1995/045

Institute of Hydrology

Natural Environment Research Council

Natural
Environment
Research
Council

DESIGN OF A FLOW MEASUREMENT STRUCTURE FOR THE SNOWDONIA ENVIRONMENTAL CHANGE NETWORK SITE

Prepared by: J.A. Hudson[1] & A. Warwick[2]

[1] Institute of Hydrology (Plynlimon), Staylittle, Llanbrynmair, Powys, SY19 7DB.
[2] Institute of Hydrology, Maclean Building, Crowmarsh Gifford, Wallingford, Oxon, OX10 8BB.

March 1995

Introduction

The ECN site currently being established by CCW on Snowdonia requires flow to be measured from a target catchment, the Teyrn. The technique chosen must fulfil the following criteria:

(1) The method must incorporate a precision engineered channel that is of a shape that will ensure sensitivity, precision and accuracy in the stage-discharge relationship throughout the flow range. The structure should preferably be one which has a British Standard or laboratory derived calibration.

(2) It must be large enough to cope with flows likely to occur during the length of the study, which is expected to be at least 30 years. It must certainly be constructed in such a way and of sufficiently durable materials as to be capable of withstanding the destructive force of extreme flows. It will preferably have a laboratory calibration that covers the range of flows from the 50 year return period drought to the 50 year flood. The ability to deal with low flows is arguably less critical, as alternative methods of discharge measurement e.g. volumetric, are available in the severest drought conditions.

(3) Because of its location within a National Park, the structure itself must be either visually unobtrusive or aesthetically pleasing, or at least be capable of disguise if neither of these is possible. Generally, thin plate weirs are easier to disguise than structures with a large plan area such as flumes or broad crested weirs, and this could possibly by achieved by using loose boulders piled against the length of the cut-off wall away from the crest.

(4) The stilling pool upstream of the weir will also be visually obvious, and will form a barrier to access at times, so its depth and areal extent needs to be minimised. This can only be achieved by minimising the height of the structure i.e by choosing a structure that has a low vertical intrusion from the bed and a minimal afflux (water level rise per unit increase in flow).

Trapezoidal flumes afford the smallest stilling pool for any given flow because they have side, not vertical, contractions and generally appear as a regularised version of the shape of the original channel. However they are difficult and expensive to build. Broad-crested weirs such as the Crump or flat-V variants of the triangular profile weir have a simpler geometry and a more reliable calibration, but the vertical intrusion of the weir block tends to increase the average depth and extent of the weir pool compared to a flume.

Under normal circumstances a thin plate weir would not be recommended in such an area simply on the grounds of sediment build up and lack of easy access to clear this when it occurs. However, in the Teyrn, sediment is not a problem as the site is just downstream of a lake. A thin plate weir therefore remains a possibility, although the downside is that the stilling pool will be more extensive than the alternatives, particularly at higher flows.

The choice for CCW is to accept the visual impact of the large, deep stilling pool associated with thin plate weirs, in the interests of keeping costs down, or to ensure less visual intrusion of the stilling pool (and generally better aesthetic appearance of the weir block itself) at the higher cost and with the constructional difficulties associated with flumes or broad-crested weirs.

Flow ranges for the structure

Before being able to predict the type and size of structure it is necessary to forecast the likely range of flows with which the structure will have to cope. There is no gauging station nearby that drains a catchment of similar characteristics or with a sufficiently long record, that can be used to estimate flow statistics in the Teyrn. It is necessary therefore to use Flood Studies Report techniques (NERC, 1975) to estimate the frequency of flows likely to be encountered during the study. As mentioned, low flows will take care of themselves provided some form of compound structure is used in order to ensure sensitivity at the low to moderate end of the flow range.

The main FSR method for use in ungauged catchments is based on flood relationships with catchment characteristics, as follows:

$$MAF = k \cdot AREA^{0.94} \cdot STRMFRQ^{0.27} \cdot S1085^{0.16} \cdot SOIL^{1.23} \cdot RSMD^{1.03} \cdot (1+LAKE)^{-0.85} \qquad (1)$$

where:
MAF is the Mean Annual Flood ($m^3.sec^{-1}$), equivalent to a return period of 2.33 years.
k is a regional constant (0.0213 for Wales)
AREA is the catchment area (km^2)
STRMFRQ is the drainage density (junctions. km^{-2})
S1085 is a measure of channel slope from 10% to 85% of the channel length upstream from the gauging station (m. km^{-1})
SOIL is an areally-weighted, dimensionless index of the winter rain acceptance potential (infiltration capacity) of the soils in the catchment
RSMD is the effective daily rainfall of 5-year return period after average winter soil moisture deficit has been satisfied (mm)
LAKE is the proportion of the catchment covered by open water.

