

 T J Booth Associates. Consulting Engineers 20A Eastgate St, Rochdale OL16 1DH	Project wharmby / lower town house farm barn, littleborough				Job Ref.	
	Section reservoir floor risk assessment				Sheet no./rev. 1 / 0	
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Reservoir Flood Risk Assessment, Lower Townhouse Barn, Littleborough, OL15 9JX.

History.

Revision	Date	Reason
0	Mar 26	original

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Background.

T J B A have been engaged by the owner Mr Wharby Esq to assess the Flood Risk aspects of a reservoir and embankment dam to the north of Lower Town House Farm Barn.

The assessment will utilise published data to determine the flood magnitude and frequency and the ability of the reservoir (Lower Reservoir) to safely cope with the flows to address Town & Country Planning Conditions for the development of the barn.

The reservoirs were originally used for water supply for the Fothergill & Harvey factory which are no longer present and currently provide a recreational fishing resources.

The reservoirs & associated upstream watercourse are modelled in the EA surface water flood extent maps, and include an allowance for climate change (*see Appendix*).

The surface water extents map shows there is a low (0.1-1% AEP/100-1000yr) to medium chance (1-3.3% AEP/30-100yr) of flooding of the watercourse feeding the lower reservoir.

The flood depth maps suggest the lower reservoir itself attenuates much of the water, with a high chance (>3.33% AEP/<30yr) of up to 300mm depth, and a low chance (0.1-1% AEP/100-1000yr) of up to 600mm depth. The flood depth maps suggests it has enough capacity to cope with excess rainfall events (including climate change) without breaching its banks.

However it is suspected that the EA modelling does not include the input from Clough Brook to the upper reservoir, so despite their published maps for the purposes of this report flood estimates are determined using published hydrometric methods based upon the Flood Estimation Handbook and Institute of Hydrology methodology.

The reservoirs are part of a water course so intrinsically are Flood Zone 3a (high probability), however the surrounding areas are classed as Flood Zone 2 (ref; *rep-tjba-wharmby-lwr townhouse fm-FRA-rev1, Jul 25*).

Purpose of Flood Risk Assessment.

The purpose of the assessment is to inform & accompany a planning application for a residential development within a designated Flood Zone 2 area.

Reference.

National Planning Policy Framework and the Planning Practice Guidance (flood risk & coastal change notes).

Flood Zone 1 is defined as land assessed as having less than a 1:1000 annual probability of river flooding (<0.1%) in any year; (land shown in white on the Flood Map). Flood Zone 3 is defined as land assessed as having a 1 in 100 or greater annual probability of river flooding (>1%) in any year; (land shown in dark blue on the Flood Map).

The Planning Practice Guidance indicates the Flood Risk Vulnerability Classification (Table 2) for the Application Site end use to be 'Less Vulnerable'. The Flood Risk 'Vulnerability' and Flood Zone 'Compatibility' matrix (Table 3) indicates that 'Less Vulnerable' development is acceptable in Flood Zone 3 areas.

Preliminary or Full Flood Risk Assessment.

Preliminary to support planning application.

Location & Catchment Plan.

Direct Catchment Plan.

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The local area is on the edge of the urban fringe, surrounded generally by rural grazing and woodland.

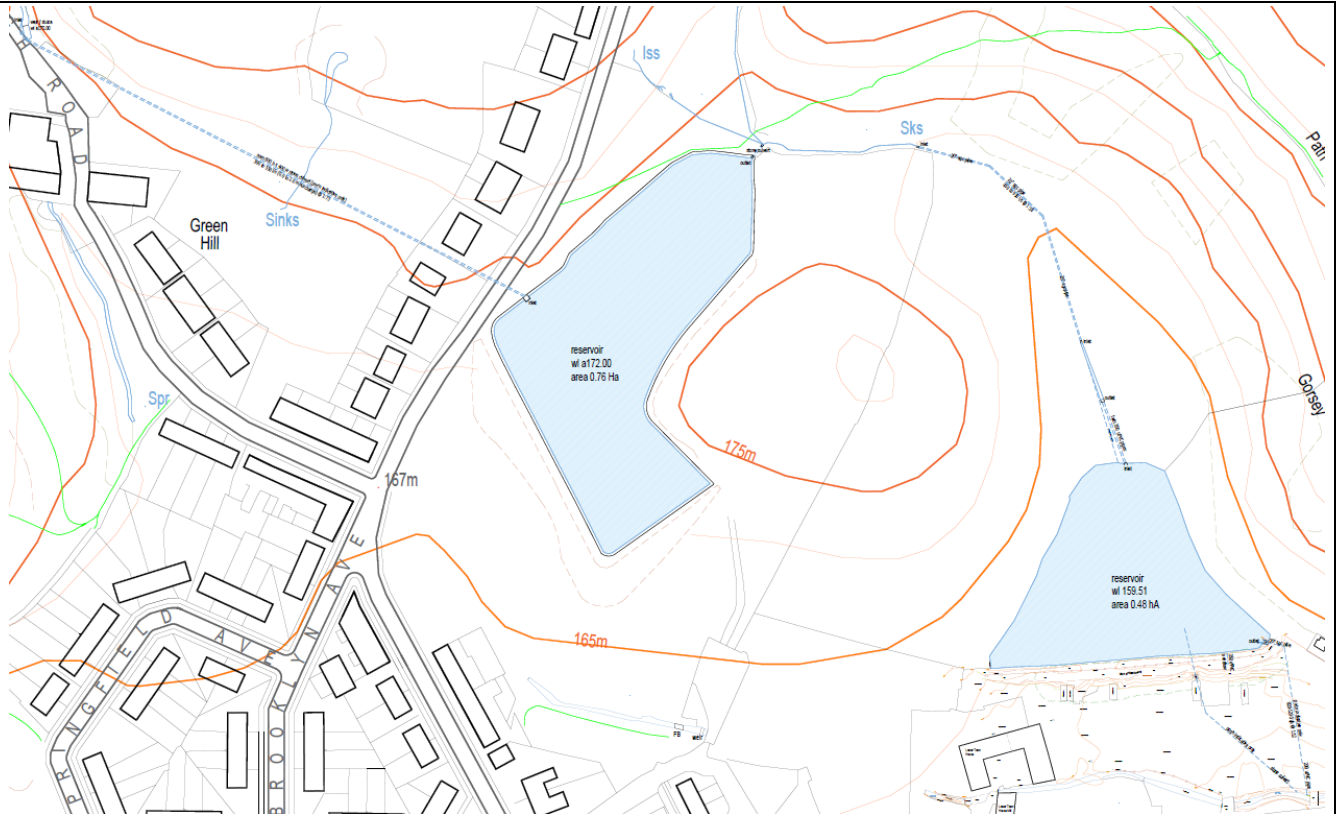
Townhouse Brook flows eastwards from on the south edge of the site and into the River Roch north of Littleborough.

