Runoff Routing Methods

xpswmm and xpstorm uses seven major types of Hydrograph Generation techniques available in Runoff.

### SWMM Runoff Non-linear Reservoir Method

Subcatchments are modeled as idealized rectangular areas with the slope of the catchment perpendicular to the width. Each subcatchment is classified into 3 or 4 subareas as indicated in the following table.

<table>
<thead>
<tr>
<th>Subarea</th>
<th>Perviousness</th>
<th>Depressions</th>
<th>Snow Cover and Extent</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Impervious</td>
<td>Yes</td>
<td>Bare</td>
</tr>
<tr>
<td>A2</td>
<td>Pervious</td>
<td>Yes</td>
<td>Constant</td>
</tr>
<tr>
<td>A3</td>
<td>Impervious</td>
<td>No</td>
<td>Bare</td>
</tr>
<tr>
<td>A4</td>
<td>Impervious</td>
<td>Yes</td>
<td>100% covered</td>
</tr>
</tbody>
</table>

Flow from each sub-area moves directly to a node and does not travel over any other sub-area. The width of the pervious sub-area, A2, is the entire subcatchment width, whereas the width of the impervious sub-areas A1, A3, A4 are in proportion to the ratio of their area to the total impervious area. Subcatchments are analyzed as spatially lumped non-linear reservoirs. The routing is performed separately for each of the four sub-areas (A1-A4) of the catchment.

### Kinematic Wave (Runoff)

The kinematic wave method for overland flow applies only the kinematic wave component of the St. Venant shallow flow equations for momentum and continuity. Similar to the Runoff procedure the subcatchments are modelled as idealised rectangular areas with the slope of the catchment perpendicular to the width. The width parameter is used as per the Runoff procedure. Both the Runoff and Kinematic wave procedures only apply either the Horton or Green Ampt loss to the pervious percentage of the subarea, as defined by the % impervious data item. The only loss applied to the impervious component is via the depression storage defined for the impervious area in the infiltration dialog. Additional depression loss (defined with infiltration information) can also be applied to the pervious component. This will be an additional loss to the Horton or Green Ampt loss.

Henderson and Wooding (1964) and Wooding (1965) developed a theory for the overland and stream hydrograph. All flows are assumed to obey the equations of continuity and momentum. The continuity and momentum equations for overland kinematic waves reduce to the following two equations, respectively:

**Continuity Equation:**

\[
\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i - \int = i_e
\]

and

**Momentum Equation:**

\[
q = ay^m = 1.49/N S_o^{1/2} y^{5/3}
\]

*Where:*

- \(y = y(x,t)\) = depth of overland flow (ft),
- \(q = q(x,t)\) = rate of overland flow/unit width,
- \(i-f = i_e\) = net rainfall rate,
- \(a = \text{conveyance factor} = (1/N)S_o^{1/2}\) when obtained from Manning’s equation,
- \(m = 5/3\) when obtained from Manning’s equation,
- \(N = \text{effective roughness coefficient},\)
\[ S_o = \text{average overland slope}, \]
\[ Y_o = \text{mean depth of overland flow}. \]

*After Bedient and Huber, 1992*

The required data for this method is similar to the EPA Runoff method including area (ha or acres), impervious %, subarea width (m or ft) and slope (m/m or ft/ft).

**Laurenson**

![Laurenson's Method: Node MH-04](image)

When using Laurenson Hydrology the subcatchment width is by default not used. The catchment roughness utilised when calculating the storage delay parameter “B” is taken from the pervious Manning “n” value for the sub-catchment included with the infiltration information. A second method is available by including the Configuration parameter RAFTS under the Configuration menu. This allows % imp to be set to 0 and 0 in the first two separate sub areas. This will allow the infiltration equation to be applied to the total subareas. Width is then used as a factor to control the “B” computation by inputting 0.0 and 100.0 respectively for the fully pervious and impervious sub areas. Appropriate infiltration can then be applied to the respective subareas.

The default procedure (without the RAFTS configuration parameter active) only applies either the Horton or Green Ampt loss to the pervious percentage of the subarea, as defined by the % impervious data item. There is currently no loss applied to the impervious component as the depression storage defined for the impervious area in the infiltration dialog is not active with either the Laurenson or time/area runoff procedures.

Routing for a particular subcatchment is carried out using the Muskingum procedure. The storage, however, is a non-linear function of the discharge, ie:

\[ s = K(q) \times q \quad \text{(equation 3)} \]

**Where:**

- \( s \) = volume of storage, (hrs \( \times \) m\(^3\)/s)
- \( q \) = instantaneous rate of runoff, (m\(^3\)/s)
- \( K(q) \) = storage delay time as a function of \( q \) (hours).

The storage function is used in the continuity equation in finite difference form thus:

\[
(i1 + i2) \times t/2 - (q1 + q2) \times t/2 = s2 - s1 \quad \text{(equation 4)}
\]

**Where:**

- \( i1, i2 \) = inflow at beginning and end of routing period (m\(^3\)/s)
- \( t \) = routing interval (hrs)
- \( q1, q2 \) = outflow from the storage at beginning and end of routing period (m\(^3\)/s)
- \( s1, s2 \) = storage volume at beginning and end of routing period (hrs \( \times \) m\(^3\)/s).

Substituting \( s2 \) and \( s1 \) in Equation (4) from Equation (3) gives:

\[ q2 = C0i2 + C1i1 + C2q1 \quad \text{(equation 5)} \]

**Where:**

- \( C0, C1, C2 \) = constants determined by the system.
C0 = C1 = \frac{t}{(2K2 + t)} \quad \text{(equation 6)}

C2 = \frac{(2K1 - t)(2K2 + t)}{2K2 + t} \quad \text{(equation 7)}

An iterative solution to Equation (5) is required due to the interrelation between C0, C1, C2, K2 and q2. K1, K2 is defined via Equation (8).

**Storage-Discharge Relationship**

Each sub-area is treated as a concentrated conceptual storage. Each storage has a storage delay time described thus:

K(q) = Bq^n \quad \text{(equation 8)}

where:

- K(q) = subarea storage delay time (hours) as a function of q
- q = discharge (m³/s)
- B = storage delay time coefficient
- n = storage non-linearity exponent.

Substituting Equation (8) into Equation (3) gives:

s = Bq^n + 1 \quad \text{(equation 9)}

**Non-linearity Exponent "n"**

The default value for the non-linearity exponent in RAFTS is -.285. RAFTS-XP provides the mechanisms to alter this value, usually in respect to rare events involving significant subcatchment overbank flood routing, by:

- directly entering an amended value of "n". A value of zero (-.001) would indicate linear catchment response and equate with unit hydrograph theory, or
- entering an "n" as a function of Q for each rating curve

It would be expected that the default "n" would prevail up to bank full and then, possibly, progressively drop back towards zero during PMF.

**Coefficients B and n**

B is either directly input for each subcatchment or estimated from Equation (10) which was derived by Aitken (1975). The value of B for each subarea is assumed to equal the average value of B for the subcatchment.

where:

- B = mean value of coefficient B for subcatchment
- A = subcatchment area (km²)
- U = fraction of catchment that is urbanised. (Where U = 1.0, the catchment is fully urbanised and when U = 0.0, the catchment is completely rural)
- Sc = main drainage slope of subcatchment (%). (The longest path of the subcatchment, starting at subcatchment outlet running up the main channel then if necessary branching off at the furthest tributary, to the top of the subcatchment.)

This equation was initially derived from six urban catchments in Australia with the following ranges applying:

- A varied from 0.8 km² to 56 km²
- U varied from 0.0 to 1.00
- Sc varied from 0.22% to 2.90%.

