FLOOD STUDIES UPDATE (FSU) PROGRAMME

WP4.2 Flood Estimation in Small and Urbanised Catchments

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FSU Work-Package 4.2

Urban and Small Catchment Flood Estimation

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1. Introduction

The Office of Public Works launched the Flood Studies Update (FSU) in 2005 which comprised several work-packages among which WP 4 was one. The WP 4.1 which is part of WP4 and which dealt with a scoping study of urban flooding issues in Ireland recommended for further research. Hence, WP4.2 is the follow on project to undertake these recommendations.

The main recommendations were:-

- To investigate whether any consistent relationships between these new soil maps and rainfall-runoff coefficients and concentration times can be established,
- To examine percolation values for a range of catchment types to investigate the relationship, if any that may exist between percolation, infiltration and runoff,
- Develop guidance on methodologies to model the spatial progression for specified flooding scenarios.
- To evaluate current flow estimation methods.

This report deals with the last recommendation.

The first part of this report examines the methods being applied in different countries to estimate urban and small catchment peak flows. The second part will screen the methods which sound applicable to Irish catchments. The methods will be scrutinized employing real data from existing stations. Peak flows computed with these methods will be related through regression either to median flood (*QMED*) or certain return period flow. The peak flows will then be converted to 2year flow (median flow) and will be compared with actual median flood of the station where the methods are tested. The methods will also be compared to the newly developed FSU methodologies. Station(s) will be erected at appropriate location(s) with urbanised catchment and small rural catchment to measure flows. The peak flow methods screened will be finally tested to these specific catchments. Based on the analysis of these results methodologies and guidelines will be developed.

2. Assessment of Urban and Small Catchment Flood Estimation Methods

A suitable and reliable technique for estimating flood magnitudes is required for effective flood-plain management, and for efficient design of attenuation storages, bridges, culverts, embankments, and flood-protection structures, whether it is urban or rural. Statistical techniques are effective tools for obtaining peak flows and their associated probabilities on gauged streams. However, most small and urban catchments where urbanization and infrastructural development take place are ungauged. In practice, most hydraulic structures to control runoff to predevelopment levels are installed in small catchments which then require flow estimation. The most widely used methods to estimate peak flows, such as Rational Method, USGS Regression Equations, NRCS Method, Unit Hydrograph Method, FEH Method, IoH Report 124, FSSR Method, TRRL/ADAS 345 methods, are discussed in the following sections.

2.1 Rational Method

The rational method has been in use for over 150 years and remains the most widely used method to estimate peak flows from urban and small rural ungauged catchments

(Watts & Hawke, 2003). It relates peak flow (m^3/s) to catchment area (km^2), rainfall intensity (mm/hr) and runoff coefficient. It has the form of:

$$Q = CiA$$

Where Q is the peak flow rate, i is the rainfall intensity, A is catchment, area and C is the runoff coefficient.

The method is based on the assumptions that rainfall intensity and storm duration is uniform over the area of study; storm duration must be equal to the time of concentration of the catchment; and that the runoff coefficient is constant during a storm (Hays & Young, 2006). The above equation is divided by 360 for SI units.

The runoff coefficient, C, is expressed as a dimensionless decimal that represents the percentage of rainfall appearing as runoff. Except for precipitation, which is accounted for in the formula by using the average rainfall intensity over some time period, all other portions of the hydrologic cycle are contained in the runoff coefficient. Therefore, C includes interception, infiltration, evaporation, depression storage, and groundwater flow. The variables needed to estimate C should include soil type, land use, degree of imperviousness, watershed slope, surface roughness, antecedent moisture condition, duration and intensity of rainfall, recurrence interval of the rainfall, interception and surface storage. The fewer of these variables used to estimate C, the less accurately the rational formula will reflect the actual hydrologic cycle. The use of average runoff coefficients for various surface types is common. In addition, C is assumed to be constant although the coefficient will increase gradually during a storm as the soil becomes saturated and depressions become filled. A suggested range of runoff coefficients are available in literature.

The rainfall intensity i is the amount of rain that has fallen per unit of time. The average rainfall intensity i can be read from an intensity-duration-frequency (IDF) curve of the catchment of interest for duration equal to the time of concentration (hr) and specified storm return period. Rainfall intensity varies with time during a given storm for different geographical regions and also for different locations specific to a region, resulting in different rainfall distributions (Nyman, 2002).

The time of concentration is defined as the travel time for a runoff to get from the most hydraulically remote point of the contributing catchment area to the point where peak flow is estimated. It can be determined using empirical formulas such as the Kirpich's equation (shown below), Kerby's Equation (Chin, 2000), or Kinematic wave equation and also from hydrographs. The calculated time of concentration is used to determine average rainfall intensity to be applied uniformly over the catchment to produce its peak flow for a specified return period using rational formula.

$$T_c = 0.0195\, L^{0.77}\, S^{-0.385}$$

Where T_c is time of concentration in min, L is maximum length of river in m, and S is the catchment gradient in m per m (the difference in elevation between the outlet and the most remote point divided by the length, L).

The extent of catchment area where Rational Method can be applied varies widely from country to country and among literatures. Please see Tables 2.1 and 2.2.

Table 2.1: Size of catchment where Rational Method is applied according to some contries

Region/Countries where Rational Method is widely applied	**Catchment size limit** (km^2)	**Remark**
- Australia: Urban[1]	5.00	
Rural[1]	25.00	
- Canada[2]	25.00	
- USA: Washington State[3]	0.40	
Maine State[4]	2.60	
Florida[5]	2.43	
New York[6]	0.08	
- US Dept. of Transportation[7]	0.80	
- Hong Kong[8]	1.50	
- Malaysia[9]	0.80	
- UK[10]	2.00 to 4.00	
- New Zealand[11]	0.50	

Table 2.2: Size of Catchment where Rational Method is applied according to literatures

Literature	**Catchment size** (km^2)	**Remark**
Debo and Reese, 1995, Municipal Storm Water Management, p. 209	0.08	
Wanielista, Kersten and Eaglin, Hydrology: Water Quantity and Quality Control, 2nd edition, 1997.	0.20 to 0.40	
Chow, V. T., Handbook of Applied Hydrology, Chow, 1964, p. 25	0.40 to 0.80	
Design and Construction of Urban Stormwater Management Systems (ASCE Manuals and Reports of Engineering Practice No. 77) ASCE, 1992, p. 90	0.40 to 0.80	
Singh, V.P., 1992, Elementary hydrology, p. 599	0.40 to 0.96	
ASCE (1996), “Urban Hydrology”, Chapter 9 in Hydrology Handbook, Manuals and Reports on Engineering Practice No.28, p. 580	1.00	Urban
Ponce, V.M., 1989, Engineering Hydrology, p.119	1.30 to 2.50	
Gray, D. M., (ed.) 1970. Handbook on the principles of hydrology, 1970, p. 8.2	2.56	
Viessman, W., and Lewis, G.L. (1996). Introduction to Hydrology, fourth edition, p. 318	2.56	
Gupta, R.S. (1989). Hydrology and Hydraulic Systems, p. 621	10.00	Rural

The Rational Method is only suitable for small catchments as the method does not account for catchment storage during flood events, thus can’t be used to produce hydrograph. The presence of flow restrictions (culverts, bridges, etc …) may affect peak flow estimated with the method. As many literatures indicated, it is appropriate for small catchments. However, the definition of small catchment is not consistent across practitioners. The runoff coefficient selection is also very subjective which in turn

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[1] Queensland Gov’t, Queensland Urban Drainage Manual, Vol. 1, 2nd Ed., 2007.
[2] Manual of Operational Hydrology in British Columbia, 2nd Ed., Coulson, C.H., 1991.
[3] Washington State DoT, Hydraulics Manual, Environmental & Engineering Services Center, 1997.
[4] USGS & Dept. of Transportation, Comparison of Peak Flow Estimation Methods for Small Drainage Basins in Maine, 2007.
[5] State of Florida, Dept. of Transportation, Hydrology Handbook, 2004.
[6] New York State, Standards and Specifications for Erosion and Sediment Control, 2004.
[7] US Dept of Transportation, Federal Highway Administration, Urban Drainage Manual, 2009.
[8] Stormwater Drainage Manual, Planning, Design and Management, Gov’t of Hong Kong, 1999.
[9] Urban stormwater management manual for Malaysia, Department of Irrigation and Drainage, 2000.
[10] Hydrology in Practice, Shaw, E.M., 2004.
[11] New Zealand, On-Site Storm Water Management Guideline, 2004.

increases the uncertainty of the peak flow estimated. Hence, the method should be used as a tool to compare outcomes from other empirical methods.

The main advantage of the Rational Method is that there is ample experience in its application over many years of its use. It is also a simple concept and computed with out use of computers.

2.2 Modified Rational Method

The traditional rational method is limited to considering storms with duration equal to the time of concentration and provides only a peak flow. It allows calculating peak flow under the assumption that rainfall intensity is uniformly distributed over the whole storm event (Hua, Liang & Zhongbo, 2003). The modified rational method can consider single event storms with changing intensities and longer durations. The modified rational method is being developed at different practicing agencies to account for the variation of rainfall intensity within same storm duration. In some instances runoff coefficient is modified to account for decrease in soil permeability as rainfall intensity increases and to adjust for increase in runoff as average slope increases. Three examples are demonstrated below.

Example 1

The one developed in California (Co. Alameda, Hydrology Manual, 2003) is of the following form:

$$Q = C'IA$$

Where C' is runoff coefficient modified by slope and rainfall intensity, and A is catchment area. Rainfall intensity I is modified as:

$$I_j = (0.33 + 0.091144MAP)(0.249 + 0.1006K_j)T_i^{-0.56253}$$

Where I_j is rainfall intensity (mm/hr) for return period j, and storm duration I, MAP is mean annual precipitation (mm), T_i is storm duration (hr) (or = T_c/60), and K_j is frequency factor to be determined per return period as shown in Table 1.1.

Table 2.3: Values of K_j, frequency factor (*Source: Alameda Hydrology & Hydraulics Manual*)

Return period (yrs)	5	10	15	25	100
Frequency Factor, K_j	0.719	1.339	1.684	2.108	3.211

The modified runoff coefficient is determined as follows:

$$C' = C + C_s + C_i$$

Where C is runoff coefficient (as in Rational Method), C_s is slope adjustment runoff coefficient (to adjust for increases in runoff as average drainage area slope increases), and C_i is rainfall intensity adjustment factor (to account for decrease in soil permeability with an increase in rainfall intensity). C_s and C_i are determined by the following equations.

$$C_s = [(0.8 - C)\ (ln(S-1)S^{0.5})]/56, \ \ for\ C \geq 0.8,\ C_s = 0$$

where S is average (weighted) slope in percent.

$$C_i = [0.8-(C+C_s)]\left[1-\frac{1}{\frac{1}{e^I}+\ln(I+1)}\right], for\ C + C_s \geq 0.8,\ C_i = 0$$

Where I is rainfall intensity (mm/hr) equal to I_j above.

