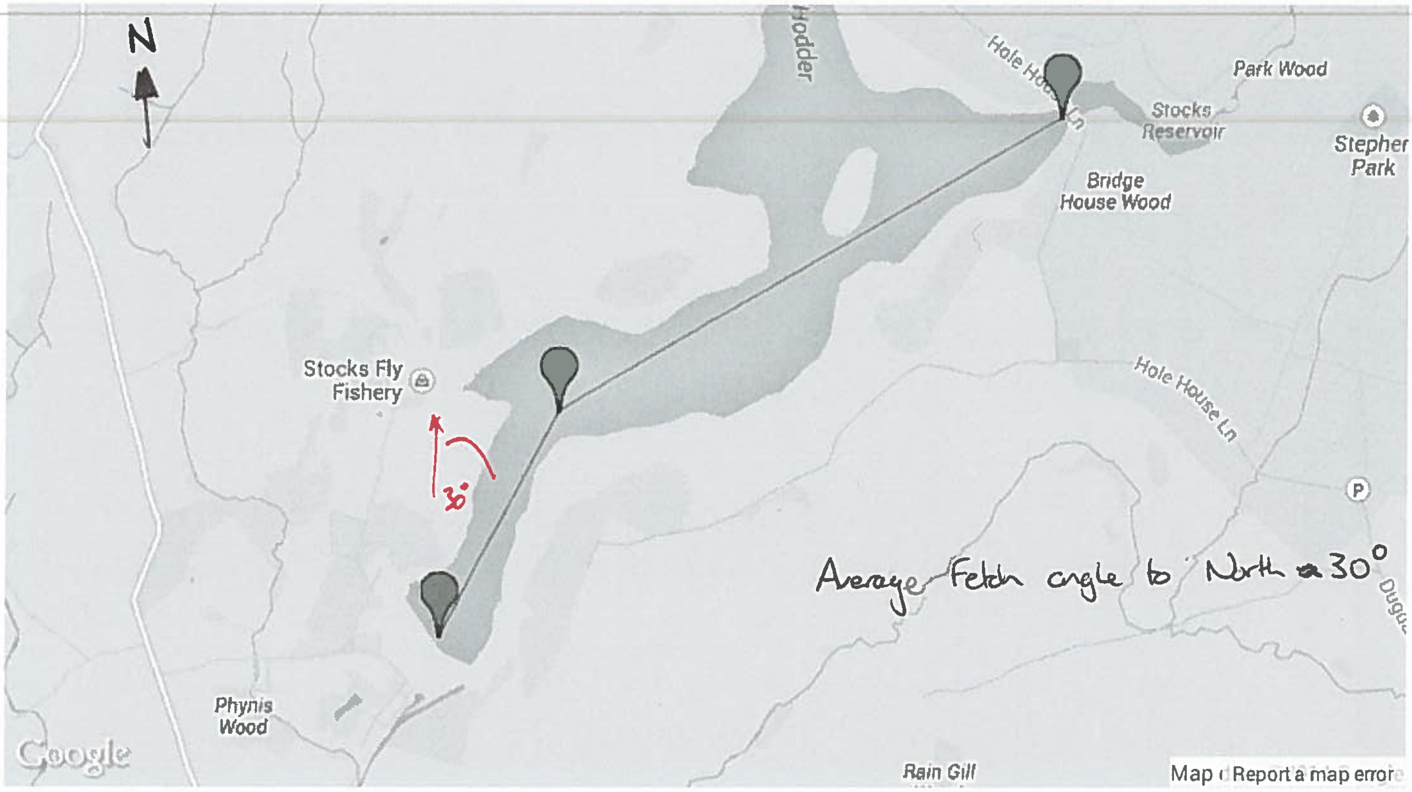
Maps you can make use of..

Measure Distance Map

Search For Location :

Search

Pan to My Location



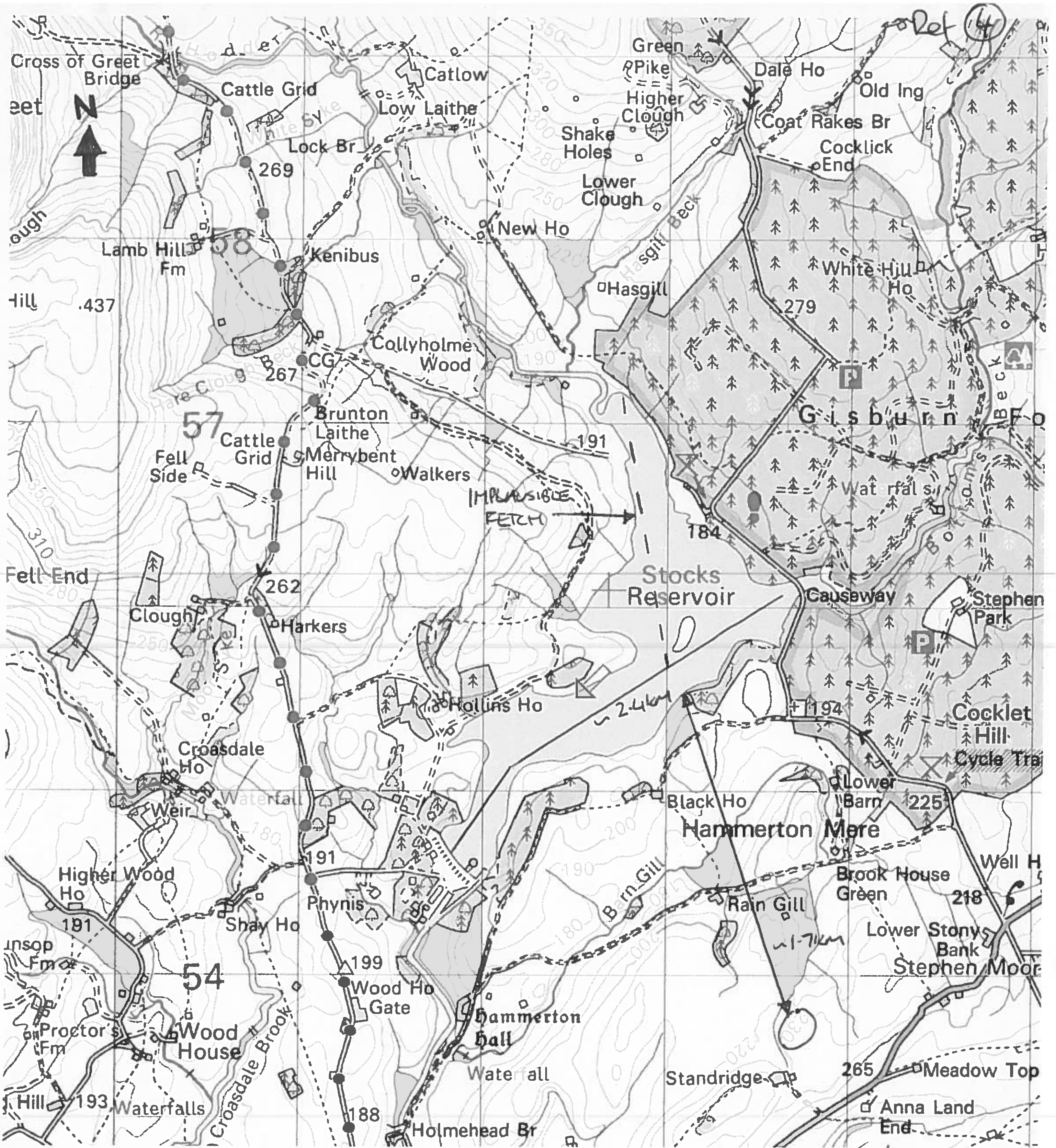
Options

[Map Height : Small - Medium - Large - Full Screen]

Total Distance 2.407 ○ Miles ◉ km ○ Nautical Miles ○ Yards

Clear Last Zoom To Fit Clear Map Toggle Markers ☑ Autopan ? ☐ Snap To Roads ?