However over the last fourteen years a wide range of areas, slopes and urbanisation outside these ranges have been tested with a high degree of success. See Sobinoff et al (1983).

For gauged catchments, deduced B values, evaluated as the average value from recorded rainfall/runoff events, should be used in preference to generalised regression estimates.

As 'U' in certain instances can be rather vague, data input in this respect has been amended to include a % impervious parameter for each subcatchment in place of the 'U' term.
The model interprets $U$ in terms of %I based on the following ratios:

<table>
<thead>
<tr>
<th>%</th>
<th>U</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>30</td>
<td>0.7</td>
</tr>
<tr>
<td>50</td>
<td>1.0</td>
</tr>
<tr>
<td>100</td>
<td>2.0*</td>
</tr>
</tbody>
</table>

* This value is extrapolated from the original data based on limited results from fully impervious areas.

**B Modification Factors**

Where gauged rainfall/runoff data is available for a range of events it should be used in preference to the above regression equation with modifying factors.

**PERN.** The original regression equation (Equation (10)) does not differentiate between catchments with the same urbanisation but different roughness. An additional empirical parameter has therefore been added to take pervious subcatchment roughness into account.

The parameter PERN is input as a Mannings 'n' representation of the average subcatchment roughness. B is then modified in accordance with the following table. If PERN is left blank then B is unchanged.

<table>
<thead>
<tr>
<th>Mannings 'n'</th>
<th>Multiplication Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>.010</td>
<td>0.4</td>
</tr>
<tr>
<td>.015</td>
<td>0.5</td>
</tr>
<tr>
<td>.025</td>
<td>1.0</td>
</tr>
<tr>
<td>.100</td>
<td>3.0</td>
</tr>
</tbody>
</table>

**BX.** During calibration of a gauged catchment an additional parameter BX in the header data is included to modify the calculated or input B by a further multiplication factor. The parameter BX will then uniformly modify all subcatchment B values previously computed, or set (Equation (10)). To invoke BX within this program it is necessary to utilise a Configuration Parameter selectable from the Tools menu.

To set a BX to a value of 1.5 for example add the Configuration Parameter BX=1.5.

**SCS Hydrology**

As an alternative to the non-linear runoff routing method employed by Runoff, hydrographs may optionally be generated by the SCS method described in the SCS National Engineering Handbook - Section 4 (NEH-4).
**Pervious Area Curve Number**

The runoff **curve number** as described in NEH-4 Chapter 9. Typical values vary from 20 for regions with high infiltration and interception capacities to 98 for impervious areas.

It is a dimensionless number depending on hydrologic soil group, cover type, treatment, hydrological condition, and antecedent moisture conditions. This number has a valid range from 0 to 100 with typical values from 60 to 90 and 98 for impervious surfaces.

This curve number is applied only to the pervious area. Internally, an area weighted composite number is generated using this number for the pervious area and 98 for the impervious area.

**Hydrograph Shape Factor**

The shape factor or peak rate factor as defined in NEH-4 Chapter 16. The typical value is 484 for a hydrograph where the volume under the falling side of the triangular unit hydrograph is equal to 1.67 times the volume under the rising limb of the curvilinear unit hydrograph. Actual values may vary from 100 in very flat swampy country to 800 in steep rocky terrain.

The Soil Conservation Service has been determined the hydrograph shape factor to be 484 for most watersheds. This was the result of analyzing many watersheds of various size and geographic location. It is used in the formulation of peak discharge and the peak of the unit hydrograph:

\[
Q_p = 484A/tp
\]

in which \( Q_p \) is the peak discharge in cfs, \( A \) is the drainage area mi\(^2\), and \( tp \) is the time to peak in hr. Several other values have been used for shape factor within Florida and other areas of the country for wetlands, sandy soils and steep terrain. Typical values in Florida include 323 or 256 and values as low as 100. Effectively, the hydrograph shape factor will lengthen the time base of the unit hydrograph and lower the peak. If you are using real rainfall data this parameter can be used for calibration. For example the falling limb time base of the hydrograph will be double when using a shape factor of 300 instead of 484.

A curvilinear or triangular shape may be chosen by selecting the appropriate button. Both shapes are supported for the entire range of hydrograph shape factors.

**Time of Concentration**

The time of concentration as described in NEH-4 Chapter 15 (in mins) is defined in two ways; the time for runoff to travel from the furthermost point of the watershed to the point in question, and the time from the end of excess rainfall to the point of inflection on the trailing limb of the unit hydrograph. \( T_c \) can be estimated from several formula such as Kinematic-wave. For a constant excess rainfall can be described as:

\[
T_c = C(n^{0.6} \frac{L^{0.6}}{i^{0.4} S^{0.3}})
\]

in which \( L \) is the distance from the upper end of the plane to the point of interest (usually the inlet) \( n \) is the Manning resistance coefficient, \( i \) is the excess rainfall rate, \( S \) is the dimensionless slope of the surface, and \( C \) is a constant that depends on units of the other variables. For \( tc \) in minutes, \( i \) in inches/hr and \( L \) in feet \( C \) equals 0.938. For \( tc \) in minutes \( i \) in mm/hr and \( L \) in meters \( C \) equals 6.99.

Another formula for determining \( tc \) is the lag equation

\[
T_c = L/0.6
\]

\[
L = I(0.8(S+1)^{0.7}/1900Y^{0.5})
\]

**where:**

- \( T_c \) = time of concentration in hours
- \( L \) = lag time in hours
- \( I \) = hydraulic length of the watershed in feet
- \( Y \) = Average land slope in percent
- \( S \) = Potential maximum retention in inches
- \( CN \) = weighted Curve Number

The time of concentration is required as input although internally the time to peak is used in creating the unit hydrograph. \( T_p = 2/3 \ T_c \) for shape factor 484.

**Calc:** The time of concentration can be calculated using the Time of Concentration Calculator
Initial Abstraction

The initial abstraction from the precipitation may be represented as an absolute number, that is the total depth of precipitation that is less (in mm or inches) or as a fraction of the amount of precipitation (between 0 to 1).

Initial Abstraction, $I_a$, contains all the loss terms prior to beginning of runoff. All though it was found to be highly variable for many small agricultural watersheds it was found that

$$I_a = 0.2 \times S$$

Where

$$S = (1000/CN)-10$$

$S$ is the potential maximum abstraction.

xp allows the user to use a fraction with .2 being the default or the Initial abstraction can be a depth in inches or mm.

Additional References:

James 1998, Editor "Modeling the Management of Stormwater Impacts" Volume 6 Chapter 23 Author Ashok Pandit and Joanie Regan What is the Impervious Area Curve Number. Publisher CHI Guelph Ontario Canada.


Unit Hydrographs

The unit hydrograph and quasi unit hydrograph procedures currently available include:

- Nash
- Rational Hydrograph (Triangular Unit Hydrograph)
- Santa Barbara Urban Hydrograph
- Snyder
- Snyder (Alameda modified)
- Time Area
- Clark

When one of the above methods requires a Time to Peak or Time of Concentration this can be calculated using the Time of Concentration Calculator.

Nash Unit Hydrograph Procedure

Nash in 1957 proposed a conceptual catchment model by considering a drainage basin with a series of identical linear reservoirs in series. By routing a unit inflow through the reservoirs, a mathematical equation for instantaneous unit hydrograph can be derived. Two items of data are required to apply this method. These include an Exponent and a Time of concentration. This method requires calibration data to base input data.