Example 2

The Rational Method generates the peak discharge that occurs when the entire catchment is contributing to the peak (at a time $t = t_c$) and ignores the effects of a storm which lasts longer than time t. Another Modified Rational Method developed in Virginia, however, considers storms with a longer duration than the catchment t_c, which may have a smaller or larger peak rate of discharge, but will produce a greater volume of runoff (area under the hydrograph) associated with the longer duration of rainfall (Virginia Dept of CR, 1999). Fig.: 2.1 shows a family of hydrographs representing storms of different durations. The storm duration which generates the greatest volume of runoff may not necessarily produce the greatest peak rate of discharge.

Note that the duration of the receding limb of the hydrograph is set to equal the time of concentration, t_c, or 1.5 times t_c. Using $1.5t_c$ in the direct solution methodology provides for a more conservative design according to the handbook sited. This is justified since it is more representative of actual storm and runoff dynamics. It is also more similar to the NRCS unit hydrograph where the receding limb extends longer than the rising limb, which will be shown later.

The modified rational method allows the designer to analyze several different storm durations to determine the one that requires the greatest storage volume with respect to the allowable release rate (which is limited to pre-development peak flow rate). This storm duration is referred to as the *critical storm duration* and is used as a storage basin sizing tool.

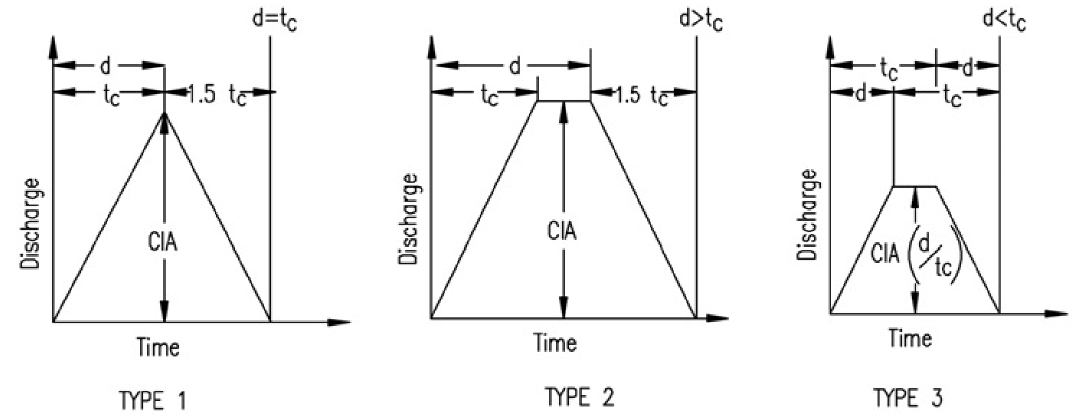

Fig. 2.1: Modified Rational Method procedures: **Type 1 -** Storm duration, *d*, is equal to the time of concentration, *tc*. **Type 2** - Storm duration, *d*, is greater than the time of concentration, *tc*. **Type 3** - Storm duration, *d*, is less than the time of concentration, *tc*. (*Source: Virginia Stormwater Management Handbook, Vol. 2, First Ed., 1999*)

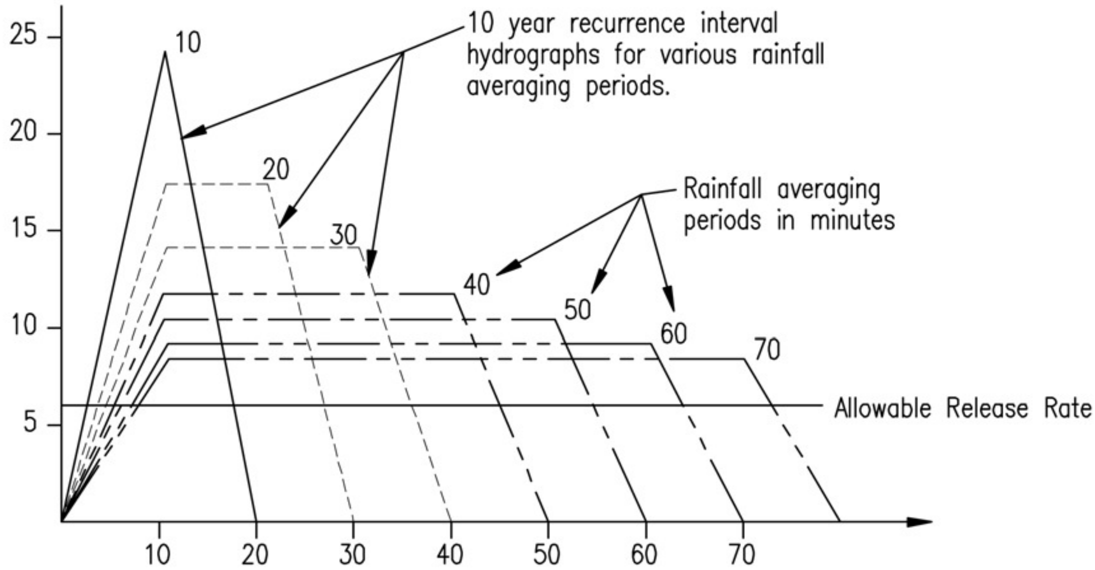

Fig. 2.2: Modified Rational Method procedures continued for several return periods. (*Source: Virginia Stormwater Management Handbook, Vol. 2, First Ed., 1999*).

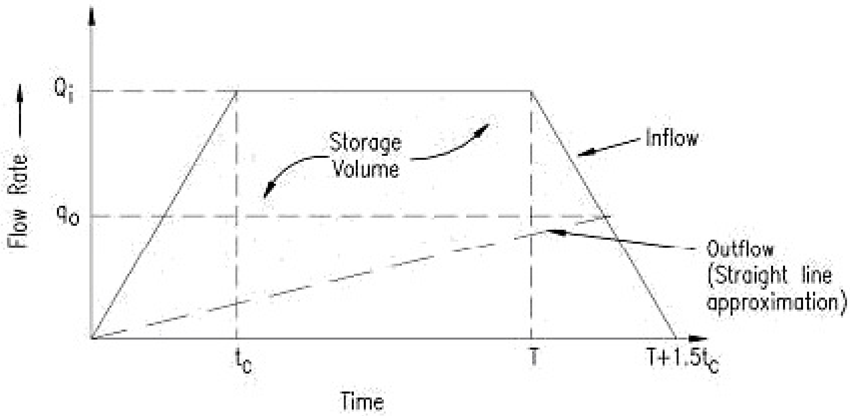

Fig. 2.3: Modified Rational Method, Trapezoidal Hydrograph Storage Volume Estimate (*Source: Virginia Stormwater Management Handbook, Volumes 2, First Edition, 1999*).

Example 3

In the 1980s, the Institute of Hydrology, Meteorological Office and HR-Wallingford refined the Rational Method and developed Modified Rational Method which is part of the "Wallingford Procedure" to be used in homogenous catchments up to 1.50km^2 (Shaw, 2004). It has the following form (Chadwick, Morfett & Borthwick, 2009):

$$Q_p = 2.78\,(C_v C_R i A)$$

Where C_v is the volumetric runoff coefficient, C_R is the routing coefficient and the remaining are same as in the Rational Method.

The recommended equation for C_v is:

$$C_v = PR/100$$

Where PR is the (urban) percentage runoff which is found from:

$$PR = 0.829PIMP + 25.0SOIL + 0.078UCWI - 20.7$$

Where *PIMP* is percentage impermeable area to total catchment area, *SOIL* is a number depending on soil type, and *UCWI* is the urban catchment wetness index (mm) related to *SAAR*.

The recommended value for C_R is a fixed value ($C_R = 1.3$) for all systems (Chadwick *et. al.* 2009). The estimation of i requires the knowledge of critical storm duration t_c. The assumption made is that this storm duration is equal to the time of concentration for the catchment, t_c, given by:

$$t_c = t_e + t_f$$

Where t_e is the time of entry into the drainage system (between 3 and 8min) and t_f is the time of flow through the drainage system.

The value of i for given return period and duration may be estimated according to the flowing procedure (Chadwick *et. al.,* 2009). First, values of Jenkinson's r (*M5* - 60min/*M5* - 2day rainfall) and *M5* - 60min (rainfall of 5year return period and 60min duration) are read from map. Next, the value of M5 - D/M5 - 60min (where D is the required duration) is read from plotted data using the value of r to obtain the required value of *M5 - D*. The value of *MT – D* (where *T* is the required return period) is found from tabulated data relating *M5 – D* to return period *T*. this value of *MT – D* (the point rainfall of required return period and duration) is next reduced by multiplying it with an areal reduction factor (*ARF*), which is plotted as a function of duration and area, to obtain the design catchment rainfall depth. Finally the design rainfall intensity (i) is found from:

$$i = (MT - D)/D$$

The value of t_f is computed from L/V (where L is channel length and V is channel velocity) and the peak flow Q_p is calculated from the equation shown above (Chadwick *et. al.* 2009).

The Rational Method is 'modified' as it has been shown in the above three practices. The runoff coefficient and rainfall intensity has been modified to account for their temporal and spatial variability during a storm event. There is no wide range of experience with these methods or the methods are limited to certain regions. It is also not clear to what size of catchment area they are applied. However, the Wallingford Procedure was found to be more accurate than the Rational Method when applied in the UK up to a catchment area of 1.50km^2 (Mitchell, *et. al.* n.d.).

2.3 USGS Regression Equations

Regional regression equations are the most commonly accepted method in the US for establishing peak flows not only at gauged sites but also at ungauged sites or sites with insufficient data. Regression equations have been developed to relate peak flow at a specified return period to the hydrology of a catchment. In the US each state is divided into regions of similar hydrologic, meteorologic, and physiographic characteristics as determined by various hydrological and statistical measures (McCuen, Johnson & Ragan, 2002).

Regional regression equations were developed by USGS as a two-step process involving ordinary and generalized least-squares regression techniques (Dillow, 1996). Ordinary least-squares (OLS) regression techniques are used in the first step to determine the best models relating catchment characteristics listed below to any *T*-year return period peak discharge estimate. In the second step, the final model identified by means of ordinary least-squares regression techniques was used in generalized least-squares regression analyses to develop equations that can be used for predictive purposes.

The catchment characteristics taken into account in the regression process are (Koltun, 2003):-

- ***A***, drainage area (km^2),
- ***S***, main channel slope (m per km),
- ***AOS***, average *maximum* overland slope of the land surface (percent),
- ***STRMFRQ***, stream frequency or drainage density (1/km),
- ***MCE***, mean catchment elevation (m),
- $\boldsymbol{MAP_c}$, mean annual precipitation at the catchment centroid (mm),
- $\boldsymbol{MAP_m}$, mean annual precipitation averaged over the catchment area (mm),
- $\boldsymbol{CF_2}$, $\boldsymbol{CF_{25}}$, and $\boldsymbol{CF_{100}}$, Climate factors with recurrence intervals of 2, 25, and 100 years, respectively (dimensionless),
- ***Water***, percentage of the catchment classified as water,
- ***Wetland***, percentage of the catchment classified as wetland (%),
- ***Urbanised***, percentage of the catchment classified as developed/urbanised (%),
- ***Undeveloped*** (grey area), percentage of the catchment classified as barren (%),
- ***Forest***, percentage of the catchment classified as forested upland (%),
- ***CR***, circularity ratio (dimensionless) - a measure of catchment shape (circular versus elongated); determined as

$$CR = P/(4\pi A)^{0.5}$$

where P is the perimeter of the catchment, in km, and A is the drainage area in km^2.