✓ checked google



It is implausible to have a fetch distance from the northern most point of the reservoir to the dam due to almost 90° change in direction of the reservoir. Additionally for waves to be funneled in a "banana effect" the south east shore would require very steep banks to contain and channel the wind in a south westerly direction, it can be seen that the south east shore is relatively flat rising from TWL ~ 180m AOD To ~ 230m AOD over ~ 1.7KM. Therefore the longest fetch is from the south east to the dam ~ 2.4KM in length

Ref (5)

Wave Overtopping

Calculation Tool

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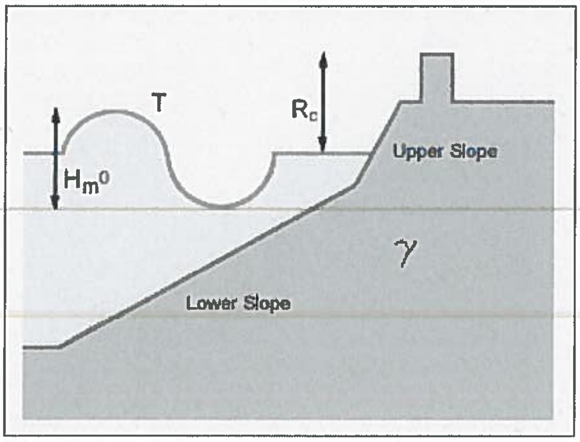
Composite Slope with (Small Vertical) Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



Beta Results

Breaking Type / Other Info

Breaking waves

Mean overtopping discharge rate per metre run of seawall (l/s/m)

0.001

T (wave period): 2.4 s ○ Tm ◉ Tp ○ Tm-1,0

H_{m0} (Wave Height at the Toe of the Structure): 0.54 m

R_c (Freeboard - The height of the crest of the wall above still water level (m)): 1.1 m

OUTPUT

Lower Slope (e.g. 1 in 2): 1 in 3

Upper Slope (e.g. 1 in 2): 1 in 3

γ (coefficient for reduction factors): Basalt (0.9)

Calculate Overtopping Rate

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Ref ⑥

Wave Overtopping

Calculation Tool

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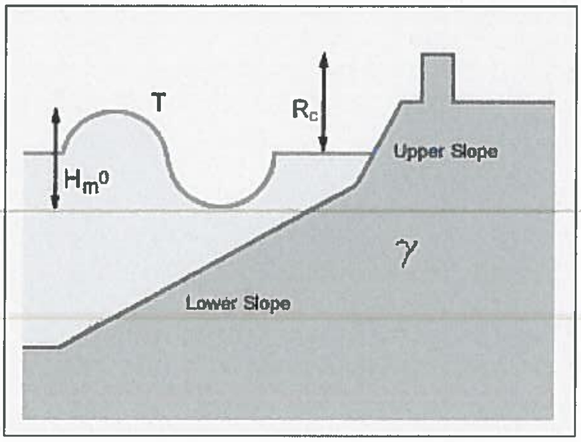
Composite Slope with (Small Vertical) Wall

Method Selection ○ Probabilistic ◉ Deterministic



T (wave period): 2.4 s ○ Tm ◉ Tp ○ Tm-1,0

H_{m0} (Wave Height at the Toe of the Structure): 0.54 m

R_c (Freeboard - The height of the crest of the wall above still water level (m)): .46 m

crest point

Lower Slope (e.g. 1 in 2): 1 in 3

Upper Slope (e.g. 1 in 2): 1 in 3

γ (coefficient for reduction factors): Basalt (0.9)

Calculate Overtopping Rate

Beta Results

Breaking Type / Other Info

Breaking waves

Mean overtopping discharge rate per metre run of seawall (l/s/m)

1.021

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Ref (7)

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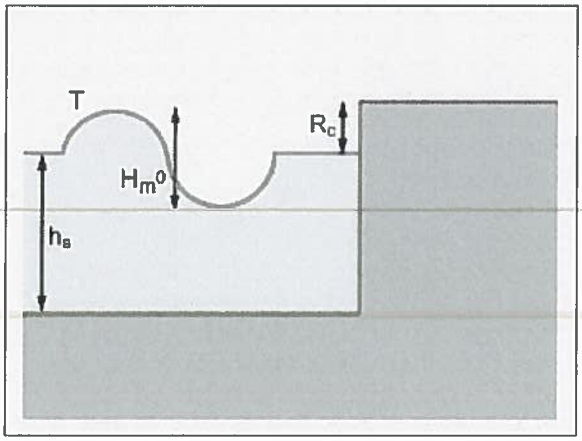
Vertical Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



Beta Results

Wave Type / Other Info

Non Impulsive

Mean overtopping discharge rate per metre run of seawall (l/s/m)

1.006

T (Wave Period)	2.4 ✓	s	○ Tm ◉ Tp ✓ ○ Tm-1,0
H_{m0} (Wave Height at toe of Structure)	.54 ✓	m	
R_c (Freeboard - the height of the crest of the wall above still water level)	1.17 ✓	m	
h_s (Water depth at toe of structure)	3 ✓	m	~~where is this from?~~ assumed

Calculate Overtopping Rate

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Ref (8)

Measure a Distance

Free Map Tools

The measure distance tool is a simple way of find the distance between two points on a map. Start Clicking!

Maps you can make use of...

Measure Distance Map

Search For Location : stocks reservoir

Search

Pan to My Location

Options

[Map Height : Small - Medium - Large - Full Screen]

Total Distance 3.175 Miles km Nautical Miles Yards

Clear Last Zoom To Fit Clear Map Toggle Markers Autopan ? Snap To Roads ?

Ref ⑨

Wave Overtopping

Calculation Tool

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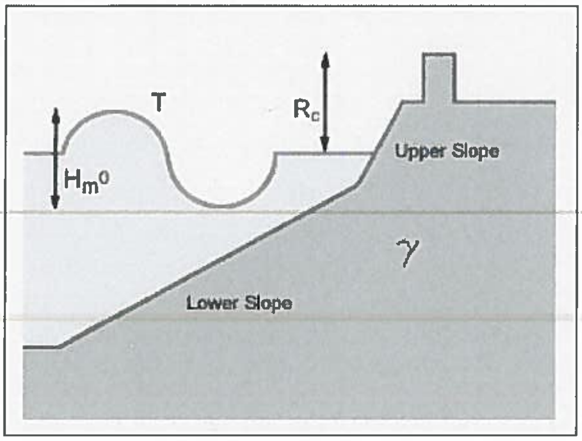
Composite Slope with (Small Vertical) Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



T (wave period)	2.7	s	○ Tm ◉ Tp ○ Tm-1,0
H_{m0} (Wave Height at the Toe of the Structure)	0.67	m	
R_c (Freeboard - The height of the crest of the wall above still water level (m))	1.4	m	output
Lower Slope (e.g. 1 in 2)	1 in 3		
Upper Slope (e.g. 1 in 2)	1 in 3		
γ (coefficient for reduction factors)	Basalt (0.9) ▼		

Calculate Overtopping Rate

Beta Results

Breaking Type / Other Info

Breaking waves

Mean overtopping discharge rate per metre run of seawall (l/s/m)

0.001

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Ref (10)

Wave Overtopping

Calculation Tool

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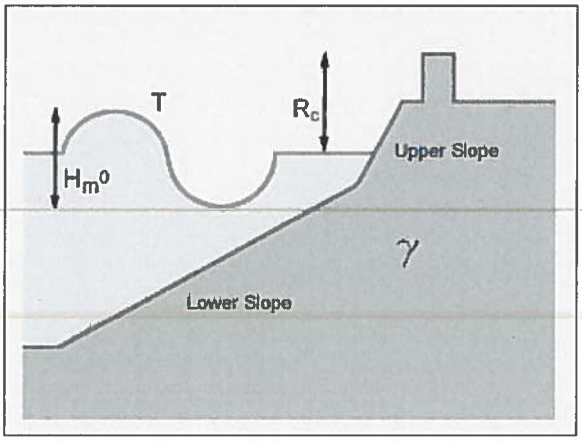
Composite Slope with (Small Vertical) Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



Beta Results

Breaking Type / Other Info

Breaking waves

Mean overtopping discharge rate per metre run of seawall (l/s/m)

0.969

T
(wave period) 2.7 s ○ Tm ◉ Tp ○ Tm-1,0

H_{m0}
(Wave Height at the Toe of the Structure) 0.67 m

R_c
(Freeboard - The height of the crest of the wall above still water level (m)) 0.62 m

Lower Slope
(e.g. 1 in 2) 1 in 3

Upper Slope
(e.g. 1 in 2) 1 in 3

γ
(coefficient for reduction factors) Basalt (0.9)

