Wimbledon Park Reservoir

Inlet Flow Confirmation Report

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For Information

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Wimbledon Park Reservoir

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List of Abreviations

Abreviation	Meaning
WB	Ward & Burke Construction Ltd.
WPL	Wimbledon Park Lake
ICE	Institution of Civil Engineers
FSR	Flood Studies Report
FEH	Flood Estimation Handbook
PMF	Probable Maximum Flood
AOD	Above Ordanance Datum
ARPE	All Reservoir Panel Engineer
QCE	Qualified Civil Engineer
BWB	Boon, Wright and Blunt Consulting
TFL	Transport For London
PMP	Peak Maximum Precipitation

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1 Introduction

1.1 Scope

Ward and Burke Construction Limited have been commissioned by Merton Borough Council to undertake the design and build of a solution to works recommended on Wimbledon Park Lake (WPL) under the Reservoirs Act 1975. These works require the upgrade of the existing spillway and construction of additional spillways to accommodate safe passage of the flood events as required under the Reservoirs Act 1975 for a Category A Reservoir. For a category A dam the following flood conditions are identified:

- **Safety check flood conditions:** Probable Maximum Flood (under just full conditions) and mean annual maximum hourly wind speed.
- **Design flood conditions:** 1 in 10000 annual probability (0.01%) of flooding and mean annual maximum hourly wind speed.

This report and the assessments and options outlined within have been prepared in consultation with Dr Andy Hughes BSc (Hons) PhD DMS CEng FICE FCIWEM MIM All Reservoirs Panel Engineer (ARPE).

1.2 Site Location

The site of the existing reservoir to be modified is in Wimbledon Park in the southwest of London. The site is located on recreational park land and neighbours the local Wimbledon Park Golf Club, Watersports and Outdoor Center and playground. The site overview drawing for this project is shown in WMBLDN-WAB-XX-XX-DR-C-010003 - Site Overview. An extract is below in Figure 1.1.