The typical regression models utilized in regional flood studies are of the form:

$$Y_T = aX_1^{b_1} X_2^{b_2} ... X_p^{b_p}$$

Where: Y_T is the dependent variable (which is the peak flow for a given return period T),
$X_1, X_2, \ldots, X_p$ are independent variables (which are the catchment characteristics,
a is the intercept coefficient (or regression coefficient), and,
$b_1, b_2, \ldots, b_p$ are regression exponents (determined using a regression analysis).

The states in US are divided into regions of similar hydrologic, meteorologic, and physiographic characteristics as determined by various hydrological and statistical measures (Koltun & Roberts, 1990) which is equivalent to the way Ireland is divided in to several hydrometric areas. When the regression analysis is complete, not all catchment characteristics would be included in the final regression equation. The variables are selected based on the influence they incur unto the dependent variable (peak flow). Then each hydrological region would have its own regression equation for a given return period. The peak flow for a 2-year return period for a certain hydrometric area could be, say:

$$Q_2 = 2.52A^{0.775} (E/1000)^{3.32} (F+1)^{-0.504}$$

And 5-year return period for the same hydrometric area could be say:

$$Q_5 = 23.00A^{0.720}(E/1000)^{3.36}(F+1)^{-0.885}, \ldots \text{ and so on.}$$

Where A is catchment area, E is mean catchment elevation, and F is forested area.

USGS Regression offers several advantages over other methods according to State of Maine urban and arterial highway design guide (Maine DoT, 2008). It is more accurate than rainfall-runoff modelling in comparable situations. It is based directly on annual maximum data, when gauged station is used, and thus does not depend on the questionable assumption (inherent in rainfall-runoff modelling) that the T-year storm produces the T-year flood event. However, the regression equations are subject to several limitations that it works better at catchment sizes greater than 2.5 km^2, and not steeper than 50m/km slope. It also works well at rural, undeveloped, and unregulated (natural) catchments.

2.4 The National Resources Conservation Service (NRCS) / TR-55 Method

The Technical Release 55 (TR-55) or NRCS method formerly known as SCS method relates rainfall, retention and effective rainfall or runoff (USDA, NRCS, 1986). Mass rainfall is converted to mass runoff by using a runoff curve number (CN). The method follows two procedures: graphical discharge method or tabular hydrograph method. When the catchment needs to be divided into sub-catchments because of widely differing curve numbers or non homogeneous slope patterns, then the tabular hydrograph approach is used, otherwise the graphical method is used. The graphical method is examined below.

The rainfall-runoff relationship in the model separates total rainfall into direct runoff, retention, and initial abstraction to yield the following equation for rainfall runoff:

$$Q_D = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Where Q_D is depth of direct runoff (mm), P is accumulated rainfall/potential maximum runoff (mm), I_a is initial abstraction, and S is retention of rainfall on the Catchment (mm).

Through researches, I_a was found to be approximated by the flowing equation:

$$I_a = 0.2S$$

The value of S is related to soil type and land cover of the catchment through the curve number, CN. CN is a function of soils type, vegetation cover, magnitude of impervious areas, interception, and surface storage. CN has a range of 0 to 100 (USDA-NRCS, 1986).

$$S = \alpha \frac{1000}{CN} - 10$$

Where α is unit conversion constant = 25.4 (or = 1 for British units)

The retention, or potential storage in the soil, is established by selecting a curve number (CN). The curve number is read from tables found in most US hydrologic books, or can be estimated if rainfall and runoff volume are known.

$$CN = 1000/[10 + 5P + 10Q_a - 10(Q_a^2 + 1.25Q_aP)^{0.5}]$$

Where P is rainfall (mm) and Q_a is rainfall volume (mm).

Table 2.4: Sample Curve Numbers (CN) according to land cover used in the US (*Source: USDA, 1986, Urban Hydrology for Small Watersheds, TR-55*)

Land cover description		***CN* for hydrologic soil group**			
Cover type and hydrologic condition	**Average % impervious area**	**A**	**B**	**C**	**D**
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc):					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential: average lot size					
Row houses, town houses, and residential with lot sizes 0.05 ha (1/8 ac) or less	65	77	85	90	92
0.10 ha	38	61	75	83	87
0.14 ha	30	57	72	81	86
0.20 ha	25	54	70	80	85
0.40 ha	20	51	68	79	84
0.81 ha	12	46	65	77	82
… *and so on* …	…	…	…	…	…

Where **A**, **B**, **C** and **D** respectively are:

Group A soils, which have a low runoff potential due to high infiltration rates.
Group B soils, which have a moderately low runoff potential due to moderate infiltration rates.
Group C soils, which have a moderately high runoff potential due to slow infiltration rates.
Group D soils, which have a high runoff potential due to very slow infiltration rates.

For multiple land use/soil type combinations within a catchment, aerial weighing is used to compute composite *CN*.

The peak flow estimation equation is:

$$Q_p = Q_u A Q_D F_p$$

Where Q_p is peak flow (m^3/s), Q_u is unit peak flow (m^3/s), A is catchment area, Q_D is runoff depth (mm) and F_p is adjustment factor given in table to reflect the storage in lakes or swamps that are not along the t_c flow path.

The unit peak flow is computed from:

$$Q_u = \alpha 10^{C_0 + C_1 \log(t_c) + C_2 [\log(t_c)]^2}$$

Where C_0, C_1 and C_2 are regression coefficients which are a function of the 24 hour rainfall distribution type and various I_a/P ratios given in table, t_c is time of concentration, and α is conversion constant = 0.000431 (or = 1 in British units).

Table 2.5: Storage adjustment factor used in the Peak flow estimation formula (*Source: USDA, 1986, Urban Hydrology for Small Watersheds, TR-55*)

Area of lake or swamp (%)	F_p
0.0	1.00
0.2	0.97
1.0	0.87
3.0	0.75
5.0	0.72

The TR-55 method has tabulations only for the US rainfall distribution maps. Therefore, non-US users need to determine whether a typical 24-hr rainfall that resembles a Type I, IA, II, III or which rainfall distribution type best matches the user's region.

While TR-55 gives special emphasis to urban and urbanizing catchments, the procedures apply to any small catchments in which certain limitations are met (USDA, 1986). The TR-55 method has a number of limitations where these conditions are not met, the accuracy of estimated peak discharges decreases. The method should be used on catchments that are homogeneous in CN; where parts of the watershed have CNs that differ by 5, the watershed should be subdivided and analyzed using a hydrograph method, such as TR-20. The TR-55 method should only be used when the CN is 50, or greater and the t_c, or is greater than 0.1 hour and less than 10 hours. Also, the computed value of I_a/P should be between 0.1 and 0.5. The method should be used only when the catchment has one main channel or when there are two main channels that have nearly equal times of concentration; otherwise, a hydrograph method should be used. Neither channel nor reservoir routing can be incorporated.

The NRCS has also released the WinTR-55 computer software package, which calculates peak flows for catchments with areas smaller than $65km^2$ (US DoT, 2008).

2.5 The NRCS Dimensionless Unit Hydrograph Method

Unit Hydrograph Methods may be used to compute storm water discharges for all sizes of catchments, where storm water discharge is produced by catchments, where storm water storages exist or are anticipated upstream of the point of interest. There are two commonly used unit hydrograph methods for peak flow estimation used in drainage design system: Snyder's Unit Hydrograph Method and the NRCS Dimensionless Unit Hydrograph Method. The later is discussed in this section.

Victor Mockus developed a dimensionless unit hydrograph based on a large number of unit hydrographs from catchments that varied in characteristics such as size and geographic location (Snider, 1972). The NRCS uses the same dimensionless unit hydrograph procedure which is well known method for deriving synthetic unit hydrographs. This dimensionless unit hydrograph, which is the result of averaging a large number of individual dimensionless unit hydrographs, has a time-to-peak located at approximately 20% of its time base and an inflection point at 1.7 times the time-to-peak. This curvilinear unit hydrograph can be approximated by a triangular unit hydrograph (UH) that has representation of excess runoff with one rise, one peak and one recession, see Fig. 2.4.

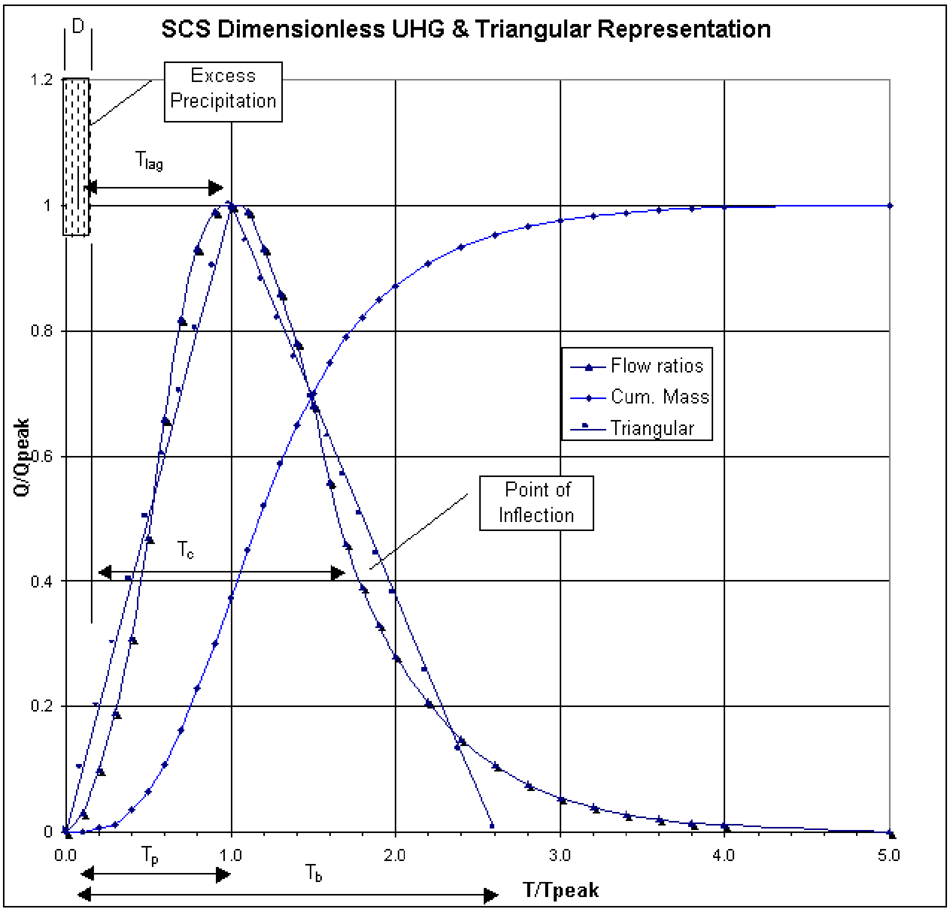

Fig. 2.4: Dimensionless Unit Hydrograph Method triangular representation (*Source: National Engineering Handbook, Section 4, Hydrology*)

Using the geometry of a triangle, one can see that the unit hydrograph has 37.5% (or 3/8) of its volume on the rising side and the remaining 62.5% (or 5/8) of the volume on the recession side. Using the dimensionless timing values on the x-axis, one can solve for the time base in terms of the time-to-peak. Recall that the unit hydrograph is the result of a unit depth (1mm) of excess rainfall (of duration D) spread uniformly over a catchment. This 1mm of excess rainfall is also indicated in Fig. 2.4 to show the definition of the timing parameters.

