PRINCE LLEWELYN RESERVOIR

Flood Study

JULY 2020

Incorporating

EC HARRIS BUILT ASSET CONSULTANCY

Hyder

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Front cover. Prince Llewelyn Reservoir

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1	29/11/2019	EB	First Issue	RG	NE
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APPENDICES

Executive Summary

Arcadis has been commissioned by Natural Resources Wales (NRW) to undertake a flood study to support the design of a new lowered spillway to enable the discontinuance of the Prince Llewelyn Reservoir under NRW's reservoir compliance programme. The reservoir, which is located in Snowdonia National Park, has a capacity of 5,477 m^3, which is well below the statutory 10,000 m^3 threshold of the Reservoirs Act 1975 (as amended by the Flood and Water Management Act 2010). Although the reservoir is not subject to any statutory safety requirements under this Act, NRW considers Prince Llewelyn Reservoir to be a high priority site and is treating it as a Category A/B dam in the spirit of the Act. In order to confirm the categorisation, NRW commissioned Arcadis to undertake flood mapping of reservoir failure (Arcadis, 2018a). The findings of this study supported a Category B classification for the dam.

This report, which is an update to an existing flood study (Arcadis, 2018b), summarises the methodology, findings and recommendations of the flood study calculation, which has been undertaken in accordance with the fourth edition of Floods and Reservoir Safety (ICE, 2015). To inform the study, a literature search and review has been carried out. A range of other data has also been collected, including historical maps, a topographic survey of the embankments and surrounding environments, and a bathymetric survey of the reservoir. In addition, a walkover survey, to identify reservoir inflows and outflows, and verify the catchment boundary, has been undertaken. The key objective of the update is to develop new spillway options that can be taken forward to design. To assess their viability, the update also includes an assessment of the downstream impacts of the options.

The bathymetry data have been used to generate a surface model of the reservoir bed (with and without silt), and a reservoir level-capacity relationship. This relationship has, in turn, been used for reservoir flood routing. In particular, a 1D hydraulic model of the reservoir and dam has been built in HEC-RAS and run for the design flood and safety check flood conditions (the 10,000 year flood and the Probable Maximum Flood (PMF), respectively). The FSR/FEH rainfall-runoff model, based on catchment descriptors, has been used to generate reservoir flood inflow hydrographs for both scenarios. The FEH13 DDF model has been used to estimate rainfall for the 10,000 year flood, while the FSR/FEH Probable Maximum Precipitation (PMP) approach has been applied to the PMF. The inflow hydrographs have been adjusted for reservoir lag time. An assessment of wave overtopping has also been made, using the overtopping prediction methods of the EurOtop Manual.

A summary table of the flood study calculation is provided in Appendix A, while key levels and volumes are illustrated in Figure i overleaf. Modelling results show that Prince Llewelyn Reservoir does not meet recommended safety standards for a Category A/B dam. In particular, there is zero freeboard and a lack of spillway capacity during both the design flood and the safety check flood. Wave overtopping discharges also far exceed the allowable rates.

NRW, as reservoir undertaker, is exploring options to retain or discontinue the reservoir. Two options have been considered as part of this study to determine a spillway configuration that is capable of replicating the flood attenuation provided by the existing arrangement. To do so, reservoir inflows for higher frequency return period events have been estimated using the Revitalised Flood Hydrograph (ReFH2) (Version 2.2) method and routed through the reservoir using the model to determine the baseline attenuation.

The preferred option, to meet the required design and safety check standards, reduces the existing retained volume of the reservoir by more than half. It involves lowering the existing spillway to an elevation of 194.7 m AOD (approx. 1.27 m), which would reduce the retained volume to 2,612 m^3 (without silt). The lowered spillway has been modelled as a 0.5 m wide and 0.3 m deep slot set within a 2 m wide spillway, with the entire embankment set at its existing average elevation (196.43 m AOD).

A second option, which looks at further reducing the retained volume, has also been identified. It involves lowering the spillway to an elevation of 193.5 m AOD, which would reduce the retained volume to 790 m^3 (without silt). The lowered spillway for this alternative option has been modelled as a 0.25 m wide and 0.45 m deep slot set within a 2 m wide spillway, with the embankment set at a minimum embankment elevation of 196.43 m AOD.

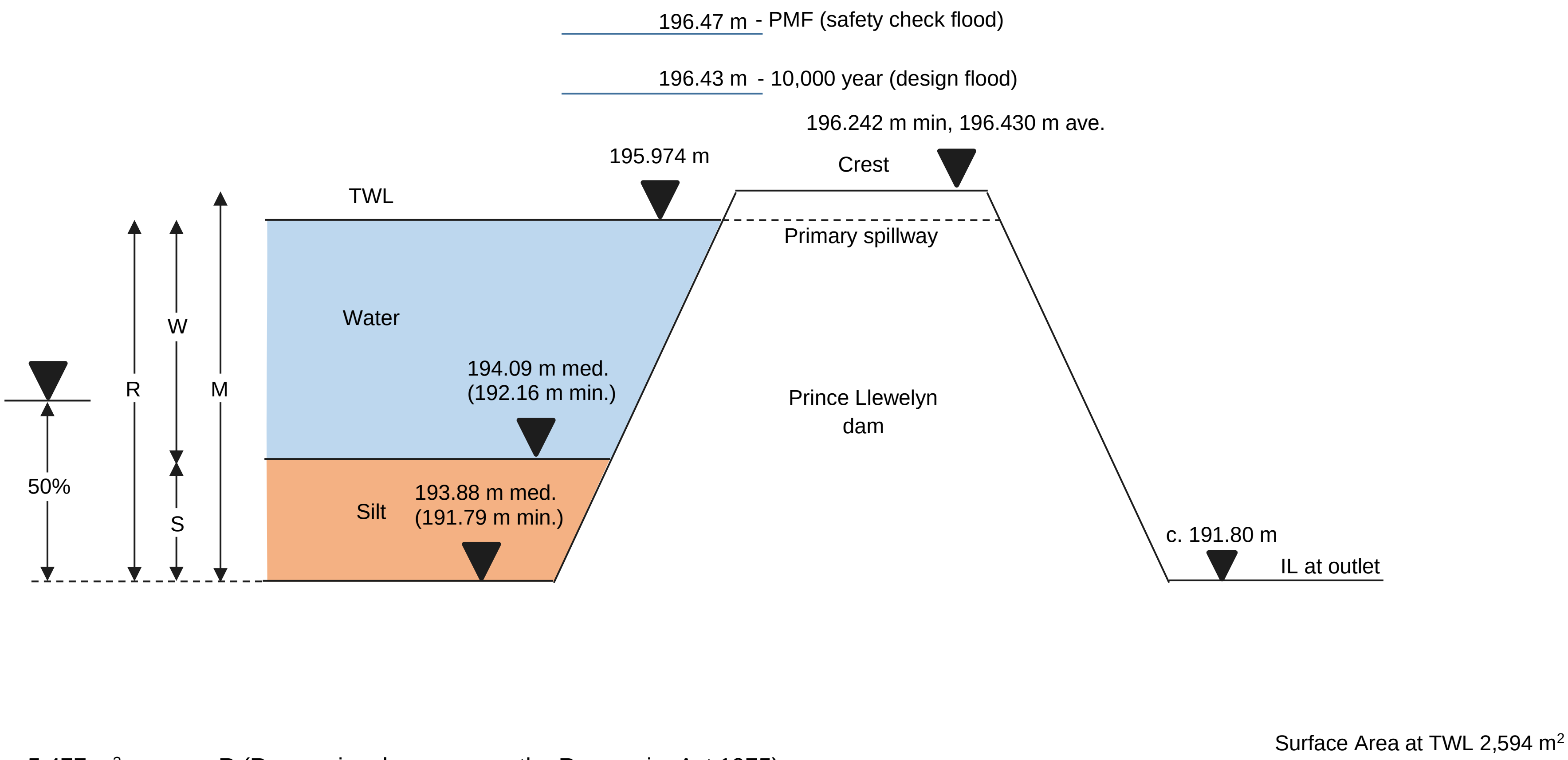
Both options meet the project requirements for attenuation and reservoir safety whilst reducing the retained volume. Option 1 is considered by Arcadis' Design Engineers and NRW to be preferable, as it would reduce the scope of removal and deconstruction works associated with the embankment and minimise environmental impacts. However, from a flood risk perspective the downstream consequences of a dam failure are further reduced with Option 2 as a result of the lower retained volume.

Updates following NRW Review of Flood Study

Following the issue of the draft Flood Study, it was agreed in consultations between NRW and the Qualified Civil Engineer (QCE) that, since Prince Llewelyn is not a Statutory Reservoir and the options investigated would further reduce the retained volume, the design flood could be changed from a 10,000 to a 1,000 year flood. Furthermore, it was also agreed that there would be no requirement to meet the safety check flood standard.

There was also a change in Arcadis' Design Engineers' and NRW's option preference, with a variation to Option 2 being favoured. This new option, 'Option 3', involves lowering the spillway to an elevation of 193.3 m AOD (lowered by 0.2m when compared to Option 2), which would reduce the retained volume to 599 m^3 (without silt). The lowered spillway for this alternative option has been modelled as a stepped spillway with the spillway crest 2 m wide and 0.6 m deep, stepping up to the existing embankment average elevation (196.43 m AOD).

Option 3 is considered by Arcadis' Design Engineers and NRW to be preferable, as it is easier to construct, maintain (reduced blockage risk) and meets the 1,000 year design flood safety standards. In terms of flood risk, it is considered to cause insignificant change downstream.



5,477 m^3	= R (Reservoir volume, as per the Reservoirs Act 1975)
622 m^3	= S (Silt volume)
4,855 m^3	= W (Water volume)
6,188 m^3	= M (Maximum volume, before reservoir overtops)
0 m	= F (Freeboard)
194.765 m	= 50% (50% volume drawdown level, 50% of R)

*All m in AOD
(ave. – average; med. – median)

Figure i. Schematic representation of Prince Llewelyn Reservoir

1 Introduction

1.1 Overview

Natural Resources Wales (NRW) owns Prince Llewelyn Reservoir, located in Snowdonia National Park, North Wales (Figure 1-1). This small raised reservoir has a capacity of less than 10,000 m^3 and is, therefore, not subject to any statutory safety requirements under the Reservoirs Act 1975 (as amended by the Flood and Water Management Act 2010). Nevertheless, NRW considers the reservoir to be a high priority site and is, consequently, treating it in the spirit of the Act (NRW, 2016a). To this end, NRW has commissioned Arcadis Consulting (UK) Ltd. (Arcadis) to carry out a flood study to support the design of a new lowered spillway for Prince Llewelyn Reservoir.

Figure 1-1. Prince Llewelyn Reservoir

1.2 Scope of Works

This report, which is an update to an existing flood study (Arcadis, 2019), summarises the methodology, findings and recommendations of the flood study, which has been undertaken in accordance with the fourth edition of Floods and Reservoir Safety (ICE, 2015). The aim of the study is to support the design of a new lowered spillway that would meet the required safety standards. The objectives are as follows:

- Undertake a historical literature review;
- Review available topographical information, in conjunction with a site visit, to verify the catchment area of the reservoir;

- Derive reservoir flood inflow hydrographs for the design flood and the safety check flood conditions;
- Generate flood hydrographs using ReFH2 for the 1-in-10 year, 1-in-30 year, 1-in-100 year with and without climate change, and 1-in-1,000 year events for the Prince Llewelyn Reservoir catchment;
- Update wave overtopping discharge calculations (mean and peak wave overtopping discharges) to take into account the updated formulae in the latest EurOtop Manual (November 2018);
- Reservoir routing of flood events (assessing attenuation and LAG) to generate reservoir outflow hydrographs for existing spillway conditions (baseline);
- Iterative routing of flood events (assessing attenuation and LAG) for up to four different spillway configurations to support the design development of a lowered spillway. This is required to develop a spillway design that results in reservoir outflow hydrographs that do not vary significantly (e.g. <5% peak flow) from the existing conditions for the 1-in-10 year, 1-in-30 year, 1-in-100 year with and without climate change, and 1-in-1,000 year events;
- Outline consideration of the possibility and consequences of flooding caused by blockage of the new spillway;
- Production of a Flood Consequences Assessment to demonstrate, based on insignificant change to the outflow hydrographs, that the proposed lowered spillway design would not result in an increase in downstream fluvial flood risk in the 1-in-10 year, 1-in-30 year, 1-in-100 year with and without climate change, and 1-in-1,000 year events;
- Production of a flood study report to confirm the potential for configuring a new lowered spillway design that would meet the recommended safety standards.

Following the issue of the draft flood study, and the agreement to change the design flood and preferred option, Arcadis' scope of work was expanded. This included the update of the flood study report, to confirm that the preferred option would meet the recommended design standard, and the update of the Flood Consequences Assessment, to confirm that the option would not result in an increase in downstream fluvial flood risk.

One small point is worth noting: in the published literature, alternative spellings Llewelyn, Llewellyn and Llywelyn are used in relation to Prince Llewelyn Reservoir and slate quarry. For consistency with NRW's project specification and local historian, Shaun Hewitt, Llewelyn has been adopted herein.

2 Study Area

2.1 Geographical and Historical Setting

Prince Llewelyn Reservoir is located on the north side of the Lledr valley, between the villages of Dolwyddelan and Betws-y-Coed (NGR SH 74232 53049; Figure 2-2). The rectangular-shaped reservoir covers an area of approximately 2,600 m^2. It is thought to be named after Llywelyn the Great (c.1173-1240), prince of Gwynedd and eventually ruler of most of Wales[1]. The reservoir dates back to the 1800s, when it is believed to have been constructed to supply water to Prince Llewelyn Slate Quarry. This open hillside quarry and pit was operational from about 1820 to 1934 (Richards, 1999 and 2007). A steam mill installed at the quarry in 1850 had been converted to water-wheel drive by the mid-1860s, suggesting that the reservoir was built around this time. Below the reservoir, in amongst the trees, there are many conduits and gullies where the water ran, and still runs a little in wet weather (Hewitt, pers. comm, 2017); local residents reportedly used the runoff to do their washing in the 1920s, after the quarry had closed. There is also the ruin of one of the water wheel houses and another smaller reservoir (now dry; Hewitt, *op. cit.*).