Clough Brook flows from the northwest and becomes Townhouse Brook after passing beneath Calderbrook Rd.

The only catchment watercourse appears to be intermittent; it originates from the open fields to the north flowing southwest and then southeast to the lower reservoir.

Drainage System Plan.

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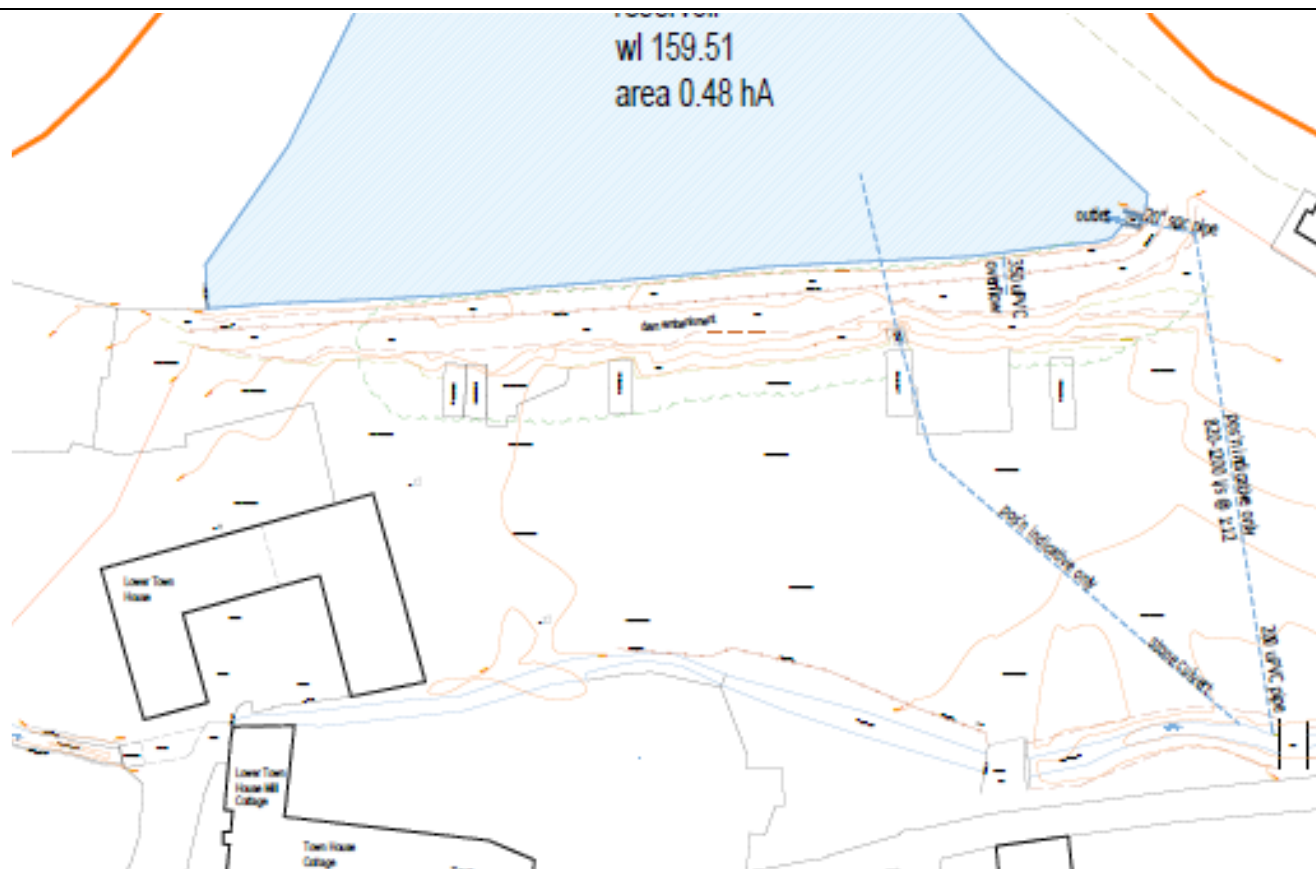
There are two reservoirs in tandem a larger upper and smaller lower reservoir. The upper reservoir is fed directly from an offtake to the west in Clough Brook conveyed via a stone culvert approx 220 m in length. The intake from Clough Brook is presently uncontrolled with a cill to the river bed approx 100 high. Thus during normal flows water is not abstracted, so overflow into the culvert occurs only during flood flows in the brook.

The upper reservoir overflows to the lower reservoir via a 225 m long part open channel and buried pipes which also receive inflow from agricultural grazing land to the north and runoff from Calderbrook rd to the north west.

Site plan.

Local Site Plan.

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The lower reservoir outlet drains directly to Townhouse Brook approx. 60 m south of the dam embankment via a 65 m long 500 mm diameter underground pipe.

The lower dam crest level varies from 160.3 to 160.0 m aOD, with a normal water level of 159.5 m aOD controlled by a cilled 500 dia outlet pipe. An 350 dia overflow pipe passes through the upper crest with an IL of 159.7 m aOD, providing 200 mm of freeboard before overflow starts.

Derivation of Data, Surveys and Models.

To obtain data for catchment and the local area flooding characteristics, a flood model of the reservoir catchment has been prepared to assess the level of flood risk from the reservoirs to the proposed development.

A flood model was developed comprising the following:

- Assessment of the rainfall & catchment characteristics up to the Application Site in order to estimate flow data for flood return period events.
- Aquisition of topographical data of the local area to inform relative ground levels at the Application Site in critical locations.
- Use the above return period data with the ground levels to assess the impact on flood levels locally.