Calculate Overtopping Rate

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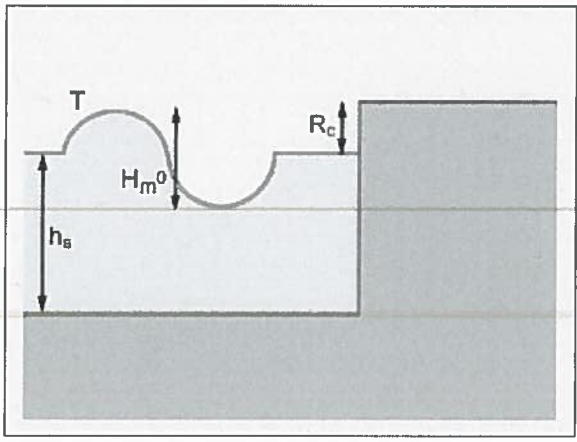
Ref (11)

Wave Overtopping

Calculation Tool

Vertical Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



T (Wave Period)	2.7	s	○ Tm ◉ Tp ○ Tm-1,0
H_{m0} (Wave Height at toe of Structure)	0.67	m	
R_c (Freeboard - the height of the crest of the wall above still water level)	1.57	m	
h_s (Water depth at toe of structure)	3	m	

Calculate Overtopping Rate

what is this line? amend

Beta Results

Wave Type / Other Info

Non Impulsive

Mean overtopping discharge rate per metre run of seawall (l/s/m)

1.012

Ref (12)

Wave Overtopping

Calculation Tool

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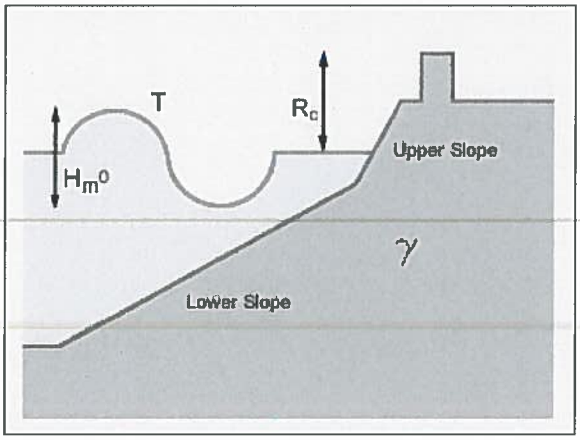
Composite Slope with (Small Vertical) Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



Beta Results

Breaking Type / Other Info

Breaking waves

Mean overtopping discharge rate per metre run of seawall (l/s/m)

1.013

T (wave period): 2.8 s ○ Tm ◉ Tp ○ Tm-1,0

H_{m0} (Wave Height at the Toe of the Structure): 0.79 m

R_c (Freeboard - The height of the crest of the wall above still water level (m)): 0.72 m

output

Lower Slope (e.g. 1 in 2): 1 in 3

Upper Slope (e.g. 1 in 2): 1 in 3

γ (coefficient for reduction factors): Basalt (0.9)

Calculate Overtopping Rate

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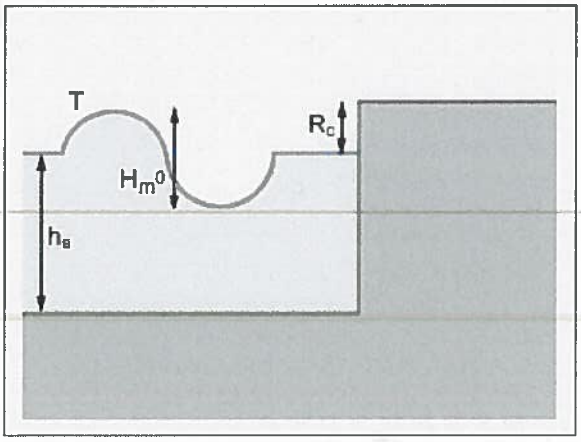
Vertical Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



Beta Results

Wave Type / Other Info

Non Impulsive

Mean overtopping discharge rate per metre run of seawall (l/s/m)

1.011

T (Wave Period)	2.8	s	○ Tm ◉ Tp ○ Tm-1,0
H_{m0} (Wave Height at toe of Structure)	0.79	m	
R_c (Freeboard - the height of the crest of the wall above still water level)	1.96	m	
h_s (Water depth at toe of structure)	3	m	where from?

Calculate Overtopping Rate

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Ref (14)

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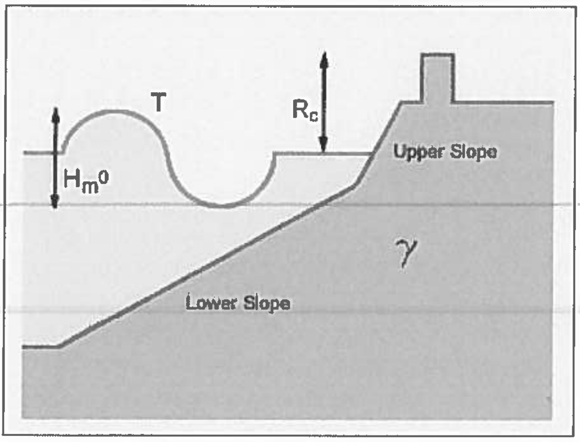
Composite Slope with (Small Vertical) Wall

Method Selection ○ **Probabilistic** ◉ **Deterministic**



Beta Results

Breaking Type / Other Info

Breaking waves

Mean overtopping discharge rate per metre run of seawall (l/s/m)

0.001

T (wave period) 2.8 s ○ Tm ◉ Tp ○ Tm-1,0

H_{m0} (Wave Height at the Toe of the Structure) 0.79 m

R_c (Freeboard - The height of the crest of the wall above still water level (m)) 1.6 m

Lower Slope (e.g. 1 in 2) 1 in 3

Upper Slope (e.g. 1 in 2) 1 in 3

γ (coefficient for reduction factors) Basalt (0.9)

Calculate Overtopping Rate

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Appendix E Reservoir Overflow Stage Discharge Calculation

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Glasgow, UK
G2 7HX
+44.(0)141 2042511 Fax +44.(0)141 2263109

CALCULATION REFERENCE SHEET

Calculation Number:	Stocks Reservoir-Cal-003	**Calculation Title:**	Stocks Reservoir: Spillweir Analysis

Purpose/Work Stage		**RELATED DOCUMENTS**
		Stocks Reservoir - Cal - 001
		Stocks Reservoir - Cal - 002
Method of Checking		**Results used in:**
		Stocks Flood Study

Document History Record

Rev	Date	Description/Reason for Issue	Orig	Chkr	Reviewer
0A	07/10/2014	For use in Stocks Flood Study	DA	DS	

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CALCULATION REFERENCE SHEET

Client: United Utilities		Job No: / Calc No:	B16000EP / Calc - 003
Project: B16000EP - Stocks Reservoir		By: D. Armour	Date: 02/10/2014
Subject: Stocks Reservoir - Spillweir Analysis		Checked: D. Steven	Date: 07/10/2014
		Reviewed:	Date:

Objectives

To examaine the spillweir hydraulics and check if the weir will remain modular up to PMF.
If does not remain modular what is the stage-discharge relationship?

Reference, Documents, Standards

2011 S10 Report
Topo Survey - 0304_NL01_A.dwg
Stocks Reservoir - Cal - 002

Design Criteria
N/A

Conclusions & Recommendations

Spillweir will not remian modular up to a PMF event.

New stage-discharge relationship for spillweir shown on page 9.

New stage-discharge relationship to be routed through ISIS by Hydrology team to find PMF outflow and stillwater floodrise at the reservoir

JACOBS CALCULATION SHEET

Office	GLASGOW	Page No.	1	Cont'n Page No.	2
Job No. & Title	B16000EP Stocks Reservoir	Originator	DA.	Date	2/10/14
Section	Stocks Reservoir – Stage Discharge Rel'ship	Checker	DS	Date	7/10/14

Task.