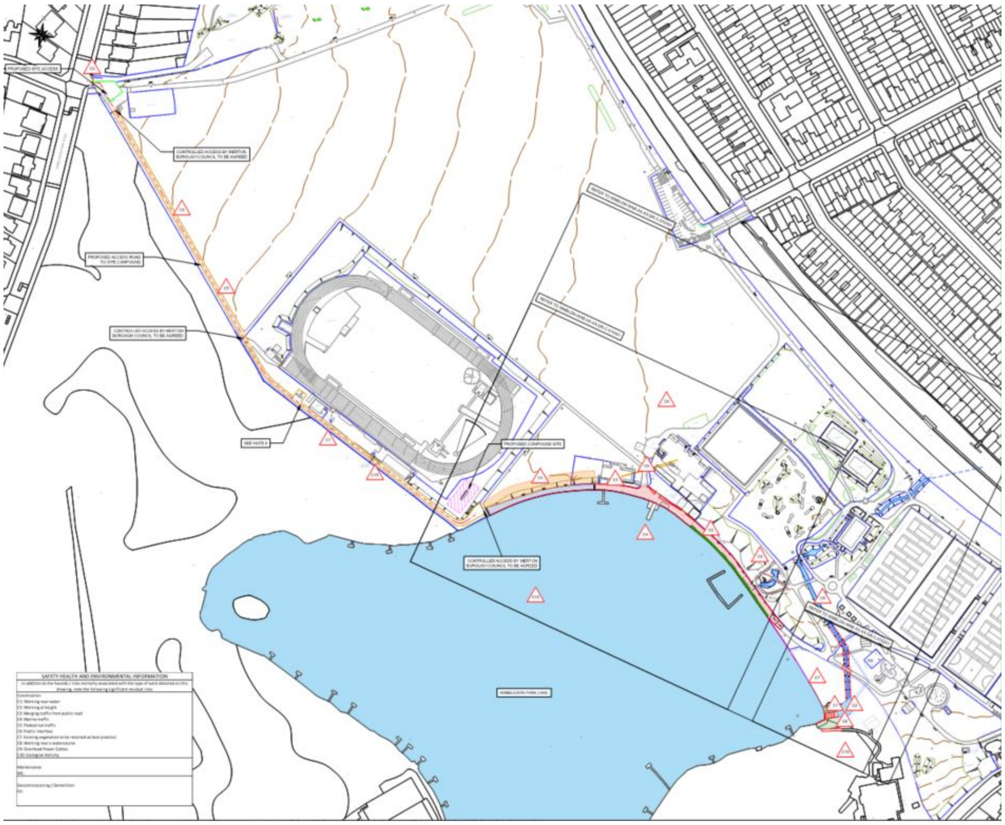

Figure 1.1: Site Location

1.3 Background

Wimbledon Park originally comprised part of the grounds of Wimbledon Manor House. In the 18th century the park was landscaped by Capability Brown who formed the reservoir by constructing a dam across an existing watercourse. The current public park and part of the reservoir is owned by the Borough Council of Merton. Wimbledon Park is home to range of public leisure facilities, with the reservoir offering water sport activities and angling. The Wimbledon Club, Wimbledon Park Golf Course and the All England Lawn Tennis and Croquet Club (AELTC) are located to the west and south of the Reservoir, and Wimbledon Athletics Track is located to the north. The Wimbledon public park is located to the north-east of reservoir, downstream of the impounding dam. The park includes a number of leisure facilities, including: a bowling green; a water sports centre; tennis courts; cafeteria; and play park. The park is bound to the northeast by London Underground railway embankment for the District Line. Vehicular access to the park is available through a tunnel beneath the railway line at the end of Revelstoke Road. The 'Southfields Grid' residential area is located beyond the railway line, and beyond that the River Wandle.

The reservoir is impounded along its north-eastern edge by an earth fill embankment approximately 320m long and up to 4m high. The crest is set at approximately 17.70m Above Ordnance Datum (AOD) and carries a public right of way for most of its length. The downstream side of the embankment is covered by a mix of bare ground with grass cover, tarmac and concrete paths and steps, scrub, established trees and buildings/boat storage. The lake has a surface area of approximately 9 hectares and a theoretical maximum storage capacity of approximately $160,000m^3$. However a bathymetric study has been carried out and it found that the total volume impounded above silt level is 78841^3.

Main inflows into the reservoir are via two 900mm diameter Thames Water surface water sewers, These serve the upstream catchment of the lake of approximately 2.54 km^2.

1.3.1 Existing Outlet/Outfall Structures

A 100mm diameter pipe is located on the southern end of the dam which is controlled by a sluice valve in a manhole near the toe. This discharges to a pool located next to the toe of the dam. This pipe acts as a drain down facility and is normally in the closed position. The pool discharges to a 450mm diameter pipe which flows in a northerly direction and outfalls to a reach of open watercourse. The watercourse flows through the park in a series of open and culverted sections, before entering a 1.0m diameter pipe beneath the railway line and joining the Thames Water surface water sewer system.

A second 100mm diameter pipe is situated near the centre of the dam and feeds the ornamental 'Waterfall Garden' set into the downstream face of the dam. There are four manholes on the crest of the embankment which are associated with this pipe, two of which contain valves for its operation. The Waterfall Garden joins the watercourse downstream of the first outlet.

A 0.7m wide stepped concrete channel arrangement located on the southern end of the dam provides the current overflow facility for the dam. The outlet pipes are not sufficient to pass the normal flow from the lake, and therefore the overflow weir controls the normal water level. Flow entering the channel is controlled by a 1.3m wide weir, whose level is maintained by a stoplog board facility. Currently the crest of the weir is set at 17.47m AOD which is approximately 230mm below the crest of the dam, and 880mm above the average silt level. However, from recent surveys this has deviated from its set level, and the water level can rise or fall below this due to rainfall or droughts. This still provides attenuation for minor rainfall events before the crest level overtops from storm events above the 1 in 1 year event.

There are currently a number of small inlets which are not listed above but are captured in the hydrological assessment.

It is reported that due to the limited freeboard between the crest of the dam and the normal water level in the lake, wave overtopping of the dam has been observed to occur, even during quite small flood events. The embankment currently has no auxiliary overflow to prevent overtopping of the dam in flood events.

2 Hydrological Assessment

An initial study of the sites hydrology has been provided in the WPL-BWB-ZZ-XX-RP-YE-0001_Reservoir Surcharge Modelling for which a summary of the peak inlet flows to WPL is provided below.

Annual Probability (1 in x (years))	Annual Probability (%)	Peak Inlet Flows (m^3s^{-1})
5	20	2.30
10	10	2.77
20	5	3.26
30	3.3	3.62
50	2	4.07
75	1.3	4.45
100	1	4.80
150	0.6	5.41
200	0.5	5.91
1000	0.1	9.09
10000	0.01	37.59
PMF	-	78.91