Hydraulic Assessment.

Catchment runoff / reservoir inflows.

Catchment analysis for existing watercourse.

Assessment of extreme flood events for conveyance and flood storage requirements.

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Catchment data has been obtained through UK Centre for Hydrology (*see Appendix*).

The reservoir catchment is shown in the OS map above.

Characteristics are:

Catchment	Main	
Area	0.16	sq km
Main stream length	0.6	km.
Standard Average rainfall	1307	mm.
5 year 1 day rainfall	85.9	mm (incl 40% CC allowance).
Soil	sandy load from carboniferous rock / glacial till (low permeability).	
Urban fraction	approx 5% (Calderbrook Rd).	

The reservoir and catchment will be assessed for the Flood Flows to 1:500 yrs RP (0.2% AEP).

Flow rates for these events were estimated using the Q/QBAR factor from river/catchment characteristics & using the appropriate UK growth curve in the Flood Studies Supplementary Report. Clough & Townhouse Brooks fall in to hydrometric area 10 & the corresponding data is given in the tables below:

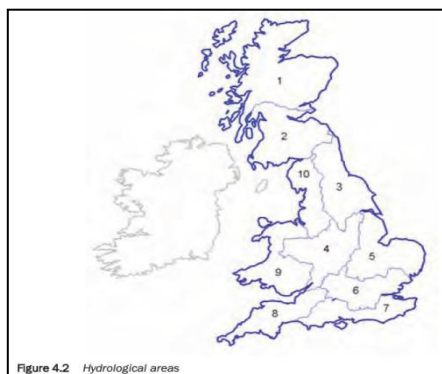


Figure 4.3 UK growth curves

Table 4.3 UK growth curve factors

Region	Hydrometric Area	Return period						
		2	5	10	25	50	100	500
NW	1	0.90	1.20	1.45	1.81	2.12	2.48	3.25
	2	0.91	1.11	1.42	1.81	2.17	2.63	3.45
	3	0.94	1.25	1.45	1.70	1.90	2.08	2.73
	9	0.93	1.21	1.42	1.71	1.94	2.18	2.86
SE	10	0.93	1.19	1.38	1.64	1.85	2.08	2.73
	4	0.89	1.23	1.49	1.87	2.20	2.57	3.62
	5	0.89	1.29	1.65	2.25	2.83	3.56	5.02
	6/7	0.88	1.28	1.62	2.14	2.62	3.19	4.49
Ireland	8	0.88	1.23	1.49	1.84	2.12	2.42	3.41
		0.95	1.20	1.37	1.60	1.77	1.96	2.40

The flows were analysed using the IH 124 applicable for catchments less than 25 sq km & the FSR methodology (*see Appendix*) in order to derive Qbar and Qmed.

An allowance of **40%** has been included in the base rainfall amounts to cover future **Climate Change** effects.

Referring to analyses in Appendix.

Using IH 124 method for small catchments, the output gave a range for **Qbar** from **0.23** (rural) to **0.26** cumecs (urban). Note the urban contribution equates to a 13% increase in flood flow.

Using revised UKCEH-FEH update, **Qmedr** and **Qmedu** are computed; **1.94** l/s and **2.25** l/s respectively.

A value for **Qbar** of **0.26** cumecs is selected for assessment.

Applying the Regional Growth Curves the **design flood flows** for differing **return periods** are tabulated below:

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ref FSR, S Report 14, fig 2

	QBAR	0.26	cumecs	
	return period	Q/QBAR	Qtp	
%AEP	TP	region 10		
	years		cumecs	l/sec
4000.0	0.025	0.01	0.00	3
2000.0	0.05	0.05	0.01	13
800.0	0.125	0.15	0.04	39
400.0	0.25	0.30	0.08	78
200.0	0.5	0.48	0.12	125
100.0	1	0.68	0.18	177
50.0	2	0.90	0.23	234
20.0	5	1.20	0.31	312
10.0	10	1.40	0.36	364
4.0	25	1.64	0.43	426
3.3	30	1.69	0.44	439
2.0	50	1.85	0.48	481
1.0	100	2.10	0.55	546
0.2	500	2.75	0.72	715

Clough Brook upper Reservoir Inflows.

The intake to the culvert feeding the upper reservoir in Clough Brook is approximately 174.5 m aOD in accord with OS maps flowing to the reservoir at a level of 172.0 m aOD.

The intake has no control other than an inlet cill approx 100 mm high off the river bed.

Construction	stone culvert nom 600 high x 400 wide.		
Base width	0.4 m.		
Area	rectangular.		
Wetted perimeter	rectangular.		
Upstream IL	174.5 m aOD	OS maps.	
Downstream IL	172.0 m aOD	OS maps.	
Length	220 m.		
Gradient	2.5 m / 220.0 m =	0.011	1:88.

Using an equivalent circular section:

$$\text{Dia} = \sqrt{(4 \times 0.6 \times 0.4 / \pi)} = 0.50 \text{ m.}$$

Inputting the above into a pipe flow calculator gives the following flow rates:

Presuming a pipe in fair condition (conservative) Hazen C = 100, Mannings N = 0.011:

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partial depth flow in circular pipes.

design flow rate	Q0	402 l/s
		0.40 cumecs
pipe diameter	D	500 mm
		0.50 m
flow depth	d	353 mm
		0.35 m
proportional depth	d/D	0.71
angle	θ	131.6 degs
flow area	A	0.15 sq m
wetted perimeter	P	1.00 m
pipe gradient	1 in	88
	s	0.01136
mannings roughness	n	0.011 concrete pipe
hydraulic radius	R	0.15 m
velocity	v	2.72 m/s
initial flow rate	Qi	0.402 cumecs
pipe length	L	220.0 m
head loss	dH	2.50 m

Pipes Flowing Full, Hazen Williams

Assumes the inlet is surcharged and full pipe flow is occurring.
 The calculation iterates the outlet loss to optimise the discharge rate.

roughness coefficient	condition	clean	average	poor
	C	150	100	70
pipe dia	D	mm	500	
roughness coefficient	C		100	
length	L	m	220.0	
inlet depth		m	2.00	
outlet depth		m	0.35	
discharge pipe inlet IL		m aD	174.500	
discharge pipe outlet IL		m aD	172.000	
pipe gradient		1 in	88.0	0.011
friction head loss	hf	m	4.15	
hydraulic gradient	i= hf/L		0.019	1 in 53
pipe inlet loss coeff	kl		0.44 sharp entry	
pipe inlet loss	hl	m	0.00	0.00
discharge rate	Q	l/s	528.1	

Partial depth flow approx 20 l/s @ 70 mm inlet depth.
 Partial depth flow approx 100 l/s @ 150 mm inlet depth.
 Pipe full flow approx 420 l/s @ 500 mm inlet depth.
 Surge flow approx 450 l/s @ 1000 mm inlet depth.
 approx 530 l/s @ 2000 mm inlet depth.