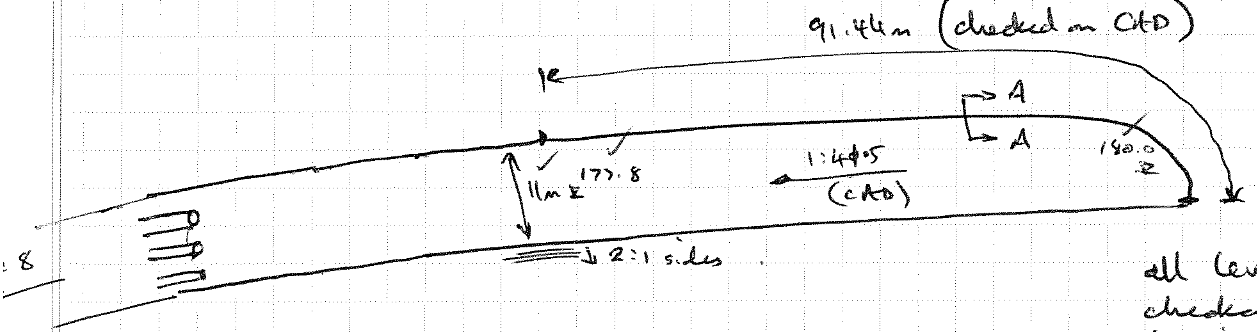
examine o'flow hydraulics and check the following

a) will the weir be modular up to PMF ? ✓ NO

b) if not modular, propose alternative stage discharge relationship ✓ see p9

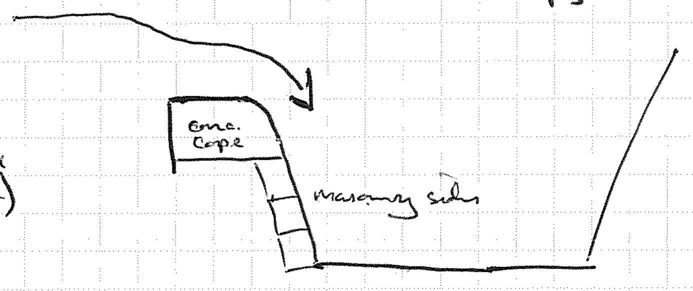
c) check downstream spillway hyd. calcs by D Stam / R Holman ✓ see attached

Weir Details



all levels / lengths etc checked on CAD drawing (topo survey) see P3

- weir crest 180.57 ✓
- dam crest 183.71 ✓
- weir C_D = 1.7 ✓ (assumed as being reasonable representation)



- PMF outflow – 356 m^3/s ✓

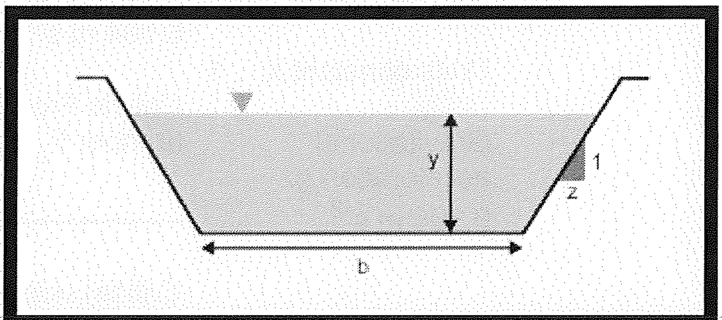
- critical slope for PMF = 1:500 ✓
- critical depth = 4.42m ✓ (V_c = 6.09 m/s)
- normal " = 1.98m ✓ (V_n = 15.03 m/s)

} see output overleaf

2

OS

onlinechannel04.php: Critical slope in a prismatic channel



Definition sketch for a prismatic channel

Formulas

$Q = (k/n) AR^{2/3}S^{1/2}$

$A = y(b + zy)$

$P = b + 2y(1 + z^2)^{1/2}$

$T = b + 2zy$

$R = A/P$

$D = A/T$

$V = Q/A$

$F = V/(gD)^{1/2}$

$S_c = gn^2 (P_c/T_c)/(k^2R_c^{1/3})$

INPUT DATA:

Select:
SI units (metric)
U.S. Customary units

Flow discharge Q: 356 $m^3 s^{-1}$

Bottom width b: 11 m

Side slope z: 0.5 m/m

Bottom slope S: 0.025 m/m

Manning's n:

INTERMEDIATE CALCS:

Normal flow depth y_n: 1.975 m

Normal flow area A_n: 23.679 m^2

Normal wetted perimeter P_n: 15.417 m

Normal top width T_n: 12.975 m

Normal hydraulic radius R_n: 1.536

OUTPUT:

Critical flow depth y_c: 4.422 m

Critical flow area A_c: 58.413 m^2

Critical wetted perimeter P_c: 20.887 m

Critical top width T_c: 15.422 m

0.014

m

Normal hydraulic depth D_n: 1.825 m

Normal flow velocity V_n: 15.034 m s^{-1}

Normal Froude number F_n: 3.554

Units constant k: 1

Gravitational acceleration g: 9.806 m s^{-2}

Critical hydraulic radius R_c: 2.797 m

Critical hydraulic depth D_c: 3.788 m

Critical flow velocity V_c: 6.094 m s^{-1}

Critical Froude number F_c: 1

Critical slope S_c: 0.001848 m/m

Your request was processed at 01:51:29 am on October 2nd, 2014 [141002 01:51:29].

Thank you for running onlinechannel_04. Please call again. [Version 1.1.1, 140618]

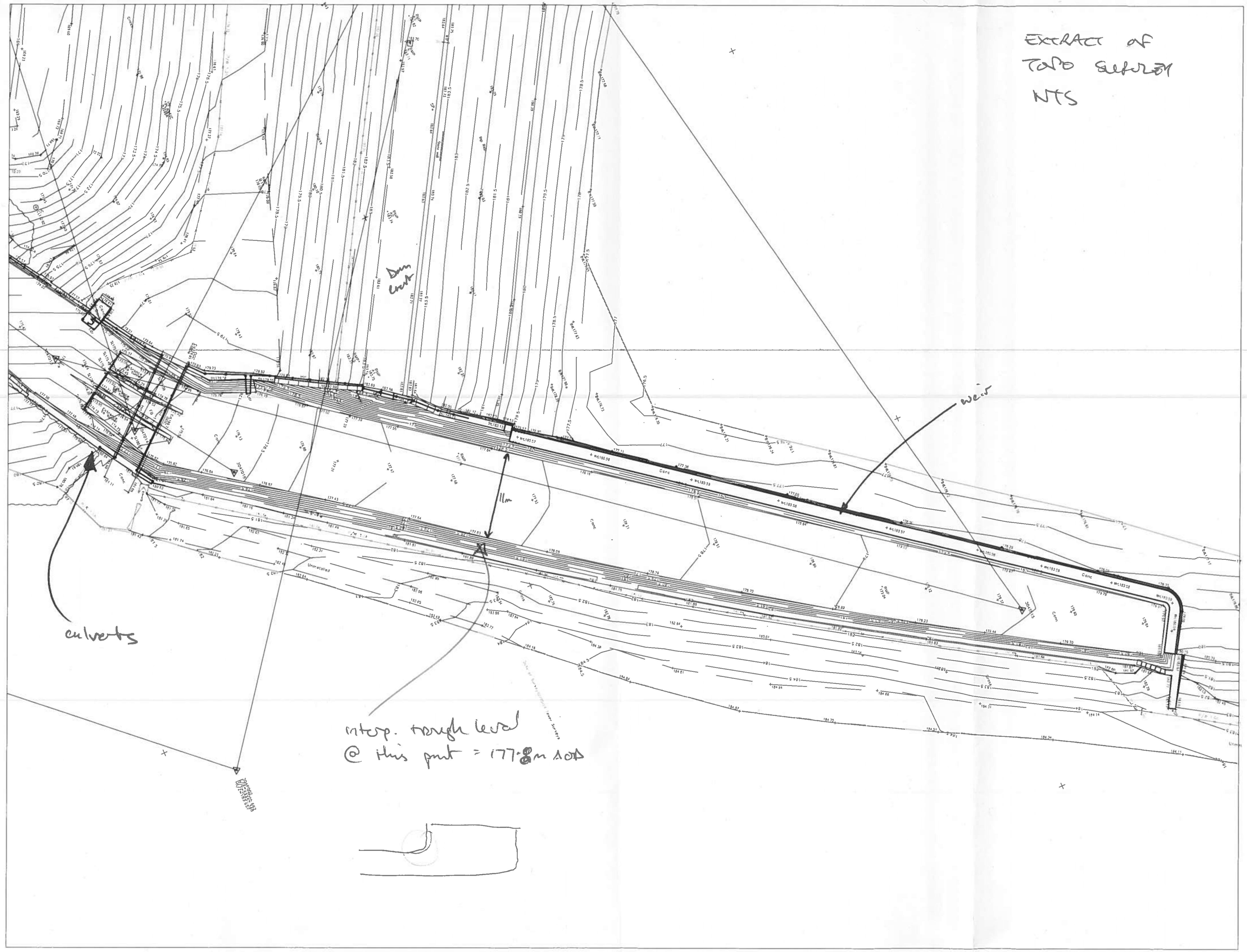
888888

online calc					
normal depth	critical depth	normal and critical depth		discharge in channel	critical slope
normal depth in culvert	critical depth in culvert	discharge in culvert		discharge sluice	discharge weir
M1 wsprofile	M2 wsprofile	M3 wsprofile	S1 wsprofile	S2 wsprofile	S3 wsprofile
C1 wsprofile	H2 wsprofile	A2 wsprofile	C3 wsprofile	H3 wsprofile	A3 wsprofile
sequent	energy loss	initial	efficiency HJ	critical width constriction	