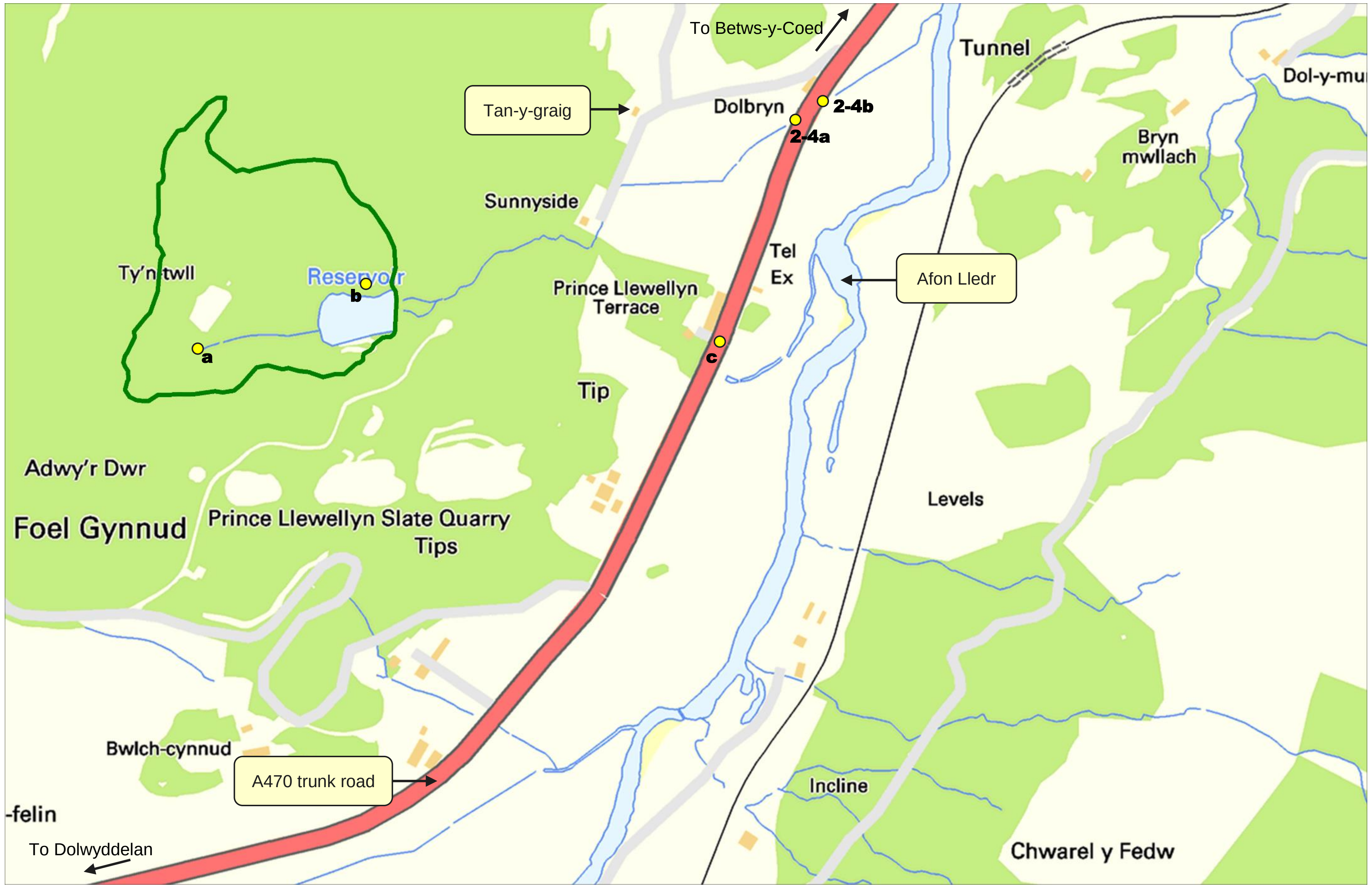
Prince Llewelyn Reservoir lies on the edge of a plantation forest and is surrounded by coniferous stands (Figure 2-2b). A handful of properties lie within 300 m of the toe of the dam, including a row of six terrace houses known as Prince Llewelyn Terrace (Figure 2-2c). The A470, the main trunk road between Cardiff and Llandudno, also passes within 260 m of the dam. In view of its surroundings, a Qualified Civil Engineer (QCE) has recommended that Prince Llewelyn Reservoir is treated as a Category A/B dam (NRW, 2016a). In other words, a dam breach “could endanger lives in a community” (ICE, 2015).

The reservoir used to be fed partly by a leat (Figure 2-2a). This leat was constructed around 1875 to carry water from the river below Llynau Diwaunydd to Prince Llewelyn Slate Quarry and the reservoir, supplementing water from the Afon Ystumiau (Figure 2-2 – map 3; North Pennines Archaeology Ltd, 2011). It was approximately 6.4 km long and crossed (and possibly intercepted) several minor streams to the south of Moel Siabod. The leat, which can be traced on old Ordnance Survey maps, was inspected in 2011 and found to be in a ruined state (Figure 2-1; North Pennines Archaeology Ltd, *op. cit.*). In light of this evidence and discussion with NRW[2], it has been assumed, for the purposes of the current study, that Prince Llewelyn Reservoir can no longer receive flows from outside of its natural topographic catchment.

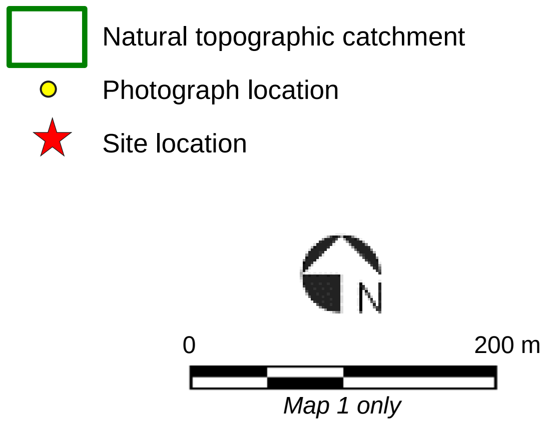
Figure 2-1. Relic leat which used to supply Prince Llewelyn Slate Quarry

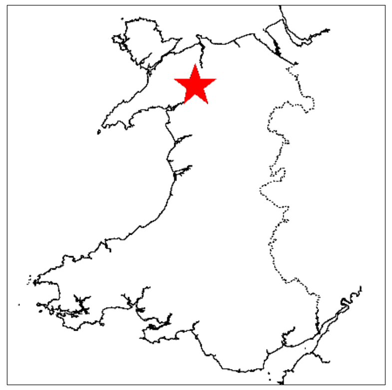
[1] Llywelyn the Great is understood to have built Dolwyddelan Castle; http://www.bbc.co.uk/wales/history/sites/themes/society/royalty_llywelyn_ab_iorwerth.shtml; http://www.walesdirectory.co.uk/Heritage_Holidays/Llywelyn_Fawr_Castles.htm

[2] NRW, pers. comm, 17 January 2017

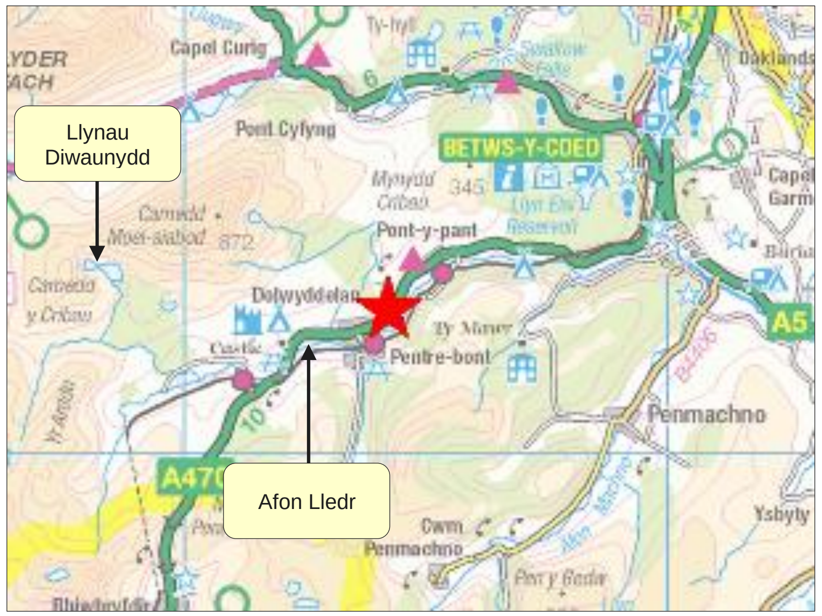


Map 1





Map 2



Map 3

a) Relic leat

b) The forested catchment draining to Prince Llewelyn Reservoir

c) Prince Llewelyn Terrace and the A470 trunk road

Figure 2-2. Study area

2.1.1 Impounding Structure

Embankment

Prince Llewelyn Reservoir is impounded at its eastern end by a 39 m long composite dam (Figure 2-3). The dam consists of two parallel stone walls infilled with clay or compacted earth, and ties into high ground at both ends. The dam crest, which is covered with grass, is around 3 m wide and elevated 4 m above the bed of the outflow channels. On the upstream side, there is a drop of approximately 1 m from the crest down to a narrow shelf, which is also contained by the aforementioned stone walls (Figure 2-3a). The dam core is believed to lie below this shelf, rather than the grassed crest (NRW, pers. comm, 2017). The dam is in poor condition, with numerous leaks evident on the downstream face (Figure 2-3c & e). Within the last couple of years, NRW has cleared the dam of trees and shrubs, the roots of which had displaced some blockwork. There are two spillways and the remains of, what appears to be, a reservoir drawdown facility. These features are described further below.

Spillways and Outlet Works

The primary spillway is positioned roughly in the centre of the dam. It is approximately 2 m wide and 400 mm deep (Figure 2-3b). A dry outflow channel is visible immediately below this spillway (channel 1, Figure 2-3f). A secondary spillway, approximately 1 m wide and 150 mm deep, is located towards the southern end of the dam.

Between these two spillways, at the downstream toe of the dam, there is a rectangular 750 × 660 mm opening, which may be part of a former drawdown facility (Figure 2-3c). A CCTV survey of this opening, conducted in November 2016, has revealed that the rectangular culvert continues straight for 4 m before there is a transition to a circular pipe (NRW, 2016a). This pipe also continues straight, for a further 10 m to a vertical plate. The plate is fitted tightly to the end of the pipe, with no sign of leakage (the water visible in the associated outflow channel (channel 2) in Figures 2-3c & f is entirely from dam leakage). On the shelf directly above the pipe is a rod of metal, which may be a spindle and part of the conjectured drawdown facility (Figure 2-3d).

Below the southern end of the dam, a relic leat, which used to take water from the reservoir to Prince Llewelyn Slate Quarry, is clearly visible (Figure 2-3f). Its position ties in with an aqueduct marked on an OS map from 1899. The leat is now disused but forms a pathway to the dam.

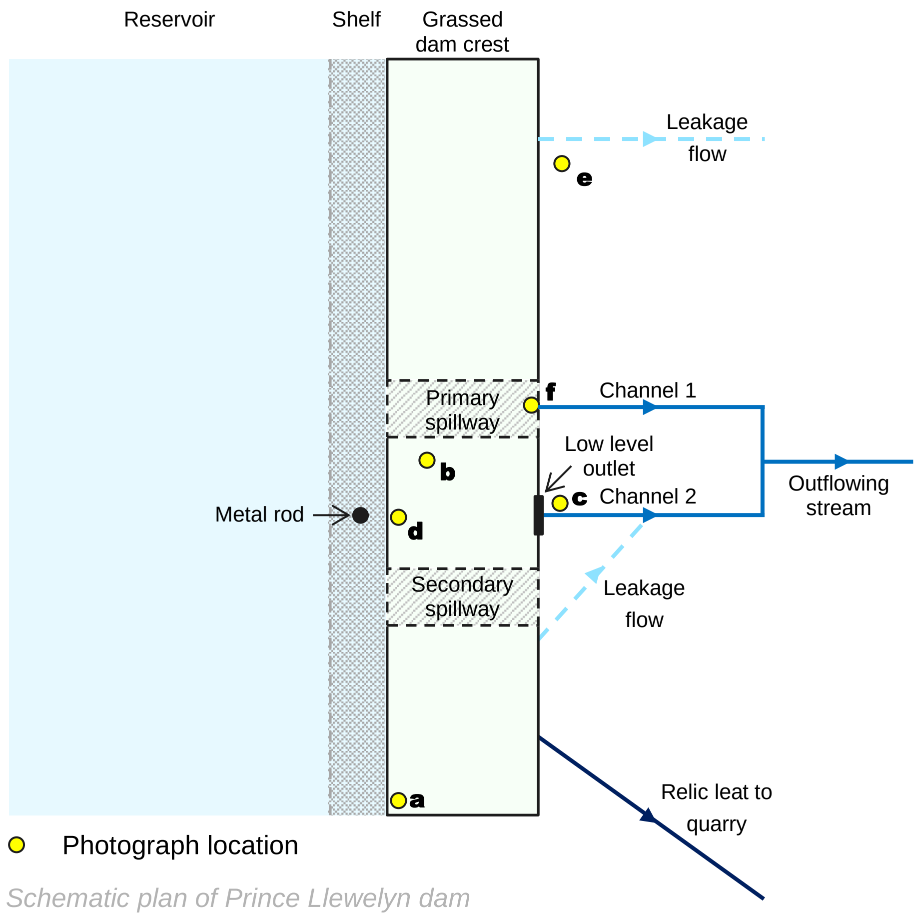
2.1.2 Recommended Standards

The recommended standards for a Category A/B fill embankment dam are given in Table 2-1.