The catchment characteristics used are shown in Table 1. Strictly, this method works better for larger catchments of area greater than 0.2 km^2, and the Teyrn falls just below this figure at 0.1898 km^2. The Teyrn is also unusual in having a non-existent drainage pattern, at least on the 1:25000 map. Instead there is internal slope drainage focusing to the lake, with the main channel starting at the lake outlet. Drainage density and channel slope are both important sources of variance in the regression model proposed by FSR, and cannot be lightly ignored. Recourse has been taken therefore, as recommended in the FSR, to assessing the characteristics of the third order basin surrounding the Teyrn and using these figures in the equation.

Table 1. Values used in the FSR equation to estimate the Mean Annual Flood

Characteristic	From OS Maps	From FSR	From Statistics
AREA (km^2)	0.1898		
STRMFRQ (juncts. km^{-2})	None in Teyrn catchment 7.36 (larger catchment)		
S1085 (m. km^{-1})	104.96 (larger catchment)		
SOIL		0.5	
LAKE	.100105		
SMD (mm)		5	
2D-M5 rainfall (mm)		150-200 (map) 177 (table)	
1D-M5 rainfall (mm)		115-154 (77% of above) 136 (77% of above)	131.236

The most significant variable in the FSR method is the daily rainfall with a 5 year return period (1D-M5). The FSR allows only the 2D-M5 to be interpolated from maps and lookup tables, so the 1D-M5 is then taken as 77% of this value. It is difficult to interpolate the value of 2D-M5 between the 150 and 200mm contours on the FSR map, particularly in an area of such spatial and altitudinal variation. A better method is to use local rainfall records where these are available. A meteorological station (Llyn Llydaw) was run just outside the Teyrn catchment from 1966 to 1977, but unfortunately the rainfall was collected in weekly rather than daily blocks. A rather crude method of interpolating the daily figures has been adopted whereby the weekly totals from the Llyn Llydaw gauge have been divided by the weekly totals for the same periods for the nearest daily gauge - at Cwm Pennant, (NGR SH534 485), and then the ratio for individual periods is multiplied by the daily Pennant value to give a scaled value of daily rainfall at Llyn Llydaw. There will obviously be some uncertainty over the values obtained, particularly for the extreme daily rainfalls that are most pertinent to this investigation. However there is no alternative with the restricted data available, and it is always possible to over- rather than under-design a structure to allow for problems of this kind.

Statistical analysis of the results (table 2) gives a good indication of the likely return periods of rainfall up to 10 years, as then shown in table 3 as a function of the quartile distribution. The statistic used is the probability of non-exceedance in any one year, F(x), approximated by the series:

$$F(x) = ((M-1)+0.69)/(N+0.38),......,((N-1)+0.69)/(N+0.38) \qquad (2)$$

where: M is the ascending ranking of maximum annual rainfalls during the period.
N is the number of years in the series.

and the reduced variate (abcissa), y, is calculated thus:

$$y = -\log\log(1/F(x)) \qquad (3)$$

Table 2. Analysis of annual daily rainfall maxima.

Year	Annual Rainfall (mm)	Max. daily rainfall (mm)	Rank (ascending order)	F(x)	y (reduced variate)
1967	2904*	97.6	4	0.3243	0.3106
1968	2223*	103.9	7	0.5879	0.6370
1969	2515	141.7	10	0.8515	1.1560
1970	3647	158.1	11	0.9394	1.5662
1971	2385	101.3	5	0.4121	0.4145
1972	2727	83.0	2	0.1485	0.0818
1973	2532	82.3	1	0.0606	-0.0855
1974	3550	141.6	9	0.7636	0.9313
1975	2653	84.2	3	0.2364	0.2032
1976	2443	124.8	8	0.6757	0.7689
1977	3076	103.4	6	0.5000	0.5214
Mean	**2836***				

** missing daily data- not included in mean*

Table 3. Return periods of daily rainfall related to quartile statistics

Quartile Statistic	Mean Value (mm)	Return period
QM1	83.1	2M (twice yearly)
(QM1+QM2)/2	90.9	
QM2	98.8	1M
(QM2+QM3)/2	106.8	M2 (once in two years)
QM3	114.8	
(QM3+QM4)/2	131.2	M5
QM4	147.6	M10

In spite of the uncertainties in the analysis, the 1D-M5 value estimated in this way, 131.2mm, is reassuringly close to the crude analysis from the FSR of 136mm for areas experiencing annual average rainfall (SAAR) between 2800 and 4000mm.