Table 2.1: BWB Inlet Flow

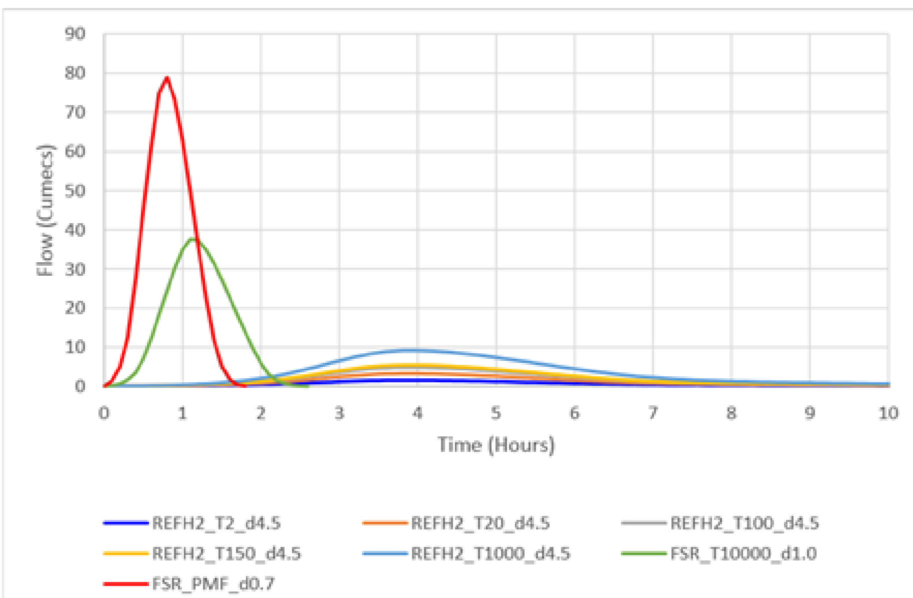

Figure 2.1: BWB Hydrographs

On initial review, the inflow hydrology appeared to be overestimated when considering the PMF flows. An analysis undertaken in accordance with Section 3 of Floods and Reservoir Safety: An Engineering Guide 2015 (ICE) (4th Edition) gave a peak inflow value of $30.26m^3s^{-1}$.

$$Q_m = 0.454A^{0.937}S_{1085}SAAR^{0.319} \tag{1}$$

To which A is the catchment area, S_{1085} is the mainstream slope (m/km), this calculation assumes that the catchment soils are impermeable and that there is no urban area in the catchment. This was reaffirmed in the most

recent Statutory Inspection under Section 10 of the Reservoirs Act (1975) (2014) which stated in Section 11.1.1 that a previous flood study suggested a PMF of under $40m^3s^{-1}$. This mentioned flood study is not available but pre-dates the most recent flood study undertaken by BWB dated April 2018.
On further analysis of the flood magnitudes relative to the flood duration's, as shown in Figure 2.1, there are 2 different methods employed to measure critical flows. According to Document WPL-BWB-ZZ-XX-RP-YE-0001_TN1 (Flood Surcharge Hydraulic Modeling) for flows less than the 1000-year event the Revitalised Flood Hydrograph (ReFH2) rainfall-runoff model was used and the FSR/FEH rainfall-Runoff model Depth Duration Frequency Model used for the 1 in 10000 and PMF events, in conjuction with the FEH13 and the FSR rainfall depth duration frequency models respectively to model the rainfall.

With the availability of the design storm depth from the FEH 2013 DDF (FEH13) it was extracted from the FEHWeb services up to the 100,000 year event, the ReFH model was subsequently used to calculate the hydrology. Floods and Reservoir Safety (4th Edition) Pg.55 Section 'FEH13 Rainfall DDF Model' recognises that the FEH13 rainfall model can provide the storm events up to the 10,000-year event.

In accordance with the guidance in Floods and Reservoir Safety 'T_p (time-to-peak) of the PMF hydrograph is taken to be two-thirds of that of a return period hydrograph, to ensure a more severe condition'. This was employed to model the PMF storm relative to the 10,000-year event. This approach does not use the FSR data as normally recommended to calculate a run-off model for the PMF and the calculation of the PMP. However, previous ReFH estimates have been found to exceed the FSR PMP models, these are given in 'Estimating the probable maxiumum flood in UK catchments ising the ReFH model' (2020).

In this case, reducing the ReFH model to a short, more intense duration, is assumed to give a conservative figure for the PMF Hydrograph. Accompanying this, the flood events for longer duration floods when using the ReFH Model were more extreme that those used in the development of the FSR/FEH model.

2.1 Design Methodology

Using the ReFh model to model flood events requires four model parameters (BL, BR, C_{max} and T_p) to be estimated for the particular catchment under consideration. As explained in the FEH supplementary report vol. 1 (Chapter 3), the recommended procedure for obtaining the model parameters depends on data availability. If sufficient flood event information is available in the form of coherent sub-daily rainfall and runoff data, then the parameters can be estimated through a joint analysis of these data. This procedure is intensive in both time and data and requires specialized software. For an ungauged site where no data is available, the ReFH model parameters can be estimated directly from catchment descriptors, which are readily available via the FEH CD-ROM.

To produce this hydrological model, a variety of hyetographs and hydrographs were produced from key parameters for the catchment area from the FEH web service. The following sections are based on the FEH and FEH supplementary report vol.1 where this can be followed. This is an iterative process due to the lag from the reservoir being taken into account, the inlet flows cannot be determined without modeling the outlet flows across the reservoirs spillways and embankments. The initial hydrology assessment under taken by WB used a time step of 6 minutes, this report and method uses a time step of 36 seconds, this increases the 'resolution' of the model.