Rational Hydrograph
The Rational method as applied by this method uses a unit hydrograph approach applying a deterministic form of the Rational formula \( Q = CIA \). The two items of data needed for this procedure are the sub-area runoff coefficient “C” and time of concentration \( t_c \). The method utilizes the rainfall concurrent to the time step being computed. The unit hydrograph incorporates a base length equal to two times the time of concentration.

This method is currently not recommended without calibration data or for models with storage nodes and significant attenuation.

This method is NOT recommended as it requires a new Rainfall event for every \( t_c \) in the model and \( t_c \)’s must be calculated by hand. If the Rational method is needed then the Rational Formula method should be used.

**Santa Barbara Urban Hydrograph Procedure**

The SBUH method, like the Soil Conservation Service Urban Hydrograph (SCSUH) method, is based on the curve number (CN) approach, and also uses SCS equations for computing soil absorption and precipitation excess. The SCSUH method works by converting the incremental runoff depths (precipitation excess) for a given basin and design storm hydrographs of equal time base according to basin time of concentration and adds them to form the runoff hydrograph. The SBUH method, on the other hand, converts the incremental runoff depths into instantaneous hydrographs which are then routed through an imaginary reservoir with a time delay equal to the basin time of concentration.

The SBUH method was developed by the Santa Barbara County Flood Control and Water Conservation District, California. The SBUH method directly computes a runoff hydrograph without going through an intermediate process (unit hydrograph) as the SCSUH method does.

The SBUH method uses two steps to synthesize the runoff hydrograph:

Step one – computing the instantaneous hydrograph, and step two – computing the runoff hydrograph.

The instantaneous hydrograph, \( I(t) \), in cfs, at each step, \( dt \), is computed as follows:

\[
I(t) = 60.5 \frac{R(t) A}{dt}
\]

where:

\( R(t) = \) total runoff depth (both impervious and pervious runoffs) at time increment \( dt \), in inches (also known as precipitation excess)

\( A = \) area in acres

\( dt = \) time interval in minutes*

*A maximum time interval of 10 minutes should be used for all design storms of 24-hour duration. A maximum time interval of 60 minutes should be used for the 100-year, 7 day design storm.

The runoff hydrograph, \( Q(t) \), is then obtained by routing the instantaneous hydrograph \( I(t) \), through an imaginary reservoir with a time delay equal to the time of concentration, \( t_c \), of the drainage basin. The following equation estimates the routed flow, \( Q(t) \):

\[
Q(t + 1) = Q(t) + w[I(t) + I(t + 1) - 2Q(t)]
\]

where:

\( w = \frac{dt}{2T + dt} \)

\( dt = \) time interval in minutes

**Snyder Unit Hydrograph Procedure**

Snyder (1938) was the first to develop a synthetic unit hydrograph based on a study of watersheds in the Appalachian Highlands. In basins ranging from 10 to 10,000 mi<sup>2</sup>. Snyder’s relationships are

\[
T_p = \frac{C_p (L_c)^{0.3}}{c_p}
\]  
(equation 1)

where

\( T_p = \) basin lag

\( L = \) length of the main stream from the outlet to the divide,

\( L_c = \) Length along the main stream to a point nearest the watershed centroid,
$C_t =$ coefficient usually ranging from 1.8 – 2.2 ($C_t$ has been found to vary from 0.4 in mountainous areas to 8.0 along the Gulf of Mexico).

$$Q_p = 640C_pA/t_p,$$  
(where 2)

Where:

$Q_p =$ peak discharge of the unit hydrograph

$A =$ drainage area

$C_p =$ storage coefficient ranging from 0.4 to 0.8 where larger values of $C_p$ are associated with smaller values of $C_t$

$T_b = 3 + t_p/8$  
(equation 3)

Where:

$T_b$ is the time base of the hydrograph. For small watersheds, Eq 3 should be replaced by multiplying $t_p$ by a value of from 3 to 5 as a better estimate of $T_b$. Eqs. 1, 2 and 3 define points for a unit hydrograph produced by an excess rainfall of duration $D = t_p/5.5$.

For other rainfall excess durations $D'$, an adjusted formula for $t_p$ becomes:

$$T' = t_p + 0.25 (D' - D)$$  
(equation 4)

Where:

$T'_p$ is the adjusted lag time for duration $D'$. Once the three quantities $t_p$, $Q_p$, and $T_b$ are known, unit hydrograph can be sketched so that the area under the curve represents 1.0 in of direct runoff from the watershed.

In this application the two items of data are:

$C_p =$ Storage coefficient plus

$T_p =$ catchment lag

The percentage of imperviousness is not an input value for the Snyder Runoff method. By design the percentage of imperviousness should be set to 0% for this reason.

Once the percentage is set to 0% it means that the whole catchment will be subject to infiltration. If using the Curve Number method for infiltration, the impervious parts of the catchment can be accounted for by modifying the curve number (CN) that is used for the infiltration accordingly. We recommend a CN value of 98 for the impervious part of the catchment. Users can also check the total infiltration volume in Table R5 of the output file.

**Snyder (Alameda) Unit Hydrograph Procedure - old**

The Snyder (Alameda) method is the Snyder procedure as applied by Alameda County in California where the individual parameters more suited to Alameda region are computed from catchment characteristics.

This method requires four items of data:

- Stream Length (L) mile
- Centroid Length (Lc) mile
- Stream Slope (S) ft/mile
- Basin “N” (roughness)

where

$$\text{Lag} = K \cdot N \left( (L \cdot Lc) / S \right)^{0.38}$$

This data allows the other Snyder parameters of $t_p$, $Q_p$, and $T_b$ to be computed directly.

**Time Area Method**
Time-area methods utilise a convolution of the rainfall excess hyetograph with a time-area diagram representing the progressive area contributions within a catchment in set time increments. Separate hydrographs are generated for the impervious and pervious surfaces within the catchment. These are combined to estimate the total flow inputs to individual sub-catchment entries to the underground urban drain network.

The time-area method dates from the research of Ross in 1922. Networked urban drainage adoptions of the procedure however only date back to 1963. This computerised program known as the TRRL Method was developed by the U.K. Transport and Road Research Laboratory, described by Watkins (1963). In the U.S. Terstriep and Stall (1974) further developed the method to include pervious runoff. In South Africa Watson (1981) made a number of additional changes particularly to the way infiltration was estimated. Between 1982 and 1986 O’Loughlin (1988) using Watson’s model as a basis carried out extensive changes once again to formulate a computerised package known as ILSAX. Over the intervening years to the present the ILSAX model has had wide usage throughout Australia particularly for routine urban drainage design. The sub-catchment runoff estimating procedure still utilises the basic time-area method to estimate both pervious and impervious portion runoff.

In this application the time/area procedure assumes a linear time area relationship for the subarea and is based on an input time of concentration. It is recommended in urban catchments to define two subarea elements one each to represent the impervious and pervious areas respectively. Values for Impervious% should both be set to 0 (zero) so the input infiltration using either Horton or Green Ampt applies to the full defined area of each. Depression storage loss as defined in the infiltration dialog is currently unused.

The only input necessary for this procedure is the time of concentration for the subarea.

**Clark Unit Hydrograph**

Enter:

- \( Tc \), time of concentration, min for the subcatchment
- \( R \), storage coefficient, hr, must be => 0.5 \times \) Runoff time step.

The Clark method (1945) requires three parameters to calculate a unit hydrograph: \( Tc \), the time of concentration for the basin, \( R \), a storage coefficient, and a time-area curve. A time-area curve defines the cumulative area of the watershed contributing runoff to the subcatchment outlet as a function of time (expressed as a proportion of \( Tc \)).