Flood Estimates

Substituting this value into the FSR equation (1) gives a mean annual flood (MAF) and floods of related return periods as shown in table 4.

Table 4. Estimated magnitude of floods of various return periods

Return period	Factor	Flow ($m^3.sec^{-1}$)
2.33 years (mean annual flood)	1.0	0.9251
5 years	1.22	1.1286
10 years	1.43	1.3229
50 years	1.94	1.7946

Using this information, it is now possible to design a structure to cope with the 50 year flood. In practice it may be better to allow for c. 2.0 m^3. sec^{-1}, or a close figure which allows the dimensions of the structure to fit in well with standard sized materials being used for the building works.

Ideal Structure Design

Assuming in the first instance that cost and ease of installation in the awkward environment afforded by the Teyrn catchment are of paramount importance; the first choice of structure would be a compound thin plate weir i.e. a rectangular notch superimposed on a V-notch, as used by IH in numerous catchments in Wales. Such structures perform well at low to moderate flows, but their simple laboratory calibrations cannot be used when both crests are in operation because of hydraulic interaction. The Barnes (1916) equation can be used, however insufficient is known about the limitations of use of this equation to recommend its use without independent calibrations from current metering or dilution gauging being done after construction.

The main problem with thin plate weirs is their relatively high crest height, high afflux and the resultant extensive stilling pool development. Design of a thin-plate structure for the Teyrn will now form the centrepoint of this exercise, but CCW will have to assess the likely visual impact of the works and decide whether a more expensive option would be more environmentally acceptable. Alternatives can be designed in this event.

The Barnes equation is as follows:

$$Q = a_v C_e (H_1^{5/2} - H_2^{5/2}) + a_r (b_r - b_v) Cd_r h_2^{3/2} \qquad (4)$$

where:

$a_v = (8/15)*(2g)^{0.5}$
$a_r = (2/3)*(2g)^{0.5}$
$Cd_r = 0.616*\ (1 - 0.1*h_2 / (b_r - b_v))$
$H = h + k_h + V_a^2 / 2g$
$k_h = 0.00085$m
$g = 9.81$ m. sec^{-2}
h_1, H_1 is the stage/total head above V invert
h_2, H_2 is the stage/total head above the rectangular crest
b_r is the total width of the rectangular crest between vertical crests
b_v is the width of the top of the confined V
h_v is the depth of the confined V
h_r is the maximum confined stage over the rectangular crest
Cd_r is the coefficient of discharge for the rectangular weir (Hamilton-Smith)
C_e is the coefficient of discharge for the V-notch (Kindsvarter and Carter)
V_a is the velocity of approach = Q/A

In the absence of any information on the limits to which this formula can be applied, those stipulated in the BS 3680: Part 4a standard have been reinterpreted to cope with superimposition of one structure on another. The V-notch chosen is a 90° which is allowed a maximum depth (h) of 0.381m to conform to the Standard. As the ratio h_V/P_V (maximum stage/weir height) should not exceed 0.4, the invert of the V needs to be 0.95m from the bed for the V-notch to operate successfully in isolation. Clearly, when head builds up on the rectangular notch h_V becomes greater than 0.381m, and the standard does not strictly apply; the need for an independent calibration becomes obvious. This can be allowed for in the Barnes equation, shown above as equation 4, by assuming that a significant velocity head develops and then taking this into account by using a velocity iteration procedure. This works by replacing V_a with Q/A, assuming $Q_{initial}$ is zero and feeding the resultant discharge estimate back into the equation repeatedly until the precision increase in the estimate is no longer significant.

Considering the rectangular notch in isolation (the V-notch flow becomes increasingly less significant as a proportion of total flow as stage rises), suggests that the head that can be measured on the rectangular crest should be restricted to a maximum h_R/P_R ratio of 0.5 and an absolute maximum of 0.6m (Hamilton-Smith equation). Given the minimum level imposed on P_R by the V-notch installation, where $P_R = h_V+P_V = 1.334$m, the full 0.6m head can be measured on the rectangular section without the h_R/P_R limit being exceeded. If greater heads need to be recorded, there are other equations available with less restricting limits.

