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1 TIPS FOR MODELLING UNGAUGED RURAL CATCHMENTS

This section outlines different methodologies for modelling ungauged rural catchments. While it is preferable to use a calibrated hydrologic model for water resources studies, especially for rural catchments, this is not always possible. Satisfactory results may still be obtained for macro level studies provided that the modeller chooses the appropriate parameters for each catchment.

The focus of this section is on the Initial Abstraction parameter, IA, and the Time to Peak parameter, TP, parameter. While the CN parameter plays a large role in determining the runoff characteristics of a particular catchment, this parameter can be readily determined and is rarely in dispute by watershed regulating authorities. Guidance is provided in this section on determining the Modified CN parameters, called CN*.

1.1 INITIAL ABSTRACTION PARAMETER, IA

1.1.1 MODIFIED CURVE NUMBER METHOD (CN*)
When using the Modified Curve Number Method the IA parameter should be set to a value in the range of 1.0 mm and 5.0 mm, depending on the circumstances. The IA value must then be used to calculate CN* (see below).

1.1.2 SCS METHOD (CN)
When using the SCS Curve Number Method, IA should be set to 0.2S where S is the soil storage (a function of CN). Bear in mind that this method may underestimate the peak flow for small storms because the initial abstraction is higher than the total rainfall, which is not accurate. A literature review of this method has found that for lower CN values, a lower IA should be used. Suggests guidelines are as follows:

\[
\begin{align*}
CN & \leq 70 & IA &= 0.075S \\
CN & > 70 \leq 80 & IA &= 0.10S \\
CN & > 80 \leq 90 & IA &= 0.15S \\
CN & > 90 & IA &= 0.2S
\end{align*}
\]

Please note that the above guidelines are for the SCS Method only, where the SCS Curve Number is used to define the soil type.

1.2 MODIFIED CURVE NUMBER, CN*

The Modified Curve Number method was first proposed by Paul Wisner & Associates in 1982, and was based on their research and monitoring of rural and urban catchments in Canada. This method has been used successfully in Canada for the past 35 years and has correlated well with measured flows.
Rather than having a varying $IA$ parameter, as in the SCS method, the $IA$ is fixed, as described above, and the $CN$ is altered. The modified $CN$, called $CN^*$, is a function of the $IA$, and total rainfall. $CN^*$ is calculated as follows:

1. Select an appropriate $IA$ (see above) for catchments being modelled.
2. Determine the SCS $CN$ value from soils maps and/or calculations. Convert the $CN$ (AMC II conditions) to a $CN$ (AMC III conditions).
3. Determine the largest precipitation volume, $P$, for a rainfall event that would just represent AMC III soil moisture conditions. In most cases this is the 100 year storm event. For example, in Markham Ontario the 100 year storm volume for the 3 hour storm is 80 mm.
4. Calculate the soil storage $S$, based on the SCS Method using $CN$ (AMC III conditions). The metric equation is $S = (25400 / CN) – 254$ and the imperial equation is $S = (1000 / CN) – 10$. This will give you the soil storage during your large storm event.
5. Calculate the $IA$ based on the SCS Method, where $IA = 0.2S$. Note that this relationship is also valid for the Modified CN Method because it is assumed that the runoff volume, $Q$, for large events is the same using both methods.
6. Determine the runoff volume, $Q$, based on the familiar:

$$Q = (P – IA)^2 / (P - IA + S)$$

7. Next calculate $S^*$ using the above equation again but this time setting IA to the value calculated for the Modified CN method (i.e. 1.0mm to 5.0mm). This IA will be the value used in the model simulations.
8. Once you have calculated $S^*$, calculate $CN^*$ from the equation:

$$S^* = (25400 / CN^*) – 254 \quad \text{metric}$$
$$S^* = (1000 / CN^*) – 10 \quad \text{imperial}$$

9. The above calculation will give you the $CN^*$ for AMC III soil conditions. You now finally determine the $CN^*$ for AMC II soil conditions by using published tables relating $CN$ for AMC II and AMC III conditions.

The above method is easily adaptable to a spreadsheet so that for future uses, you can easily and quickly calculate the $CN^*$ once you know the $IA$, $P$, and $CN$.

This process has been incorporated in the Convert to $CN^*$ tool in Visual OTTHYMO. For more information, see Appendix A.1 in the User’s Manual.

1.3 **TIME TO PEAK PARAMETER, $TP$**

Unlike the urban catchments hydrographs, rural catchment unit hydrographs do not calculate the time to peak $TP$ as a function of the other variables. The $TP$ parameter must therefore be determined by the modeller. It should be noted that most methods of estimate $TP$, start by calculating the time of concentration, $t_c$. Time of concentration is the time at which the centroid of the flow reaches the bottom of a catchment. $TP$ is usually a fixed ratio of $t_c$, depending on the unit hydrograph chosen.