P3

DS✓

EXTRACT OF
TOPO SURVEY
NTS



JACOBS CALCULATION SHEET

Office	9a4	Page No.	4	Cont'n Page No.	5
Job No. & Title	B16000ER Stocks	Originator	DA	Date	2/10/14
Section	weir check	Checker	DS	Date	7/10/14

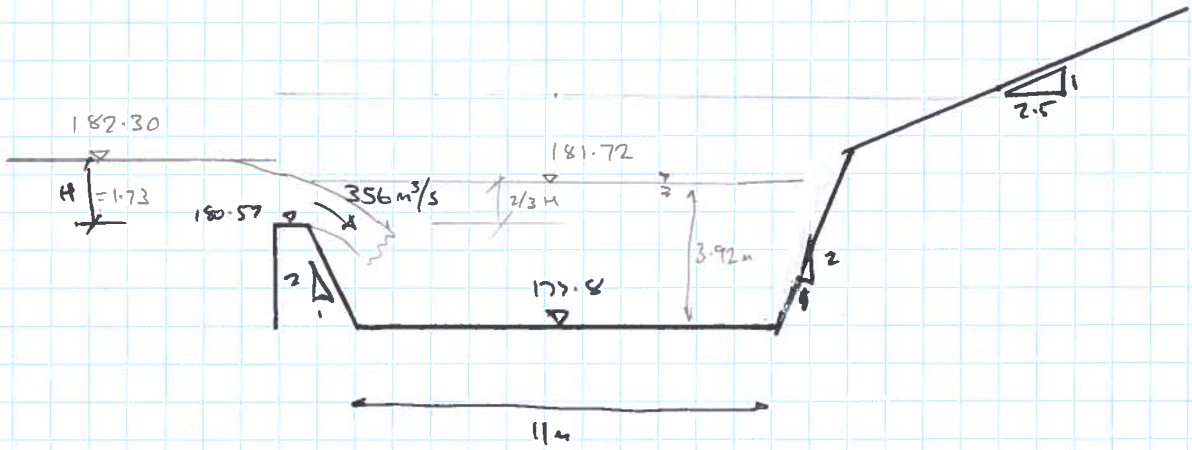
Current Reservoir Routing calc, assuming 'free' weir discharge estimates flood rise as 182.3m AOD. For weir to remain modular, the trough level must be no more than 2/3 × H* , i.e. downstream trough level
= (182.3 − 180.57) × 2/3 + 180.57 = 181.72m AOD (see sketch below)

At this flow, 356m³/s, the flow area = 3.92 × 11 + $\left(\frac{3.92}{2} \times 3.92\right)$ = 50.8m²

Velocity = Q/A = 356/50.8 = 7m/s

Velocity head = 2.5m. ($v^2/2g$ = 49/20 ~ 2.5)

Total head = 181.72 + 2.5 = 184.22 m > Reservoir level ∴ not possible.



This simple check shows that weir is not modular up to PMF.
and is likely to be drowned.

* based on CIRIA TN 134, page 92 fig 3.10 (middle curve)

JACOBS CALCULATION SHEET

Office	GLA	Page No.	5	Cont'n Page No.	6
Job No. & Title	B16000 EQ Stocks	Originator	DA	Date	2/10/14
Section	weir check	Checker	DS	Date	7/10/14

Similarly, taking the trough channel slope (1:40) and assumed mannings no 0.014, assume critical depth @ u/s end and run a grad. varied flow profile for S2 curve (ie supercritical flow since 1:40 > critical slope (1:500)) See output overleaf.

By around 90m (ie weir length), the flow could be 2.8m deep, and with vel. of 10.23 m/s (vel. head = 5.3m) ✓
similarly total head is > res. level ∴ not possible.

Assume conservatively that critical depth occurs at the d/s end of the weir and depth of water in trough behind this point is 1.5 x crit depth (ie crit depth + vel. head) prepare a spreadsheet showing trough level to show when channel takes control of flows over the weir. see output over following pages

Verification check on output.

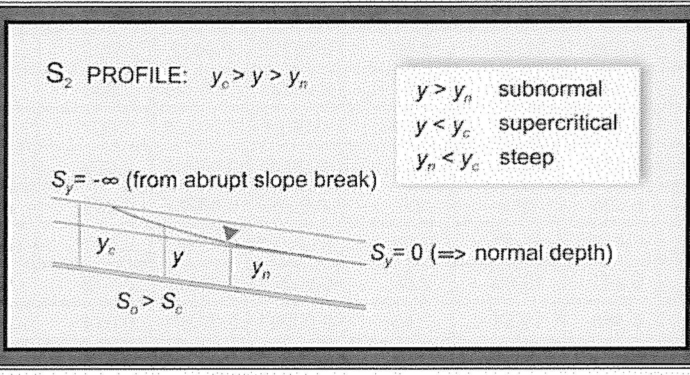
#1 : latest SIO reports – flood routing, PMF 256.88 m³/s is equivalent to a res level of 182.99. using new stage discharge for the same level, flow = 240 m³/s ∴ within 6%. Also, for same flow, level = 183.22 ie. 230mm difference (<10%) ~~it is likely that the~~ ✓

#2 : 1969 hydraulic model shows weir is modular up to 150 m³/s then channel controlled beyond this. This is similar to spreadsheet output. It appears as if the original weir level was ~ 4ft higher than currently ~~[illegible]~~ is*. The original channel appears the same as current.

* suggests weir should drown sooner!

DS

online_wsprofiles_25: S_2 water-surface profile



Description

Example

Summary

Reference

Disclaimer

INPUT DATA:

Select: SI units (metric) / U.S. Customary units [Choose S.I. Units or

Enter discharge Q (m^3/s) [cfs] : 356 Enter bottom width B (m) [ft] : 11 Enter side slope z (z H:1 V): 0.5

Enter Manning's n: 0.014 Enter bottom slope S_o (m/m) [ft/ft] : 0.025

Enter number of computational intervals n (suggested range 50-200) [If left blank, a default value of 100 will be used]:

Enter number of tabular output intervals m (suggested range 10-50) [If left blank, a default value of 10 will be used]: 50

Enter flow depth at the upstream boundary y_d (m) [ft] [If left blank, program will use critical depth]:

To calculate critical depth, the program requires the following hydraulic and geometric data for the upstream channel:
[Leave any box blank if the value is the same as the corresponding value entered above].

Enter u/s discharge $Q_{u/s}$ (m^3/s) [cfs] : Enter u/s bottom width $B_{u/s}$ (m) [ft] : Enter u/s size slope $z_{u/s}$ (z H:1 V):

ECHO OF INPUT:

Discharge Q = 356 m^3 s^{-1} Bottom width B = 11 m Side slope z = 0.5 m/m

Manning's n = 0.014 Bottom slope S_o = 0.025 m/m

Number of computational intervals n = 100 Number of tabular output intervals m = 50

Flow depth at the upstream boundary y_c = 4.42108 m [y_c was calculated by default]

Discharge $Q_{u/s}$ = 356 m^3 s^{-1} Bottom width $B_{u/s}$ = 11 m Side slope $z_{u/s}$ = 0.5 m/m

---

OUTPUT:

Computational depth interval Δy = 0.024 m Tabular output depth interval $(\Delta y)_t$ = 0.049 m

Normal depth y_n = 1.975 m Normal-depth Froude number F_n = 3.553

k	Depth (m)	Area (m^2)	Velocity (m s^{-1})	Velocity head (m)	Specific head (m)	Wetted perimeter (m)	Hydraulic radius (m)	Friction slope (m/m)	Average slope (m/m)	Specific head difference (m/m)	Length increment (m)	Total length (m)