To cope with the 50 year return period flood of 1.7946 $m^3 sec^{-1}$ the weir width, b, can be varied, provided this remains much less than the width of the channel. Widening of the rectangular notch will also have the effect of usefully reducing the required height of the cut-off wall for a given flow, and reducing the size of the stilling pool at high flows, but as such flows occur quite infrequently there seems little point in trading off precision in flow measurement for the latter. It would be convenient to choose a weir width, b, of 8' (c. 2.4m) in order that a single standard imperial-sized sheet of duralumin can form the crest.

Suggestions for a Practical Weir Design

It is apparent that the British Standard requirements for a sharp crested thin plate weir cannot be accommodated without having an excessively high weir that will cause unacceptable visual intrusion at this sensitive site in the National Park. It is recommended therefore that the standard height of the invert of the V of 0.95m ie. $h/P < 0.4$ be relaxed to 0.42m ie. $h/P =$ c. 1.0. In this way the height of the rectangular crest from the channel invert can be reduced from 1.334m to 0.8m. The rectangular section will then operate successfully to the BS calibration up to a maximum head of 0.4m ie. $h_R/P_R = 0.5$. However it should be made plain that this course of action will cause some calibration problems above a head of 0.168m on the V-notch as well as above 0.4m on the rectangular, so allowance will have to be made for non-trivial velocity heads in the approach channel. This can be done by using the iterative calibration procedure or by performing an independent calibration using current meters or dilution gauging, a procedure that has been recommended already in this report in any case.

It is difficult, and probably unnecessary at this stage, to use the Barnes equation in the optimisation procedure for determining the height of the rectangular notch above invert. An adequate approximation can be made by assuming that the discharge over the V will not increase significantly as head rises over the rectangular crest, giving a V-full discharge of c. 0.123 m^3 sec^{-1}. The rectangular weir can then be treated as the simple form. Thus for a 2.4m wide weir the 50 year flood of 1.8 m^3 sec^{-1}, and shorter return period flows, can be accommodated with heads shown in table 5.

Table 5. Head requirements on the rectangular crest for floods of various return periods.

Return period (years)	Flow (m^3 sec^{-1})	Head (m)
5	1.1286	0.365
10	1.3229	0.41
50	1.7946	0.5

The 50 year flood can thus be accommodated by a 0.5m head on the rectangular weir, giving an overall maximum height to the weir of 1.3m. In practice, most of the ECN sites will be taking advantage of existing structures, and many of these will not be able to cope with a 50 year flood, probably only a ten year flood or lower. If visual impact objections to the weir are made it would be possible to reduce the entrainable head to 0.4m which will contain only the 10 year flood, but it is doubtful whether this 0.1m reduction would have a noticeable effect in comparison to the scientific benefits lost. The other alternative is to increase the weir width, b; however this could reduce the sensitivity of flow estimation over the rectangular weir to an unacceptable degree and is not therefore recommended.

Materials and Costs

The final recommended dimensions for the weir, and the way this will be made up in duralumin sheets is shown in figure 1. Final material requirements and estimated building costs are as shown in table 6.

It should be stressed that these costs include only the initial construction, transportation of materials to Snowdonia and installation of weir and cut-off walls on site. It is assumed that CCW will arrange for the site to be cleared of overburden down to bedrock along the line of the weir, for materials to be transported from the road head to the actual construction site, for the weir to be concreted in to provide a watertight seal and for all landscaping works to comply with National Park and Local Planning requirements.

Of these, delivery of materials from road to site will be most problematical and may require erection of an overhead winch system or use of a helicopter. The overburden on the site may also be deep in places and it is important to be scrupulously clean to ensure a seal.