Over the past 40 years there have been numerous studies in both the United States and Canada in which empirical, semi-empirical, and mathematical relationships for $t_c$ have been derived. Most of the relationships state that $t_c$ is a function of catchment slope, catchment area, and ground
cover. While no single method can be used for every situation we have included the most common methods in this manual so that the modeller can choose what is appropriate for their situation.

Listed below are five methods for calculating TP. We have included both the source of the method as well as the context in which it was derived. This way the modeller should be able to choose a method that was derived for a similar situation as their own.

1.3.1 UPLAND’S METHOD

With Upland’s Method the average overland flow velocity is determined for a catchment based on the catchment slope and ground type, as shown in Figure 1. Once the velocity has been determined then the time of concentration is determined by dividing the catchment length by the overland flow velocity.

1.3.2 BRANSBY - WILLIAM’S FORMULA

In catchments where the runoff coefficient, C, is greater than 0.40, the Bransby Williams formula is a popular choice. The method calculates time of concentration as a function of catchment area, length, and slope as follows:

\[
t_c = \frac{0.057 \times L}{S_w^{0.2} \times A^{0.1}}
\]

where:

- \( t_c \) = time of concentration (min)
- \( L \) = catchment length, (m)
- \( S_w \) = catchment slope (%)
- \( A \) = catchment area (ha)

1.3.3 AIRPORT METHOD

For catchments where the runoff coefficient, C, is less than 0.40, the Airport formula may provide a better estimate of the time of concentration. This method was developed for airfields and calculates time of concentration as a function of runoff coefficient, length, and slope as follows:

\[
t_c = \frac{3.26 \times (1.1 - C) \times L^{0.5}}{S_w^{0.33}}
\]

where:

- \( t_c \) = time of concentration (min)
- \( C \) = runoff coefficient
- \( L \) = catchment length, (m)
- \( S_w \) = catchment slope (%)

1.3.4 **Williams' Equation (1977)**

Williams, who co-developed the William's Unit Hydrograph (**WILHYD** in Visual OTTHYMO) with Hann in 1973 later derived empirical relationships for both the $K$ and $T_P$ variables in **WILHYD**. These relationships are:

\[ K = 16.14A^{0.24}S^{-0.84} \]  \hspace{1cm} (3)

\[ t_P = 6.54A^{0.39}S^{-0.50} \]  \hspace{1cm} (4)

The above relationships were derived for watersheds in the southern United States. Refer to the Theory Reference section of this manual for more information on the derivation of the **WILHYD** unit hydrograph.
2 TIPS FOR MODELLING UNGAUGED URBAN CATCHMENTS

This section provides direction for modellers who are modelling ungauged urban catchments. In most cases, urban catchments are not gauged since the response to rainfall can be accurately simulated. However, like any model the user should be aware that the inappropriate selection of parameters can lead to erroneous output. This section will guide the modeller in selecting parameters that have been successfully used in the water resources industry.

2.1 IMPERVIOUSNESS

There are two impervious ratios required, the amount of directly connected imperviousness, \textit{XIMP}, and the total imperviousness, \textit{TIMP}. \textit{XIMP} must be less than or equal to \textit{TIMP}.

\textit{TIMP} is a function of the land use of the catchment. Land use is a planning term that describes the approved, or proposed, use for the catchment (e.g. residential, commercial, industrial). Water resources studies are generally tied to planning applications and depending on the level of planning application, (i.e. Secondary Plan, Official Plan Amendment, Draft Plan), the modeller will have a little or a lot of information about the land use. Therefore it is important to select a conservative value for the imperviousness when performing more macro level studies so that when the subsequent more detailed studies are completed, the more refined land use calculations will still be valid in the overall model.

The following table gives examples of suggested \textit{TIMP} and \textit{XIMP} values, based on land use, for the macro-level studies. These values can be used with the information supplied by the planner to determine area weighted values for the catchment of interest.

<table>
<thead>
<tr>
<th>Land Use</th>
<th>XIMP</th>
<th>TIMP</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estate Residential</td>
<td>20</td>
<td>40</td>
</tr>
<tr>
<td>Low Density Residential (e.g. Single Units)</td>
<td>25</td>
<td>50</td>
</tr>
<tr>
<td>Medium Density Residential (e.g. Semi-detached Units)</td>
<td>35</td>
<td>55</td>
</tr>
<tr>
<td>High Density Residential (e.g. Townhouse Units)</td>
<td>50</td>
<td>60</td>
</tr>
<tr>
<td>School</td>
<td>55</td>
<td>55</td>
</tr>
<tr>
<td>Commercial</td>
<td>85</td>
<td>85</td>
</tr>
<tr>
<td>Park</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

For more detailed level studies (i.e. Site Plan), there should be more information available so that the \textit{XIMP} and \textit{TIMP} can be calculated.
2.2 Loss Routine

In both the United States and Canada, either the Horton’s Method (LOSS = 1) or the CN Method (LOSS = 2) are commonly used for urban catchments. The Proportional Loss Method (LOSS = 3) has been successfully used in France for urban catchments. While the selection of Loss Routine can be somewhat arbitrary and at the discretion of the user, there are a few things to keep in mind when choosing a loss routine.