Combined Assessment Flows.

The above section presents the following flow regime for the reservoirs (incl 40% CC allowance for rainfall):

		Catchment	Clough brook	Total	Note
Flow Event	Q	cumecs	cumecs	cumecs	
Low Flow 95% ile	Q ₍₉₉₎	Not used		-	
Mean annual daily flow	Q _(mean)	0.002	0.00	0.02	Assume 0.17 m flow depth @ clough brook
100% AEP (1 yr)	Q1	0.18	0.01	0.19	Assume 0.25 m flow depth @ clough brook
50% AEP (2 yr)	Q2	0.23	0.42	0.45	Assume 0.5 m flow depth @ clough brook
20% AEP (5 yr)	Q5	0.31		-	
3.33% AEP (30 yr)	Q30	0.44		-	
2% AEP (50 yr)	Q50	0.48	0.45	0.93	Assume 1.0 m flow depth @ clough brook
1% AEP (100 yr)	Q100	0.55		-	
0.2% AEP (500 yr)	Q500	0.72	0.53	1.25	Assume 2.0 m flow depth @ clough brook

The inputs to the upper reservoir from Clough Brook have been proportioned over the flood return periods to represent the likely depth at the inlet.

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Reservoir Outlet Flow Capacity.

The Lower reservoir is served by a single 500 dia outlet pipe nom 65.5 m long with a gradient of approx 1:13. Should this outlet be overwhelmed a 350 dia overflow pipe is present approx 200 mm higher. The dam crest level is 160.00 apprx 0.5 m above the outlet cill level.

Presuming the inlet is full to overflow level (ie full depth surcharge) and the outlet into Townhouse Brook is surcharged by flood flow. Referring to EA modelling data for Townhouse Brook (*see Appendix*).

Reservoir outlet Pipe Flowing Full, Hazen Williams					
<i>Assumes the inlet is surcharged and full pipe flow is occurring.</i>					
<i>The calculation iterates the outlet loss to optimise the discharge rate.</i>					
roughness coefficient	condition	clean	average	poor	
	C	150	100	70	
pipe dia		D	mm	500	
roughness coefficient		C		100	clean water, SGC pipe
length		L	m	64.0	
inlet depth			m	1.05	resvr at overflow lvl
outlet depth			m	1.50	townhse br Q500 lvl
discharge pipe inlet IL			m aD	159.00	
discharge pipe outlet IL			m aD	154.00	
pipe gradient			1 in	12.8	0.078
friction head loss		hf	m	4.43	
hydraulic gradient		i= hf/L		0.069	1 in 14
pipe inlet loss coeff		kl		0.44	sharp entry
pipe inlet loss		hl	m	0.12	0.122
discharge rate		Q	l/s	1065.4	

The main outlet pipe capacity is assessed at 1065 l/s when flowing full from a surcharged reservoir into a flooded Townhouse Brook.

Reservoir overflow Pipe Flowing Full, Hazen Williams					
<i>Assumes the inlet is surcharged and full pipe flow is occurring.</i>					
<i>The calculation iterates the outlet loss to optimise the discharge rate.</i>					
roughness coefficient	condition	clean	average	poor	
	C	150	100	70	
pipe dia		D	mm	350	
roughness coefficient		C		100	clean water, SGC pipe
length		L	m	5.0	
inlet depth			m	0.35	resvr at overflow lvl
outlet depth			m	0.00	unrestricted discharge
discharge pipe inlet IL			m aD	159.70	
discharge pipe outlet IL			m aD	159.60	
pipe gradient			1 in	50.0	0.020
friction head loss		hf	m	0.35	
hydraulic gradient		i= hf/L		0.070	1 in 14
pipe inlet loss coeff		kl		0.44	sharp entry
pipe inlet loss		hl	m	0.10	0.098
discharge rate		Q	l/s	420.9	

The overflow pipe capacity is assessed at 420 l/s when flowing full from a surcharged reservoir onto the downstream side of the embankment.

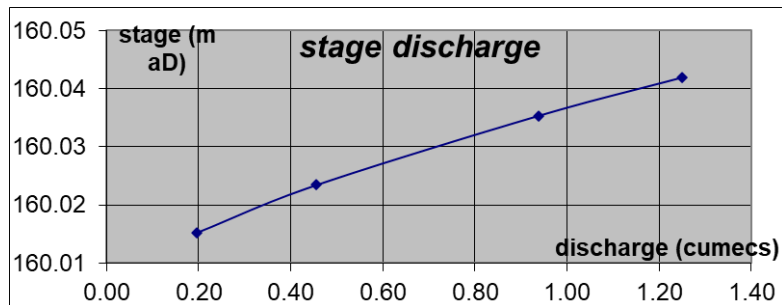
Total discharge capacity from the flooded lower reservoir is 1065 l/s via the main outlet + 420 l/s from the overflow giving **1485 l/s**.

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Reservoir Crest Capacity.

The Lower reservoir has a nominally 100 m long crest by approx 2.0 m wide at a level of 160 m aOD with a sloping embankment on the downstream side.

Analysis of the depth of flow over the crest in the situation where the outlet and overflow are blocked has been undertaken. (Refer to Appendix).



The above stage discharge chart shows the water depth against flow.

At the 0.2% AEP (1 in 500 year) flow event (1.25 cumeec) the water depth on the crest is 40 mm.

Summary.