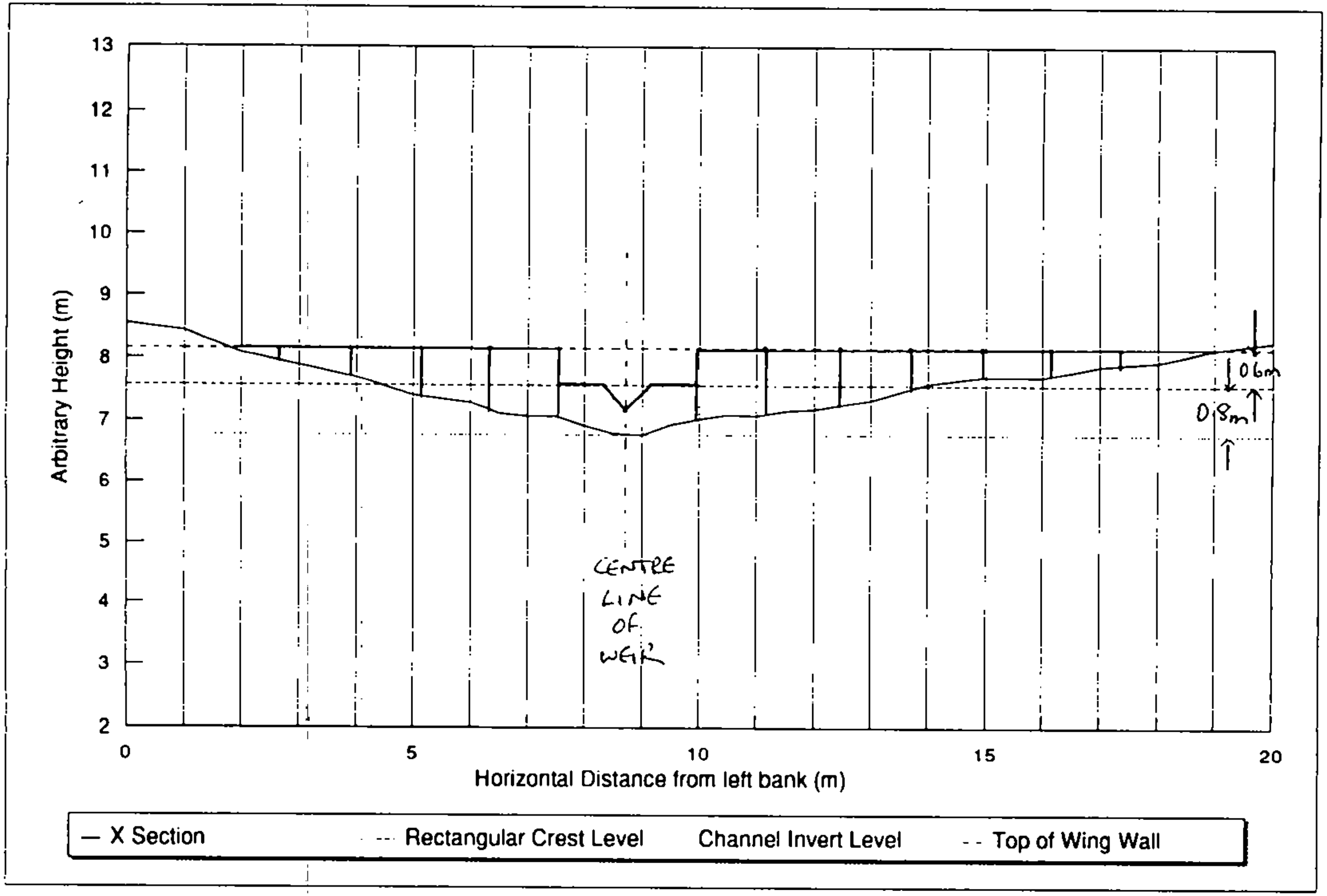

Figure 1. Cross section of stream channel with proposed weir imposed

Table 6. Costs and timescale of construction

TASK	TIME AND UNIT PRICE	COST
Workshop construction, original erection of weir plates and bracing and dismantling for transit.	25 man days at £164.50 per day	4662.50
Materials		2377.00
Installation	(5 days * 2 men) + O/TIME	2500.00
Subsistence		516.50
Van Hire		600.00
Wood		25.00
TOTAL		**10681.00**
VAT		1869.00
TOTAL (inc. VAT)		**12550.00**

References

Barnes (1916) Hydraulic Flow Reviewed. *Spon*, London.

British Standards Institution (1965) The measurement of liquid flow in open channels. B.S. 3680: Part 4a. Thin plate weirs and venturi flumes. *BSI*, London.

Natural Environment Research Council (1975) The Flood Studies Report. *Institute of Hydrology*, Wallingford, Oxon, OX10 8BB.