Horton’s Method is what is used in the SWMM model, therefore if the user is comparing results with a SWMM based model, or working in a watershed where the overall model used was SWMM, then this method may be the most appropriate. However, the user should bear in mind that for longer duration storms (greater than or equal to 12 hours) the Horton’s Method may not accurately predict the runoff from pervious areas. We have seen cases where the model simulates no runoff from a pervious area during a 12 hour 100 year storm. This is clearly erroneous. The CN Method does not have any limitations with respect to storm length and often yields more conservative results as compared to Horton’s Method.

If the user selects the CN Method, then the \( \text{IA} \) parameter should be set somewhere between 1.5 mm and 5 mm. Note that this is a different value than what would be used for a rural catchment with the same \( \text{CN} \) value. An urban catchment generally has less pervious depression storage than the same catchment in its rural state.

2.3 Parameters for the Pervious Component

The pervious slope, \( \text{SLPP} \), is the average slope of the pervious areas. This is not the catchment slope from highest point to lowest point, but an average when considering only the pervious areas. For example, if the catchment consists of a residential subdivision, this value would represent the average slope of the pervious lot surface. In this example the slope would not be less than 2\% or whatever the municipal minimum is.

The overland flow length, \( \text{LGP} \), should be set to the representative value for the pervious areas. It is not the length of the catchment from high point to low point. This value represents the average length over which flows from pervious areas would travel before being intercepted by channels, sewers, or roads. For example, in a residential subdivision this value might be the representative lot length which is typically 40 m.

The Manning’s roughness coefficient for pervious surfaces, \( \text{MNP} \), should be selected based on sheet flow and not channel flow. This is a common mistake for modellers. Most listed values of Manning’s values are for channel flow, whereas the pervious runoff simulated is sheet flow. Therefore, if we assumed a grassed surface then the sheet flow Manning’s roughness coefficient would be approximately 0.25, whereas the channel roughness coefficient for the same material might be 0.025.

For an ungauged urban catchment, the pervious storage coefficient, \( \text{SCP} \), should be set to 0, which will let the program determine the storage coefficient.
2.4 Parameters for the Impervious Component

The impervious depression storage, $DPSI$, should be set to an appropriate value for the representative impervious surface. For roads, driveways, and roofs, this value is typically between 0.8 mm and 1.5 mm.

The impervious slope, $SLPI$, is the average slope of impervious areas. This is not the catchment slope from highest point to lowest point, but an average when considering only the impervious areas. For example, if the catchment consists of a residential subdivision, this value would represent the average slope of the impervious road surfaces. In this example the slope would not be less than whatever the municipal minimum is. Typically, $SLPI$ ranges between 0.5 to 2.0.

The impervious length, $LGI$, is one of the most important parameters for modelling urban catchments. A common mistake when modelling ungauged urban catchments is to set $LGI$ equal to the measured catchment length. Previous studies by Paul Wisner Associates Inc. have determined that $LGI$ is related to the catchment area based on the following equation:

$$A = 1.5LGI^2$$ \hspace{1cm} (5)

where:

- $A$ = catchment area ($m^2$)
- $LGI$ = impervious length ($m$)

This relationship will yield runoff characteristics similar to those which would be measured. The $LGI$ parameter should only be adjusted from this relationship if the model is being calibrated.

The Manning’s roughness coefficient for impervious surfaces, $MNI$, should be selected based on channel flow, not sheet flow as in $MNP$. For example, if the representative impervious surface were a road, then the $MNI$ should be set around 0.013.

For an ungauged urban catchment the impervious storage coefficient, $SCI$, should be set to 0, which will let the program determine the storage coefficient.
3 SWM POND MODELLING

Probably the single biggest use for Visual OTTHYMO is to help create water resources strategies whereby stormwater management ponds are implemented to address issues of water quality control, erosion control, and water quantity (i.e. flooding) control. Visual OTTHYMO can be utilized to examine many scenarios that help water resources planners and engineers determine the most effective strategy, on a watershed or sub-watershed basis.