The design storm flows into the lower reservoir originate from the local catchment plus the inflow via the upper reservoir from Clough Brook, the mean annual flood amount is 0.45 cumeecs (450 l/s) and the 0.2% AEP (500 year) event 1.25 cumeecs (1250 l/s).

The combined main outlet and overflow will convey 1.45 cumeecs which is 16% greater than the 0.2% AEP flood flow.

Should the outlet and overflow become blocked during an extreme flood event the reservoir crest depth would be less than 50 mm of surface flow.

Flood Risk Assessments.

Existing Site Surface Water System.

The lower reservoir is operating as a private fishery and provided with a dedicated outlet and overflow pipes. The outlet is protected with a debris screen which is regularly maintained and the overflow is unimpeded.

The outlet from the overflow was obscured by vegetation so will need to be validated as serviceable as part of this FRA.

Proposed Site Surface Water System.

This assessment is in support of a planning application for the change of use of an agricultural barn to a dwelling.

The development will not change the runoff or any physical aspect of the reservoir.

Fluvial Morphological Impacts.

The proposed development will not impact fluvial morphology.

Potential Sources of Flooding.

The location has two potential sources of flooding, Townhouse Brook classed as a main river passing to the south side and the upper & lower reservoirs to the north.

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Flooding related to Townhouse Brook has been addressed in a separate FRA; ref *rep-tjba-lwr townhouse farm-FRA- rev1*.

Flooding from the reservoirs is addressed in this FRA.

Artificial Watercourse/Canals.

The Rochdale Canal is approx. 900m distant from the Application Site.

Loss of overland flow.

Overland flow routes will not be affected as part of the development.

Loss of Floodplain.

Loss of floodplain will not be decreased as part of the development.

Springs and high groundwater levels.

Ground water levels are controlled primarily by river levels in the local area and therefore present no more of a risk.

Surcharging of local drainage & sewerage systems.

The reservoirs are not connected to local underground drainage or sewerage systems.

Urban areas have been incorporated into the fluvial flood model using the higher and more conservative urban QBAR runoff figure as the basis for model data, and thus the model is considered conservative including runoff from urban areas.

Reservoirs.

Flooding from the reservoirs is addressed in this FRA.

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Conclusion & Residual risks.

The lower reservoir was constructed using an earth dam across a minor watercourse at the truncation of the valley at the main valley of Townhouse Brook.

This assessment is specifically looking at the hydromorphological aspects of the reservoir and catchment and does not look at the structural aspects of the embankment.

The water management of the reservoir appeared to be in place with regular inspection (both by the Fishery and the Landowner) to ensure that adequate drainage is maintained.

The consequence of partial blockage of the main outlet would lead to a 200 mm rise in water level and operation of the overflow.

Should both outlets become blocked during an extreme flood event the flow over the length of the reservoir crest would be in the order of 40 mm deep.

The conclusions of this Flood Risk Assessment is that the present lower reservoir does not present a flood risk to the proposed development.

Advice going forwards:

- The outlet of the overflow pipe was obscured during the inspection, this should be cleared and a robust pathway from it to Townhouse Brook provided to prevent risk of scour to the embankment.
- To improved reliability of flood water discharge installation of a simple broad notch overflow into the crest flowing into an armoured channel down the embankment face to Townhouse Brook would be an improvement.
- It was noted that the former scour / draw-off valve was not serviceable, so it should be noted that should the reservoir need to be drained for repairs / maintenance a temporary pumped siphon arrangement would have to be installed.

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Appendix.

Rainfall Analysis & Catchment Data.

Refer to CEH catchment descriptors below.

Ref 1	A guide to the flood studies report, Institute of Hydrology, No 49					
catchment area	AREA	0.16	sqkm		1:50000 maps	
main stream length	MSL	0.60	km		1:25000 maps	
stream level aD @ 85% length	H85%	180	m	ref 1	1:25000 maps	
stream level aD @ 10% length	H10%	160	m		1:25000 maps	
channel slope	S1085	44.4	m/km			
stream junctions	N	1.0	no		1:25000 maps	
stream frequency	STMFRQ	6.41		ref 1	N/AREA	
total catchment stream length	NETLEN	0.50	km	1:50000 OS maps		
base flow index	BFIHOST	0.65		ref 1 FEH Web Service		
standard average annual rainfall (1916-50)	SAAR9120	1307	mm	ref 1 FEH Web Service		
M5 1day rainfall		61.38	mm	FEH Web Service		
climatechange adjustment	CC	40	%			
M5 1day rainfall incl CC adj		85.9	mm			
ratio of M5 60min to M5 2day	r	0.25		see tab		
areal reduction factor	ARF	0.979		ref 1, fig 6.58		
mean soil moisture deficit	SMDBAR	19	mm	fig 4.9 FSR1, chap4		
residual soil moisture deficit	RSMD	65.1	mm	M5 1day x ARF - SMDBAR (ref 1, app A		
catchment wetness index	CWI	124.2		ref 1, fig 6.62, see tab CWI		
catchment soil index	SOIL	0.45		obtain from WINDES IH124 module, but z		
catchment lake area		0.012	sqkm	lakes directly on line 1:25000 maps		
lake subcatchment area		0.15	sqkm	1:25000 maps		
flood attenuation by reservoirs & lakes	FARL	0.73				
proportional lake area	lake	0.08		1:25000 maps		
area of urban catchment	URBAN	0.01	sqkm	1:50000 OS maps		
area of suburban catchment	SUBURBAN	0.00	sqkm	1:50000 OS maps		
urban development fraction of AREA	URBANEXT	0.06		urban / area +0.5xsuburban/area		
	URBT	1.06		1+URBAN		
hydrometric region		10		FSR fig 4.15, CIRIA 168		
regional constant	C	0.0213		FSR fig 4.15, CIRIA 168		

Climate Change Impacts.