0	4.421	58.4	6.1	1.894	6.315	20.89	2.8	0.00185	-	-	-	0
2	4.372	57.65	6.18	1.943	6.316	20.78	2.77	0.00192	0.0019	0.001	0.03	0.04
4	4.323	56.9	6.26	1.995	6.318	20.67	2.75	0.00199	0.00197	0.002	0.07	0.16
6	4.274	56.15	6.34	2.049	6.323	20.56	2.73	0.00206	0.00204	0.003	0.11	0.36
8	4.225	55.41	6.43	2.104	6.33	20.45	2.71	0.00214	0.00212	0.004	0.16	0.65
10	4.176	54.66	6.51	2.162	6.338	20.34	2.69	0.00222	0.0022	0.005	0.2	1.03
12	4.128	53.92	6.6	2.222	6.349	20.23	2.67	0.00231	0.00229	0.006	0.25	1.51
14	4.079	53.18	6.69	2.284	6.362	20.12	2.64	0.0024	0.00238	0.007	0.31	2.09
16	4.03	52.45	6.79	2.348	6.378	20.01	2.62	0.0025	0.00247	0.008	0.36	2.79
18	3.981	51.71	6.88	2.415	6.396	19.9	2.6	0.0026	0.00257	0.009	0.42	3.6
20	3.932	50.98	6.98	2.485	6.417	19.79	2.58	0.00271	0.00268	0.011	0.48	4.53
22	3.883	50.25	7.08	2.558	6.441	19.68	2.55	0.00282	0.00279	0.012	0.55	5.6
24	3.834	49.53	7.19	2.634	6.468	19.57	2.53	0.00294	0.00291	0.014	0.62	6.81
26	3.785	48.8	7.29	2.712	6.498	19.46	2.51	0.00306	0.00303	0.015	0.7	8.17
28	3.736	48.08	7.4	2.794	6.531	19.35	2.48	0.00319	0.00316	0.017	0.78	9.68
30	3.687	47.36	7.52	2.88	6.567	19.25	2.46	0.00333	0.0033	0.019	0.87	11.37
32	3.638	46.64	7.63	2.969	6.608	19.14	2.44	0.00348	0.00344	0.021	0.96	13.24
34	3.59	45.93	7.75	3.062	6.652	19.03	2.41	0.00364	0.0036	0.023	1.06	15.3
36	3.541	45.21	7.87	3.16	6.7	18.92	2.39	0.0038	0.00376	0.025	1.16	17.57
38	3.492	44.5	8	3.261	6.753	18.81	2.37	0.00398	0.00393	0.027	1.28	20.07
40	3.443	43.8	8.13	3.368	6.81	18.7	2.34	0.00416	0.00412	0.029	1.4	22.81
42	3.394	43.09	8.26	3.479	6.873	18.59	2.32	0.00436	0.00431	0.032	1.53	25.81
44	3.345	42.39	8.4	3.595	6.94	18.48	2.29	0.00457	0.00452	0.034	1.68	29.09
46	3.296	41.69	8.54	3.717	7.013	18.37	2.27	0.00479	0.00474	0.037	1.83	32.68
48	3.247	40.99	8.69	3.845	7.092	18.26	2.24	0.00503	0.00497	0.04	2	36.6
50	3.198	40.29	8.84	3.978	7.177	18.15	2.22	0.00528	0.00522	0.043	2.19	40.89
52	3.149	39.6	8.99	4.119	7.268	18.04	2.19	0.00555	0.00548	0.047	2.39	45.57
54	3.1	38.91	9.15	4.267	7.367	17.93	2.17	0.00584	0.00577	0.05	2.61	50.68
56	3.051	38.22	9.31	4.422	7.473	17.82	2.14	0.00615	0.00607	0.054	2.86	56.26
58	3.003	37.54	9.48	4.585	7.587	17.71	2.12	0.00648	0.00639	0.058	3.13	62.38
60	2.954	36.85	9.66	4.757	7.71	17.6	2.09	0.00683	0.00674	0.063	3.42	69.07
62	2.905	36.17	9.84	4.937	7.842	17.5	2.07	0.00721	0.00711	0.067	3.76	76.41
64	2.856	35.49	10.03	5.128	7.984	17.39	2.04	0.00762	0.00751	0.072	4.13	84.47
66	2.807	34.81	10.23	5.329	8.136	17.28	2.02	0.00805	0.00794	0.078	4.54	93.35
68	2.758	34.14	10.43	5.542	8.3	17.17	1.99	0.00852	0.0084	0.083	5.02	103.14
70	2.709	33.47	10.64	5.767	8.476	17.06	1.96	0.00903	0.0089	0.089	5.56	113.97
72	2.66	32.8	10.85	6.004	8.665	16.95	1.94	0.00957	0.00944	0.096	6.18	126.01
74	2.611	32.13	11.08	6.256	8.868	16.84	1.91	0.01016	0.01001	0.103	6.89	139.42
76	2.562	31.47	11.31	6.523	9.086	16.73	1.88	0.0108	0.01064	0.111	7.73	154.45
78	2.513	30.81	11.56	6.807	9.32	16.62	1.85	0.0115	0.01132	0.119	8.73	171.39
80	2.464	30.15	11.81	7.108	9.573	16.51	1.83	0.01225	0.01206	0.128	9.93	190.61
82	2.416	29.49	12.07	7.429	9.844	16.4	1.8	0.01307	0.01286	0.138	11.39	212.62
84	2.367	28.83	12.35	7.77	10.137	16.29	1.77	0.01396	0.01373	0.149	13.22	238.09
86	2.318	28.18	12.63	8.134	10.452	16.18	1.74	0.01493	0.01468	0.161	15.56	267.95
88	2.269	27.53	12.93	8.523	10.792	16.07	1.71	0.01599	0.01572	0.173	18.66	303.61
90	2.22	26.88	13.24	8.939	11.158	15.96	1.68	0.01716	0.01686	0.187	22.96	347.19
92	2.171	26.24	13.57	9.384	11.555	15.85	1.65	0.01844	0.01811	0.202	29.31	402.3
94	2.122	25.59	13.91	9.861	11.983	15.75	1.63	0.01984	0.01948	0.219	39.59	475.65

B16000Stocks Reservoir - Calculation of Stage Discharge Relationship

originated D Armour 02/10/2014

checked/ reviewed D Steven 7/10/14

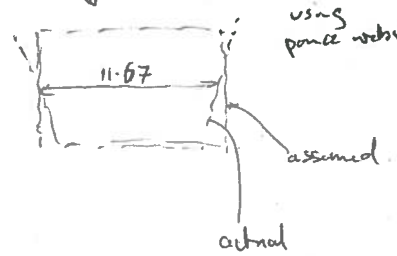
dam crest	183.71	mAOD	source (topo survey)
Weir crest	180.57	mAOD	source (topo survey)
Trough invert	177.8	mAOD	source (topo survey)
Weir Cd	1.7		source (hydrology calcs, and concurred by originator)
Weir length	91.44	mAOD	source (topo survey)
Side slopes (z)	0.5		source (topo survey)
Channel width (base)	11	mAOD	source (topo survey),

		free flow	channel flow (no trans losses included since channel is straight)								Output (1)			Dam overtopping		Output (2)
Flow	Weir Head	Res level	Crit depth	rev'd channel width	Crit depth (refined)	control level	flow area	vel head	u/s level	submergence	final reservoir leve	flow description	Dam overtopping?	head over dam crest	Adjusted level	final reservoir level
0	0	180.57	0.00	11.00	0.00	177.80	0.00	0.00	177.80	n/a	180.57	weir control	flood contained	n/a	n/a	180.57
25	0.30	180.87	0.81	11.20	0.80	178.60	8.94	0.40	179.00	-5.31	180.87	weir control	flood contained	n/a	n/a	180.87
50	0.47	181.04	1.28	11.32	1.26	179.06	14.23	0.63	179.69	-1.88	181.04	weir control	flood contained	n/a	n/a	181.04
75	0.62	181.19	1.68	11.42	1.64	179.44	18.70	0.82	180.26	-0.51	181.19	weir control	flood contained	n/a	n/a	181.19
100	0.75	181.32	2.03	11.51	1.97	179.77	22.71	0.99	180.76	0.26	181.32	weir control	flood contained	n/a	n/a	181.32
125	0.86	181.43	2.36	11.59	2.28	180.08	26.41	1.14	181.22	0.75	181.43	weir control	flood contained	n/a	n/a	181.43
150	0.98	181.55	2.66	11.67	2.56	180.36	29.88	1.28	181.65	1.10	181.65	channel control	flood contained	n/a	n/a	181.65
175	1.08	181.65	2.95	11.74	2.83	180.63	33.18	1.42	182.04	1.36	182.04	channel control	flood contained	n/a	n/a	182.04
200	1.18	181.75	3.23	11.81	3.08	180.88	36.34	1.54	182.42	1.57	182.42	channel control	flood contained	n/a	n/a	182.42
225	1.28	181.85	3.49	11.87	3.32	181.12	39.38	1.66	182.78	1.73	182.78	channel control	flood contained	n/a	n/a	182.78
250	1.37	181.94	3.74	11.94	3.55	181.35	42.31	1.78	183.12	1.86	183.12	channel control	flood contained	n/a	n/a	183.12
275	1.46	182.03	3.99	12.00	3.76	181.56	45.16	1.89	183.45	1.97	183.45	channel control	flood contained	n/a	n/a	183.45
300	1.55	182.12	4.23	12.06	3.98	181.78	47.94	2.00	183.77	2.07	183.77	channel control	dam overtopped	n/a	n/a	183.77
325	1.63	182.20	4.46	12.11	4.18	181.98	50.64	2.10	184.08	2.15	184.08	channel control	dam overtopped	0.129197408	183.8391974	183.84
350	1.72	182.29	4.68	12.17	4.38	182.18	53.29	2.20	184.38	2.22	184.38	channel control	dam overtopped	0.204993353	183.9149934	183.91
375	1.80	182.37	4.90	12.23	4.57	182.37	55.88	2.30	184.67	2.28	184.67	channel control	dam overtopped	0.268544683	183.9785447	183.98
400	1.88	182.45	5.12	12.28	4.76	182.56	58.42	2.39	184.95	2.33	184.95	channel control	dam overtopped	0.325256329	184.0352563	184.04
425	1.95	182.52	5.33	12.33	4.94	182.74	60.91	2.48	185.22	2.38	185.22	channel control	dam overtopped	0.377370393	184.0873704	184.09
450	2.03	182.60	5.54	12.38	5.12	182.92	63.36	2.57	185.49	2.42	185.49	channel control	dam overtopped	0.42609117	184.1360912	184.14
475	2.10	182.67	5.74	12.43	5.29	183.09	65.78	2.66	185.75	2.46	185.75	channel control	dam overtopped	0.472159896	184.1821599	184.18
500	2.18	182.75	5.94	12.48	5.46	183.26	68.15	2.74	186.00	2.49	186.00	channel control	dam overtopped	0.516073707	184.2260737	184.23