2.1.1 Key Catchment parameters

Review of the Key Catchment Descriptors used for the catchment in both the Ward and Burke analysis and the BWB analysis show minor discrepancies between the data. These do not have a significant impact on the output data and for this analysis Ward and Burke have used the data as given in the FEHWeb Services shown in table 2.2. The flood attenuation from reservoirs and lakes (FARL) was set to 1 to discount the attenuating effect of the lake, as this will be assessed hydraulically.

Parameter	Symbol	Value
Base Flow Index	BFI_{Host}	0.435
Flood Attenuation from Reservoirs and Lakes	FARL	1
Floodplain extent	FPEXT	0.05
Proportion of time soils are wet	PROPWET	0.290
Average annual rainfall (1961-1990)	$SAAR_{6190}$	613
Standard percentage runoff	SPR_{Host}	40.310
Factor of urban extent (2000)	$URBEXT_{2000}$	0.4396
Mean of all internodal slopes	DPS_{Bar}	45.9
Mean distance between each node	DPL_{Bar}	1.130

Table 2.2: Key Catchment Parameters

2.1.2 Design storm profile

The storm duration analysis does not consider reservoir lag in determining the time to peak, however this is used in the total storm duration, as per volume 4 of the FEH. However, the ReFH equation was used to calculate the time to peak under the analysis which differs to the approach adopted by BWB (FSR/FEH Methodology).

In accordance with the Floods and Reservoir Safety (4th edition) to ensure more severe condition the T_p of the PMF hydrograph was taken to be 2/3 of that of a return period hydrograph. In which case, the calculated storm duration for the PMF event was modified to suit.

One key difference in the PMF calculation is the selected rainfall volume, which BWB selected using the guidance from the FEH handbook, PMP calculation for a 2hr duration event. Given that this events duration is significantly less than the posed PMP for the 2 hour storm duration, the data from FEH13 was used for the design hydrograph to provide a more reflective intensity of rainfall.

The design rainfall for the PMF was taken from the FEH web service for a 100,000 year event, this provides the maximum estimation of rainfall from the web service. BWB discounted the reservoir lag, where as WB have calculated it to be between 18 minutes and 90 minutes depending on the storm return period. Due to short time to peak of the PMF storm an 18 minute lag is comparatively sizable and can not be discounted.

$$T_p = 1.56PROPWET^{-1.09}DPLBAR^{0.60}(1 + URBEXT_{2000})^{-3.34}DPSBAR^{-0.28} \quad (2)$$

$$T_p = 4.270PROPWET^{-0.80}DPLBAR^{0.54}(1 + URBEXT_{2000})^{-5.77}DPSBAR^{-0.35} \quad (3)$$

The ReFH and FSR/FEH equations for time to peak T_p respectively.

The rainfall volume selected was done so using FEH13 rainfall data in both instances. The BWB report states that this volume was selected for a 2.6-hour duration storm which differs from the duration outlined in their initial calculation. Both durations were calculated using the FSR/FEH equation:

$$D = (T_p + ReservoirLag)(1 + \frac{SAAR_{6190}}{1000}) \tag{4}$$

Where: D is the storm duration in hours, $SAAR_{6190}$ is the standard average annual runoff rainfall (mm), T_p is the time to peak of the unit hydrograph and the reservoir lag is the time difference in the peak inflow and outflow of the reservoir.

Seasonal correction factors were adopted, and a storm profile was formed on the basis of a 50% summer profile. This is in line with the guidance outlined in Flood Estimation Handbook Supplementary Report No. 1 stating 'On catchments characterised as being urbanised ($0.125 < URBEXT < 0.50$) the 50% summer profile has been adopted, which is on average more peaked than 50% of observed UK summer storms.' The adjustment for climate change for the 100 year storm has been only made to the rainfall, and the duration or lag has not been adjusted as per the FEH.

Return Period	WB Rainfall (mm)	WB Duration (hrs)	WB Reservoir Lag (hrs)	WB Time to Peak (hrs)	BWB Rainfall (mm)	BWB Duration (hrs)	BWB Reservoir Lag (hrs)	BWB Time to Peak (hrs)
1	18.18	3.4782	1.5	0.6563	-	-	-	0.65
10	37.10	2.9942	1.2	0.6563	-	-	-	0.65
100	58.94	2.5104	0.9	0.6563	-	-	-	0.65
100+70%	100.198	2.5104	0.9	0.6563	-	-	-	0.65
1000	92.84	1.8652	0.5	0.6563	-	-	-	0.65
10000	128.61	1.5426	0.3	0.6563	115.32	1.048*	0.125**	0.65
PMF	156.66	1.1897	0.3	0.4375	190	0.7***	0.125**	0.452

Table 2.3: Storm data

*BWB calculated a storm duration of 0.967 hours but used a duration of 1.048 in their software and chose the design rainfall from a 2.6 hour storm.
**Discounted for being too small.
*** The storm is said to be 0.7 hours in length, however the design rainfall was selected for a 2 hour storm.