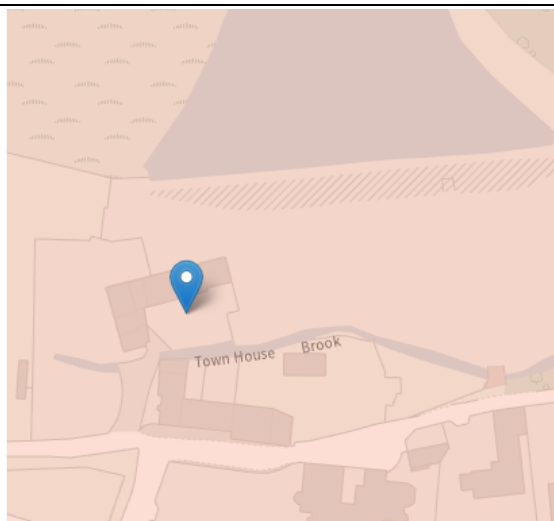
Climate change impacts on fluvial flooding are measured on the potential increase through the expected development lifetime. With a longer development lifetime, the assessment must consider a greater chance of river level increase.

Residential developments are considered to have the longest potential development lifetimes. Commercial developments have shorter lifetimes owing to higher turnover of changes in use types etc, and therefore a higher frequency of associated redesign & updated flood risk assessment.

The proposed development is residential, and as identified in Section 1, the vulnerability of the development is 'more vulnerable'. The Planning Flood Maps designate the Site as being within flood zone 2.

EA guidance specifies climate change allowances by management catchment specific areas. Peak flow river maps have been produced which specify the climate change allowances to be used in a flood risk assessment:

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Irwell Management Catchment peak river flow allowances

	Central	Higher
2020s	12%	15%
2050s	19%	26%
2080s	35%	46%

EA guidance indicates the peak flow rates that should be used for the vulnerability classification:

- essential infrastructure – use the higher central allowance.
- highly vulnerable – use central allowance (development should not be permitted in flood zone 3a).
- more vulnerable – use the central allowance.
- less vulnerable – use the central allowance.
- water compatible – use the central allowance.

Given the more vulnerable development risk category, the central allowance climate change multiplier should be used as the basis for assessing the potential impact of climate change over the development lifetime.

As the Application Site is to be used for a residential end use, maximum development lifetimes apply, and the '2080s' timeline is appropriate. Comparing the '2080s' timeline with the central allowance from the peak river flow predictions above, suggests a 35% climate change allowance for the Site.

Mean Annual Flood (Qbar) & Median Flow (Qmed).

Flood Studies Report, rural (Large and small catchments)

The mean annual flood is determined using the regional constant, catchment area, stream frequency, soil index, soil moisture deficit, stream gradient (m/km) and proportional lake area.

Flood Studies Report, rural (<25km²)

The mean annual flood is calculated in a very similar manner to the IH124 equation, though the soil index variable is instead changed to a SPR (Standard Percentage Runoff) variable.

Flood Estimation Handbook

The FEH calculates the median annual flood instead of the mean. This means that a conversion is factored into the equation after. The Qmed is calculated using the catchment area, standard annual average rainfall (mm), BFIHOST which is the base flow index derived from the hydrology of soil types, and FARL. FARL is a reservoir reduction aspect, set at 1.0 and can effectively be ignored. This means that areas with waterbodies that reduce runoff will cause the greenfield runoff rate to be overestimated. The Qmed then undergoes a conversion factor to produce the Qbar.

Methodology Conclusion

The most appropriate methodology for this catchment appears to be the IH124 calculation. This is primarily because of its middle ground between the results given by the FEH and the FSR rural for <25Km². The 'FSR rural for large and small catchments' is omitted due to its tendency to overestimate. This is shown in IH124-Flood Studies Supplementary Reports.

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computation of median flow (QMED)

QMEDr & QMEDu ungauged catchments

ref UKCEH-FEH statistical update-issue 1.4-oct 2025

$QMEDr = 6.8274 \times AREA^{0.8499} \times 0.178^{(1000/SAAR9120)} \times FARL^{3.045} \times 0.0321 \times BFIHOST^{0.2}$

$QMEDu = QMEDr \times (1 + 0.3 \times URBEXT)^{1.8838} \times (1 + 0.3 \times URBEXT \times ((70 / (67.0674 - 63.82 \times BFIHOST)) - 1))$

catchment area	AREA	0.156	sqkm	
lakes / reservoirs	FARL	0.73		
	BFIHOST19	0.65		
annual aver rainfall	SAAR9120	1307	mm	
area of urban catchment	URBANEXT	0.1	sqkm	
	QMEDr	0.00	cumecs	1.94 l/s
	QMEDu	0.00	cumecs	2.25 l/s

computation of mean annual flood (QBAR)

QBAR rural (large & small catchment areas)

using 6 element regression eqn from FSR 1

$QBARr = C \times AREA^{0.94} \times STMFRQ^{0.27} \times SOIL^{1.23} \times RSMD^{1.03} \times S1085^{0.16} \times (1 + LAKE)^{-0.85}$

catchment area	area	0.156	sqkm	
stream frequency	stmfrq	6.41		
soil type	soil	0.45		
soil moisture deficit	rsmd	65.1	mm	
stream gradient	s1085	44.4	m/km	
proportional lake area	lake	0.08		
regional constant	C	0.0213		
	QBARr	0.29	cumecs	

QBAR rural (small catchments less than 25 sqkm)

using 3 element regression eqn (IH 124)

$QBARr = 0.00108 \times AREA^{0.69} \times SAAR^{1.17} \times SOIL^{2.17}$

		79.5		
catchment area	area	0.156	sqkm	
annual av rainfall	saar	1307		
soil type	soil	0.45		
	QBARr	0.23	cumecs	

QBAR urban (eqn 7.4 IH124)

$QBARu = QBARr \times URB^{2NC} [1 + URBAN((21/CIND) - 0.3)]$

$CIND = 102.4 \times SOIL + 0.28 \times (CWI - 125)$

$NC = 0.92 - 0.00024 \times SAAR$ 500 < SAAR < 1100

$NC = 0.74 - 0.000082 \times SAAR$ 1100 < SAAR < 3000

QBARr		0.29	0.23	
soil type	soil	0.45		
catchment wetness	cwi	124.2		
urban proportion	urban	0.06		
1+urban proportion	urbt	1.06		
annual av rainfall	saar	1307		
	nc	0.63		
	cind	45.856		
	$URB^{2NC} [1 + URBAN((21/CIND) - 0.3)]$	1.093		
	QBARu	0.32	0.26	