\( xswmm \) utilizes a dimensionless time area curve:

\[
\begin{align*}
A &= 1.414 \ T^{1.5} & \text{for } 0 \leq T < 0.5 \\
1 - A &= 1.414 \ (1 - T)^{1.5} & \text{for } 0.5 < T < 1
\end{align*}
\]

where:

- \( A \) = cumulative area as a fraction of total subcatchment area and
- \( T \) = fraction of time of concentration.

The ordinates of the time-area curve are converted to volume of runoff per second for unit excess and interpolated to the given time interval. The resulting translation hydrograph is then routed through a linear reservoir to simulate the storage effects of the basin; and the resulting unit hydrograph for instantaneous excess is averaged to produce the hydrograph for unit excess occurring in the given time interval.

The linear reservoir routing is accomplished using the general equation:

\[
Q(2) = CA \times I \times CB \times Q(1)
\]

The routing coefficients are calculated from:

\[
CA = \frac{\Delta T}{(R + 0.5 \times \Delta T)}
\]

\[
CB = 1 - CA
\]

\[
Qungr = 0.5 \ [Q(1) + Q(2)]
\]

where:

- \( Q(2) \) = instantaneous flow at end of period,
- \( Q(1) \) = instantaneous flow at the beginning of period,
- \( I \) = ordinate of the translation hydrograph,
- \( \Delta T \) = computation time interval in hours (also duration of unit excess),
- \( R \) = basin storage factor in hours, and
\( Q_{unr} \) = the unit hydrograph ordinate at end of computation interval.

The computation of unit hydrograph ordinates is terminated when its volume exceeds 0.995 inch (mm) or 150 ordinates, whichever occurs first.

Rational Formula Hydrology

### Runoff Coefficient

**Pervious Runoff C.** The Runoff Coefficient will be required if the Direct or Alameda method for calculating C is selected in the Rational Formula Global Database.

**Runoff Curve Number.** The Runoff Curve Number will only be required if AR&R 1977 (the Ordon Curve method) is selected in the Rational Formula Global Database.

### TC Calculation

**Additional Travel Time.** Travel time from outlet of subcatchment to the connecting node (minutes). This option allows for defining odd shaped catchments, or catchments whose flow concentrates at points remote to the node itself. This time does NOT represent the gutter time component of a time of concentration, but the additional time beyond the time of concentration. The typical use of this value is to represent a remote catchment connected by a pipe or drain without explicitly modeling the connecting conduit itself. To allow for gutter travel time, an equivalent length of overland path should be added to the overland flow length.

**Time of Concentration.** Time of Concentration, \( T_c \) can be input as a constant value (minutes) or calculated using one of the equations listed in the TC Calculator. A minimum Time of Concentration value of 5 minutes is customary and required.

**Time of Constant Flow.** The program creates a hydrograph from the rational formula. The duration of the rising and recession limbs of the hydrograph are each equal to the time of concentration. The duration of the period of constant flow is entered in this field (in minutes). To simulate constant flow enter a duration equal to or longer than the Job Control simulation time in this field.

When using the Rational Formula, \( Q = kCIA \), where \( k=1.008 \) for English units and is often omitted from the equation. In SI units, however, \( k=0.00278 \) and should not be omitted from the equation. It is also critical to enter proper values for C and I.

### Time of Concentration Methods

**Friend’s Equation**

\[
T_c = \frac{BnL^{0.333}}{S^{0.2}}
\]
where:

- \( B \) = Const (72 US Units, 107 metric)
- \( S \) = Average catchment slope (Percent, %)
- \( L \) = Length of overland flow (ft, m)
- \( n \) = Horton’s roughness (similar but not identical to Manning’s n)
  - Paved 0.015
  - Bare Soil .0275
  - Poorly Grassed .035
  - Average Grass .045
  - Dense grass 0.06

**Kinematic Wave**

\[
t_c = 0.93 \left( \frac{L^{0.6} n^{0.6}}{i^{0.4} S^{0.3}} \right)
\]

where:

- \( L \) = Length of overland flow (ft, m)
- \( S \) = Average catchment slope (ft/ft, m/m)
- \( i \) = Rainfall Intensity (in/hr, mm/hr)
- \( n \) = Surface roughness (similar but not identical to Manning’s n)
  - Concrete or Asphalt 0.010 - 0.013
  - Bare Sand 0.010 - 0.016
  - Graveled Surface 0.012 - 0.030
  - Bare Clay-Loam Soil (eroded) 0.012 - 0.033
  - Sparse Vegetation 0.053 - 0.130
  - Short Grass Prairie 0.100 - 0.200
  - Lawns 0.170 - 0.480

**Izzard’s Formula**

\[
t_c = \frac{41KL^{0.333}}{t_i^{0.667}}
\]

Where:

\[
K = \frac{0.0007i + C_r}{S^{0.333}}
\]

- \( L \) = Length of overland flow (ft, m)
- \( S \) = Average catchment slope (ft/ft, m/m)
- \( i \) = Rainfall Intensity (in/hr, mm/hr)
- \( C_r \) = Retardance Coefficient
  - Very smooth asphalt 0.007
  - Tar and sand pavement 0.0075
  - Concrete 0.012
  - Tar and gravel pavement 0.017
  - Short grass 0.046
  - Dense grass 0.06

**Alameda Method**

\[
t_c = \frac{L}{60V}
\]
where:

\( L \) = Length of overland flow (ft, m)

\( V \) = Overland flow velocity (m/s, ft/s)

**Kerby’s Equation**

\[
t_c = 0.83 \left( \frac{L}{S^{0.5}} \right)^{0.467}
\]

where:

\( L \) = Length of overland flow (ft, m)

\( S \) = Average catchment slope (ft/ft, m/m)

\( n \) = Retardance Roughness

- Smooth pavement 0.02
- Poor grass, bare soil 0.30
- Average grass 0.40
- Dense grass 0.80

**Kirpich’s Equation**

\[
t_c = 0.0078 \left( \frac{L}{S^{0.385}} \right)^{0.77}
\]

where:

\( L \) = Length of overland flow (ft, m)

\( S \) = Average catchment slope (ft/ft, m/m)

**Bransby Williams Method**

\[
t_c = \frac{21.3}{5280} \left( \frac{L}{\frac{A}{640} S^{0.2}} \right)
\]

where:

\( L \) = Length of overland flow (ft)

\( A \) = Catchment area (acres)

\( S \) = Average catchment slope (ft/ft)

---

⚠️ When working in metric units (of meters, hectares, and dimensionless slope m/m), the input is converted automatically to US units for use in this equation. Use meters, hectares and dimensionless slope for a metric unit model.

**US Federal Aviation Authority**

\[
t_c = \frac{1.8(1.1 - C)L^{0.5}}{S^{0.33}}
\]

where:

\( L \) = Length of overland flow (ft, m)

\( S \) = Average catchment slope (ft/ft, m/m)
C = Runoff coefficient

- Business district 0.70 - 0.95
- Single family residential 0.30 - 0.50
- Asphalt/Concrete 0.70 - 0.95
- Sandy soil lawn 0.05 - 0.20
- Heavy soil lawn 0.13 - 0.35
- Brick 0.70 - 0.85

TR-55 Methods

From the Technical Release 55 (TR-55) – Urban Hydrology for Small Watersheds manual – Chapter 3, we have included Sheet Flow, Shallow Concentrated Flow and Channel Flow Time of Concentration calculation options. Additional details and recommendations on these methods and parameters can be reviewed by browsing to the USDA NRCS TR55 Manual.