The following relationships are made and will be useful in further developing the peak flow relationships. Note that the time base, T_b, of the triangular unit hydrograph extends form 0 to 2.67 and the time to peak, T_p, is at 1.0, thus the time base is 2.67 times the time to peak or:

$$T_b = 2.67T_p$$

And that the recession limb time, T_r, is then 1.67 times the time to peak.

$$T_r = T_b - T_p = 1.67T_p$$

Using the geometric relationships of the triangular unit hydrograph of Fig. 2.4, the total volume under the hydrograph is found by (area under two triangles):

$$Q_D = \frac{1}{2}Q_p T_p + \frac{1}{2}Q_p T_r = \frac{1}{2}Q_p(T_p + T_r)$$

Then,

$$Q_p = \frac{2Q_D}{T_p + T_r}$$

Where Q_D is the volume of direct runoff (area under hydrograph) which equals to 1 for a unit hydrograph.

The peak flow in terms of catchment area (A) will become:

$$Q_p = \frac{2.083AQ_D}{T_p}$$

Where: Q_p = Peak flow (m^3/s),
A = Drainage area (km^2),
Q_D = Volume of direct runoff (1mm), and
T_p = Time to peak (hr).

The constant 2.083 (or 484 in British units), which is also known as peak rate factor, reflects a unit hydrograph that has a 3/8 of its area under the rising limb. For catchments which are mountainous the fraction could be as high as 2.6 (600 in British units). At catchments which are flat and swampy, the constant could be in the order of 1.3 (100 in British units) (Brown, *et. al.,* 2009).

Time to peak can also be represented in terms of duration of unit excess rainfall (D) and time of concentration (T_c) (Texas DoT, 2009).

$$T_p = \frac{D}{2} + 0.6T_c,$$

Where lag = $0.6T_c$
And, $D = 0.133T_c$

Other unit hydrograph methods are available such as Snyder Synthetic Unit Hydrograph, and Natural Unit Hydrograph methods. But they either require the value of curve number (*CN*) or Rainfall distribution type identified.

2.6 The FSSR 6, 3-variable Method

The Flood Studies Supplementary Report No. 6 was introduced to overcome the shortcomings in the estimation of mean annual floods from small catchment through the use of FSR. It is worth noting that the FSR investigation included catchment areas in the range of $0.05km^2$ to $9868km^2$. Although its application is limited to areas between 0.50 and $20km^2$ (Balmforth *et. al.,* 2006), small catchments (less than $20km^2$) did not feature prominently in the study (Marshall & Bayliss, 1994). A total of fifty-three catchments with less than $20km^2$ catchment area were used in the regression analysis during this study (Institute of Hydrology, 1978).

FSSR 6 provides *QBAR* equations for possible use on catchments of less than $20km^2$:

$QBAR = 0.00066\, \mathbf{AREA}^{0.92}\, \mathbf{SAAR}^{1.22}\, \mathbf{SOIL}^{2.0}$ *or*

$$QBAR = 0.0288\, \boldsymbol{AREA}^{0.90}\, \boldsymbol{RSMD}^{1.23}\, \boldsymbol{SOIL}^{1.77} \boldsymbol{STMFRQ}^{0.23}$$

According to Cawley (2003), the above equations performed well for sample catchments with *SOIL* indices greater than 0.45 (soil types 4 and 5), but were poor with catchments with *SOIL* types 1, 2 and 3. The CIRIA report C635 (Balmforth *et. al.,* 2006) stated that although the three parameter equation is easier to use, it was established that the accuracy was not significantly improved from the general six parameter equation for all catchments.

2.7 The Institute of Hydrology Report 124 (IoH 124)

The IoH 124 Report was a research to examine the response of small catchments, less than 25km^2, to rainfall and to derive an improved flood estimation equation (Marshall & Bayliss, 1994). A total of 84 sites were used to validate the method (UK Dept of Transport, 2004). The report has developed new equation to estimate time to peak (T_p(0)) of instantaneous unit hydrograph for part urban and rural catchments of less than 25km^2.

$$T_p(0) = T_p(0)_{rural}\, (1 + \boldsymbol{URBAN})^B$$

Where $T_p(0)_{rural} = 283.0\boldsymbol{S1085}^{-0.33}\, \boldsymbol{SAAR}^{-0.54}\, \boldsymbol{MSL}^{0.23}$

And $B = -1.0 - 3.0 \exp\left[-\left(\frac{T_p(0)_{rural}}{7.0}\right)^2\right]$

It also worked out an equation to estimate mean annual flood, *QBAR*, for small rural and urban catchments.

$$QBAR_{Rural} = 0.00108\, \boldsymbol{AREA}^{0.89}\, \boldsymbol{SAAR}^{1.17}\, \boldsymbol{SOIL}^{2.17}$$

And

$$QBAR_{Uban}/QBAR_{Rural} = (1 + \boldsymbol{URBAN})^{2NC}[1 + \boldsymbol{URBAN}\{(21/\boldsymbol{CIND}) - 0.31]$$

Where: $NC = 0.92 - 0.00024\boldsymbol{SAAR}$, for $500 \leq \boldsymbol{SAAR} \leq 1100$mm,
$NC = 0.74 - 0.000082\boldsymbol{SAAR}$, for $1100 \leq \boldsymbol{SAAR} \leq 3000$mm, and
$CIND = 102.4SOIL + 0.28\,(CWI - 125)$,

$$SOIL = \frac{(0.10S_1 + 0.30S_2 + 0.37S_3 + 0.47S_4 + 0.53S_5}{S_1 + S_2 + S_3 + S_4 + S_5}$$

Where: *NC* is rainfall continentality factor, *CIND* is catchment index and *CWI* is catchment wetness index as in FSR (1975), and *QBAR* (m^3/s), *AREA* (km^2), and *SAAR* (mm).

The *QBAR* computed has an estimated return period of 2 ⅓ years. The estimated *QBAR* is then multiplied by a growth factor of 1.96 (FSR 1975) to get 100-year peak flow.

2.8 ADAS 345 and TRRL Methods

The Agricultural Development and Advisory Service, ADAS developed in 1982 a method primarily for the sizing of field drainage pipes, which was based on the Transport and Road Research Laboratory, TRRL, method (Balkham *et.al.* 2010). The ADAS method is applicable to very small catchment areas up to 0.3km^2. In other words the method applies to one drainage unit, i.e. one pipe, in a system (Shaw 2004). This method takes into account the design storm rainfall and time of concentration for the required return period by using the Bilham formula. For a 75 year return period the design flow, Q (m^3/s) can be determined from (UK Dept of Transport, 2010):

$$Q = AREA(0.0443SAAR - 11.19)SOIL^{2.0}\left[\frac{18.79T^{0.28} - 1}{10T}\right]$$

Where: $AREA$ (km^2) is the catchment plan area, $SAAR$ (mm) is the standard average annual rainfall for the particular location, and T is the time of concentration (hrs) and is given by:

$$T = 0.1677\frac{W^{0.78}}{Z^{0.39}}$$

Where: W is the maximum catchment width in metres, Z is the average height of the upstream catchment divide in metres above the discharge level.

The estimated 75-year return period design flow value is divided by a scaling factor of 1.88 (regional growth factor given by FSR 1975) to obtain the mean annual design flow. The mean annual design flow in turn is multiplied by a growth factor of 1.96 to get 100-year design flood flow. According to the Design Manual for Roads and Bridges (UK DoT, 2010), it is recommended to use the IoH 124 Method for catchments greater than $0.4km^2$ and the ADAS Method for catchments less than or equal to $0.4km^2$. There is a slightly different version (shown below) to estimate peak flow in the CIRIA report C635 (Balmforth *et. al.,* 2006).

$$Q = 13.73S_m^2 F_A A\frac{R_B}{T_c}$$

Where: Q is peak flow (l/s), S_m is soil index (as in WRAP), A is catchment area in ha, F_A is annual rainfall factor (F_A = 0.00127$SAAR$ – 0.321), R_B is rainfall depth (defined by Bilham formula), and T_c is time of concentration (hr).

$T_c = 2.48(LN)^{0.39}$

Where: L is catchment length (km) form upstream divide being measured approximately along the middle of the catchment, N is dimensionless number equal to the ratio L/Z, where Z is the rise from the outfall to the average height of the upstream divide (km).

The TRRL method also known as the Young & Prudhoe method (Young & Prudhoe 1973), predecessor to ADAS 345, was developed from rainfall and runoff data over several years of monitoring specifically for small catchments to allow estimation of peak flows for sizing of road culverts (Balmforth *et. al.,* 2006). Peak flow can be predicted by:

$$Q_p = \frac{F_A A R_B}{3.6T_c}$$

Where: F_A is dimensionless annual rainfall factor (= 0.00127R_A – 0.321), R_A is the average annual rainfall (mm),
A is catchment area (km^2), R_D is expected rainfall depth given in tabulated form in Young and Prudhoe (1973) based on Bilham formulae (mm), R_D can be derived for value of T_c and return period T, $10/T = 1.25T_c(0.0349R_B + 0.1)^{-3.55}$, and T_c equation above.

The TRRL method was considered to be suitable for predominantly clay type of soil catchments (Balmforth *et. al.,* 2006).

2.9 FEH Statistical with revised equation for Q_{MED}

The creation of improved database because of the HiFlows-UK Project and the feedbacks from users of FEH lead to the launch of this method. A total of 602 rural catchment in the UK were used in the development of this method, and it is applicable to catchments greater than $0.5km^2$ (Kjeldsen, Jones & Bayliss, 2008).

The method brought, unto the FEH, the following key improvements:

- A new regression model for estimating the median annual maximum flood (*QMED*) at ungauged catchments,
- An improved procedure for the use of donor catchments for estimation of *QMED* at ungauged catchments, and,
- An improved procedure for formation of pooling groups and estimation of pooled growth curves.

The method also introduced new catchment descriptors, and a technique of weighting donor catchments using geographical distance.

The final model for prediction of *QMED* at ungauged sites is given by (Kjeldsen *et. al.,* 2008):

$$QMED = 8.3062AREA^{0.851}0.1536^{(\frac{1000}{SAAR})}FARL^{3.4451}0.046^{BFIHOST^2}$$

Details can be found in the Science Report: SC050050 by Thomas R. Kjeldsen, David A. Jones and Adrian C. Bayliss, (2008).

2.10 The Revitalised Flood Hydrograph (ReFH) Model

The Revitalised Flood Hydrograph (ReFH) model has been developed to improve the way that observed flood events are modelled and has a number of advantages over the FSR/FEH unit hydrograph and losses model (Kjeldsen, Stewart, Packman, Folwell & Bayliss, 2005). The key improvements are:

- a new baseflow model which provides a more objective method of separating total runoff into baseflow and direct runoff;
- a loss model based on the uniform Probability Distributed Model of Moore (1985);
- a more flexible unit hydrograph shape;
- improved handling of antecedent soil moisture conditions.