2.1.3 Baseflow Model

The baseflow model implemented in the ReFH model is described in the FEH supplementary report volume 1. The equations selected for each variable are shown below. These are lifted from the FEH, in which they are described fully.

$$BL = 25.5BFIHOST^{0.47}DPLBAR^{0.21}PROPWET^{-0.53}(1 + URBEXT_{2000}^{-3.01} \tag{5}$$

$$BR = 3.75BFIHOST^{1.08}PROPWET^{0.36} \tag{6}$$

$$k_1 = \frac{BR}{1+BR} * (\frac{BL(1-k_3)}{t(1+BR)} - k_3) \tag{7}$$

$$k_2 = \frac{BR}{1+BR} * (1 - \frac{BL(1-k_3)}{t(1+BR)}) \tag{8}$$

$$k_3 = exp(\frac{-t(1+BR)}{BL}) \tag{9}$$

$$C_{max} = 596.7BFIHOST^{0.95}PROPWET^{-0.24} \tag{10}$$

$$C_{ini,summer} = \frac{C_{max}}{2}(0.90 - 0.82BFIHOST - 0.43PROPWET) \tag{11}$$

$$BF_{0,summer} = 33.9 * (C_{int} - 85.4) + 3.14 * SAAR_{6190}10^{-5}A \tag{12}$$

The above equations are then used in each time step to calculate the percentage runoff and hence the Nett precipitation and Soil Moisture, which are then further used to calculate the total inflows to the reservoir.

2.1.4 Hydrographs

The data from the storm profile was used to undertake an analysis of base magnitude during the storm using the guidance set out in Flood Estimation Handbook Supplementary Report No. 1. This data was also used to form the Unit Hydrograph of the event, again, in line with Flood Estimation Handbook Supplementary Report No. 1. See the following subsection for the hydrographs produced for each storm.

2.2 Design Output

The models produced are shown below, where they show the duration and intensity of rain over the catchment area over multiple time steps and how this impacts the inflow to the reservoir. A table highlighting the peak flows is also produced.