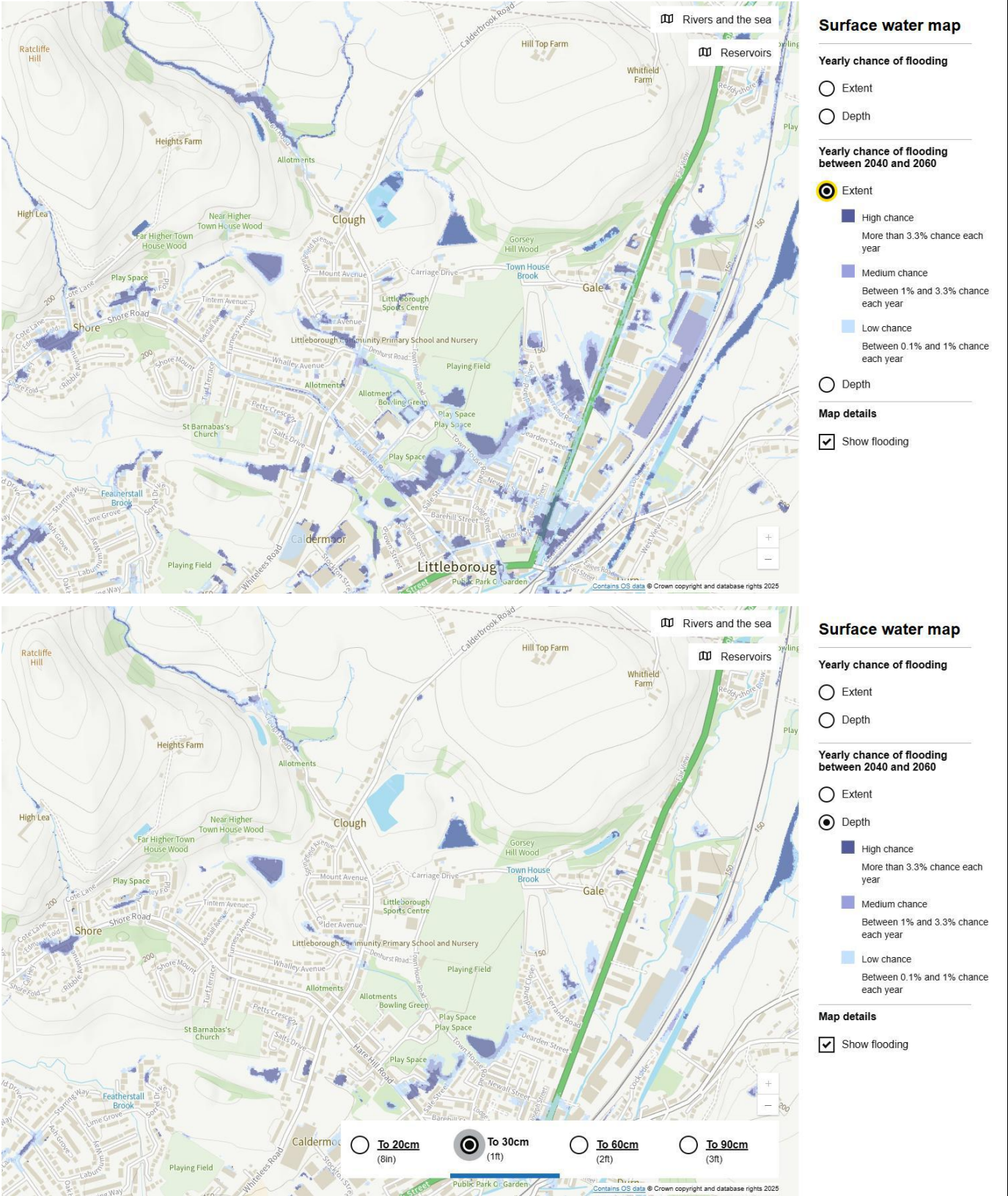
WHS Flow Analysis (Qube Report).

Cube report not used, due to small catchment size and extra catchment input from Clough Brook.

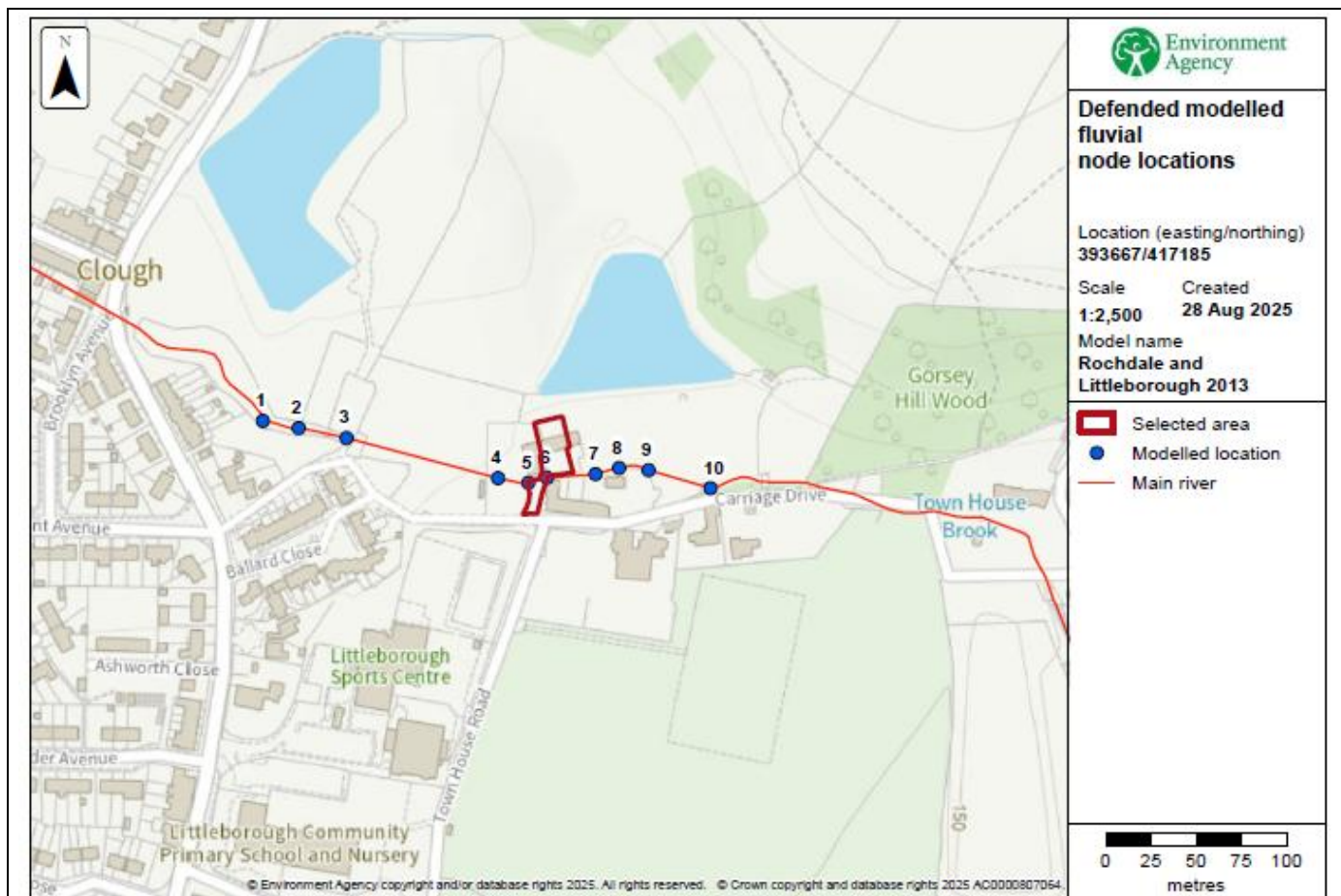
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Environment Agency Flood Models.