The ReFH model consists of the same three main components as the original FSR/FEH model: a loss model, a routing model and a baseflow model (Kjeldsen *et. al.,* 2005). When simulating a flood event, the loss model is used to estimate the fraction of total runoff turned into direct runoff. The direct runoff is then routed to the catchment outlet using the unit hydrograph convolution in the routing model and, finally, the baseflow is added to the direct runoff to obtain total runoff (Kjeldsen *et. al.,* 2005). In other words the ReFH model transforms a design rainfall event into a design flood. The method requires a number of catchment descriptors as input in order to give outputs as shown below, Fig. 2.5.

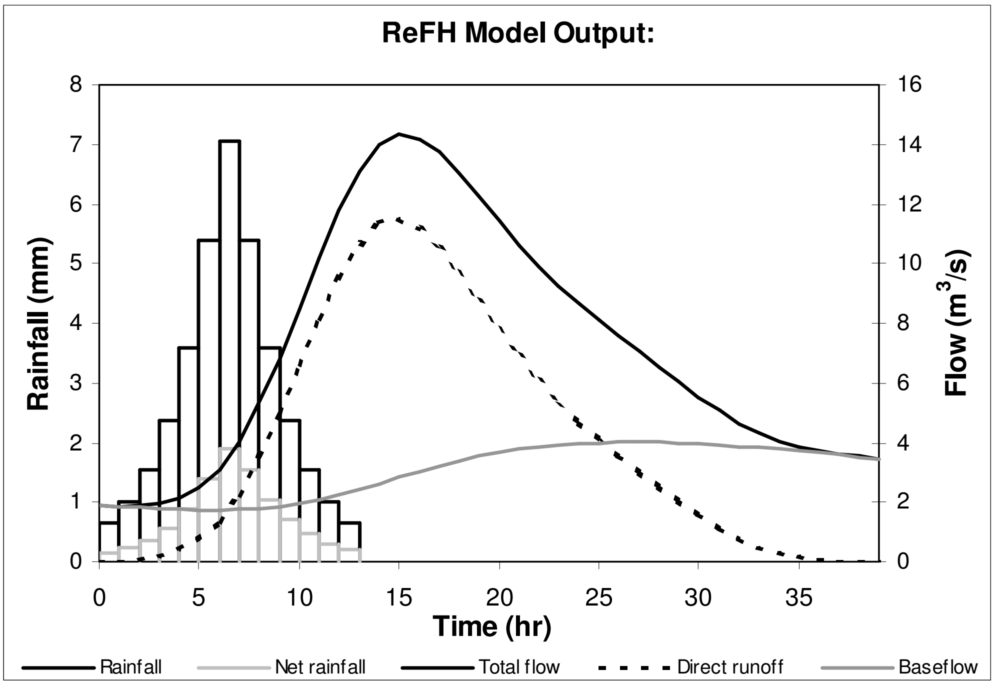

Fig. 2.5: ReFH Method spreadsheet sample output of the MS Excel software

The ReFH Method is considered to provide a more realistic representation of the flood hydrology than that in the FSR/FEH method, where direct runoff and baseflow are treated as independent components. Software in the form of an Excel spreadsheet has been developed to allow implementation of the ReFH design method. This spreadsheet is a user friendly implementation of the method to be applied for design flood estimation in the UK (Kjeldsen *et. al.,* 2005).

It is worth noting that the model was developed and calibrated using data from 101 UK catchments and at catchment areas with magnitude ranging from 3.5km^2 to 511km^2. The supplementary report also explains that the ReFH method can be used for catchment areas 0.5km2 to 1000km^2 (Kjeldsen, 2007). However, there are cautious notes from the Operational Instruction 197_09 (EA 2009) not to use ReFH, firstly, to estimate peak flows on heavily urbanised catchments, i.e. $URBEXT_{1990}$ greater than 0.5. Secondly, the same document advises not to use ReFH to estimate peak flows on permeable catchments, where *BFIHOST* is greater than 0.65 (EA, 2009).

3. Discussion on selection of methods applicable to Ireland

In the following section it is endeavoured to look the pros and cons of the methods in relation to their application on Irish catchments.

3.1 The Rational Method and Modified Rational Method

The most serious drawback of the Rational Method, according to Pitt *et. al.* (2007), is that it gives only peak discharge and provides no information on the time distribution of the storm runoff. It allows no routing of hydrographs through the drainage system or storage structures. Besides, the selection of "C" and "T_c" when choosing "i" in the method is more of judgment than a precise account of the antecedent moisture or a real distribution of rainfall intensity (NY State NPSC and SWG, 2004). Modifications of the Rational Method have also similar limitations.

Furthermore, according to American Public Works Association Special Report no. 43 (cited in NY State NPSC and SWG, 2004) and Pitt *et. al.* (2007), use of the Rational Method should be limited to drainage areas of less than $0.08km^2$ (8ha). While ASCE (1992) advocates that the method is not recommended for drainage areas larger than $0.81km^2$ (81ha). In the majority of literatures the magnitude of "small catchment" is not as large as $25km^2$, the figure commonly quoted in Ireland.

The Canadian Association of Transportation (2004) advises the method to be used for return periods of 5 to 10 years, mainly for design of small culverts and very small bridges and best restricted to small areas of relatively low permeability. Even though there are differences from region to region where to apply the method, there is an overall agreement that it is a crude approach which requires cautious judgment.

On the positive side, the method is easy to apply and can provide a rough first value especially in small uniform urban areas (Shaw, 2004). The inputs required; drainage area, land use, soil type, rainfall intensity for selected return period, and the distance and elevation between the remotest point and the point of interest of the catchment are available. Runoff coefficient, and in some instances Manning's coefficient, are also available in hydrology/hydraulics books. However finding runoff coefficients for the number of sites which will be examined in this research is practically cumbersome. Hence it is not a primary choice.

3.2 The USGS Regression Equations

The U.S. Geological Survey regional regression equations were developed to estimate peak discharges for storms of a given recurrence interval. The FSR, FSU (WP2.2) and FEH-statistical *Qmed* equations were also developed in the same procedure. The difference is while in Ireland there is one regression equation for the whole country, in the case of the US there is regression equation for each hydrologic area. A hydrologic area could be a single catchment or combination of catchments with similar hydrologic, meteorologic and physiographic characteristics.

The similarity in the approaches is that the catchment descriptors used in the development of the US regression equations are also mostly available in Ireland. Perhaps this approach could be an option leading to the introduction of more refined peak flow estimation method.

3.3 The NRCS – TR55 Method

The TR-55 method follows two procedures: graphical discharge method and tabular hydrograph method. Both methods input requirements are time of concentration, drainage area, 24-hr rainfall distribution type, curve number and hydrologic soil conditions. An investigation is required whether there is an Irish equivalent to rainfall distribution type and hydrologic soil groupings.

The advantage of this method is, it is developed to estimate runoffs and peak flows from mainly small catchments with emphasis to urban and urbanizing catchments. It also is accompanied with computer software package. The disadvantage of this method is the hydrological data used to generate the curve numbers are from the US. No curve number or equivalent is introduced in Ireland. Hence, there is less interest in this option.

3.4 The NRCS Dimensionless Unit Hydrograph Method

The NRCS dimensionless unit hydrograph (NRCS DUH) is a hydrograph developed to represent several unit hydrographs; plotted using the ratio of the basic units time to peak and peak rate (USDA-NRCS, 2007). The NRCS DUH plots a triangular hydrograph to practically represent excess runoff with one rise, one peak, and one recession. Its geometric makeup can be easily described mathematically, which makes it very useful in the processes of estimating discharge rates. The main input variables required for the computation are drainage area, volume of direct runoff, time of concentration, duration of unit excess rainfall and peak rate factor.

The method is not difficult to apply on Irish catchments for most of the necessary input variables are readily available. However, examination and determination of Peak Rate Factor for Ireland catchment will be required. Thus this option could be constrained.

3.5 The FSSR 6, 3-variable Method and Institute of Hydrology Report 124

Both the above methods developed for UK catchments have been in use in Ireland since their publication (O'Sullivan *et. al.,* 2010). The IoH 124 is relatively recent compared to the FSSR 6, while the FSSR 6 is now replaced by newer methods such as FEH. The reservation against IoH 124 is that the catchment descriptors *SOIL* and *SAAR* were not represented proportionally in the number of catchments used during the research that lead to the report (Marshall & Bayliss, 1994).

The catchment descriptors used by both methods to estimate peak flows are also available for Irish catchments. Balmforth *et. al.* (2006) recommends FSSR 6 for use for catchment magnitudes between 0.05 to 20km^2. The same report also suggests using IoH 124 for catchments up to 25km^2 and the CIRIA culvert design report (Balkham *et. al.,* 2010) recommends it more for small rural (Greenfield) catchments. Thus, it is worth examining this two methods on existing gauged stations in Ireland and compare the results with all the methods described in this report.

3.6 TRRL and ADAS 345

The ADAS 345 method, a precursor to TRRL 565 method, is widely used both in Ireland and the UK. The TRRL method predates the FSR method and is mainly applied on clay soil type. Similarly, the ADAS 345 was developed for designing pipes for agricultural drainage systems. The National SUDS working group in the UK recommends the use of ADAS 345 for catchment sizes up to 0.5km^2 (Balkham *et. al.*, 2010). The descriptors *SOIL* and *SAAR* in ADAS weren't proportionally represented in the project which led to develop the method. However, it is worth experimenting the method on Irish catchments.

3.7 FEH Statistical with revised equation for Q_{MED}

This method is developed on UK catchments and is relatively new. It is applied on catchments of the size greater than 0.5km^2 and is recommended up to 25km^2. Although there is a need for verification, non official reports indicated conflicting concerns that on one hand this method may not be appropriate for heavily urbanized catchments due to drainage network effects. On the other hand that it produces results close to the actual measured *QMED*. Even though it doesn't take in to account arterial drainage, it is an option worth trying on Irish catchments.

3.8 The Revitalised Flood Hydrograph (ReFH) Model

The ReFH method is recommended for calculating a design hydrograph (Kjeldsen, 2007). It has been calibrated to provide estimates that are broadly consistent with statistical estimates of peak flow for return periods of around 100 years (Balkham et. al., 2010). The model is freely available in a spreadsheet application format and it requires obtaining FEH digital catchment data. Finding DDF model parameters (C, D_1, D_2, D_3, E and F) for Irish catchments could be a hindrance. There is also some reservation among few consultants not to use it on highly permeable (BFIHOST >0.65) catchments, and for it is only calibrated on only seven catchments. Until the DDF model parameters for Irish catchments are available, this method won't be experimented.

3.9 Methods Selected

According to the above discussions it is plausible to examine the following methods on Irish data:

- FSSR 6 (3 variable),
- IH 124,
- TRRL (Young & Prudhoe) method,
- ADAS 345,
- FEH Statistical, and,
- FSU methods (similar to USGS Regression Equation).

4. Application of Selected Methods

In order to carry out the tests Amax series data for catchments up to 30km^2 were collected from OPW and EPA hydrometrics sections. Physical catchment descriptors were also gathered.