Sheet Flow

For sheet flow of less than 300 feet, use Manning’s kinematic solution (Overtop and Meadows 1976) to compute $T_f$:

$$T_f = \frac{0.0077(L_{0.5}n^{0.6}s^{0.4})}{P_2^{0.5}}$$  \hspace{1cm} [eq. 3-3]

where:

- $T_f$ = travel time (hr),
- $n$ = Manning’s roughness coefficient (table 3-1),
- $L$ = flow length (ft),
- $P_2$ = 2-year, 24-hour rainfall (in),
- $s$ = slope of hydraulic grade line (land slope, ft/ft)

This simplified form of the Manning’s kinematic solution is based on the following: (1) shallow steady uniform flow, (2) constant intensity of rainfall excess (that part of a rain available for runoff), (3) rainfall duration of 24 hours, and (4) minor effect of infiltration on travel time. Rainfall depth can be obtained from appendix B.

Shallow

Travel time ($T_f$) is the ratio of flow length to flow velocity:

$$T_f = \frac{L}{3600V}$$  \hspace{1cm} [eq. 3-1]

where:

- $T_f$ = travel time (hr),
- $L$ = flow length (ft),
- $V$ = average velocity (ft/s)
- 3600 = conversion factor from seconds to hours.

Channel Flow
Manning’s equation is:

\[ V = \frac{2}{n} \left( \frac{1}{r} \right) \left( \frac{a}{p_w} \right) \left( \frac{s}{f} \right) \]  

[eq. 3.4]

where:

- \( V \) = average velocity (ft/s)
- \( r \) = hydraulic radius (ft) and is equal to \( \sqrt{a/p_w} \)
- \( a \) = cross sectional flow area (ft²)
- \( p_w \) = wetted perimeter (ft)
- \( s \) = slope of the hydraulic grade line (channel slope, ft/ft)
- \( n \) = Manning’s roughness coefficient for open channel flow.

Manning’s \( n \) values for open channel flow can be obtained from standard textbooks such as Chow (1959) or Linsley et al. (1982). After average velocity is computed using equation 3.4, \( T_i \) for the channel segment can be estimated using equation 3.1.

**UK Hydrology**

There are five major hydrology methods available within UK Hydrology. The following methods were derived in the UK though may be applicable elsewhere.

1. Variable PR Method (Variable PR)
2. Wallingford Procedure (Wallingford)
3. Revitalised FSR/FEH Method (ReFH)
4. Flood Estimation Handbook (FEH)
5. Flood Studies Report (FSR)

The following references are applicable for each method:


It is important to note that some data fields may be shared between hydrology methods. These shared data field values may not be the same for all hydrology methods and should be tailored for the specific UK Hydrology method selected.
Variable PR Method (Variable PR)

The Variable PR runoff method, sometimes known as New PR or New UK method, uses the non-linear reservoir routing hydrology method as per the Wallingford Procedure runoff method. The volume of percentage runoff varies during the rainfall event to take account of changes to the catchment wetness during the storm. The Variable PR method routing equation is shown below:

\[ PR = IF \times PIMP + (100 - IF \times PIMP) \times NAPI/PF \]

Routing Coefficient (KR). The Variable PR runoff method routing coefficient (KR) is typically input as a value 0.5 or less when used in rural catchments.

Effective Impervious Factor (IF). The Effective Impervious Factor is used to give an effective contributing impervious area based on the impervious area runoff. Recommended Values for the Effective Impervious Factor (IF) are shown below:

<table>
<thead>
<tr>
<th>Surface condition</th>
<th>Effective impervious area factor, IF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poor</td>
<td>0.45</td>
</tr>
<tr>
<td>Fair</td>
<td>0.60</td>
</tr>
<tr>
<td>Good</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Maximum Moisture Depth (PF). The Maximum Moisture Depth is typically set at 200mm.

Net Antecedent Precipitation Index (NAPI). The Net Antecedent Precipitation Index (NAPI) is the 30 day antecedent precipitation index, which is directly dependent on the soil type. Starting NAPI and Maximum Soil Moisture are required. The Net Antecedent Precipitation Index is defined by the following equation:

\[ NAPI = k^{1.24}NP_{-1} + k^{0.56}NAPL_{-1} \]

Where:

<table>
<thead>
<tr>
<th>k</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.10</td>
<td>1</td>
</tr>
<tr>
<td>0.50</td>
<td>2</td>
</tr>
<tr>
<td>0.70</td>
<td>3</td>
</tr>
<tr>
<td>0.90</td>
<td>4</td>
</tr>
<tr>
<td>0.99</td>
<td>5</td>
</tr>
</tbody>
</table>

\[ t \] - model timestep (hours)
\[ NP \] - Rain less unsatisfied DEPSTOG in previous timestep (DEPSTOG – depression storage).

After the depression storage is filled, evaporation will be subtracted from the rainfall and the remaining amount will be routed through the same reservoir than in the Wallingford Method. Qvol is multiplied with AREA to get the outflow volume in cubic meter.

WRAP Soil (%). The WRAP SOIL percentages define the index of the water holding capacity of the soil (0.15 to 0.5), based on the FSR WRAP parameter obtained from FSR. The formula for calculating the WRAP Soil index is shown below:

\[ WRAP \text{ Soil Index} = (\text{Soil 1 \% } * 0.15) + (\text{Soil 2 \% } * 0.3) + (\text{Soil 3 \% } * 0.4) + (\text{Soil 4 \% } * 0.45) + (\text{Soil 5 \% } * 0.5). \]

There are five possible WRAP SOIL percentages which can be input to define the sub-catchment soil conditions. The WRAP SOIL percentages are input as fractions and the sum of which must equal 100\% (i.e.: Soil 1 \% + Soil 2 \%...+ Soil 5 \% = 1.0). The W.R.A.P. soil classifications are defined as follows:

<table>
<thead>
<tr>
<th>Classification</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>
|   | a. Well drained permeable sandy or loamy soils and shallower analogues over highly permeable limestone, chalk, sandstone or related drifts.  
|   | b. Earthy peat soils drained by dikes and pumps  
|   | c. Less permeable loamy over clayey soils on plateaux adjacent to very permeable soils in valleys. |
|   | 2. |
|   | a. Very permeable soils with shallow ground-water  
|   | b. Permeable soils over rock or fragipan, commonly on slopes in western Britain associated with smaller areas of less permeable wet soils  
|   | c. Moderately permeable soils, some with slowly permeable subsoils |
|   | 3. |
|   | a. Relatively impermeable soils in boulder and sedimentary clays, and in alluvium, especially in eastern England  
|   | b. Permeable soils with shallow ground-water in low lying areas  
|   | c. Mixed areas of permeable and impermeable soils, in approximately equal proportions |
|   | 4. |
|   | Clayey, or loamy over clayey soils with an impermeable layer at shallow depth |
|   | 5. |
|   | Soils of the wet uplands with:  
|   | a. peaty or humose surface horizons and impermeable layers at shallow depth  
|   | b. deep raw peat associated with gentle upland slopes or basin sites  
|   | c. bare rock cliffs and screes, and  
|   | d. shallow, permeable rocky soils on steep slopes |

**Wallingford Procedure (Wallingford)**

The Wallingford Procedure runoff method (Wallingford) is a non-linear reservoir routing method. This method can be simulated with any runoff time step though using the appropriate time step is essential for simulating the correct reservoir routing. The percentage of runoff is fixed throughout the storm. The Wallingford Procedure runoff method routing equation is shown below:

\[
PR = 0.829PIMP + 25SOIL + 0.078UCWI - 20.7
\]

**Routing Coefficient (KR).** The Variable PR runoff method routing coefficient (KR) is typically input as a value 0.5 or less when used in rural catchments.