$Q = C_d B H^{3/2}$

assumes rectang. channel i.e. (simplified)

eg. for 150 m³/s



re-checked using ponce website

no transition losses assumed since there is no narrowing of channel

$Q = C_d B H^{3/2}$

B = Length of dam = 360 m

$C_d = 1.5$

considers dam crest as aux. overflow

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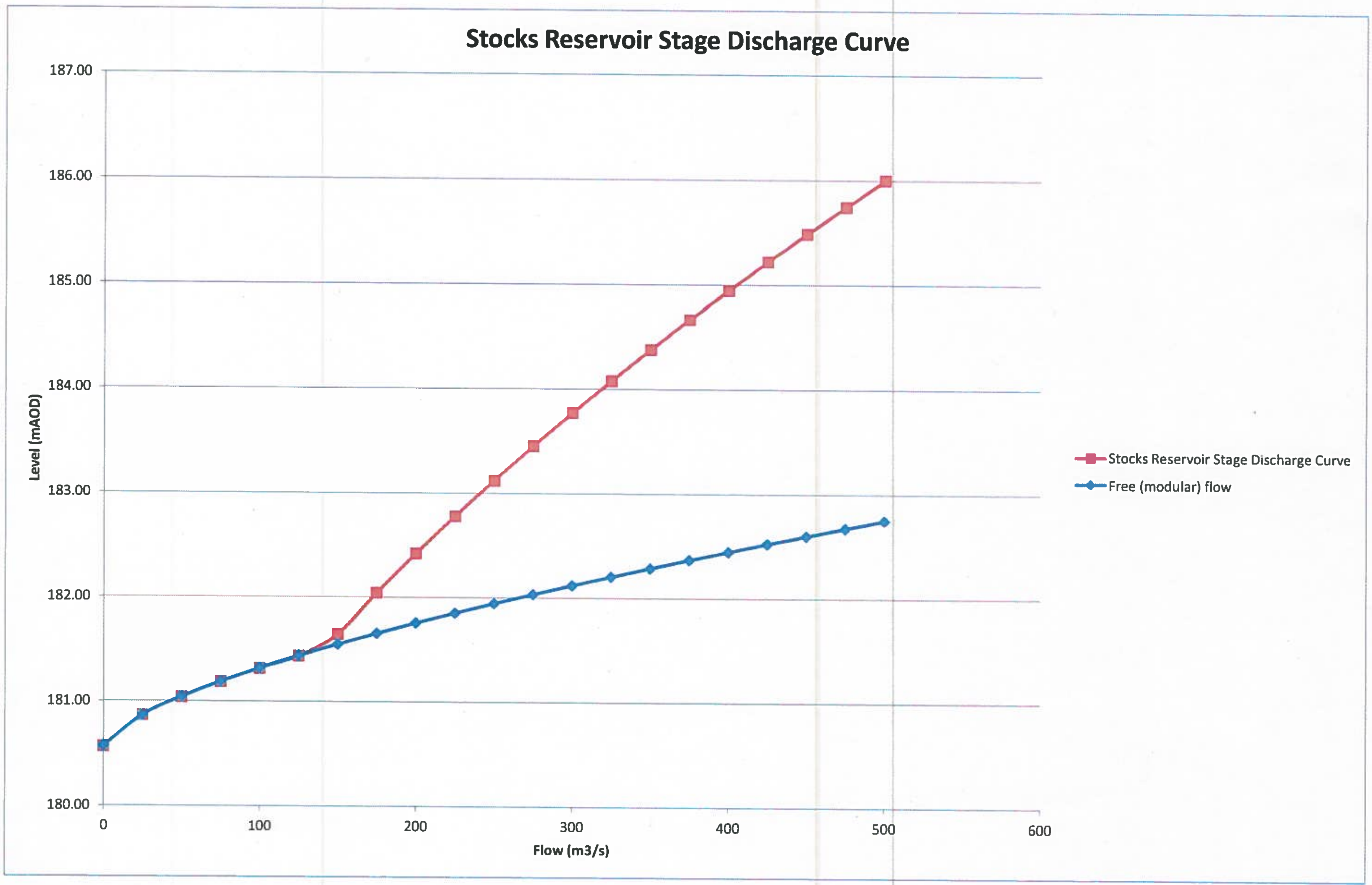
weir calcs



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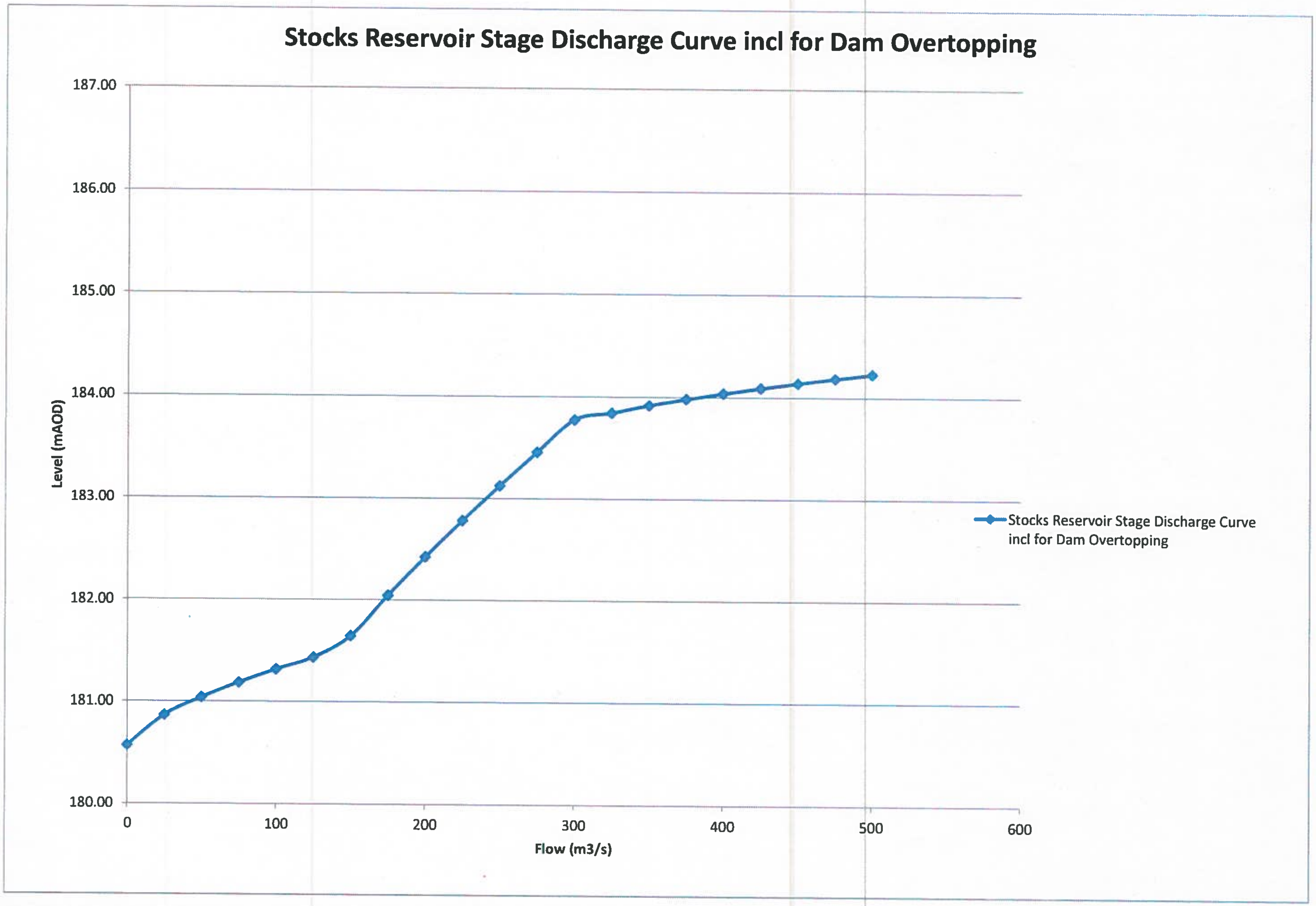
SDR (free, subm)