4.1 Data Collection and Screening

The 2011 Hydrometric register (EPA) comprehensive data set was used as a starting point. The list included EPA and OPW stations and few from Northern Ireland. In the first phase gauging stations at catchment areas larger than 50km^2 were collected. Then stations installed at out fall of reservoir/lakes, treatment plants and also tidal ones were removed. In the second phase older stations with less than 7years record length were also removed. Then stations located in the same stream were checked, and only one was kept to avoid double account. The effect of arterial drainage was also investigated and hydrometric years coinciding with drainage period were removed. A total of 84 stations were listed with area of ≤50km^2 and a minimum of 5years record length. After annual maximum series (Amax) flows for the screened stations was checked, there left 41 stations with area ≤30km^2 or 37 stations with area of ≤25km^2 or 16 stations with area ≤10km^2, all having a minimum of 7years record length. Physical catchment descriptors for each station were determined from FSU databases. The flow estimation methods selected above were applied on to these three categories of candidate stations.

4.2 Catchment Descriptions

The final dataset of 41 gauging stations consists a catchment as small as 2.8km^2 and as large as 28.6km^2, the average being 13.3km^2. About 16 of these stations have area of $\leq$10km^2 and 37 of them have area of $\leq$24.3km^2.

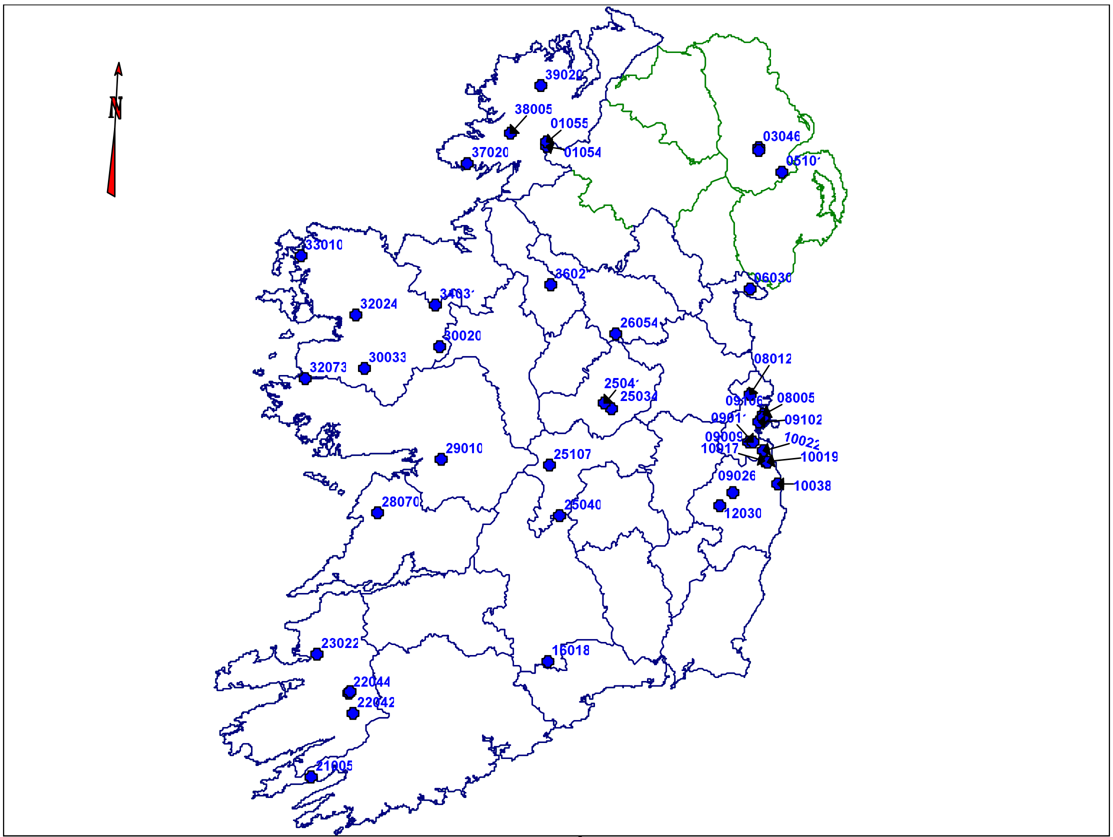

Fig. 4.1: The 41 gauging stations with less than 30km^2 included in the study.

The slope in these catchments varies between 1.2 to 90.1m/km, the average being 26.1m/km. More than 60% of the stations have slope of <30m/km. Only four stations have slope $\geq$80m/km.

The *SAAR* among the 41 candidate stations vary between 475 and 2583mm. Nearly all of the 41stations, except 2, have *SAAR* between approximately 500 and 2000mm, and 70% of them have *SAAR* between 500 and 1500mm.

The *FARL* index range is from 0.63 and 1.00, where 1.00 indicates of no attenuation within the catchment. About 63% of the stations have *FARL* index of 1.00.

BFIsoil index values vary among the 41 stations between 0.28 and 0.72, the average being 0.51. More than half of the catchments are above the medium range.

About 25 of the station have *URBEXT* value of zero and the remaining 16 stations have between 0.42% and 68.33%. Over 90% of the stations have an *URBEXT* value of $\leq$0.025 which makes them predominantly rural or greenfield catchments.

4.3 Data Analysis

Each method were used to estimate peak flow and then converted to *Qmed* at each station. *Qmed* estimated was compared to *Qmed* computed from Amax series. The test results are tabulated below.

FSSR-6, IoH124, ADAS, TRRL and FSU-7variable methods overestimated by up to 500% and underestimated between by up to 90% at nearly half of the 41 stations. FEH-statistical overestimated by up to 170% and underestimated by up to 95% at 17 of the 41 stations, while FSU-3variable overestimated by up to 185% and underestimated by up to 85% at 15 of the 41 stations. The first five methods hugely overestimates compared to FEH and FSU-3v methods, while all methods tend to underestimate to the same degree.

The seven models applied on the first category of 41stations, with catchment area up to 30km^2 and minimum record length of 7years, were evaluated through quantitative statistical tests. The R^2 values of FSSR-6, IoH124 and TRRL shows a linear relationship between *Qmed* estimated from Amax termed as *Qmed* observed from this point on and *Qmed* predicted, though not strong enough. However all the models produce higher factorial standard errors, all above 3.00.

Table 4.1: Outputs of statistical measures for 41 stations

Flow (*Qmed*) estimation methods (≤**30**km^2 & ≥7yrs)	Coeff. of determination (R^2)	SE of estimate	RMSE	NSE	Mean Bias	FSE
FSSR_6	0.548	18.727	0.395	0.315	-0.307	3.570
IoH 124	0.539	19.302	0.406	0.272	-0.204	3.617
ADAS 345	0.544	17.729	0.423	0.386	-0.387	3.486
TRRL (Young & Prudhoe)	0.547	18.255	0.380	0.349	-0.656	3.530
FEH – Statistical	0.359	20.141	*0.361*	0.207	*0.055*	3.684
FSU (7 variables)	0.318	19.573	0.402	0.252	-0.363	3.639
FSU (3 variables)	0.488	20.831	*0.369*	0.152	*0.154*	3.739

Examining the tests and scatter plots (Fig. 4.2a), six outlier stations were noted and removed from the dataset. Further analysis showed that there is significant improvement in the values of R^2, *RMSE*, *mean bias* and *fse* as shown on Table 4.2 (Fig. 4.2b) below. Accordingly, the FEH-Statistical and FSU-3v methods seem to perform better than the rest. It shows that there is a strong linear relationship between *Qmed* observed and *Qmed* predicted. The R^2 values indicate that more than 67% of the *Qmed* observed can be explained by the FEH-Statistical and FSU-3v models.

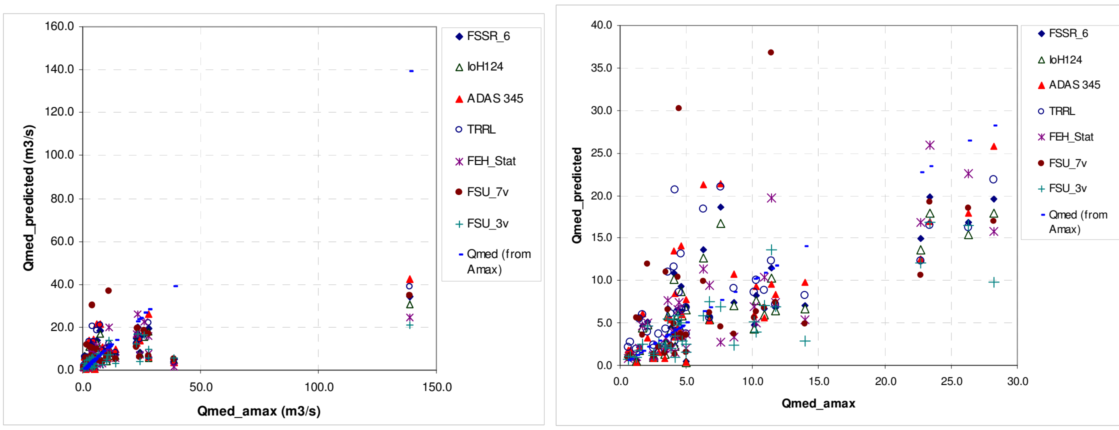

Fig. 4.2 (a, b): Predicted *Qmed* versus *Qmed* from Amax; (a) for 41 stations and (b) for 35 stations.

Table 4.2a: Outputs of statistical measures for 35 stations

Flow (*Qmed*) estimation methods ($\leq$**30**km^2 & $\geq$7yrs)	Coeff. of determination (R^2)	SE of estimate	RMSE	NSE	Mean Bias	FSE
FSSR_6	0.628	4.550	0.313	*0.622*	-0.193	1.931
IoH 124	0.629	4.708	0.327	*0.595*	-0.098	1.960
ADAS 345	0.502	5.438	0.357	0.460	-0.279	2.086
TRRL (Young & Prudhoe)	0.484	5.381	0.292	0.471	-0.594	2.077
FEH – Statistical	*0.706*	4.159	*0.254*	*0.684*	*-0.007*	*1.857*
FSU (7 variables)	0.223	8.042	0.367	-0.18	-0.489	2.473
FSU (3 variables)	*0.673*	5.274	*0.285*	0.492	*0.164*	2.059

Furthermore, stations with record length lesser than 10years were removed. The values of R^2 for both FEH-stat and FSU-3v improved and their performance was enhanced, while R^2 decreased for the rest of the methods to below 0.60, see Table 4.2b. But the number of stationed was reduced to 28, too small to be representative. Hence, the analysis was pursued with 35 stations. At this stage it sounds that FEH-statistical and FSU-3varibale methods are better compared to the rest.

Table 4.2b: Outputs of statistical measures for 28 stations

Flow (*Qmed*) estimation methods ($\leq$**30**km^2 & $\geq$10yrs)	Coeff. of determination (R^2)	SE of estimate	RMSE	NSE	Mean Bias	FSE
FSSR_6	*0.589*	4.549	0.316	*0.576*	*-0.108*	1.931
IoH 124	*0.593*	4.668	0.334	*0.554*	*-0.014*	1.952
ADAS 345	0.405	5.738	0.367	0.326	*-0.191*	2.136
TRRL (Young & Prudhoe)	0.379	5.655	0.290	0.345	-0.571	2.122
FEH – Statistical	*0.737*	*3.846*	*0.243*	*0.697*	*0.070*	*1.795*
FSU (7 variables)	0.189	8.543	0.364	-0.49	-0.449	2.539
FSU (3 variables)	*0.749*	4.638	*0.272*	*0.560*	0.228	1.947

FEH – Statistical equation:

$$QMED = 8.3062 AREA^{0.851} 0.1536^{\left(\frac{1000}{SAAR}\right)} FARL^{3.4451} 0.046^{BFIHOST^2}$$

FSU – 3variables equation:

$$Qmed = 0.000302*(AREA^{\,0.829})*(SAAR^{0.898})*(BFI^{-1.539})$$

Both methods were further scrutinised. FEH-Statistical wasn't performing well at stations with steep slope. Similarly, the FSU-3v wasn't performing well at catchments where there is lake or water storage, and under-predicting at mid-range grade of slope and over-predicting at lower slope catchments. This could be that the FEH equation doesn't take in to account slope (*S1085*) and the FSU equation doesn't take into account *FARL* and *S1085*. See Fig. 4.3 *a* to *h*.