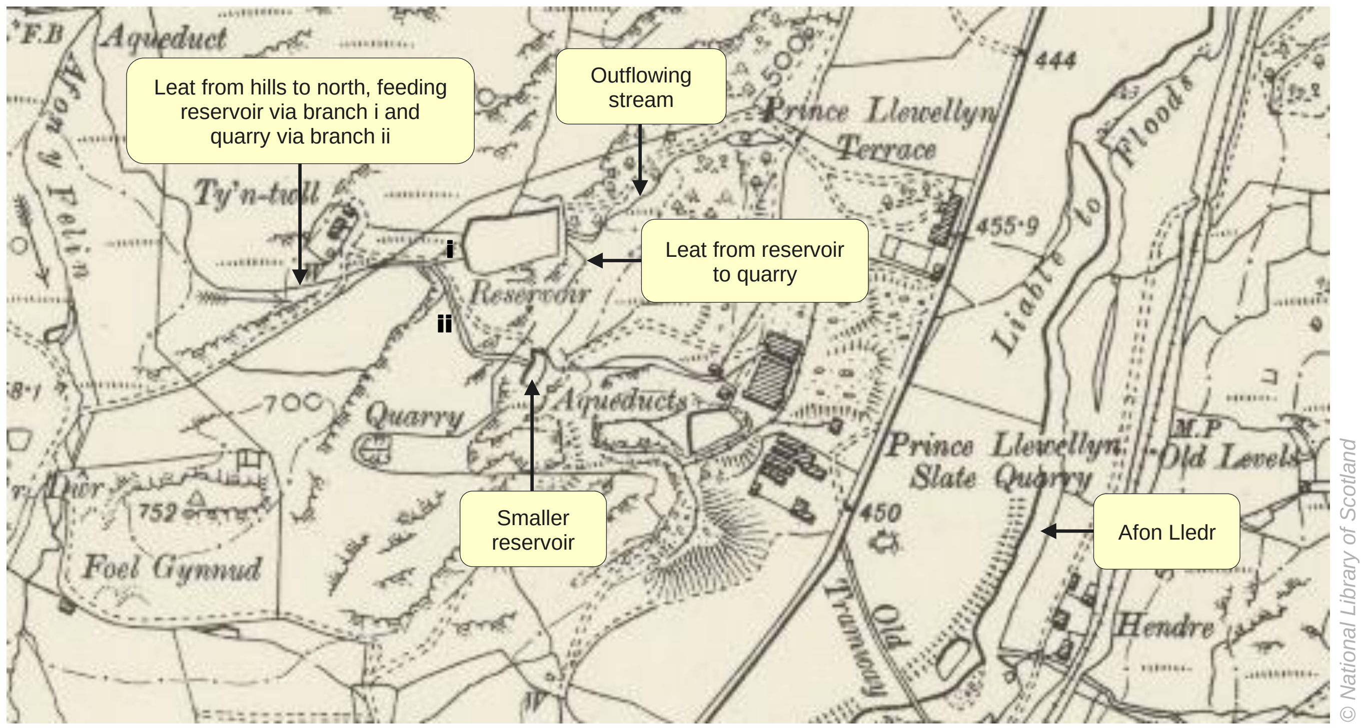
Table 2-1. Recommended standards for a Category A/B fill embankment dam (ICE, 2015)

Flood condition	Recommended standards
Design flood	• Design flood surcharge combined with wave action must not lead to any overtopping (a mean wave overtopping discharge rate of 0.001 l/s/m may be taken as zero); • Minimum flood freeboard of 600 mm.
Safety check flood	• Allowable wave overtopping discharge of 0.1 l/s/m, based on a dam crest and downstream face of bare clay fill, grass covered erodible fill or poor grass cover; • Safety check flood surcharge level should not exceed the top of the dam. If the flood peak is particularly prolonged, the flood surcharge level may have to be lower still to avoid harmful leakage through the crest materials above the dam core; • Safety check flood surcharge combined with wave action must not lead to a wave overtopping discharge in excess of 0.1 l/s/m.

Prince Llewelyn Reservoir

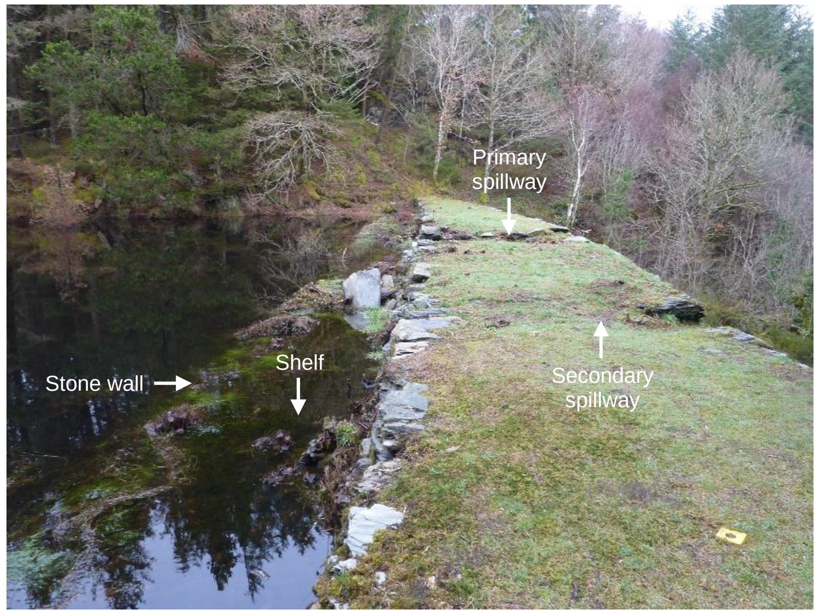


Schematic plan of Prince Llewelyn dam



1899 map of the study area

Figure 2-3. Impoundment features of Prince Llewelyn Reservoir



a) Prince Llewelyn dam


b) Primary spillway

c) Low level outlet, associated with possible former drawdown facility


d) Metal rod (circled), associated with possible former drawdown facility


e) Leakage through the northern end of the dam

f) Outflowing streams

2.2 Catchment Hydrology

The natural catchment of Prince Llewelyn Reservoir is very small, covering an area of approximately 0.046 km^2. It receives an average annual rainfall of 2,010 mm (CEH, 2018). The altitude of the catchment ranges from 194 to 238 m AOD, while the mean drainage path slope (DPSBAR) is 179 m/km (based on LiDAR data). Forestry is the primary land use and the catchment is, consequently, classified as essentially rural (URBEXT1990 value of zero). The geology is dominated by mudstone and siltstone of the Cwm Eigiau Formation (BGS, 2017). It is overlain by the Manod soil association, which consists largely of free draining fine loamy soils (Mackney *et al*, 1984). The good surface drainage is reflected in the catchment's relatively low SPRHOST (33.1) and high BFIHOST (0.572) values.

The reservoir is fed by a number of small hillside streams, an example of which can be seen in Figure 2-2b. The two outflows from the reservoir (channels 1 and 2, Figure 2-3f) converge approximately 35 m to the east of the dam. The merged watercourse continues in a north-east direction, passing close to a residential property named Sunnyside and crossing under the A470 trunk road (Figure 2-4). It then discharges into the Afon Lledr, approximately 600 m downstream of the dam. The Afon Lledr is a designated main river, which flows parallel to the A470 before merging with the River Conwy near Betws-y-Coed.

a) View looking south-west along the line of the outflowing stream

b) View looking north-east along the line of the outflowing stream, towards the Afon Lledr

Figure 2-4. The outflowing stream, viewed from the A470 road crossing (the location of these photographs is shown on Figure 2-2)

3 Data Collection

NRW has provided a range of data to support this project, while additional information has been sourced from the internet. The different types of data obtained are summarised in Table 3-1.

Table 3-1. Data sources

Data type	Data item	Source
Historical maps	OS 6-inch Great Britain, revised 1899, published 1901	National Library of Scotland's 'Map Images' website (http://maps.nls.uk/geo/explore/#zoom=17&lat=53.0591&lon=-3.8769&layers=6&b=1)
Present-day maps	OS Street View	OS OpenData
	1:250,000 Scale Colour Raster	
	OS VectorMap District	
Topographic data	OS Terrain 50	
	1 m horizontal resolution LiDAR data	Lle Geo-Portal (http://lle.gov.wales/home)
	A topographic survey of the embankments and surrounding environments, together with a bathymetric survey of the reservoir	Commissioned by NRW and undertaken by NHTB Consultancy Ltd. in April 2016
Publications and previous reports	Diving survey report	Commissioned by NRW and conducted by Salvesen (UK) Ltd. in March 2016
	The Lledr Valley and Dolwyddelan, by Shaun V. Hewitt, published in 2016	Hard copies purchased as part of the previous studies
	The Slate Regions of North and Mid Wales and their Railways, by Alun John Richards, published in 1999	
	Slate Quarrying in Wales, by Alun John Richards, published in 2006	
	Gazeteer (sic) of Slate Quarrying in Wales, by Alun John Richards, published in 2007	
	Information about Prince Llewelyn Slate Quarry and aqueduct	historicwales.gov.uk and Coflein
Field reconnaissance	A walkover survey of the reservoir and surrounding environments, including verification of the catchment boundary	Undertaken by Arcadis, accompanied by Peter Oxbury from NRW, on 17 January 2017
Consultations	During the study, consultations have been undertaken with Peter Oxbury	Reservoir Keeper, NRW
	Correspondence with Shaun Hewitt	Local historian and resident of Prince Llewelyn Terrace

4 Reservoir Flood Assessment

This section summarises the methodology and findings of the flood study calculation for Prince Llewelyn Reservoir, based on existing conditions. The calculation involves three key stages, as illustrated in Figure 4-1; the corresponding report section number is also shown. A summary table for the flood study calculation is provided in Appendix A.

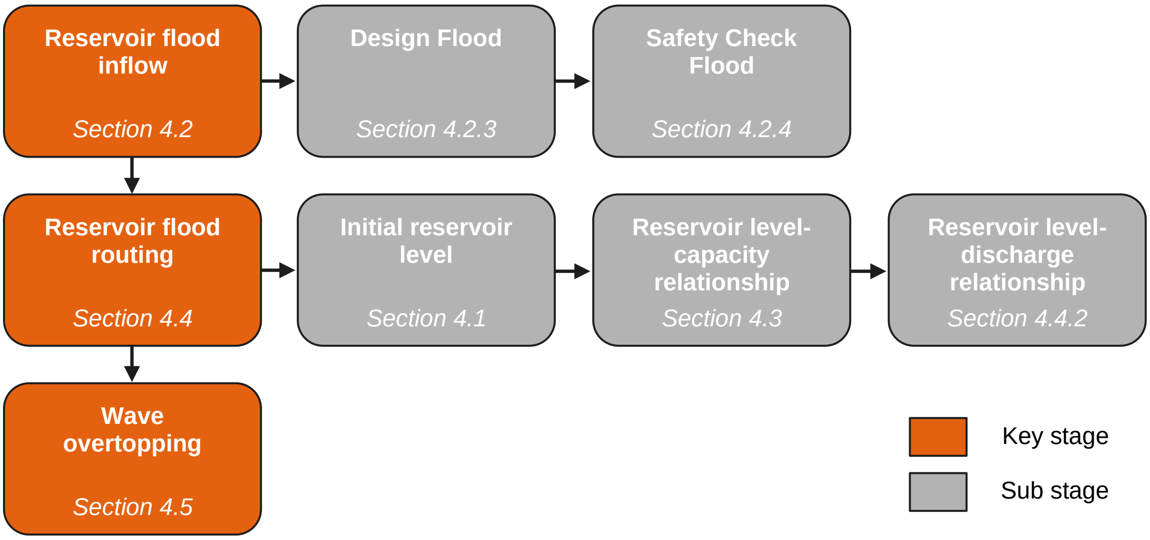


Figure 4-1. Key components of the flood study calculation

4.1 Initial Reservoir Level and Other Key Parameters

The reservoir level at the start of a flood influences the reservoir's ability to contain the flood inflow and is, therefore, an important consideration when assessing reservoir safety. The initial reservoir condition standard for a Category A/B dam is just full, i.e. no spill (ICE, 2015). This equates to 195.974 m AOD for Prince Llewelyn Reservoir, reflecting the minimum crest height of the primary spillway. Other key reservoir parameters are provided in Table 4-1.

Table 4-1. Key reservoir parameters, based on existing conditions

Parameter	Value	Information source
Surface area	2,594 m²	Reservoir level-capacity relationship presented in Section 4.3
Initial reservoir level	195.974 m AOD	
Spillway level (primary spillway)	195.974 m AOD	
Spillway width	2 m	NHTB Consultancy Ltd.'s (2016) topographic survey
Minimum dam crest level (secondary spillway)	196.242 m AOD	
Average dam crest level	196.430 m AOD	

4.2 Reservoir Flood Inflow

4.2.1 Verification of Catchment Boundary

The catchment boundary, presented in Figure 2-2, has been defined manually, based on observations made by Arcadis during the walkover survey, supplemented by OS Terrain 50 and LiDAR data. Catchment descriptor values, which have also been calculated manually, are given in Table 4-2.

Table 4-2. Key catchment descriptors for Prince Llewelyn Reservoir

Descriptor	Value	Derivation method
AREA (km²)	0.046	Site observations and 1 m horizontal resolution LiDAR data
SAAR (mm)	2,010	FEH Web Service, based on the point data for NGR SH 74231 53046*
PROPWET	0.71	
URBEXT 1990	0	Urban Extent 1990 layer on the FEH Web Service
DPLBAR (km)	0.18	Calculated according to equation 7.1 of the FEH vol. 5 and checked against LiDAR
DPSBAR (m/km)	179	Estimated from LiDAR data
BFIHOST	0.572	Institute of Hydrology Report No. 126
SPRHOST (%)	33.05	

* Rainfall Depth-Duration-Frequency (DDF) model parameters have also been taken from this FEH catchment.