**Antecedent Wetness Condition (UCWI).** The Antecedent Wetness Condition (Urban Catchment Wetness Index - UCWI), which is a composite of two antecedent wetness parameters and is given by:

\[
UCWI = 125 + 8API - SMD
\]

**Where**
API\_5 = 5-day antecedent precipitation index (mm)

SMD = soil moisture deficit (mm)

Alternatively, design values are provided by referring to average annual rainfall for a given location.

**WRAP SOIL (%)**. The WRAP SOIL percentages define the index of the water holding capacity of the soil (0.15 tot 0.5), based on the FSR WRAP parameter obtained from FSR. The formula for calculating the WRAP Soil index is shown below:

\[
\text{WRAP SOIL Index} = (\text{Soil 1} \times 0.15) + (\text{Soil 2} \times 0.3) + (\text{Soil 3} \times 0.4) + (\text{Soil 4} \times 0.45) + (\text{Soil 5} \times 0.5).
\]

There are five possible WRAP SOIL percentages which can be input to define the sub-catchment soil conditions. The WRAP SOIL percentages are input as fractions and the sum of which must equal 100% (i.e.: Soil 1 % + Soil 2 %...+ Soil 5 % = 1.0). The W.R.A.P. soil classifications are defined as follows:

<table>
<thead>
<tr>
<th>Classification</th>
<th>Description</th>
</tr>
</thead>
</table>
| 1              | 1. a. i. Well drained permeable sandy or loamy soils and shallower analogues over highly permeable limestone, chalk, sandstone or related drifts.  
   ii. Earthy peat soils drained by dikes and pumps  
   iii. Less permeable loamy over clayey soils on plateaux adjacent to very permeable soils in valleys. |
| 2              | 1. a. i. Very permeable soils with shallow ground-water  
   ii. Permeable soils over rock or fragipan, commonly on slopes in western Britain associated with smaller areas of less permeable wet soils  
   iii. Moderately permeable soils, some with slowly permeable subsoils |
| 3              | 1. a. i. Relatively impermeable soils in boulder and sedimentary clays, and in alluvium, especially in eastern England  
   ii. Permeable soils with shallow ground-water in low lying areas  
   iii. Mixed areas of permeable and impermeable soils, in approximately equal proportions |
| 4              | Clayey, or loamy over clayey soils with an impermeable layer at shallow depth |
| 5              | Soils of the wet uplands with:  
   1. a. i. peaty or humose surface horizons and impermeable layers at shallow depth  
   ii. deep raw peat associated with gentle upland slopes or basin sites  
   iii. bare rock cliffs and screes, and  
   iv. shallow, permeable rocky soils on steep slopes |

It is noted that while using the Wallingford Procedure runoff method the following sub-catchment slope values are recommended:

- 1.25% - mild slope
- 2.75% - medium slope
- 4.0% - steep slope

**Revitalised FSR/FEH Method (ReFH)**
The Revitalised FSR/FEH Method is a unit hydrograph method which refines the original FSR/FEH Method. The base flow component is dynamic in this method, it is computed by a routing model of the runoff. The runoff time step has to fulfill the condition $0.02*TP \leq \text{timestep} \leq 0.2*TP$. In contrast to the original FSR method this method updates the percentage runoff in each time step. The difference between net and effective rain is infiltration.

**Season and Return Period.** This input field allows the user to specify in which season the simulation will occur. Recommended seasons are: Winter - November to April, and Summer - May to October. The Return Period represents the number of months in the season. The initial soil moisture and initial baseflow values will vary depending on the season selected.

**Mean Drainage Path Slope (DPSBAR).** The Mean Drainage Path Slope represents the average sub-catchment drainage path slope.

**Extent of Urban and Suburban Land Cover (URBEXT).** For the Extent of Urban and Suburban Land Cover either select the 1990 or 2000 values if available or simply enter a user-defined value. Please note that the year 2000 values are only available if using the FEH CD-ROM Version 2. If values for 1990 or 2000 are zero, please check the values as they may not be available from the FEH data.

**Proportion of Time when Soil Moisture Deficit was below 6mm (PROPWET).** This parameter represents the proportion of time when the soil moisture deficit was below 6mm during the years between 1961-1990, as defined by Met Office Rainfall and Evaporation Calculation System (MORECS).

**Mean Drainage Path Length (DPLBAR).** The Mean Drainage Path Length represents the average sub-catchment drainage distance.

**Average Annual Rainfall (SAAR).** This input parameter is the average annual rainfall value from between 1961 to 1990.

**Base Flow Index (BFIHOST).** The Base Flow Index represents the baseflow index derived from Hydrology of Soil Types (HOST) soils data.

**Overrule Option.** The Time to Peak and Baseflow calculations can be specified using the Overrule option. The user input of these parameters is optional. If they are not specified by the user, the engine will generate its own values based on other input parameters. Note that if a user defined baseflow is used it is a constant baseflow and does not vary with the rainfall.

**Loss Model (Rainfall).** The Loss Model used to derive Net Rainfall

Net rainfall, $p_t$, measured in mm, is needed for the convolution of the unit hydrograph. It is derived from the equation below where $p_t/P$ represents percentage runoff using $P$, the design rainfall derived above. Calculate the following parameters:

1. Calculate Maximum Soil Moisture Capacity, $C_{\text{max}}$ (mm):

   $$C_{\text{max}} = 596.7 \times BFIHOST^{0.95} \times PROPWET^{-0.24}$$

   $r^2 = 0.55, \ fse = 0.161, \ n = 100$

2. Calculate Initial Soil Moisture Capacity, $C_{\text{ini}}$ (mm):
If season is Winter use:

\[
\frac{C_{ini,\text{winter}}}{0.5C_{\text{max}}} = 1.20 - 1.70BFIHOST + 0.82\text{PROPWET}
\]

\[r^2 = 0.53, \quad n = 93\quad \text{Eq.7}\]

If season is Summer use:

\[
\frac{C_{ini,\text{summer}}}{0.5C_{\text{max}}} = 0.90 - 0.82BFIHOST - 0.43\text{PROPWET}
\]

\[r^2 = 0.49, \quad n = 7\quad \text{Eq.8}\]

If \(C_{ini}/0.5C_{\text{max}} < 1\), then \(C_{ini} = 0\).

The soil moisture storage \(C_t\) (mm) is calculated from:

\[
C_t = C_{t-1} + P_t
\]

\[\text{Eq.9}\]

With \(C_t\) at time zero equal to \(\text{Alpha}_T \times C_{ini}\)

Correction factor, \(\text{Alpha}_T\):

\(\text{Alpha}_T = 1\) for return periods less than 5 years

If season is Winter use:

\[
\text{Alpha}_T = 1.166T^{-0.073}
\]

\[\text{Eq.10}\]

If season is Summer use:

\[
\text{Alpha}_T = 1.444T^{-0.182}
\]

\[\text{Eq.11}\]

Calculate the net rainfall, \(P_t\):

If \(C_t < C_{\text{max}}\) use the following equations:

For \(t = 1\):

\[
\frac{P_t}{P_i} = \alpha_t \left(\frac{C_{ini}}{C_{\text{max}}}\right) + \left(\frac{P_t}{2C_{\text{max}}}\right)
\]

\[\text{Eq.12}\]

For \(t = 2, 3, \ldots\):

\[
\frac{P_t}{P_i} = \frac{\left(\frac{C_{t-1}}{C_{\text{max}}}\right) + \left(\frac{P_t}{2C_{\text{max}}}\right)}{\alpha_t}
\]

\[\text{Eq.13}\]

If soil storage filled during the event, where \(C_t = C_{\text{max}}\) use:

\[
\frac{P_t}{P_i} = 1 - C_{\text{max}} \left(\frac{1 - C_{t-1}/C_{\text{max}}}{2P_t}\right)
\]

\[\text{Eq.14}\]

Clarification for calculation of the first two timesteps:

At time zero, \(t = 0\):

\[
\alpha_t \times C_{ini} = C_0
\]

\[\text{Eq.15}\]

First timestep: \(t = 1\)

\[
C_1 = C_0 + P_1
\]

\[\text{Eq.16}\]

using \(C_0\) calculated at \(t = 0\), giving:
\[ \frac{P_1}{P_1} = \left( \frac{C_0}{C_{\text{max}}} \right) + \left( \frac{P_1}{2C_{\text{max}}} \right) \]  
Eq. 17

Second timestep: \( t = 2 \)

\[ C_2 = C_1 + P_2 \]  
Eq. 18

using \( C_t \) calculated at \( t = 1 \), giving:

\[ \frac{P_2}{P_2} = \left( \frac{C_1}{C_{\text{max}}} \right) + \left( \frac{P_2}{2C_{\text{max}}} \right) \]  
Eq. 19

and so on.