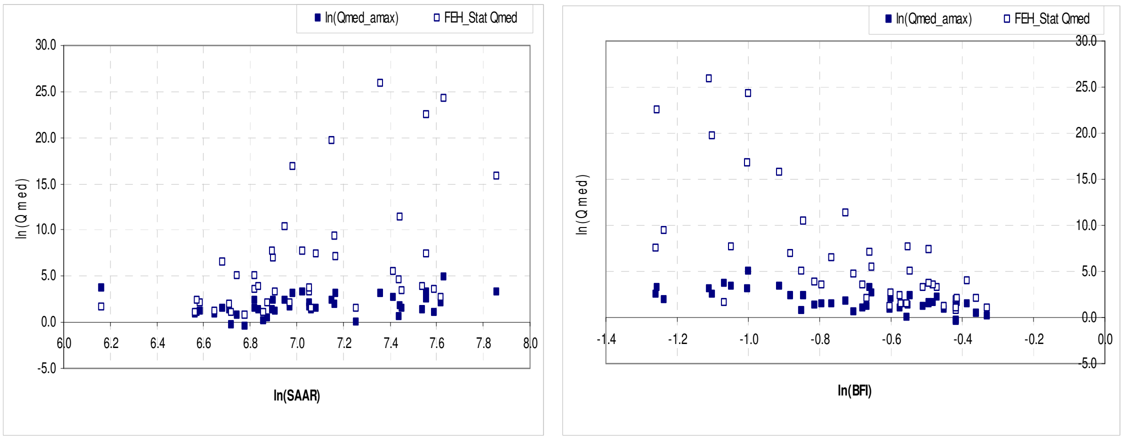

Fig. 4.3 (a and b): Trend of *Qmed* estimated with FEH-Statistical with values of *SAAR* and *BFIsoil* index at 41stations.

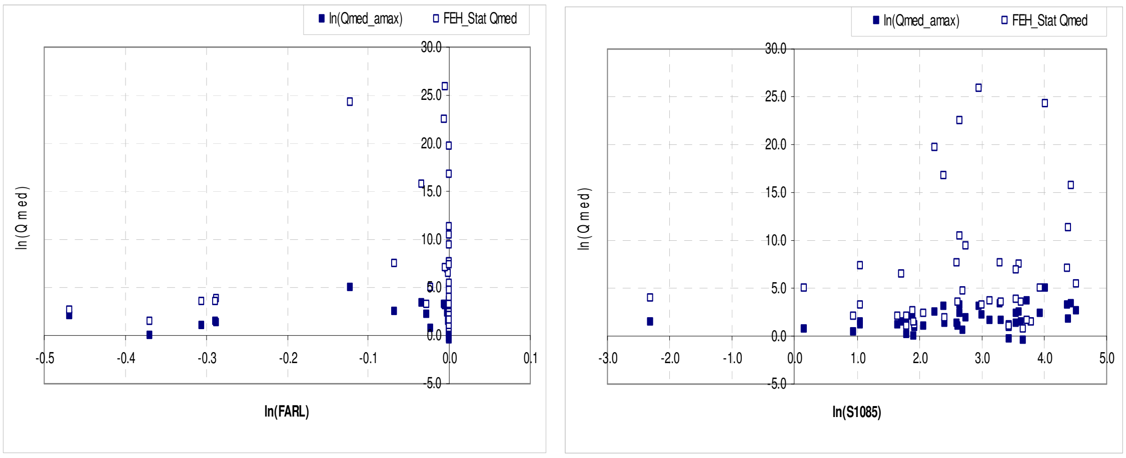

Fig. 4.3(c and d): Trend of *Qmed* estimated with FEH-Statistical with values of *FARL* and *S1085* at 41stations.

Looking at the above plots (a, b, c and d) FEH statistical tends to hugely overestimate *Qmed* at values of *SAAR* greater than 940mm [*ln*(6.85)] and at values of *BFI* less than 0.5 [*ln*(-0.65)]. Similarly, it overly estimates *Qmed* for values of *FARL* equal to 1 [*ln*(0.0)], at catchments with no lake. It is not easy though to explain its performance in relation to slope. It seems to overestimate at slopes between 10 and 80m/km.

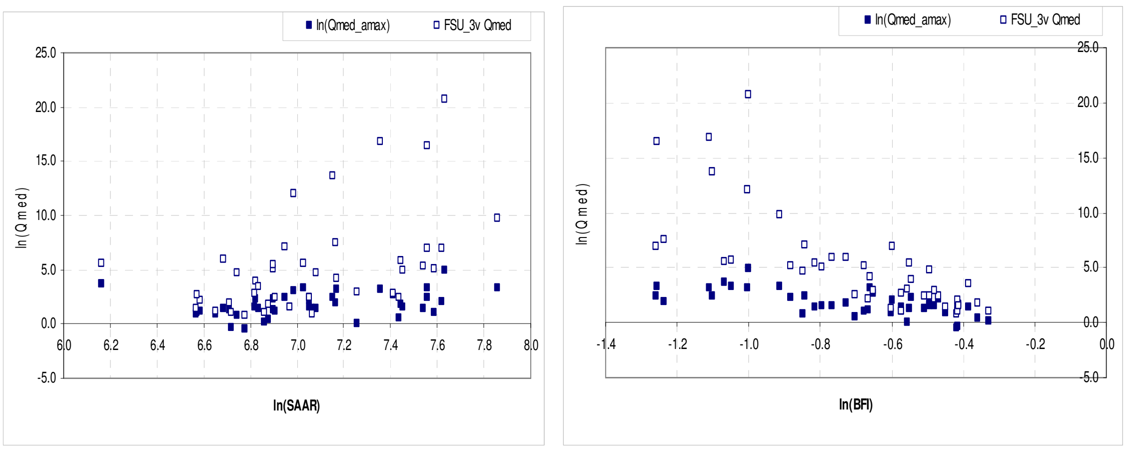

Fig. 4.3 (e and f): Trend of *Qmed* estimated with FSU-3variable equation with values of *SAAR* and *BFIsoil* index at 41stations.

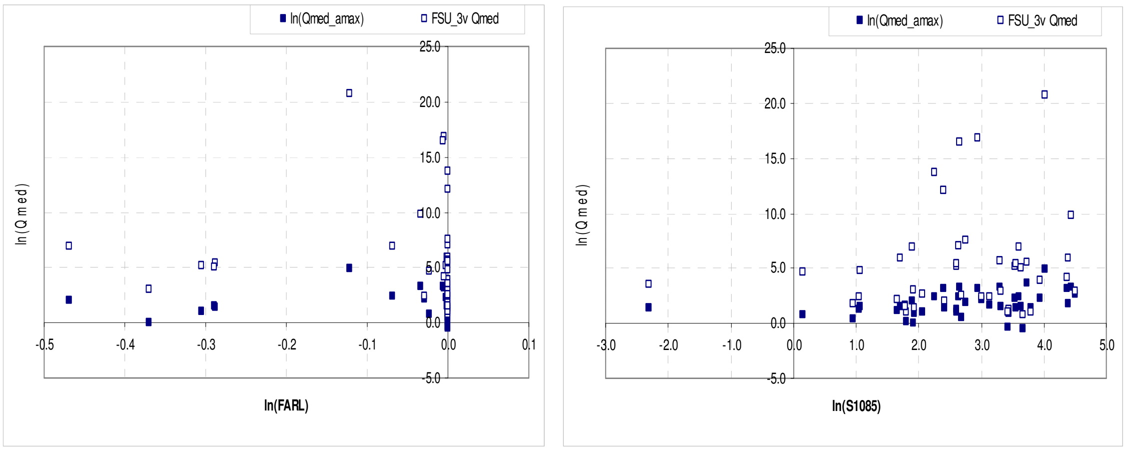

Fig. 4.3 (g and h): Trend of *Qmed* estimated with FSU-3variable equation with values of *FARL* and *S1085* at 41stations.

The behaviour of the FSU 3varibale equation is also similar to the FEH method with regard to *SAAR*, *BFI*, *FARL* and *S1085*. However the degree of overestimation of the FSU method is lesser than that of FEH (see plots e, f, g and h). It was therefore envisaged to develop a new regression equation which takes in to account *S1085* and *FARL* in addition to the *PCDs* already contained in the two methods.

Table 4.3a: Outputs from regression analysis with 41 stations

Coefficients	*Parameter*	*Standard Error*	*t-Stat*	*p-value*
Intercept	-7.7554	2.4816	-3.1252	0.0036
Ln[AREA]	0.9817	0.1888	5.1993	0.0000
Ln[SAAR]	0.7022	0.3631	1.9338	0.0613
Ln[BFI]	-1.5592	0.4469	-3.4889	0.0013
Ln[FARL]	1.7007	1.0294	1.6521	0.1075
Ln[S1085]	0.4098	0.1047	3.9154	0.0004

The regression equation developed with the above parameters and used on 41 stations has R^2 = 0.743, se = 0.622, fse = 1.863; note the p-values for SAAR and FARL.

The statistical measures are compared between the existing methods and the new equation, see Table 4.3b.

Table 4.3b: Comparing the new regression equation with FEH and FSU-3v applied to 41 stations

Flow (*Qmed*) estimation methods (≤**30**km^2 & ≥7yrs)	Coeff. of determination (R^2)	SE of estimate	RMSE	Mean Bias	FSE
FEH – Statistical	0.360	20.14	*0.361*	*0.055*	3.68
FSU (3 variables)	0.488	20.83	*0.369*	*0.154*	3.74
FSU4.2a’ (new)	0.683	*15.82*	*0.250*	*-0.174*	*3.32*

Looking at Table 4.3a, the *p*-values for *SAAR* and *FARL* are above the critical value of 0.05 significance level, a cut off point usually adopted in statistical analysis. A *p*-value of 0.1075 (approx 0.10) tells that there’s a 1 in 10 chance that *FARL* might not explain well the estimated value of *Qmed*. The same concept applies to the *p*-value of *SAAR*. However it has to be noted that significance tests do not usually tell us whether the difference is of practical importance. The regression equation might require further investigation for more improvement. Removal of the same outlier stations actually improves the regression, see Table 4.4a.