4.2.2 Method Overview

The flood protection standards for a Category A dam, and the rainfall depth-duration-frequency (DDF) and rainfall-runoff models to be used, are specified in the fourth edition of Floods and Reservoir Safety. Since this guide was published prior to the release of the FEH13 DDF rainfall model and the ReFH2 software package, clarification of the latest situation is provided in Table 4-3.

Table 4-3. Flood protection standards, and associated hydrological models, for a Category A dam

Criteria	Design flood condition	Safety check flood condition
Reservoir flood inflow	10,000 year	Probable Maximum Flood (PMF)
Rainfall DDF model	**ICE (2015) recommendation:** FSR until the new DDF is issued.	**ICE (2015) recommendation:** FSR (PMP).
	Latest situation: The FEH13 DDF rainfall model was released in November 2015 and should be used to derive the 10,000 year reservoir flood inflow.	**Latest situation:** There have been no recent changes to the method for estimating PMF, and the FSR (PMP) remains current best-practice.
Rainfall-runoff model	**ICE (2015) recommendation:** FSR/FEH until ReFH is extended.	**ICE (2015) recommendation:** FSR/FEH until ReFH is extended.
	Latest situation: ReFH has not yet been extended beyond the 1,000 year event. The FSR/FEH rainfall-runoff method should, therefore, continue to be used to derive both the 10,000 year reservoir flood inflow and the PMF.	

Arcadis has consulted Wallingford HydroSolutions (WHS) on the compatibility of the FSR/FEH method and the FEH13 DDF rainfall model, since the technical guide to the ReFH 2.2 software states that the two models should not be used together (WHS, 2016). WHS has advised that, whilst the FSR/FEH method was originally conditioned on the use of FSR rainfall and using it with FEH13 rainfall has unknown implications, the specifics of the rainfall loss model become less important at very long return periods. Consequently, WHS has confirmed that the two models can be used together for the purposes of reservoir safety.

As well as flood inflows, Q50 (the 50 percentile flow) has been estimated using LowFlows 2. This estimate, which is presented in Appendix D, will be used by NRW in drawdown calculations.

4.2.3 Design Flood

The FEH Boundary unit in Flood Modeller Pro (version 4.3) has been used to generate the reservoir flood inflow hydrograph for the design flood condition[3]. The rainfall-runoff model is based on the catchment descriptor values given in Table 4-2. A design rainfall depth of 159 mm has been obtained from the FEH13 DDF model (given the very small size of the study catchment, no areal reduction factor has been applied). A winter design storm of 1.75 hours' duration has been adopted, based on a reservoir lag time (RLAG) of 0.2 hours (derivation of RLAG is described in Section 4.4.1). The resulting hydrograph is presented in Figure 4-2. The peak 10,000 year reservoir flood inflow is 0.92 $m^3\ s^{-1}$.

4.2.4 Safety Check Flood

The same rainfall-runoff model has been used to derive the safety check flood hydrograph. The input parameter values applied to the estimation of the PMF are given in Table 4-4. The time to peak, Tp(0), has been reduced by one-third, to represent the more rapid and intense response that is believed to occur in exceptional conditions. It is worth noting that the estimated maximum 2-hour rainfall given by the FSR method is less than the design rainfall generated by the FEH13 DDF model for a 10,000 year storm of 1.75 hours' duration.

In line with recommended practice, summer and winter probable maximum precipitation (PMP) events have been considered separately. An adjustment for frozen ground in winter has been applied (SPR set to 53%). The winter PMP, based on a storm duration of 1.25 hours and an RLAG of 0.15 hours, has been found to give the highest value of PMF and has been adopted herein. The resulting hydrograph is again presented in Figure 4-2. The PMF peak flow is 1.21 $m^3\ s^{-1}$. It is estimated to have a return period of 10^6 years, according to the methodology-based, and 10^7 years according to the geometry-based estimates, of return period (FEH Vol. 4, Section 4.5.1).

A full list of the model input parameters for both the design flood and safety check flood is given in Appendix B.

Table 4-4. Input parameter values for PMF estimation

Input parameter		Value
All-year point estimated maximum precipitations (EMPs) of 2-hour and 24-hour duration	Em-2h	137 mm
	Em-24h	350 mm
Snowmelt rate*		42 mm/day
100-year snow depth water equivalent (S100)*		200 mm

* Winter PMP only

[3] Since the reservoir occupies only 6 per cent of the catchment area, it has been treated as part of the catchment, i.e. rain falling on the reservoir has been passed through the rainfall-runoff model.

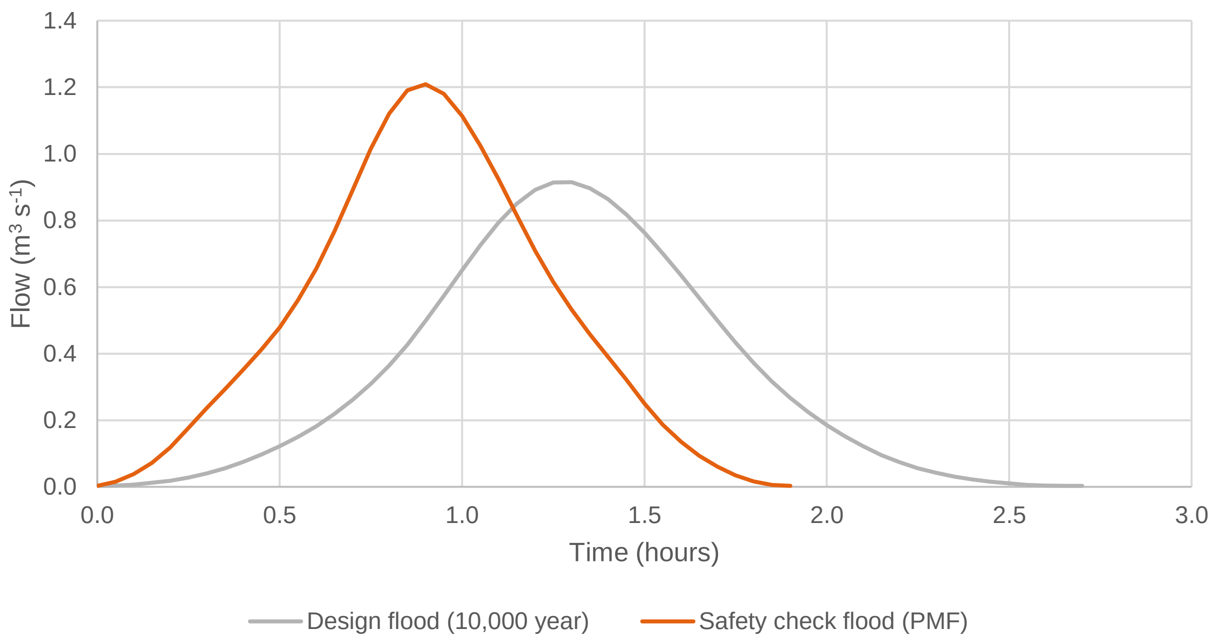


Figure 4-2. Reservoir flood inflow hydrographs

4.2.5 Higher Frequency Events

Inflows for a number of higher frequency events have been estimated to inform the proposed options for lowering the spillway (i.e. to quantify the impact of the options on downstream peak flood flows). Hydrographs have been derived using the ReFH 2.2 software for the 10 year, 30 year, 100 year and 1,000 year return periods. To account for the effects of climate change, an increase in peak flow of 30% and 75% has been applied to the 100 year return period in accordance with ‘Flood Consequence Assessments: Climate change allowances’ (NRW, 2016). The catchment descriptors from the FEH Web Service have been used to define inflow hydrographs with an initial duration of 3.15 hours as summarised in Table 4-5 below. Subsequent derivation of RLAG and update of the inflow hydrograph is described in Section 4.4.1.

Table 4-5. Summary of inflows for higher frequency events

Return Period (years)	Season	Peak Flow ($m^3 s^{-1}$)
10	Winter	0.07
30	Winter	0.10
100	Winter	0.14
100 plus 30% climate change	Winter	0.18
100 plus 75% climate change	Winter	0.24
1000	Winter	0.25

4.3 Reservoir Level-Capacity Relationship

4.3.1 Without Silt

A surface raster model of the reservoir's hard bed has been generated from the bathymetry data; the presence of silt deposits has been ignored[4]. The model has been used to compute a reservoir level-capacity relationship, which is illustrated in Figure 4-3. Based on this relationship, the capacity of the reservoir, at the existing minimum crest level of the primary spillway, is 5,477 m^3. This is well below the statutory 10,000 m^3 threshold of the Reservoirs Act 1975 and confirms that Prince Llewelyn Reservoir does not fall within the scope of the Act. Raising the primary spillway to the same level as the secondary spillway (196.242 m AOD) would increase the capacity of the impoundment by approximately 13 per cent (to 6,188 m^3). Other key statistics, obtained from the reservoir level-capacity relationship, are given in Table 4-6.

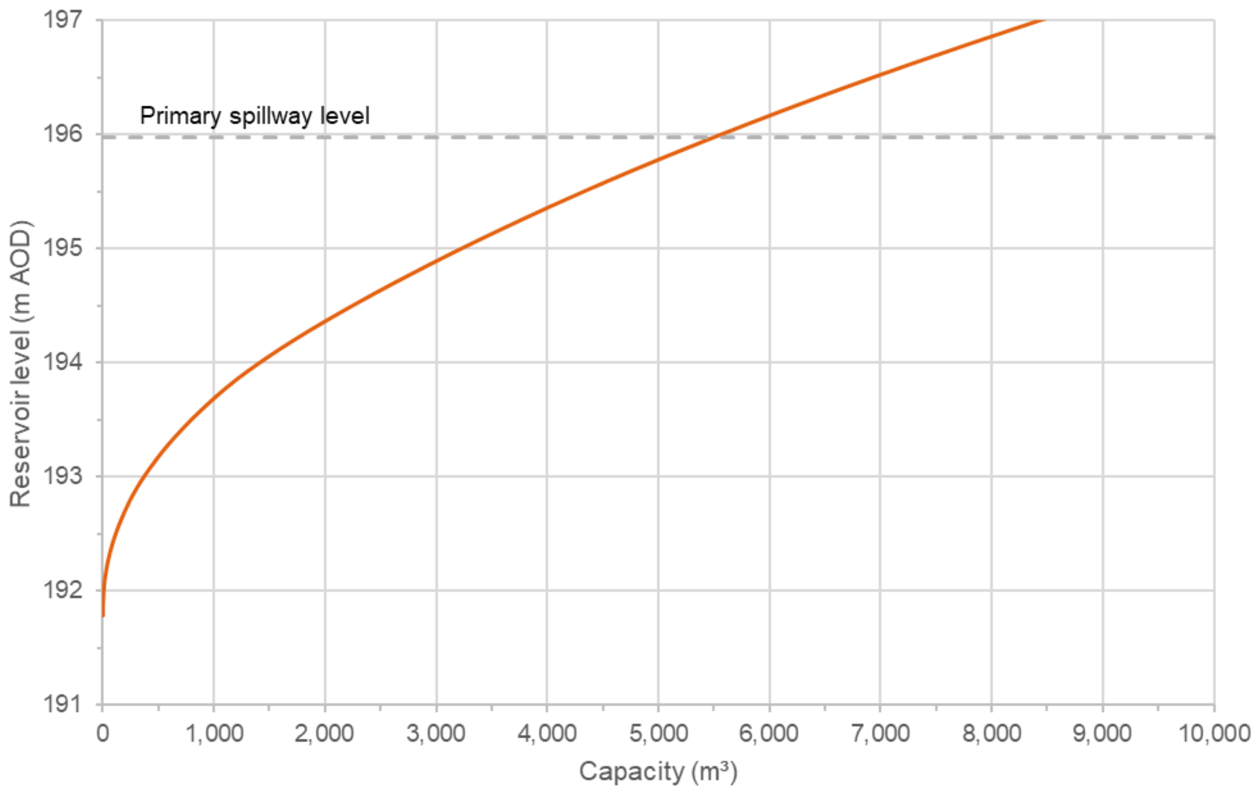


Figure 4-3. Reservoir level-capacity relationship (without silt[5])

Table 4-6. Key reservoir level-capacity statistics

Reservoir level	Surface area (m^2)	Capacity (m^3)
Primary spillway level (195.974 m AOD)	2,594	5,477
Minimum crest level / secondary spillway (196.242 m AOD)	2,730	6,188
Average crest level (196.430 m AOD)	2,837	6,709

[4] Under the Reservoirs Act 1975, silt must be included as though it is water and the original capacity is unchanged, unless a QCE certifies it differently (NRW, 2016b)

[5] 'without silt' means without silt included as silt - i.e. silt is included as if it were water, to meet the capacity definition under the Act.

It is worth mentioning that NHTB Consultancy Ltd. has estimated a slightly smaller original reservoir capacity (5,000 m^3) than Arcadis. This is likely to be due to differences between the interpolation methods used by NHTB and Arcadis to create the surface raster model. It is also important to bear in mind that there are uncertainties associated with both sets of estimates, due to the use of point data to define a continuous surface (maximum spacing between surveyed points is about 10 m).

4.3.2 With Silt

The total volume of silt within the reservoir is estimated to be 622 m^3. This means that the actual storage capacity of the reservoir is approximately 4,855 m^3. For comparison, NHTB has estimated the silt volume to be 500 m^3.