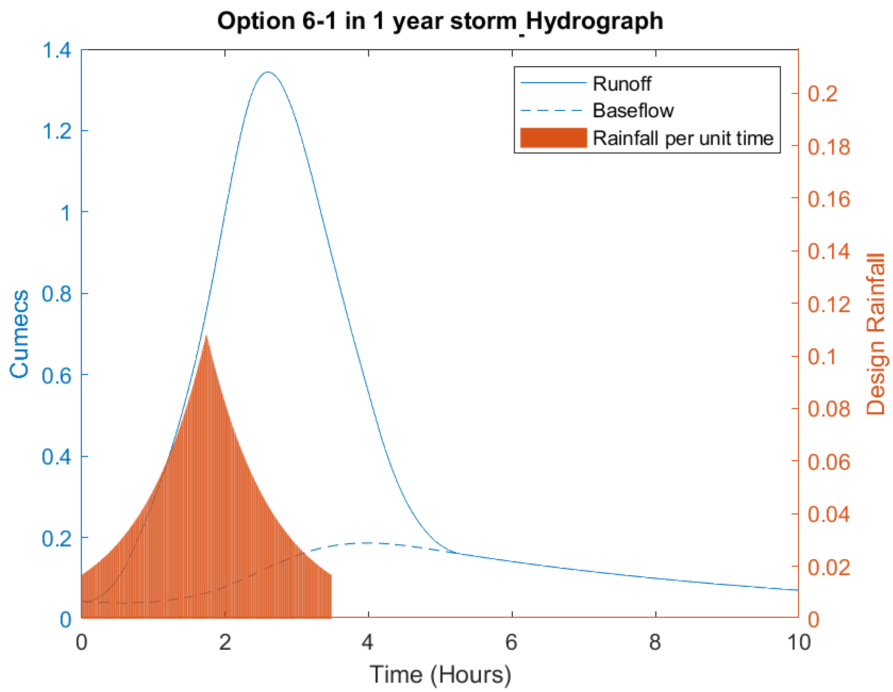

Figure 2.2: Hydrograph for a 1 in 1 year storm event

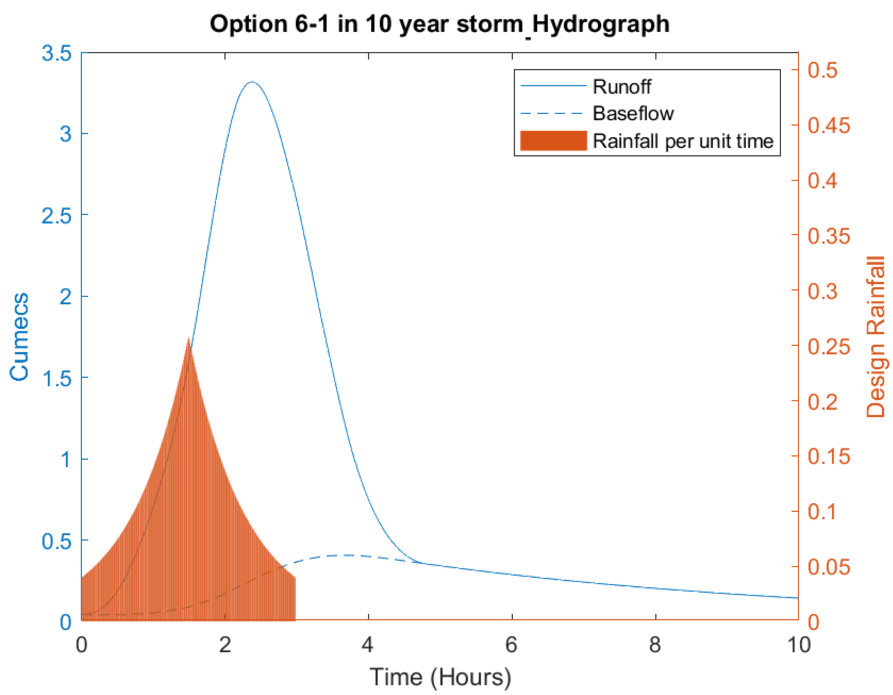

Figure 2.3: Hydrograph for a 1 in 10 year storm event