**Convert net rainfall values to correct units**

Net rainfall values should be divided by 10 to convert them from mm into cm for convolution with the unit hydrograph.

**Hydrograph**

*Instantaneous Unit Hydrograph Calculations:*

**Calculate Time to Peak, \( T_p \) from Eq. 1 above:**

\[ T_p = 1.563 \text{PROFWET}^{-1.99} \text{DPLBAR}^{0.09} \left( 1 + \text{URBEXT} \right)^{-0.31} \text{DPSBAR}^{-0.29} \]

\( r^2 = 0.81, \ fse = 1.32, \ n = 101 \)  
Eq. 1

**Calculate other parameters necessary to draw the ReFH Kinked Triangle unit hydrograph.**

\[ U_k = 0.80 \]  
Eq. 20

\[ U_p = 0.65 \]  
Eq. 21

\[ TB_t = \frac{2T_p}{U_p} \]  
Eq. 22

\[ U_c = U_p \left( TB_t - 2T_p \right) / \left( TB_t - T_p \right) \]  
Eq. 23

\[ U_{2Tp} = U_k U_c \]  
Eq. 24

\[ TB = \left( 1 + 2 \left( 1 - U_p \right) / \left( U_k U_c \right) \right) T_p \]  
Eq. 25
Plot the unit hydrograph using points \((T_P, U_p), (2T_P, U_{2T_P}), (TB, 0)\)

Finding the unit hydrograph ordinates

\[ u_t = \frac{U_p}{T_P} \quad \text{Eq. 26} \]

If \( T_P \) use:

\[ u_t = \frac{U_p}{T_P} t \quad \text{Eq. 27} \]

If \( T_P \) use:

\[ u_t = \frac{U_{2T_P} - U_p}{T_P} (t - T_P) + U_p \quad \text{Eq. 28} \]

Converting the IUH to a TUH

To convert the IUH to a TUH (which is the UH for a finite rainfall duration) calculate the area under the graph between timesteps effectively applying an S-Curve to the hydrograph. If the peak and kink of the unit hydrograph do not fall on an exact timestep the area will need to be corrected. These areas should be used for the convolution, for example the area under the graph between timestep 0 and timestep 1 should be used as the unit hydrograph ordinate for timestep 1.

The area under the entire curve should be equal to \( T_P \).

Before convoluting the unit hydrograph, the areas must be multiplied by a factor of \( 2.78 \times \frac{AR}{EA} \). This converts the units into \( m^3/s/cm \) ready for convolution.

**Baseflow Model**

Calculate Baseflow Equation.

**Calculate Baseflow Recharge Parameter, \( BR \)**

\[ BR = 3.751BFIHOST^{1.08} \text{PROPWET}^{0.36} \]

\[ r^2 = 0.34, \quad fse = 2.04, \quad n = 100 \quad \text{Eq. 30} \]

Calculate Baseflow Lag, \( BL \) (hours):

\[ BL = 25.47BFIHOST^{0.46} \text{DPLBAR}^{0.23} \text{PROPWET}^{0.33}(1 + \text{URBEXT})^{-0.14} \]

\[ r^2 = 0.41, \quad fse = 2.03, \quad n = 100 \quad \text{Eq. 31} \]

\( BL \) is calculated in hours but should be divided through by 24 to give it in days to be used in the baseflow equation above and in the baseflow constant equations below.

**Calculate Initial Baseflow, \( BF_0 \) \( m^3/s)\):**

The following equation can be used for either season:

\[ BF_{0,\text{ALL}} = (41.36(C_{\text{in}} - 115.89) + 4.56SAAR) \times 10^{-5} \text{AREA} \]

\[ n = 1183, \quad r^2 = 0.45 \quad \text{Eq. 32} \]

If season is Winter use:

\[ BF_{0,\text{WINTER}} = (63.79(C_{\text{in}} - 120.79) + 5.54SAAR) \times 10^{-5} \text{AREA} \]

\[ n = 752, \quad r^2 = 0.47 \quad \text{Eq. 33} \]
If season is *Summer* use:

\[ BF_{S,\text{SUMMER}} = (33.94(C_{sl} - 85.42) + 3.14\text{SAAR})10^{-5}\text{ AREA} \]

\[ n = 431, \quad n^3 = 0.42 \]  

Eq. 34

Baseflow Constant Equations:

\[ K_1 = \frac{BR}{1 + BR}\left(\frac{(BL/\Delta)(1-K_3)}{1 + BR} - K_3\right) \]  

Eq. 35

\[ K_2 = \frac{BR}{1 + BR}\left(1 - \frac{(BL/\Delta)(1-K_3)}{1 + BR}\right) \]  

Eq. 36

\[ K_3 = \exp\left(-\frac{(1 + BR)\Delta}{BL}\right) \]  

Eq. 37

Where \(\Delta\) is the timestep in days.

Use these parameters to calculate the baseflow profile:

\[ BF_t = \frac{1}{1 - K_2}\left[K_1q_{t-1} + K_2q_t + (K_1 + K_2)BF_{t-1}\right] \quad \text{for } t = 1, 2, 3, \ldots \]  

Eq. 38

where \(q_t\) is direct runoff from the routing model and \(K_1, K_2, K_3\) are constants and functions of the baseflow parameters \(BR\) and \(BL\).

**Flood Estimation Handbook (FEH)**

The Flood Estimate Handbook (FEH) hydrology method is a unit hydrograph method where a constant baseflow is added to the runoff, similar to the Flood Studies Report hydrology method (FSR). The catchment descriptors can be obtained from the FEH CD-ROM which is available from [http://www.hydrosolutions.co.uk/fehcd.html](http://www.hydrosolutions.co.uk/fehcd.html). The runoff time step has to fulfill the condition \(0.02\text{TP} \leq \text{time step} \leq 0.2\text{TP}\).

**Mean Drainage Path Slope (DPSBAR).** The Mean Drainage Path Slope represents the average sub-catchment drainage path slope.

**Extent of Urban and Suburban Land Cover (URBEXT).** For the Extent of Urban and Suburban Land Cover either select the 1990 or 2000 values if available or simply enter a user-defined value. Please note that the year 2000 values are only available if using the FEH CD-ROM Version 2. If values for 1990 or 2000 are zero, please check the values as they may not be available from the FEH data.
Standard Runoff (%) \((\text{SPRHOST})\). The digital FEH data has 29 soil categories while the FSR had only 5.

\[
\text{SPR} = \text{SPRHOST} = \text{SPR}_{\text{HOST}1} + \ldots + \text{SPR}_{\text{HOST}29}
\]

SPRHOST is calculated by the FEH CD program and written to the .CSV file.