4.4 Reservoir Routing

4.4.1 Method

A 1D hydraulic model has been built, using HEC-RAS version 5.0.7, to route the 10,000 year and PMF inflow hydrographs through Prince Llewelyn reservoir. The reservoir has been represented in the model using a storage area, based on the reservoir level-capacity relationship presented in Section 4.3.1, while the dam has been simulated as a broad crested weir. The longitudinal elevation profile of the embankment has been defined using NHTB's topographic survey. A weir coefficient of 1.0 has been adopted for the embankment and spillway, reflecting the generally rough surface. The sensitivity of model output to the value of this weir coefficient, as well as the bed elevation of the outflowing stream, has been tested.

As a validity check of the HEC-RAS output, a model of Prince Llewelyn Reservoir has also been built using Flood Modeller 4.3. Although the set-up of the two models is different[6], they give very similar results. The difference in predicted stillwater flood levels between the two models is 6 mm during the design flood and 5 mm during the safety check flood, lending confidence to the HEC-RAS results presented herein.

Reservoir Lag Time

The HEC-RAS model has been used to establish the lag effect of Prince Llewelyn Reservoir. In particular, RLAG has been calculated as follows:

- The reservoir flood inflow hydrographs, based on an initial RLAG value of 0, have been routed through the model. The time between the peak of the inflow and the peak of the outflow hydrographs has then been used to redefine the value of RLAG;
- The revised RLAG value has, in turn, been used to adjust the storm duration, in accordance with equation 8.1 of the FEH Vol. 4;
- The inflow hydrographs have then been re-generated, based on the adjusted storm durations, and re-routed through the model. The RLAG value has again been established;
- This iterative process has been repeated until the RLAG value does not change between successive model runs and, hence, the design storm duration has stabilised.

The final RLAG value is 0.2 hours for the 10,000 year event and 0.15 hours for the PMF.

[6] For example:
- In HEC-RAS, the outflowing stream is represented by cross-sections along a river reach, while in Flood Modeller, the outflow discharges into a reservoir;
- In Flood Modeller, the dam has been represented using a spill unit, rather than as a broad crested weir.

4.4.2 Results

Reservoir level-discharge relationship

A reservoir level-discharge relationship for Prince Llewelyn dam has been generated using the HEC-RAS model and is presented in Figure 4-4.

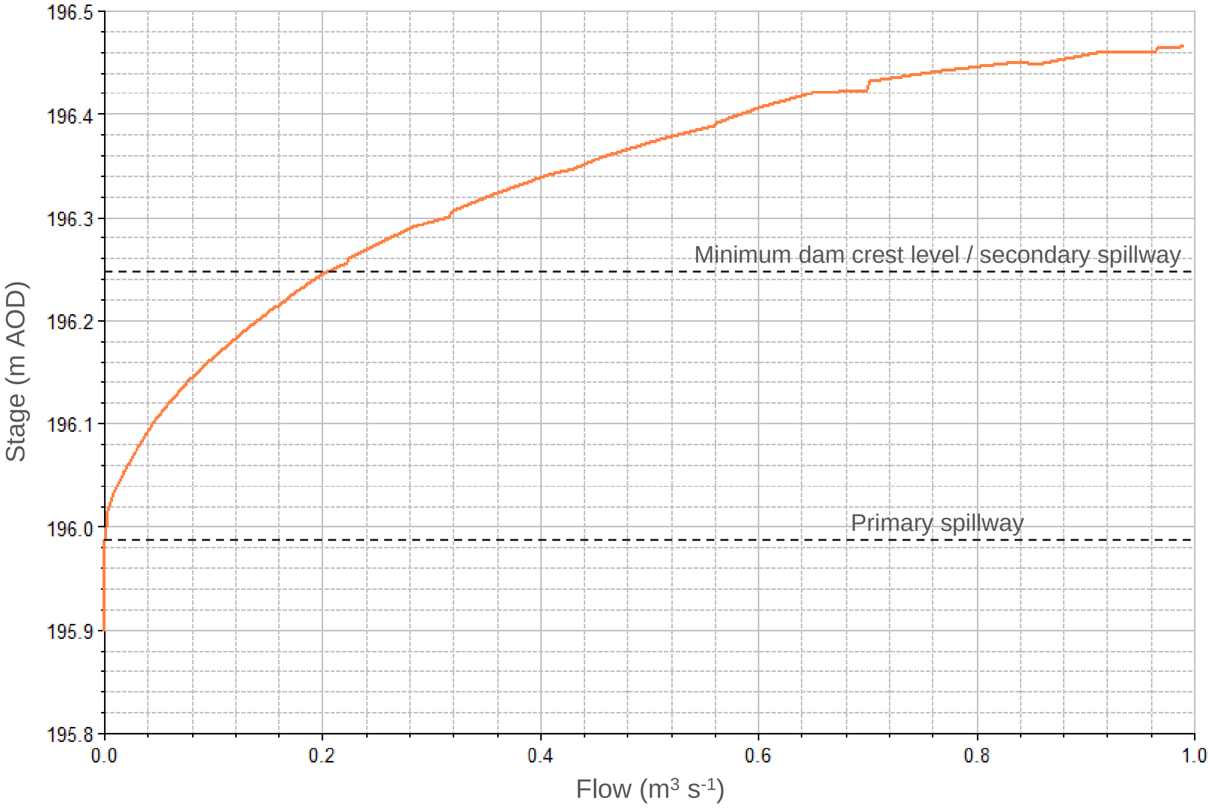


Figure 4-4. Reservoir level-discharge relationship

Reservoir Attenuation

The inflow and routed outflows for the Design Flood (10,000 year) and Safety Check Flood (PMF) are shown in Figure 4-5 and Figure 4-6, respectively. Table 4-7 states the inflow and outflow peaks along with the difference for each of these events.

Table 4-7. Peak Inflows and routed outflows for the Design Flood and Safety Check Flood.

Flood Condition	Peak Inflow (m^3 s^{-1})	Peak Routed Outflow	Difference (m^3 s^{-1})
Design Flood (10,000 year)	0.92	0.74	0.18
Safety Check Flood (PMF)	1.21	0.99	0.22

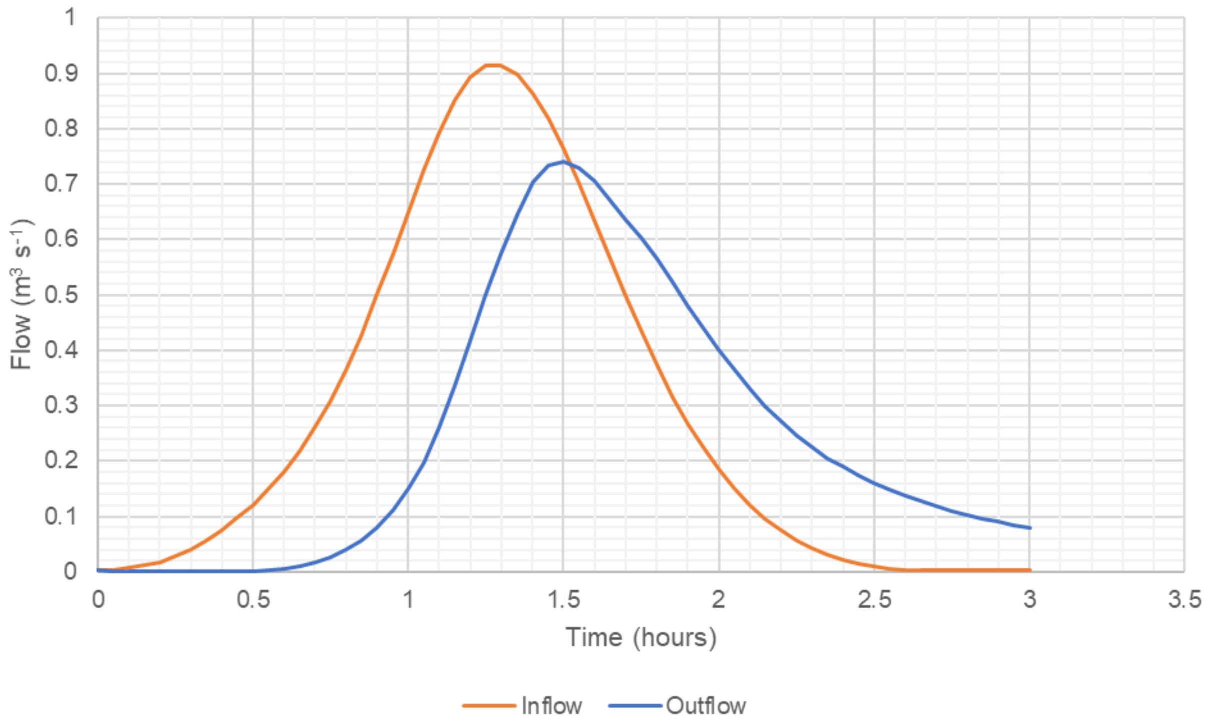


Figure 4-5. Design Flood (10,000 year) inflow and routed outflow hydrographs

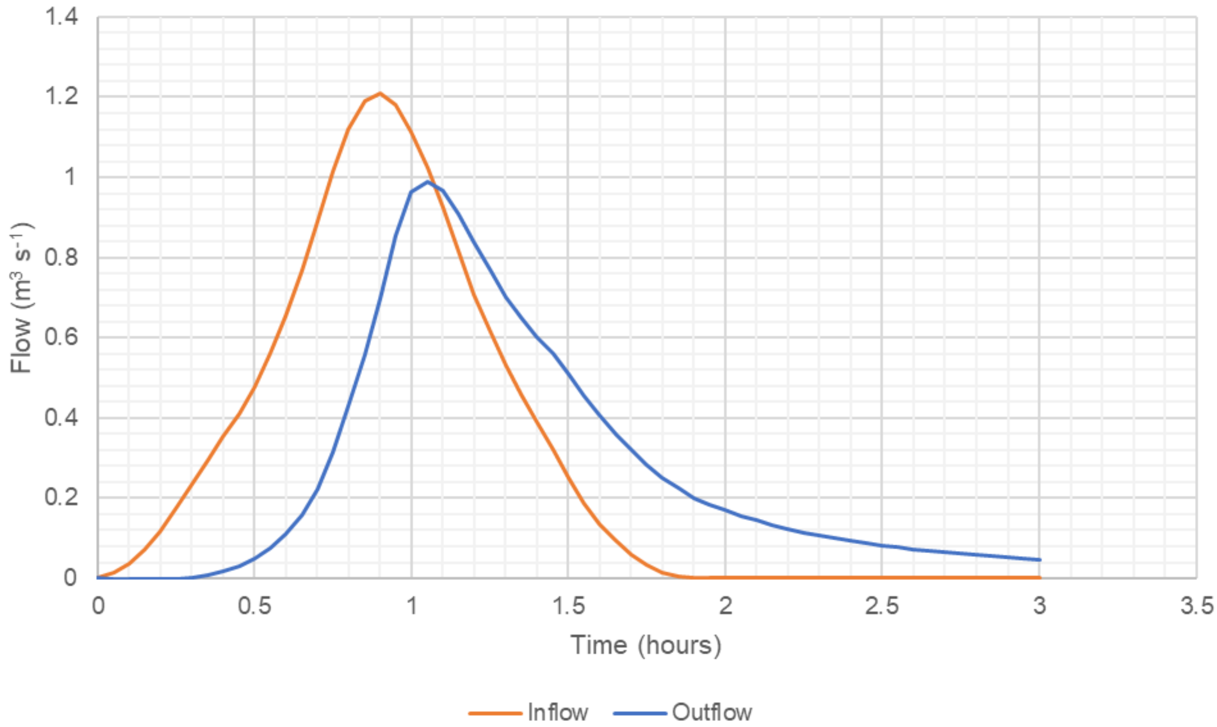


Figure 4-6. Safety Check Flood (PMF) inflow and routed outflow hydrographs

Stillwater flood level

Predicted stillwater flood levels are presented in Figures 4-7 and 4-8. They show that overflowing of the dam occurs, for around 1.25 hours, during both the 10,000 year event and the PMF. At the low spot on the dam crest (corresponding to the location of the secondary spillway), the depth of overtopping during these two events is predicted to reach a maximum of 188 mm and 228 mm, respectively. In other words, there is a lack of spillway capacity and zero freeboard under both the design flood and the safety check flood conditions. Prince Llewelyn Reservoir, therefore, does not meet the recommended standards for a Category A/B dam (given in Section 2.1.2).