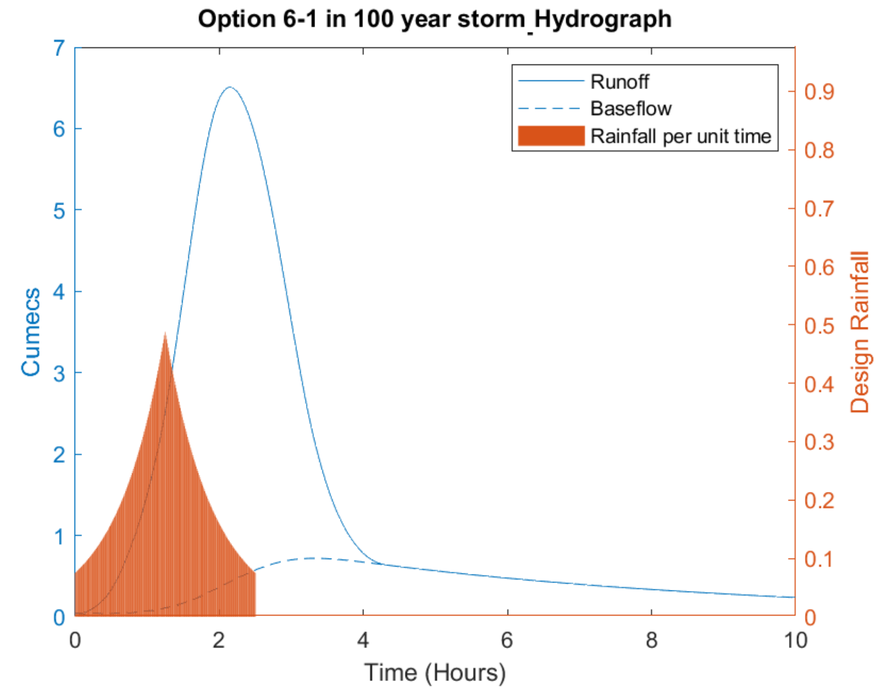

Figure 2.4: Hydrograph for a 1 in 100 year storm event

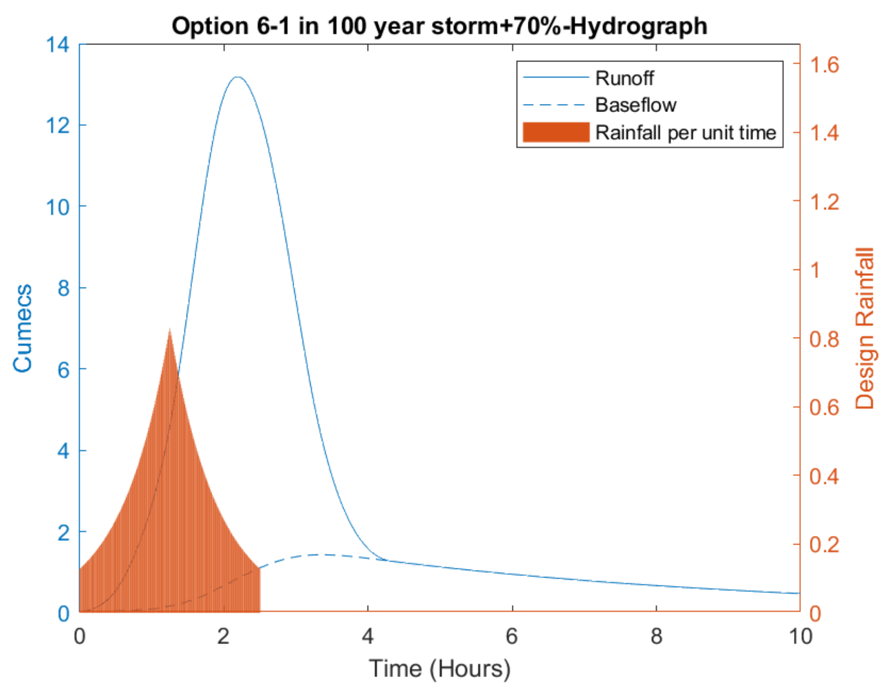

Figure 2.5: Hydrograph for a 1 in 100 year storm event (adjusted for climate change)