Where measured flow records are available the SPR may be calculated from the Base Flow Index (BFI) from research published in FSSR 16 and it is not covered here – see also FEH, Volume 4, Chapter 1.

At the moment XP-SWMM only offers direct input for SPR

Proportion of Time when Soil Moisture Deficit was below 6mm \((\text{PROPWET})\). This parameter represents the proportion of time when the soil moisture deficit was below 6mm during the years between 1961-1990, as defined by Met Office Rainfall and Evaporation Calculation System (MORECS).

Mean Drainage Path Length \((\text{DPLBAR})\). The Mean Drainage Path Length represents the average sub-catchment drainage distance.

Average Annual Rainfall \((\text{SAAR})\). This input parameter is the average annual rainfall value from between 1961 to 1990.

Overrule Option. The Time to Peak, Catchment Wetness Index (CWI) and Baseflow calculations can be specified using the Overrule option. The user input of these parameters is optional. If they are not specified by the user, the engine will generate its own values based on other input parameters. Note that if a user defined baseflow is used it is a constant baseflow and does not vary with the rainfall.

Computed Variables and Hydrograph

\(\text{TP0 (hrs)}\)

Time to peak is computed based on the following formula:

\[
\text{TP0} = 4.27 \times \text{DPSBAR}^{0.35} \times \text{PROPWET}^{0.5} \times \text{DPLBAR}^{0.54} \times (1+\text{URBEXT})^{-0.35}
\]

\(\text{TPt (hrs)}\)

Time to rise of the unit hydrograph as defined by:

\[
\text{TPt} = \text{TP0} + \frac{T}{2}, \ T \text{ time step}
\]

\(\text{QP (m}^3\text{s} \ 10\text{mm)}\)

The peak flow for the unit hydrograph as calculated by:

\[
\text{QP} = \frac{(220)}{(\text{TPt})} \times \frac{(\text{AREA})}{100}
\]

\(\text{TB (hrs)}\)

Time base of the unit hydrograph as calculated by:

\[
\text{TB} = 2.52 \times \text{TPt}
\]

Base Flow \((\text{m}^3\text{s})\)

Base flow for the unit hydrograph as calculated by:

\[
\text{BASEFLOW} = \text{AREA} \times \left(33 \times (\text{CWI} - 125) + 3 \times \text{SAAR} + 5.5 \right) \times 10^5
\]

\(\text{PR} (%)\)

Total percentage run off of the catchment is defined as follows:

\[
\text{PR}_{\text{total}} = \text{PR}_{\text{rural}} \times (1 - 0.615 \times \text{URBEXT}) + 70 \times 0.615 \times \text{URBEXT}
\]

where

\[
\text{PR}_{\text{rural}} = \text{SPR} + \text{DPR}_{\text{CWI}} + \text{DPR}_{\text{rain}} \quad \text{and}
\]

\[
\text{DPR}_{\text{rain}} = 0.45 \times (P - 40)^{0.7}, \ \text{if} \ P > 40\text{mm and} \quad \text{DPR}_{\text{rain}} = 0
\]
otherwise, \( P \) is the rainfall depth.

\[
DPR_{cw1} = 0.25 \times (CWI - 125).
\]

**Flood Studies Report (FSR)**

The Flood Studies Report runoff method (FSR) is a unit hydrograph method where a constant baseflow is added to the runoff. Catchment descriptors should be obtained from measured data. The runoff time step has to fulfill the condition 0.02*TP <= time step <= 0.2*TP.

**Height at 85% from Catchment Outlet (H85).** This input parameter, in conjunction with the Height at 10% from Catchment Outlet (H10), allows the main channel slope (S1085) to be calculated. The values are the heights at 10% and 85% of the main channel length upstream from the catchment outlet.

**Height at 10% from Catchment Outlet (H10).** This input parameter, in conjunction with the Height at 85% from Catchment Outlet (H85), allows the main channel slope (S1085) to be calculated. The values are the heights at 10% and 85% of the main channel length upstream from the catchment outlet.

**Extent of Urban and Suburban Land Cover**

**Standard Runoff (%) (SPR%).** This value can be calculated from Figure I.4.18 (first revision) (Institute of Hydrology, 1978) which shows the various soil types. The soil classifications are also available from the Walingford Procedure, Volume 3, Maps, “Winter rain acceptance potential”. The equation was first published in FSSR 16, 1985.

\[
SPR = 10S1 + 30S2 + 37S3 + 47S4 + 50S5
\]

where

\[ S1 - S5 \] are the proportions of the catchment covered by soil classes 1-5 respectively.
Where measured flow records are available, the SPR may be calculated from the Base Flow Index (BFI) from research published in FSSR 16 and it is not covered here – see also FEH, Volume 4, Chapter 1.

**Length of Main Channel (MSL).** TheLength of the Main Channel or Stream (MSL) measured in metres on a 1:25000 Ordnance Survey Map. It is normally measured in kilometres (and represented in the formula in km) but it is input in metres to be consistent with other data input.

**Average Annual Rainfall (SAAR).** This input parameter is the average annual rainfall value from 1941 to 1970 from FSR Figure II.3.1 (Natural Environment Research Council, 1975). The data is also available from the Wallingford Procedure, Volume 3 Maps and the FEH CD.

**Overrule Option.** The Time to Peak, Catchment Wetness Index (CWI) and Baseflow calculations can be specified using the Overrule option. The user input of these parameters is optional. If they are not specified by the user, the engine will generate its own values based on other input parameters. Note that if a user defined baseflow is used it is a constant baseflow and does not vary with the rainfall.

**Computed Variables and Hydrograph**

**TP0 (hrs)** This is the instantaneous time to peak of the unit hydrograph. Ciria14 uses the equation from FSSR 16 published in 1985. However IH 124 published a more recent equation that outperforms FSSR 16 on small, partly urbanised catchments. The IH equation also performed well on the FSSR 16 data set and therefore it is recommended for general use i.e. on small and large catchments, rural and partly urban. For these reasons the IH 124 formula has been used in the software.

Time to peak when calculated from catchment characteristics is:

\[
S1085 = \frac{(H85 - H10)}{(0.75 \times MSL)}
\]

**TPt (hrs).** Time to rise of the unit hydrograph as defined by:

\[
TPt = TP0 + T/2, T \text{ time step}
\]

**QP (m\(^3\)/s 10mm).** The peak flow for the unit hydrograph as calculated by:

\[
QP = \left(\frac{220}{TPt}\right) \times \left(\frac{AREA}{100}\right)
\]

**TB (hrs).** Time base of the unit hydrograph as calculated by:

\[
TB = 2.52 \times TPt
\]

**Base Flow (m\(^3\)/s).** Base flow for the unit hydrograph as calculated by:

\[
BASEFLOW = AREA \times [33 \times (CWI - 125) + 3 \times SAAR + 5.5] \times 10^{-2}
\]

**PR (%).** Total percentage run off of the catchment is defined as follows:

\[
PR_{\text{total}} = PR_{\text{rural}} \times (1 - 0.3 \times URBAN) + 70 \times 0.3 \times URBAN
\]

where

\[
PR_{\text{rural}} = SPR + DPR_{\text{CWI}} + DPR_{\text{rain}}\quad \text{and}
\]

\[
DPR_{\text{rain}} = 0.45 \times (P - 40)^{0.7}, \quad \text{if } P > 40\text{mm and}
\]

\[
DPR_{\text{CWI}} = 0.25 \times (CWI - 125)
\]

otherwise P is the rainfall depth.