Condition	Peak stillwater level (m AOD)
Design flood (10,000 year)	196.43
Safety check flood (PMF)	196.47

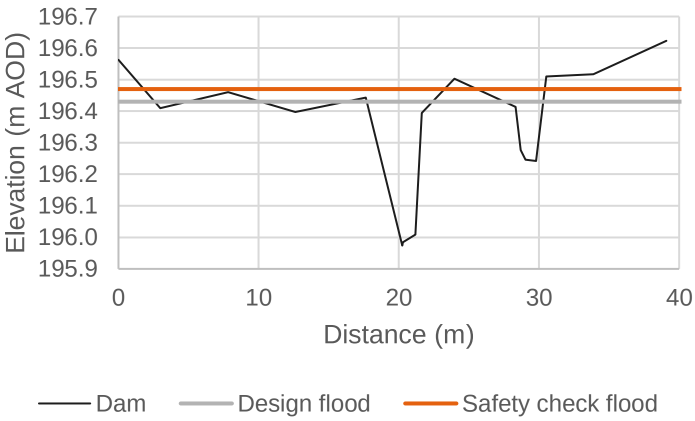


Figure 4-7. Stillwater flood level predictions

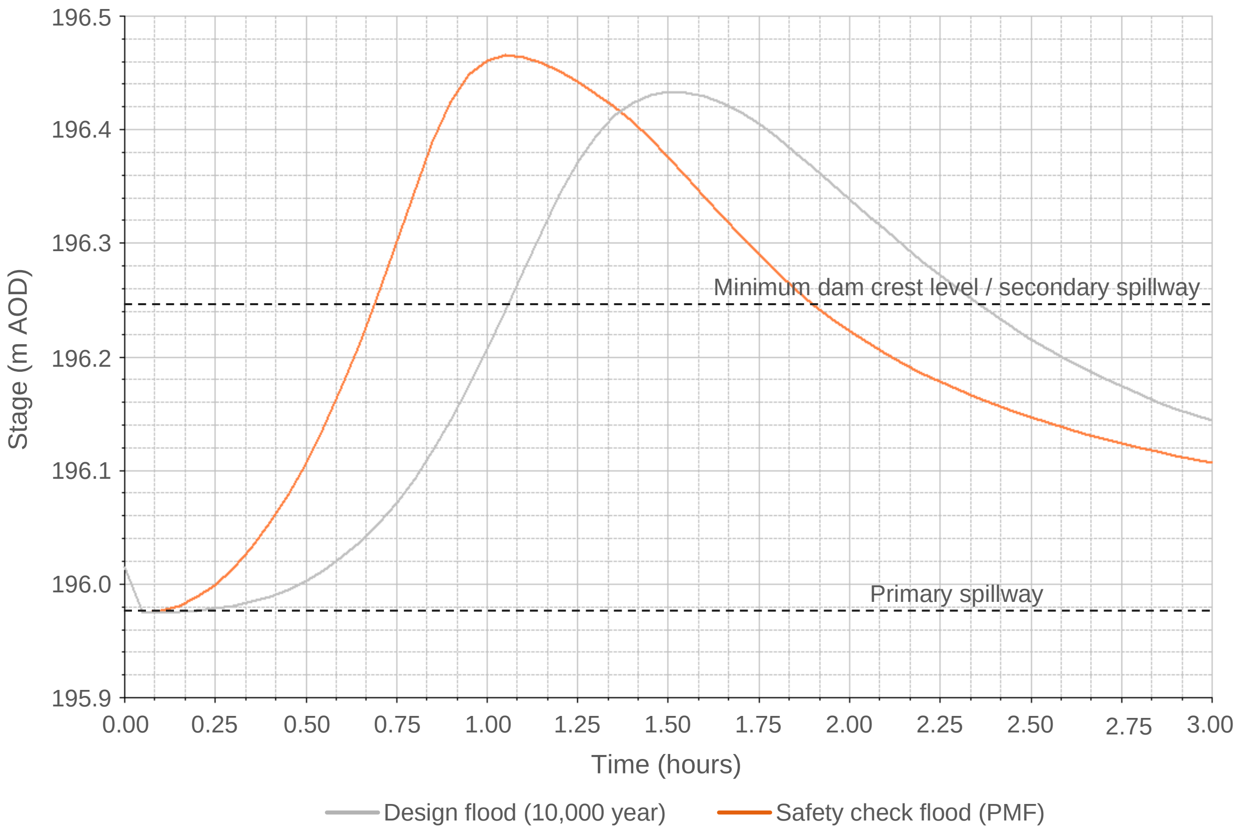


Figure 4-8. Modelled stage hydrographs

Sensitivity analysis

As mentioned in Section 4.4.1, the sensitivity of model output to selected model input parameters has been tested. One parameter value has been modified at a time as follows:

- *Weir coefficient.* The weir coefficient, applied to Prince Llewelyn dam, has been adjusted by ±20 per cent. A higher value of 1.7, which is typical for round nosed weirs, has also been tested. The results, which are presented in Table 4-8, show that stillwater flood level predictions are not very sensitive to this input parameter: a 20 per cent change in the coefficient value results in a maximum 7 per cent change in the depth of water flowing over the spillway.
- *Downstream bed level.* The modelling results also show that adjusting the bed level of the outflowing stream by ±0.5 m has no impact on stillwater flood level predictions for Prince Llewelyn Reservoir.

Table 4-8. Sensitivity test results

Parameter	Peak stillwater level (m AOD)		Overflow depth (m)		Difference* (m)	
	10,000 year	PMF	10,000 year	PMF	10,000 year	PMF
Weir coefficient -20%	196.46	196.49	0.486	0.516	0.03 *(7%)*	0.02 *(4%)*
Weir coefficient +20%	196.41	196.45	0.436	0.476	-0.02 *(-4%)*	-0.02 *(-4%)*
Weir coefficient 1.7	196.36	196.40	0.386	0.426	-0.07 *(-15%)*	-0.07 *(-14%)*
Downstream bed elevation –0.5 m	196.43	196.47	0.456	0.496	0	0
Downstream bed elevation +0.5 m	196.43	196.47	0.456	0.496	0	0

* Relative to the overflow depth under existing conditions

4.5 Wave Overtopping

Wave overtopping discharges have been calculated following the procedures set out in the fourth edition of Floods and Reservoir Safety (Chapter 5). First, significant wave height and wave period have been predicted for a range of fetch lengths, measured from the lowest point along the embankment. The predictions are based on mean annual maximum hourly wind speed, which varies with fetch direction. A combination of a fetch length of 73 m and direction of 254°N creates the largest waves.

Second, mean and maximum overtopping discharges have been derived using the overtopping prediction methods of the EurOtop Manual (EurOtop, 2018). A full list of the input parameters, together with the results, is presented in Appendix C.

A mean wave overtopping discharge of 7.6 l/s/m is predicted, based on existing conditions (i.e. zero freeboard / overflow conditions). This far exceeds the recommended standards for an embankment dam (mean wave overtopping discharge of less than 0.001 l/s/m during the design flood condition and 0.1 l/s/m during the safety check flood condition).

In order for these standards to be met, a minimum freeboard of 240 mm and 140 mm, above the design flood and safety check flood stillwater level, would be required, respectively. This is based on the assumption that the upstream face of the dam will be grassed (i.e. roughness of the dam (yf) of 0.38). However, the 240 mm freeboard required for overtopping is less than the minimum freeboard of 600 mm required to meet the design flood standard (Table 2-1).

4.6 Flood Attenuation

As identified in Section 4.4.2, the reservoir provides an attenuating effect on flood flows that would ordinarily result in a reduction in flood risk to downstream. However, in reality owing to the size of the upstream catchment (0.046 km^2) the inflows and outflows from the reservoir are very small.

In order to establish the attenuation offered by the existing reservoir and spillway arrangement during higher frequency events the model has been run with the inflow hydrographs and peak flows described in Section 4.2.5. A summary of the results is provided in Table 4-9.

Table 4-9. Summary of flood attenuation provided during lower return periods.

Return Period (years)	Peak Inflow ($m^3 s^{-1}$)*	Peak Outflow ($m^3 s^{-1}$)	Volume of Temporary Storage (m^3)	Peak Stage (m AOD)	Attenuation
10	0.083	0.068	409	196.13	18.1%
30	0.112	0.095	489	196.16	15.2%
100	0.154	0.134	570	196.19	13.0%
100 plus 30% climate change	0.199	0.177	678	196.23	11.1%
100 plus 75% climate change	0.265	0.242	787	196.27	8.7%
1000	0.264	0.240	787	196.27	9.1%

*Note the reported peak inflows differ to those reported in Section 4.2.5 given the storm duration refinement following RLAG adjustment.

5 Potential Options

As confirmed in Section 4, Prince Llewelyn Reservoir holds 5,477 m^3 of water, based on the existing configuration of the primary spillway. This is well below the statutory 10,000 m^3 threshold of the Reservoirs Act 1975. As such, the reservoir could, in theory, be classed as discontinued. However, the modelling results, presented in Section 4.4.2, show that the dam lacks sufficient freeboard, as well as spillway capacity, to provide safe passage of both the design flood and the safety check flood.

NRW is therefore exploring options to further reduce the capacity of Prince Llewelyn Reservoir; however, doing so could affect the flood attenuation offered by the reservoir. Therefore, options have been assessed with the objective of reducing the retained volume in the reservoir whilst providing a similar level of attenuation, so that the outflows are not more than approximately 5% greater than the baseline outflows for the higher frequency events.

This section discusses two options to reduce the retained volume. These options were initially developed using the findings of the NRW option selection workshop (January 2019) and in consultation with NRW and Arcadis' Design Engineers. Preliminary calculations of the spillway dimensions and elevations were undertaken, which were then tested in the model and iteratively adjusted through model re-runs in order to optimise the spillway configurations to those discussed below.

5.1 Option 1

The first option reduces the existing retained volume by more than half. It involves lowering the spillway to an elevation of 194.7 m AOD (by approx. 1.27 m), which would reduce the retained volume to 2,612 m^3 without silt. The lowered spillway has been modelled as a 0.5 m wide and 0.3 m deep slot set within a 2 m wide 'upper' spillway, with the entire embankment set at its existing average elevation (196.43 m AOD), as shown in Figure 5-1. A weir coefficient of 1.7 (equivalent to a broad-crested weir) has been used. This coefficient is higher than the one adopted to represent the existing spillway and reflects the improvement that can be achieved through the provision of a more efficient and well maintained (stone / concrete) spillway weir.

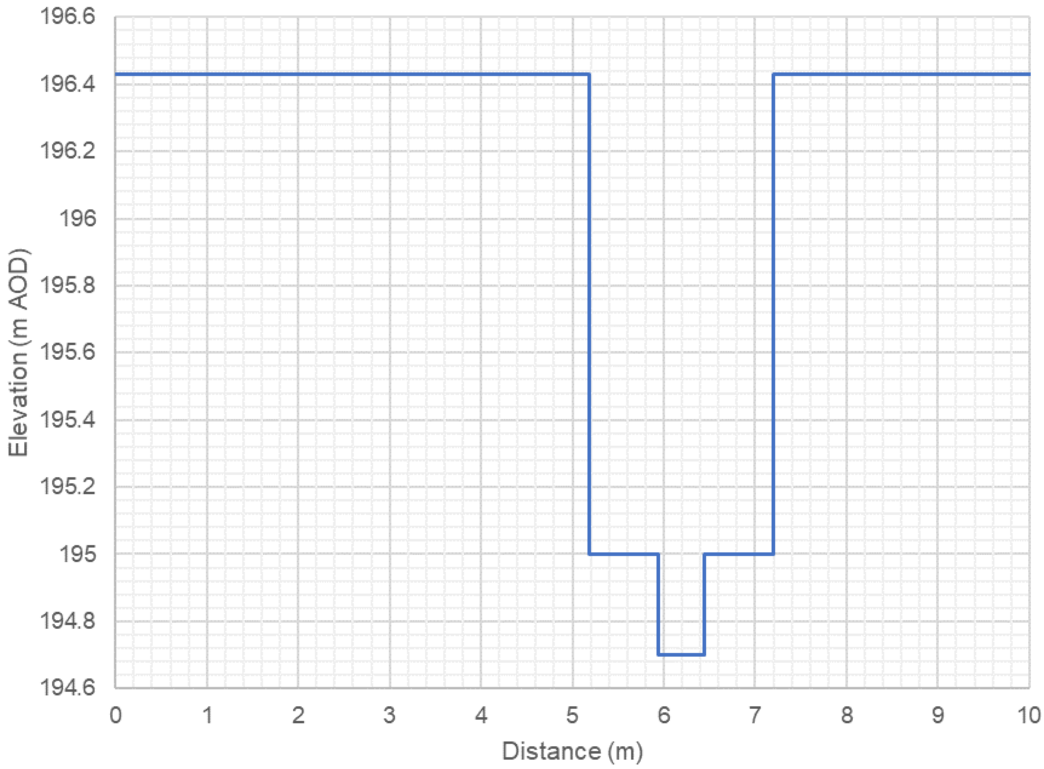


Figure 5-1. Schematic of Option 1.

The results for Option 1 are summarised in Table 5-1 and indicate that for all the modelled return periods except the 1000 year and PMF event the peak outflow is less than the baseline. During the 1000 year the peak outflow increases by 1.3%. As this is less than 5%, it is considered that Option 1 would represent an insignificant change in flood risk. Furthermore, with the embankment elevation set at a minimum of 196.43 m AOD, the embankment would be 1.19m above the predicted design flood level (i.e. freeboard greater than 0.6 m) and 1 m above the safety check flood (i.e. freeboard greater than the 0.14 m required for wave overtopping), thus indicating that the required freeboard would be maintained in these events. Given the freeboard available, there is scope to reduce the embankment elevation to around 195.84 m AOD (a 0.59 m reduction) whilst still maintaining the design and safety check flood requirements.

Table 5-1. Summary of results for Option 1.

Return Period (years)	Peak Inflow ($m^3 s^{-1}$)	Peak Outflow ($m^3 s^{-1}$)	Volume of Temporary Storage (m^3)	Peak Stage (m AOD)	Attenuation	Change in Peak Outflow from Baseline (%)
10	0.083	0.068	375	194.89	18.1%	0.0
30	0.111	0.093	457	194.93	16.2%	-2.1
100	0.154	0.130	580	194.99	15.6%	-3.0
100 plus 30% climate change	0.197	0.174	663	195.03	11.7%	-1.7
100 plus 75% climate change	0.261	0.240	747	195.07	8.1%	-0.8
1000	0.264	0.243	747	195.07	8.0%	1.3
10,000	0.717	0.648	1,113	195.24	9.6%	-12.6
PMF	1.474	1.257	1,538	195.43	14.7%	27.1

A 2 m wide slot is considered to be particularly prone to blockage given the forested nature of the catchment. A 67% blockage of the spillway has been modelled by reducing the width of the slot to 0.165 m and the 'upper' spillway to 0.66 m, shown in Figure 5-2.