Table 4.4a: Outputs from regression analysis with 35 stations when outlier stations removed

Coefficients	*Parameter*	*Standard Error*	*t-Stat*	*p-value*
Intercept	-10.7733	2.6551	-4.0576	0.0003
Ln[AREA]	0.9245	0.1796	5.1469	0.0000
Ln[SAAR]	1.2695	0.4013	3.1637	0.0036
Ln[BFI]	-0.9030	0.4390	-2.0569	0.0488
Ln[FARL]	2.3163	0.9497	2.4390	0.0211
Ln[S1085]	0.2513	0.1007	2.4965	0.0185

The FSU4.2a regression equation redeveloped with the above parameters used on 35 stations has $R^2 = 0.758$, *se* = 0.521, *and fse* = 1.684.

The final form of the FSU4.2a regression equation is as follows:

$$Qmed = (2.0951*10^{-5})*(AREA^{0.9245})*(SAAR^{1.2695})*(BFI^{-0.9030})*(FARL^{2.3163})*(S1085^{0.2513})$$

Table 4.4b: Comparing the new regression equation with FEH and FSU-3v applied to 35 stations.

Flow (*Qmed*) estimation methods (≤**30**km² & ≥7yrs)	Coeff. of determination (R^2)	SE of estimate	RMSE	Mean Bias	FSE
FEH – Statistical	0.706	4.16	0.254	-0.007	1.857
FSU (3 variables)	0.673	5.27	0.285	0.164	2.059
FSU4.2a (new)	0.800	3.33	0.213	-0.145	1.686

The equation has R^2 value of 0.80, a *se* below 5.50 and *fse* of less than 2.50 when outlier stations are removed. The new equation, compared to FEH –Statistical and FSU-3v methods, it over estimates Q_{med} to a lesser degree, by up to 167% (as opposed to 185%) and underestimates by up to 83% (as opposed to 85%) at 13 (as opposed to 15) of the 41 stations. Thus, while it takes more *PCDs*, it is performing well.

The regression equation can take in to account urban extent within a catchment in the same manner as in the FSU method with seven variables.

$$Q_{MEDfinal} = Qmed\,(1 + URBEXT)^{1.482}$$

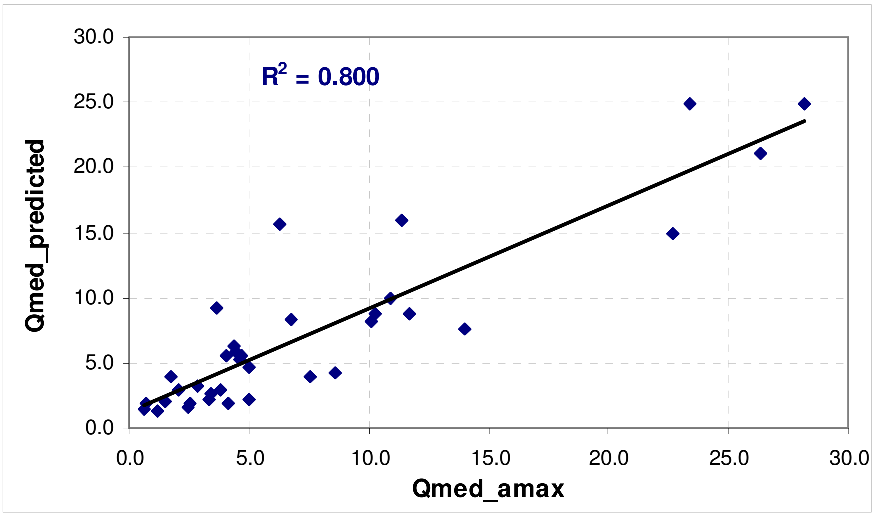

Fig. 4.4: Plot of Qmed from Amax series versus *Qmed* estimated with FSU4.2a equation.

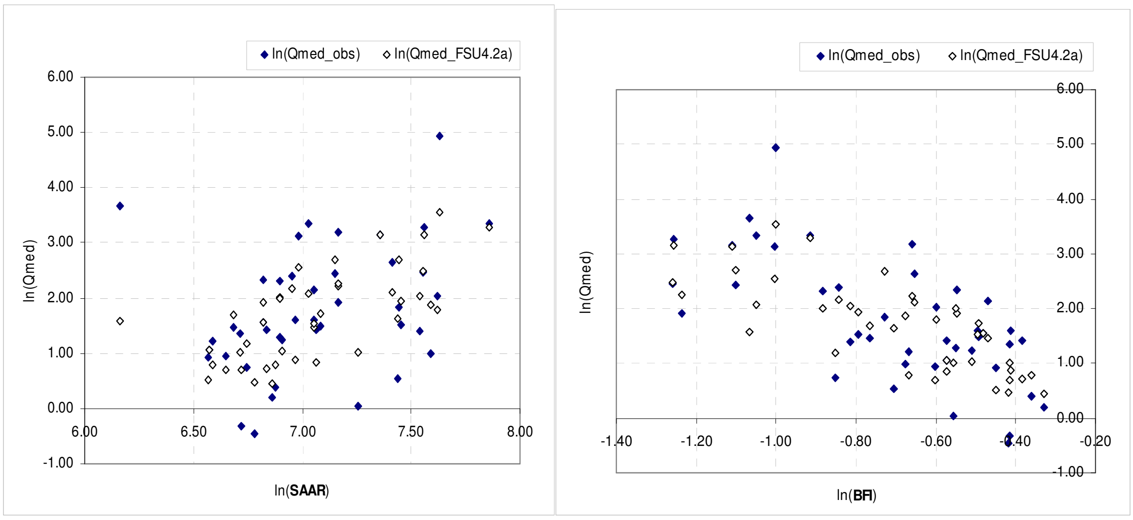

Fig. 4.5 (a and b): Trend of *Qmed* estimated with FSU4.2a regression equation with values of *SAAR* and *BFIsoil* at 41stations.

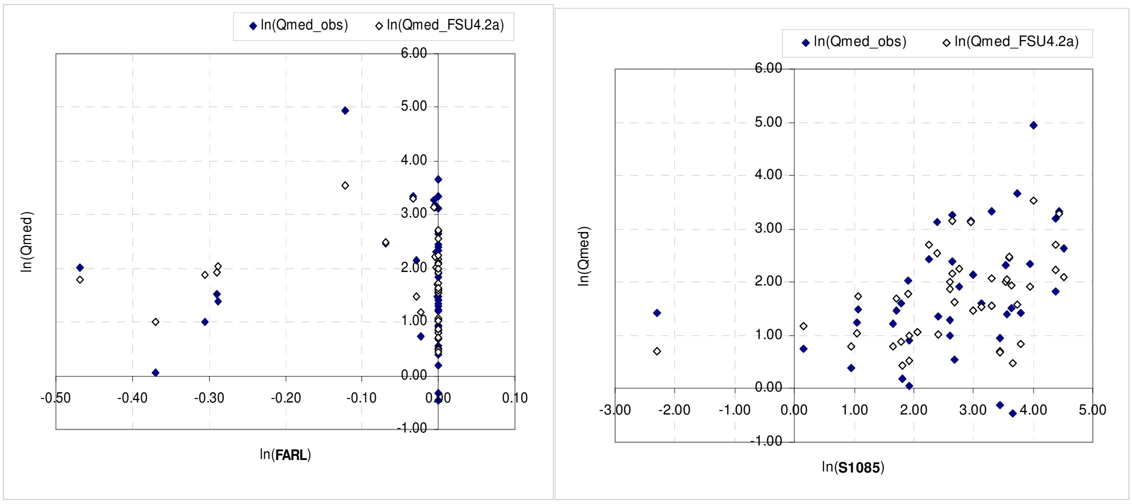

Fig. 4.5 (c and d): Trend of *Qmed* estimated with FSU4.2a regression equation with values of *FARL* and *S1085*at 41stations.

Examining the above four plots, although there is an overall underestimation by the new regression equation, it follows the trend of *Qmed* from Amax series unlike FEH-statistical and FSU-3variable equations.

5. Discussion and Findings

The first set of data comprises 41 stations with record lengths between 7 and 32years, the average being 18years. Their catchment area varies between 2.80 to 28.63km^2 and *Qmed* from Amax varies between 0.63 to 139.00m^3/s, the average being 13.32km^2 and 12.16m^3/s respectively.

The traditionally used IoH124 method overestimates by 509% and underestimates by up to 90% when applied to the 41 stations. Although FSSR-6, TRRL and IoH124 were the only methods where the R^2 value was >0.50, there were a couple of outlier stations which needed to be removed.

When Stations 01055, 16018, 21005, 23022, 28070 and 30033 were removed from the data set, R^2 values for FEH-Statistical and FSU-3v improved to 0.71 and 0.67

respectively and 0.63 for both IoH124 and FSSR-6. The number of stations stands now at 35. The *RMSE* and *Mean Bias* value for FEH-Statistical were 0.25 and -0.01 and that of FSU-3v were 0.28 and 0.16.

Further investigation led to removing stations with less than 10years of record length, thus the number of stations reduced to 28. Consequently, the values of R^2 for both FEH-stat and FSU-3v improved, while it decreased for the rest of the methods to below 0.60. The *RMSE* and *mean Bias* of FEH-statistical were 0.24 and 0.07 and that of FSU-3v were 0.27 and 0.23, thus, no significant change. Although the FEH-statistical and FSU-3v methods were better they still overestimated by up to 185% and underestimated by up to 95%. This indicated there is more room for improvement. Thus, a new regression equation was developed within reasonable statistical measures. The new regression equation FSU4.2a performs better than FEH-statistical and FSU-3v. Besides it is developed employing Irish Amax series data and PCDs.

6. Conclusion and Recommendations

OPW runs nearly 430 stations, and over 200 stations are run by EPA (local authorities). Only 41 stations were found to have the parameters required to carry out urban and small catchment flow estimation analysis, though the data quality from some of them are still questionable. The number of small catchments is dwindling perhaps for budget reason. This is one aspect that has to be reconsidered, i.e. to keep some gauging stations running and maintain data quality for the sake of research.

Seven existing methods were investigated. Some of the methods overestimate hugely, including IoH124 which is widely used in Ireland. FEH-statistical and FSU-3v overestimate and underestimate moderately relative to the rest of methods, and are currently the better ones according to this research. As an option a new regression equation was developed taking into account five variables, *AREA*, *SAAR*, *BFI*, *FARL* and *S1085* to overcome the shortcomings of the two later methods. The results from the new method are encouraging. However it needed to be tested rigorously at more gauging stations with good quality data before it is released for use.

The FSU-3varibale method wasn't developed with small catchment in mind and hasn't been tested. Similarly, the FEH-statistical wasn't particularly developed for small catchments, though it showed promising results compared to the traditionally used methods (Faulkner *et. al.* 2011). It was also developed with UK catchment characteristics and it hasn't been tested in Ireland. The onus is to strengthen the new regression equation through testing, perhaps by erecting more new project gauging stations or revisiting existing non-functional gauges at small and urban catchments.

7. Abbreviations

PCD:	physical catchment descriptors,
AREA:	catchment area,
SAAR:	standard period average annual rainfall, (usually from 1961 to 1990),
BFIsoil:	base flow index for soils,
FARL:	flood attenuation by reservoirs and lakes,
FLATWET:	index of catchment wetness,
DRAIND:	drainage density,
S1085:	mainstream slope,
ARTDRAIN2:	percentage of the catchment river network that is included in the drainage schemes,
URBEXT:	index of urban extent,
SOIL:	soil index or winter rain acceptance potential,

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