a

DS



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SDR (incl dam o'top)

Calc Ref ① (see p10)

FYLDE WATER BOARD

REPORT

ON

STOCKS RESERVOIR

OCTOBER 1969

F. LAW, B.Sc. Tech., F.I.C.E., M.I.W.E.,
Engineer,
Fylde Water Board,
Sefton Street,
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BINNIE & PARTNERS,
Chartered Engineers,
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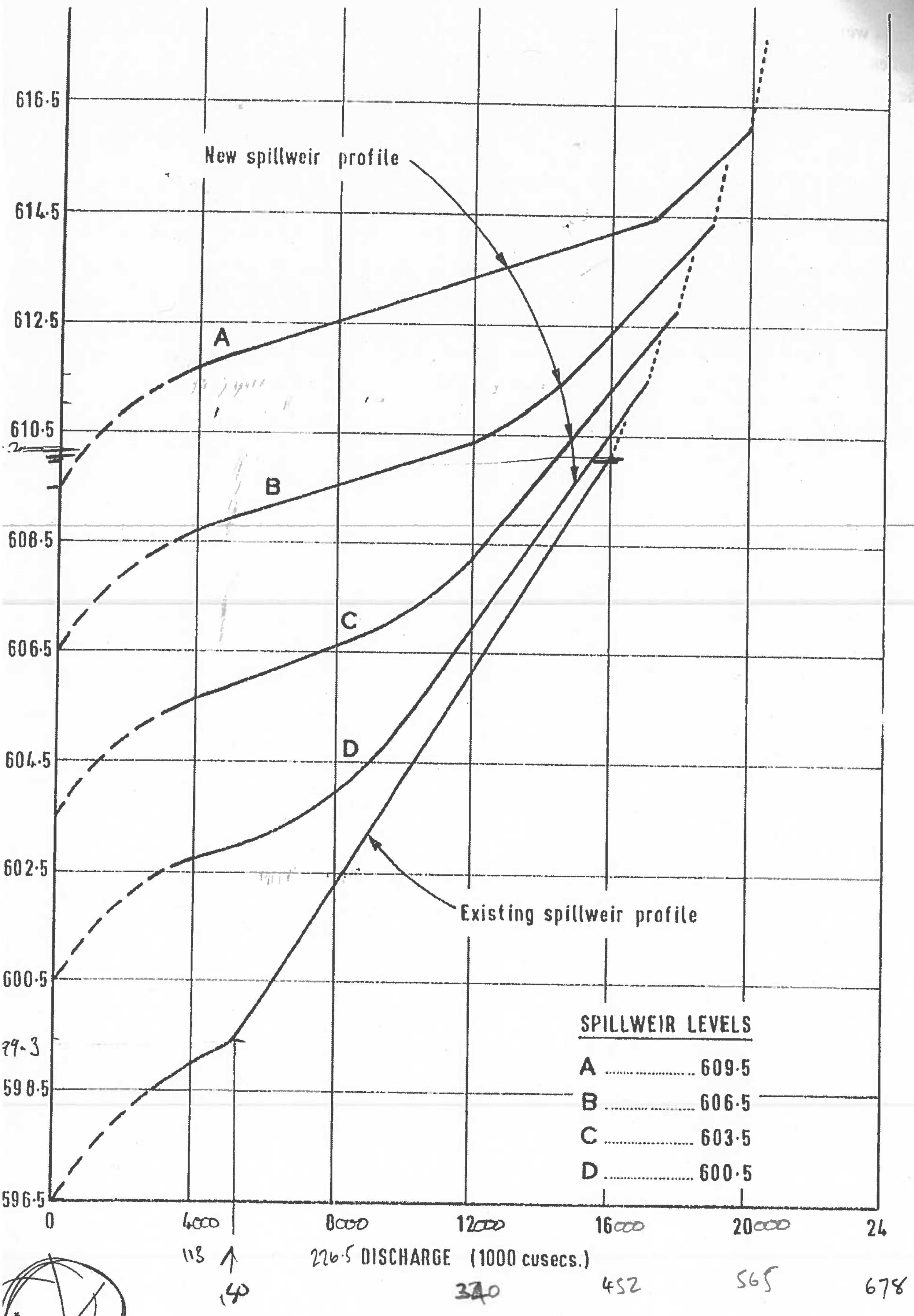

SPILLWAY RATING CURVES

Calc Ref 2

See P20

UNITED UTILITIES

STOCKS RESERVOIR

Report of the Result of an Inspection made under Section 10 of the Reservoirs Act 1975 on 24 June 2011
by P J Williams CEng, FICE

July 2011

periodical inspection of 1998 a review was made of the flood hydrology for the reservoir and the wave surcharge allowances calculated. The assessment showed the worst case PMF flood event to be the winter PMF with snow melt and gave the reservoir water levels and flows summarised below.

Winter PMF peak in flow	=	513.58 m^3/sec
Routed peak out flow	=	256.88 m^3/sec
Top water level	=	180.57 m AoD
Maximum still water flood level	=	182.99 m AoD
Dam crest level (minimum)	=	183.82 m AoD
Stillwater freeboard to dam crest level	=	0.83m
Wave surcharge allowance (calculated)	=	1.55m
Total surcharge level (flood + wave)	=	184.54 m AoD
Top of wave wall level	=	184.60 m AoD

Freeboard to top of wave wall under flood and wave allowance = 0.06m

With the newly constructed wave wall, the dam will retain the PMF flood event with the maximum wave surcharge allowance as calculated by the previous Inspecting Engineer (Mr BH Rofe). The wave surcharge allowance calculated is considered to be particularly prudent, as it allows for refraction of the wave along the reservoir. There is no recommendation to amend the calculated flood or wave surcharge levels at this stage.

11.6 *Discharge Outlets*

The reservoir water level can be lowered by means of the draw off pipe work, the scour outlet and the compensation water pipe work. A drawdown report was prepared by MWH in Oct 2002 and investigated the drawdown rates using a range of outlet pipes. The combined capacity of the scour outlet, compensation outlet and the lower draw-off main show that dewatering can be made at rates in excess of 10 m^3/sec with the reservoir at top water level. It is estimated that the reservoir could be drained to 50% of its depth in about 13 days using the scour, compensation and lower draw-off outlets. If just the scour outlet is used it would take about 25 days to drain to 50% depth. Draining the reservoir to 50% depth would remove approximately 89% of the stored water volume. These drawdown rates are seen as acceptable for an emergency however, it would be prudent to limit the rate of drawdown to no more than 0.6m/day in other cases.

B. Photos

Figure 6.1: Photo looking downstream showing end of spillway and compensation channel and natural channel downstream

Source: MMB, 2019

Figure 6.2: Photo looking upstream showing outfall of spillway, adjacent to compensation channel (foreground)

Source: MMB, 2019

Figure 6.3: Photo looking upstream showing compensation channel adjacent to site

Source: MMB, 2019

Figure 6.4: Photo showing outfall of compensation flow culvert

Source: MMB, 2019