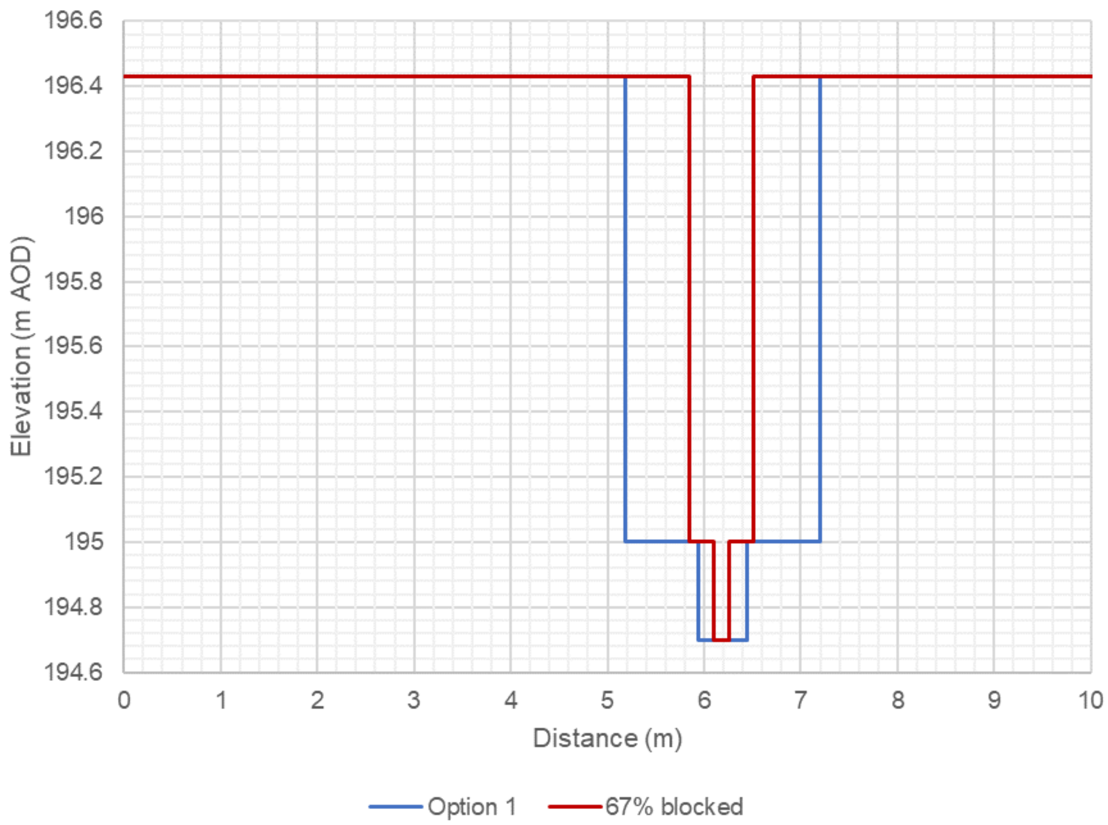


Figure 5-2. Schematic of Option 1 and Blockage.

During the 1000 year event the peak outflow is reduced to 0.216 $m^3 s^{-1}$, a 10% reduction when compared to the baseline event. The peak water level in the reservoir during the blockage scenario is 195.25 m AOD (0.18 m increase), which is 1.02 m lower than in the baseline scenario.

Results of the blockage scenario for this option indicate that there would be no implications in terms of dam safety and no increase in peak outflow in the 1,000 year event. Depending on the proportion of the blockage there is potential for outflows to increase in the higher frequency events (e.g. 10 year event); however, as the sizes of the upstream catchment and corresponding flows are relatively small, it is judged that the consequence of a blockage on downstream flood risk is negligible.

5.2 Option 2

The second option looks at further reducing the retained volume. It involves lowering the spillway to an elevation of 193.5 m AOD, which would reduce the retained volume to 790 m^3 without silt. The lowered spillway has been modelled as a 0.25 m wide and 0.45 m deep slot set within a 2 m wide 'upper' spillway, with the embankment set at a minimum embankment elevation of 196.43 m AOD, as shown in Figure 5-3. A weir coefficient of 1.7 (equivalent to a broad-crested weir) has been used. This reflects the improvement that can be achieved through the provision of a more efficient and well maintained (stone / concrete) spillway weir.

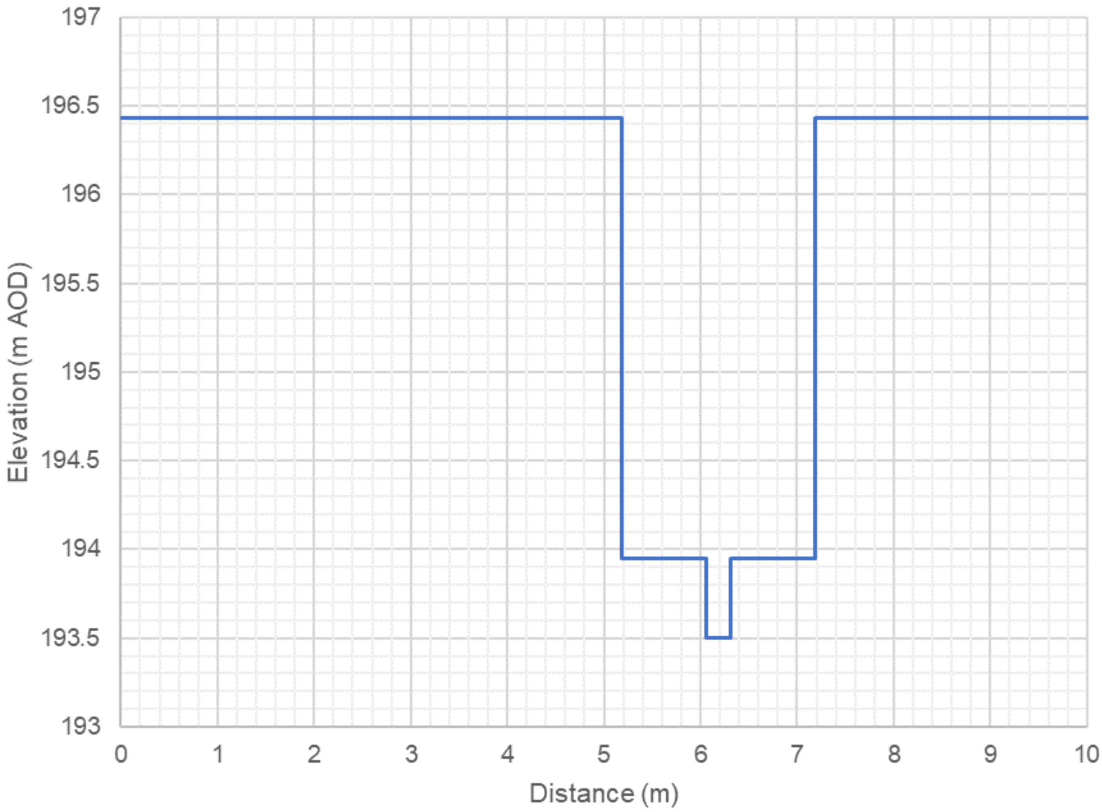


Figure 5-3. Schematic of Option 2.

The results for Option 2 are summarised in Table 5-2 and indicate that for all the modelled return periods except the 10 year, 100 year plus 75% climate change, 1000 year and safety check flood (PMF) events peak outflow is less than the baseline. During the 10 year, 100 year plus 75% climate change and the 1000 year events the peak outflow increases by less than 5% and these increases are considered to represent an insignificant change in flood risk.

With the embankment elevation set at a minimum of 196.43 m AOD, the embankment would be 2.2 m above the predicted design flood level (i.e. freeboard greater than 0.6 m) and 2.0 m above the safety check flood (i.e. freeboard greater than 0.14 m), thus indicating that the required freeboard would be maintained in these events. Given the freeboard available, there is scope to reduce the embankment elevation to around 194.83 m AOD (a 1.6 m reduction) whilst still maintaining the design and safety check flood requirements.

Table 5-2. Summary of results for Option 2.

Return Period (years)	Peak Inflow ($m^3\ s^{-1}$)	Peak Outflow ($m^3\ s^{-1}$)	Volume of Temporary Storage (m^3)	Peak Stage (m AOD)	Attenuation	Change in Peak Outflow from Baseline (%)
10	0.083	0.069	338	193.80	16.9%	1.5
30	0.112	0.094	415	193.86	16.1%	-1.1
100	0.154	0.130	540	193.95	15.6%	-3.0
100 plus 30% climate change	0.196	0.176	598	193.99	10.2%	-0.6
100 plus 75% climate change	0.258	0.243	672	194.04	5.8%	0.4
1000	0.259	0.244	672	194.04	5.8%	1.7
10,000	0.738	0.692	973	194.23	6.2%	-6.6
PMF	1.463	1.305	1,283	194.41	10.8%	32.0

5.3 Further Reduction on Embankment Elevation

Following initial presentation and discussion of options, NRW requested details of the potential for further reducing the embankment elevation if there was no requirement to meet the design and safety check flood requirements. If it was confirmed that there was no requirement to meet these standards, then the embankment in Option 1 could be lowered to around 195.07 m AOD (0.77 m lower than minimum required to meet design and safety standards), in Option 2 could be lowered to around 194.04 m AOD (0.79 m lower). With these embankment elevations the existing attenuation would still be replicated in flood events up to and including the 1000 year.

5.4 Construction Considerations

In order to facilitate / simplify construction of the slot, the ‘upper’ spillway in Option 1 and Option 2 (both 2 m wide) could potentially be lowered down to the invert of the slot, and then a stop log weir added to create the slot and original ‘upper’ spillway profile proposed within the two options (Option 1: slot – 0.5 m wide and 0.3 m; deep).

5.5 Flood Risk Considerations

To support the flood study a Flood Consequences Assessment has been produced (Appendix E). This summarises evidence presented in this report to demonstrate that the two options would not result in an increase in downstream fluvial flood risk and would significantly reduce the likelihood and consequences of flooding due to reservoir failure.

5.6 Discussion

A summary of the assessed options and their performance against the project requirements is provided in Table 5-3.

Table 5-3. Summary of option results

Option	Spillway Elevation (m AOD)	Retained Volume (m^3)	Spillway Width		Passes Requirements?		
			Slot (m)	Upper (m)	Attenuation	Design Flood	Safety Check Flood
Option 1	194.70	2,612	0.50	2	Yes	Yes	Yes
Option 2	193.50	790	0.25	2	Yes	Yes	Yes

Both options meet the project requirements for attenuation and reservoir safety whilst reducing the retained volume. Option 1 is considered by Arcadis' Design Engineers and NRW to be preferable, as it would reduce the scope of removal and deconstruction works associated with the embankment. Discussions with NRW indicate that this option would minimise environmental impacts. However, from a flood risk perspective the downstream consequences of a dam failure are further reduced with Option 2 as a result of the lower retained volume.

6 Update following NRW Review of Flood Study

Following the issue of the draft Flood Study, it was agreed in consultations between NRW and the QCE that, since Prince Llewelyn is not a Statutory Reservoir and the options investigated would further reduce the retained volume, the design flood could be changed from a 10,000 to a 1,000 year flood. Furthermore, it was agreed that there would be no requirement to meet the safety check flood standard.

There was also a change in Arcadis' Design Engineers' and NRW's option preference, with a variation to Option 2 being favoured. This new option, 'Option 3', is presented below.

6.1 Option 3

This option, which has been developed by Arcadis' Design Engineers, looks at further reducing the retained volume. It involves lowering the spillway to an elevation of 193.3 m AOD (lowered by 0.2m when compared to Option 2), which would reduce the retained volume to 599 m^3 (without silt). The lowered spillway has been modelled as a stepped spillway with the crest 2m wide and 0.6m deep stepping up to the existing embankment average elevation (196.43 m AOD) as shown in Figure 6-1. A weir coefficient of 1.7 (equivalent to a broad-crested weir) has been used. This reflects the improvement that can be achieved through the provision of a more efficient and well maintained (stone / concrete) spillway weir.

The spillway has been designed so that the crest is above the silt in the reservoir, thus preventing the mobilisation of the silt downstream. Other environmental and ecological aspects related to the spillway design are detailed within the environmental action plan (NRW, 2020)

Figure 6-1. Schematic of Option 3.

The results for Option 3 are summarised in Table 6-1 and indicate that for all the modelled return periods the peak outflow is slightly greater than the baseline. Although in the 10 year, 30 year, 100 year and 1000 year events the percentage change in outflow is greater than 5%, in reality the actual change in flow is relatively small (between 4.5 l s^{-1} and 13.3 l s^{-1}). This level of increase is considered to result in an insignificant change to downstream flood risk. With the embankment elevation set at a minimum of 196.43 m AOD, the embankment is set 2.96m above the predicted design (1000 year) flood level, thus confirming that sufficient freeboard would be maintained in this event.

Table 6-1. Summary of results for Option 3.

Return Period (years)	Peak Inflow (m^3 s^{-1})	Peak Outflow (m^3 s^{-1})	Volume of Temporary Storage (m^3)	Peak Stage (m AOD)	Attenuation	Change in Peak Outflow from Baseline (m^3 s^{-1})	Change in Peak Outflow from Baseline (%)
10	0.075	0.073	73	193.38	1.7%	0.0050	7.3
30	0.102	0.101	83	193.39	1.5%	0.0056	5.9
100	0.144	0.142	112	193.42	1.3%	0.0082	6.1
100 plus 30% climate change	0.186	0.184	131	193.44	1.1%	0.0075	4.3
100 plus 75% climate change	0.249	0.247	161	193.47	1.0%	0.0045	1.9
1000	0.256	0.253	161	193.47	0.9%	0.0133	5.6

In summary Option 3 is considered by Arcadis' Design Engineers and NRW to now be preferable, as it is easier to construct than either Option 1 or 2, maintain (reduced blockage risk) and meets the 1,000 year design flood safety standards. In terms of flood risk, it is considered to cause insignificant change downstream.

7 Conclusions and Recommendations

This flood study has been undertaken to support the development of the design of a new lowered spillway that would meet the required safety standards. The following conclusions can be drawn from the study.

7.1 Current Situation

Prince Llewelyn Reservoir holds 5,477 m^3 of water, based on the existing configuration of the primary spillway. This is well below the statutory 10,000 m^3 threshold of the Reservoirs Act 1975. Nevertheless, NRW is treating the reservoir in the spirit of the Act, and modelling results show that the reservoir does not meet the recommended safety standards for a Category A/B dam. In particular, the dam, which has a minimum crest elevation of 196.242 m AOD, lacks sufficient spillway capacity and freeboard to provide safe passage of both the design flood and the safety check flood. Stillwater flood levels are predicted to reach 196.43 m AOD during the 10,000 year event and 196.47 m AOD during the PMF. Estimates of wave overtopping discharges also far exceed the allowable rates.