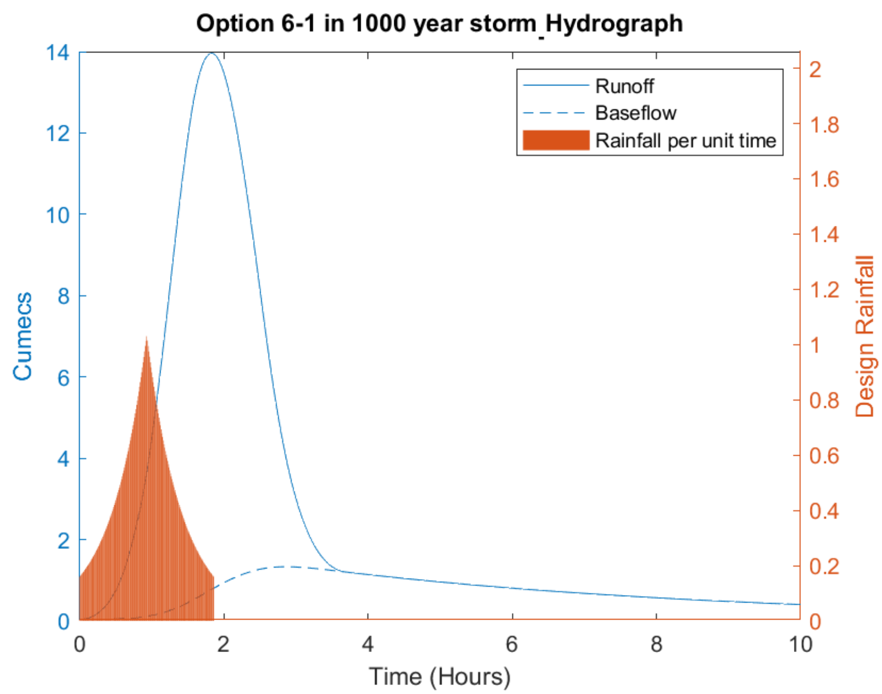

Figure 2.6: Hydrograph for a 1 in 1000 year storm event

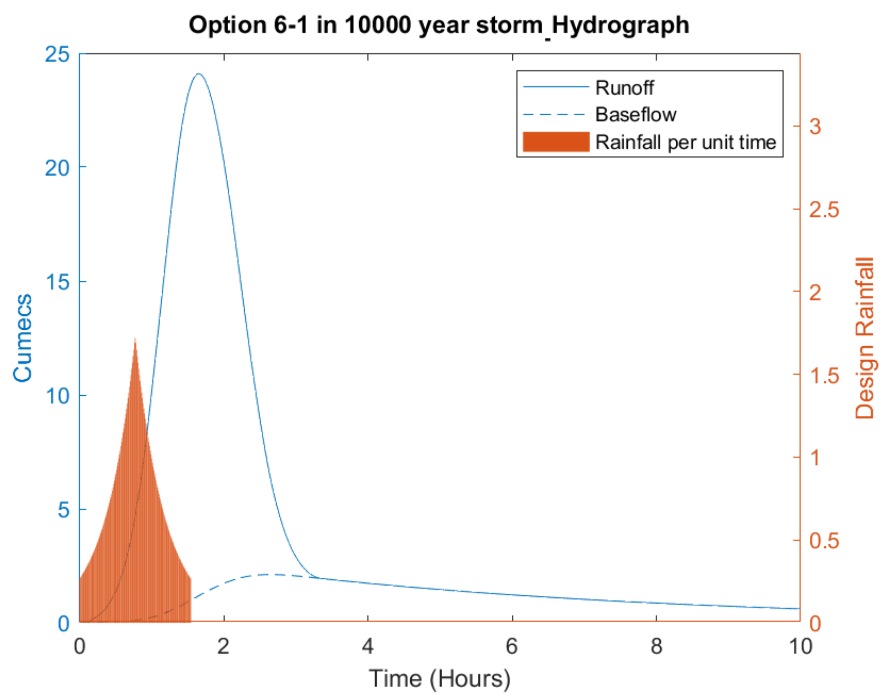

Figure 2.7: Hydrograph for a 1 in 10000 year storm event

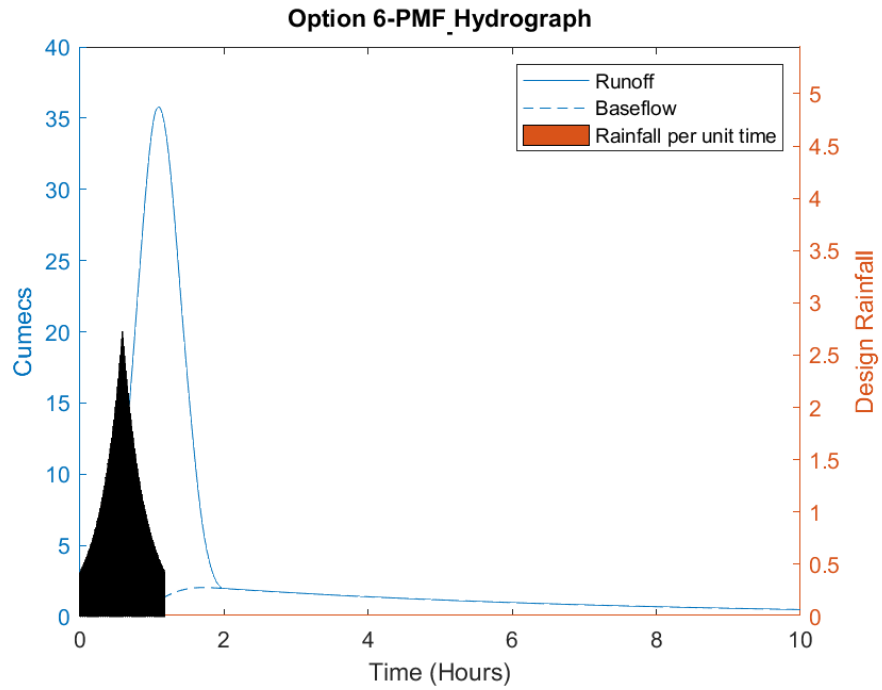

Figure 2.8: Hydrograph for a PMF storm event

Annual Probability (1 in x (years))	Annual Probability (%)	Peak Inlet Flows (m^3s^-1)
1	100	1.3438
10	10	3.3172
100	1	6.5069
100 + 70%	1	13.2089
1000	0.1	13.9566
10000	0.01	24.0972
PMF	-	35.791

Table 2.4: WB - Inlet Flow Information

2.3 Conclusion of Hydrology

The Ward and Burke calculations provide a more conservative estimate of the storm events up to and including the 1 in 1000 year storm in comparison to the estimates provided by BWB as shown by tables 2.1 and 2.4.
The flows calculated for the 10,000 year return and PMF return are larger than those estimated by BWB but appear to be more reflective to the flow identified when using the ReFEH method for the lower return periods. Given the size of this catchment, and the very short storm duration, the data extracted from the FEH/FRS method used by BWB, which is more commonly used for longer return period, gives over conservative flood volumes. One main example of this is the rainfall volume of the 10000 year and PMF storrms, which have been chosen for a much longer storm that the duration calculated by BWB. This creates design parameters which may not reflect the true condition of the catchment.

This flood study allows a design which can be more considerate to the park to be carried out and constructed instead of those listed in BWB's Stage 2 Outline Options Report, one of which involved decreasing the level of the lake by over two metres, in addition to other works.

References

1. Floods and Reservoirs safety [ICE](1999)
2. Hydraulic Structures [P. Novak et al.]
3. Open Channel Hydraulics [V. Te Chow et al.](1959)
4. Flood Estimation Handbook [Institute of Hydrology](1999)
5. Flood Estimation Handbook:Supplementary Report No.1 [Institute of Hydrology](2007)
6. Flood Estimation handbook Web Service (https://fehweb.ceh.ac.uk/)
7. Wimbledon Park Lake Flood Study: Stage 1 Flood Surcharge Hydraulic Modelling [BWB](2018)
8. Wimbledon Park Lake Flood Study: Stage 2 Outline Options Report [BWB](2019)
9. Estimating the probable maximum flood in UK catchments using the ReFH model [S. Pucknell et al.] (2020)