Environment Agency Technical Surface Water Climate Change Flood Risk Extent & Depth Maps.



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Defended modelled
fluvial
node locations

Location (easting/northing)
393667/417185

Scale Created
1:2,500 28 Aug 2025

Model name
Rochdale and
Littleborough 2013

-  Selected area
 Modelled location
 Main river



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Modelled node locations data

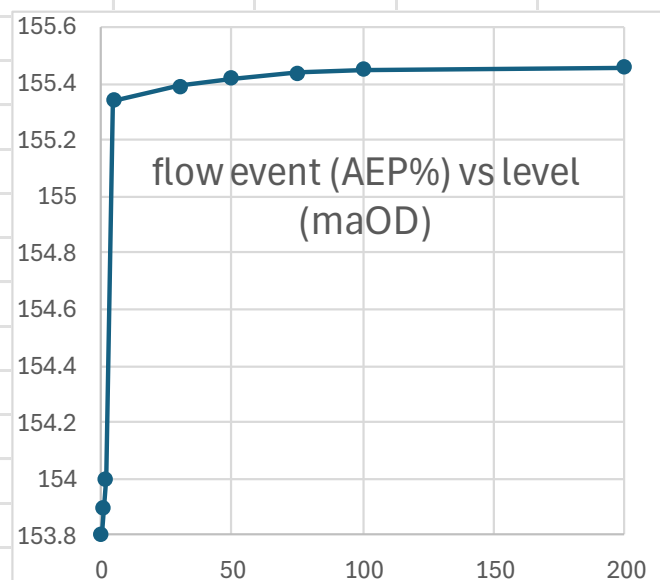
Defended

Label	Modelled location ID	Easting	Northing	5% AEP	3.33% AEP	1.33% AEP	1% AEP	0.5% AEP	0.1% AEP
				Level	Level	Level	Level	Level	Level
1	1674170	393509	417205	162.57	162.60	162.66	162.67	162.68	162.73
2	1674169	393528	417201	162.18	162.21	162.28	162.29	162.31	162.38
3	1674254	393555	417196	161.46	161.49	161.55	161.56	161.58	161.63
4	1674654	393639	417174	158.02	158.10	158.24	158.26	158.30	158.42
5	1674181	393655	417171	157.85	157.95	158.11	158.14	158.19	158.39
6	1673796	393666	417174	157.05	157.07	157.13	157.14	157.15	157.19
7	1674435	393693	417176	156.68	156.73	156.77	156.77	156.77	156.78
8	1674089	393706	417179	156.69	156.74	156.79	156.79	156.80	156.81
9	1674732	393722	417178	156.70	156.76	156.82	156.83	156.84	156.86
10	1673811	393756	417168	155.34	155.39	155.44	155.45	155.46	155.50

Data in this table comes from the Rochdale and Littleborough 2013 model.
 Level values are shown in mAOD, and flow values are shown in cubic metres per second.
 Any blank cells show where a particular scenario has not been modelled for this location.

EA Flood Model, Townhouse Brook (Node 10)

Flow Event	Q	Level (maOD)
Mean annual daily flow	Q _(mean)	
Low Flow 95% ile	0	153.8
100% AEP (1 yr)	1	153.90
50% AEP (2 yr)	2	154.00
5% AEP (20 yr)	5	155.34
3.3% AEP (30 yr)	30	155.39
2% AEP (50 yr)	50	155.42
1.3% AEP (75 yr)	75	155.44
1% AEP (100 yr)	100	155.45
0.5% AEP (200 yr)	200	155.46
0.2% AEP (500 yr)	500	155.52
0.1% AEP (1000 yr)	1000	155.50



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Reservoir Crest Flows.

return period		years	500	50	2	1
flow rate		cumecs	1.25	0.93	0.45	0.19
initial u/s head above crest (h)		m	0.04	0.04	0.02	0.02
breadth of crest (B)		m	100.00	100.00	100.00	100.00
width of crest (L)		m	2.00	2.00	2.00	2.00
u/s depth below crest (p)		m	3.00	3.00	3.00	3.00
crest level		m aD	160.00	160.00	160.00	160.00
water level on crest		m aD	160.04	160.04	160.02	160.02
Cd			0.86	0.83	0.74	0.61
design flow rate (Q)	soln 1	cumecs	1.25	0.94	0.46	0.20
u/s velocity (v)		m/s	0.00	0.00	0.00	0.00
vel head $v^2/2g$		m	0.00	0.00	0.00	0.00
total head (H)		m	0.04	0.04	0.02	0.02
check $L/1.75 < H$			1.14	1.14	1.14	1.14