7.2 Potential Options

To address this issue, NRW has explored options that meet the required safety standards and reduce the retained volume, whilst also maintaining the flood attenuation provided by the existing arrangement.

Option 1 reduces the retained volume to 2,612 m^3 (without silt), whilst providing similar attenuation of higher frequency (10 year, 30 year, 100 year and 1,000 year return periods) flood flows (classified in this study as no increase in peak outflow of more than 5% compared against the baseline scenario).

Option 2 further reduces the retained volume down to 790 m^3 (without silt), whilst again maintaining similar attenuation of flood flows.

Option 1 was initially considered by Arcadis' Design Engineers and NRW to be preferable, as it would reduce the scope of removal and deconstruction works associated with the embankment and minimise environmental impacts. However, from a flood risk perspective the downstream consequences of a dam failure are further reduced with Option 2 as a result of the lower retained volume.

7.3 Option 3

Following the issue of the draft flood study, a third option was developed. This preferred option was a variation on Option 2, and was developed to take into account an NRW/QCE agreed change to the design and safety check flood standard (used to configure the option); and Arcadis' Design Engineers proposed refinements to facilitate construction and maintenance (reduced blockage risk).

Option 3 reduces the retained volume to 599 m^3 (without silt). Although the percentage change in outflow will slightly increase (maximum +7.3%), in reality the actual change in downstream flow will be relatively small (between 4.5 $l\ s^{-1}$ and 13.3 $l\ s^{-1}$). This level of increase is considered to result in an insignificant change to downstream flood risk.

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APPENDIX A

Flood Study Summary Table - Flood Capacity and Freeboard Assessment

Existing Condition (E) of Prince Llewelyn Reservoir

E1.0	Site Information and Data	Response
E1.1	Site	Prince Llewelyn Reservoir
E1.2	Grid reference	SH 74232 53049
E1.3	Dam category	A/B
E1.4	Inflow flood (Table 2.1 - Floods and Reservoir Safety (ICE, 2015) guide)	Design Flood: 10,000 year Safety Check Flood: PMF
E1.5	Dam type	Composite (two stone walls infilled with clay or compacted earth)
E1.6	Surface of dam (embankment) crest	Grass
E1.7	Downstream slope material	Quarried stone blocks
E1.8	Upstream slope material - Roughness influence factor (γ_f)	Simple Slope 0.38 For wave heights less than 0.75 m, grass influences the run-up process and lowers the γ_f. For wave heights greater than 0.75 m, γ_f equals 1). As the wave height is less than 0.75 m (see Appendix D) Equation 5.23 (EurOtop Manual, 2018) has been applied.
E1.9	Upstream embankment slope	1 (V) : 1.5 (H)
E1.10	Top of dam level - Minimum embankment crest level - Average embankment crest level	 196.242 m AOD 196.430 m AOD
E1.11	Wave wall top level	-
E1.12	Core/level	Unknown
E1.13	Spillway crest elevation	195.974 m AOD
E1.14	Spillway crest width	2 m
E1.15	Spillway weir coefficient	1.0

E2.0	Catchment and Rainfall Data	Response
E2.1	Catchment area, A	0.046 km^2 (see Section 4.2.1)
E2.2	DPSBAR	179.4 m/km
E2.3	Standard Average Annual Rainfall (SAAR)	2,010 mm

E3.0	Flood Peak Inflow	Response
E3.1	Peak of 10,000 year flood	0.92 m^3/s (see Section 4.2.3)
E3.2	Peak of PMF	1.21 m^3/s (see Section 4.2.4)

E4.0	Estimated Stillwater Rise	Response
E4.1	Stillwater 10,000 year flood level	196.43 m AOD (see Section 4.4.2)
E4.2	Stillwater PMF level	196.47 m AOD (see Section 4.4.2)

E5.0	Wave Surcharge	Response
E5.1	Mean wave overtopping discharge	7.6 l/s/m (see Section 4.5)

E6.0	Conclusions	Response
E6.1	Check provision of adequate dam freeboard - Design Flood Conditions	Stillwater flood rise is expected to exceed the crest level of Prince Llewelyn dam. Therefore, current freeboard is inadequate.
E6.2	Check provision of adequate dam freeboard - Safety Check Flood Conditions	

Proposed Condition (P) of Prince Llewelyn Reservoir

P1.0	Site Information and Data	Response
P1.1	Site	Prince Llewelyn Reservoir
P1.2	Grid reference	SH 74232 53049
P1.3	Dam category	A/B
P1.4	Inflow flood (Table 2.1 - Floods and Reservoir Safety (ICE, 2015) guide)	Design Flood: 10,000 year (Option 1 & 2), 1,000 year (Option 3) Safety Check Flood: PMF (Option 1 & 2), N/A (Option 3)
P1.5	Dam type	Composite (two stone walls infilled with clay or compacted earth)
P1.6	Surface of dam (embankment) crest	Concrete, asphalt or similar
P1.7	Downstream slope material	Quarried stone blocks
P1.8	Upstream slope material - Roughness influence factor (γ_f)	Simple Slope 0.38 (existing roughness of dam face maintained)
P1.9	Upstream embankment slope	1 (V) : 1.5 (H)
P1.10	Top of dam level	Range of options from 194.04 to 196.43m AOD (see Section 5 & 6)
P1.11	Wave wall top level	N/A
P1.12	Core/level	Unknown
P1.13	Spillway crest elevation	Option 1 194.7m AOD, Option 2 193.5m AOD and Option 3 193.3m AOD
P1.14	Spillway crest width	2m (see Section 5.1, 5.2 & 6.1),
P1.15	Spillway weir coefficient	1.7

P2.0	Catchment and Rainfall Data	Response
P2.1	Catchment area, A	0.046 km² (see Section 4.2.1)
P2.2	DPSBAR	179.4 m/km
P2.3	Standard Average Annual Rainfall (SAAR)	2,010 mm

P3.0	Flood Peak Inflow	Response
P3.1	Peak of 10,000 year flood	Option 1 - 0.65 m³/s & Option 2 - 0.69 m³/s, (see Section 5.1 & 5.2)

	Peak 1,000 year flood	Option 3 – 0.24 m³/s (see Section 6.1)
P3.2	Peak of PMF	Option 1 - 1.26 m³/s & Option 2 - 1.31 m³/s, (see Section 5.1 & 5.2) Option 3 – N/A

P4.0	**Estimated Stillwater Rise**	**Response**
P4.1	Stillwater 10,000 year flood level Stillwater 1,000 year flood level	Option 1 – 195.24m AOD & Option 2 – 194.23m AOD, (see Section 5.1 & 5.2) Option 3 – 194.04 m AOD (see Section 6.1)
P4.2	Stillwater PMF level	Option 1 – 195.43m AOD & Option 2 – 194.41m AOD, (see Sections 5.1 & 5.2) Option 3 – N/A

P5.0	**Wave Surcharge**	**Response**
P5.1	Mean wave overtopping discharge	0.001 l/s/m or less for the design flood and less than 0.1 l/s/m for the safety check flood. (see Section 4.5)

P6.0	**Conclusions**	**Response**
P6.1	Check provision of adequate dam freeboard - Design Flood Conditions	Stillwater flood rise is predicted to be more than 600 mm below the proposed crest of the dam, which satisfies the recommended safety standards.
P6.2	Check provision of adequate dam freeboard - Safety Check Flood Conditions	Stillwater flood rise is predicted to be below the proposed crest of the dam and there is no overtopping discharge. This satisfies the recommended safety standards. No requirement to satisfies the recommended safety standards for Option 3.

APPENDIX B

Reservoir Flood Inflow Calculations

APPENDIX C

Wave Overtopping Calculations

Calculation of Significant Wave Height and Wave Period

1.0	Parameter	Value	Reference
1.1	Fetch length	Between: 27 m (A) and 74 m (F)	Figure D1
1.2	Wind direction	Between: 240° (I) and 299° (A)	Figure D1
1.3	Reservoir altitude	196 m AOD	-
1.4	Altitude Adjustment (f_A)	1.20	f_A = 1.0 + (0.001 x alt)
1.5	Over-Water Adjustment (f_W)	1.00	Table 5.2 - Floods and Reservoir Safety (ICE, 2015) guide
1.6	Duration Adjustment (f_D)	1.05	Recommended value for UK reservoir lengths ≤ 2 km
1.7	Wind speed (U_{50})	23 m/s	Figure 5.2 - Floods and Reservoir Safety (ICE, 2015) guide
1.8	Direction adjustment (f_N)	Between: 0.91 (A) and 1 (I)	Table 5.3 - Floods and Reservoir Safety (ICE, 2015) guide
1.9	Return period adjustment for mean annual wind speed (f_T)	0.79	Table 5.1 - Floods and Reservoir Safety (ICE, 2015) guide

1.10

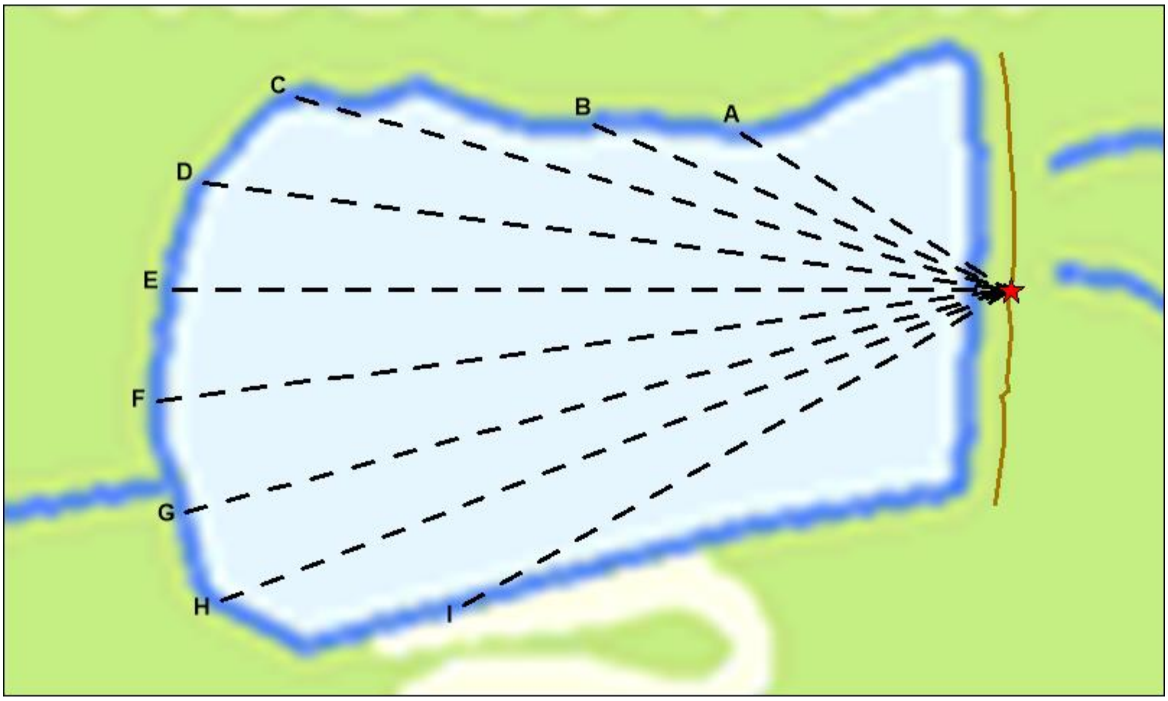

Figure D1. Fetch directions

Calculation of mean (qmean) and maximum (qmax) overtopping discharges

2.0	Parameter	Value	Reference
2.1	Berm influence factor (γ_b)	1	
2.2	Roughness influence factor (γ_f)	0.38	Chapter 5 - Floods and Reservoir Safety (ICE, 2015) guide and Chapter 5 EurOtop Manual (EurOtop, 2018)
2.3	Wave obliquity factor (γ_β)	1	
2.4	Wavewall influence factor (γ_v)	1	
2.5	Upstream embankment slope	34 degrees	(approximately 1 in 1.5)
2.6	Freeboard (Rc)	Overflow is predicted in both the design and safety check flood	

Results

In order to satisfy the required standards, a minimum freeboard of 240mm and 140mm, above the design flood and safety check flood stillwater level, would be required respectively. Details are provided below (critical fetch length are highlighted in bold).

Ref.	Fetch length (m)	Wind direction (°N)	f_N	Wind speed, U (m/s)	Wave height, H_s (m)	Peak wave period, T_p (s)	Design Flood		Safety Flood	
							qmean (l/s/m)	qmax (l/s/m)	qmean (l/s/m)	qmax (l/s/m)
A	27	299	0.91	20.8	0.06	0.64	0.000	0.00	0.000	0.00
B	39	291	0.94	21.4	0.08	0.72	0.000	0.00	0.002	0.00
C	64	284	0.96	21.9	0.10	0.85	0.000	0.00	0.030	0.00
D	70	277	0.98	22.3	0.11	0.88	0.000	0.00	0.052	0.01
E	72	270	0.99	22.6	0.11	0.89	0.001	0.00	0.065	0.01
F	74	262	1	22.8	0.11	0.90	0.001	0.00	0.077	0.01
G	**73**	**254**	**1**	**22.9**	**0.11**	**0.90**	**0.001**	**0.00**	**0.078**	**0.01**
H	73	248	1	22.9	0.11	0.90	0.001	0.00	0.076	0.01
I	54	240	1	22.8	0.10	0.82	0.000	0.00	0.019	0.00

APPENDIX D

Q50 Inflow

APPENDIX E

Flood Consequences Assessment

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