3.1 HOW TO BUILD A RATING CURVE USING ROUTE RESERVOIR

A rating curve for any stormwater management pond describes how the pond operates. In Visual OTTHYMO the command ROUTE RESERVOIR is used to enter a pond rating curve and simulate routing. The rating curve is described by the Discharge (i.e. outflow) and Storage relationship. Note that the Stage or water depth variable is taken out of the input, since both Discharge and Storage are a function of Stage. The Stage-Storage and Stage-Discharge rating curves are essentially combined into one Discharge-Storage curve. An example of a Discharge-Storage Curve is as follows:

<table>
<thead>
<tr>
<th>Discharge (m³/s)</th>
<th>Storage (ha-m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.06</td>
<td>0.34</td>
</tr>
<tr>
<td>0.21</td>
<td>0.48</td>
</tr>
<tr>
<td>0.37</td>
<td>0.60</td>
</tr>
<tr>
<td>0.66</td>
<td>0.83</td>
</tr>
<tr>
<td>0.94</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Designing a Discharge-Storage curve, at the watershed or sub-watershed planning level, involves determining each storage ordinate for every given discharge ordinate. Discharge ordinates are usually known or can readily be determined. They may represent allowable flows or release rates that when combined with other flows are the allowable flows at key locations. Storage ordinates are what the modeller is trying to calculate in order to meet the discharge targets.

For single event analysis the Discharge-Storage curve is built from the smallest to largest values, which corresponds to the smallest to largest rainfall events. For example, the above Discharge-Storage curve was based on the following design storm events.

<table>
<thead>
<tr>
<th>Discharge (m³/s)</th>
<th>Storage (ha-m)</th>
<th>Design Storm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>25 mm</td>
</tr>
<tr>
<td>0.06</td>
<td>0.34</td>
<td>2 year</td>
</tr>
<tr>
<td>0.21</td>
<td>0.48</td>
<td>5 year</td>
</tr>
<tr>
<td>0.37</td>
<td>0.60</td>
<td>25 year</td>
</tr>
<tr>
<td>0.66</td>
<td>0.83</td>
<td>100 year</td>
</tr>
</tbody>
</table>
When building a curve the storms must be run from smallest to largest and the storage iterated until the pond outflow matches that of the target value in the Discharge-Storage curve. Only then can the modeller move onto the next largest storm. The proper pond sizing methodology is therefore:

1. The modeller enters the first 2 sets of points on the curve, (0,0) and the first target flows (e.g. 0.06). The modeller guesses a storage value and then runs the model with the storm that corresponds to the target flows.
2. The modeller checks the outflow and compares it with the target. If the outflow is too high then the modeller must increase the storage. If the outflow is too low then the modeller must decrease the storage. Note that if the storage curve has been exceeded then the outflow may be erroneous. It is better to iterate from a large storage value to the correct storage than from a small storage value.
3. The modeller iterates step 2 until the calculates outflow matches (or is slightly less) than the target outflow. At this point the calculated storage should also match the storage in the input table.
4. The modeller then enters the next discharge ordinate for the next largest storm, guesses a new storage and runs the model.
5. Steps 2 and 3 are repeated until the outflow and storage are matched.
6. Step 4 is repeated with the next largest storm until the final storm is reached.
7. Once the last storm is iterated then the Discharge-Storage curve is complete. (e.g. when the (0.94,1.00) point is determined in the above example curve).

If the modeller is designing a SWM pond based on a real storm, or is analyzing an existing pond with design storms, then the actual discharge storage curve must be used. This can be obtained by combining the pond’s Stage-Storage curve (i.e. geometric relationship) and the Stage-Discharge curve (i.e. hydraulic relationship).

Also, a SWM pond’s actual Discharge-Storage curve must be used when creating a detail pond design, to ensure that the outflows match the targets from the design curve that was determined in the watershed or sub-watershed analysis.
4 COMPUTATION OF RAINFALL LOSSES

4.1 CRITICAL REVIEW OF SCS CURVE NUMBER PROCEDURE

4.1.1 CRITICAL REVIEW OF SCS CURVE NUMBER PROCEDURE

The SCS CN procedure is based on the equation

\[ Q = \frac{(P - I_a)^2}{(P - I_a + S)} \]  (6)

It is assumed in the procedure that the initial abstraction \( I_a = 0.2 \) S. This results in the equation

\[ Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \]  (7)

The curve numbers CN are functionally related to S by

\[ CN = \frac{1000}{S - 10} \]  (8)

CN can be obtained from tables based on land use, soil type and soil moisture conditions. However, the soil moisture is determined only for three antecedent moisture conditions (AMC), classified on the basis of precipitation in the previous 5 days. CN has no intrinsic meaning but is only a non-linear transformation of S, which is a storage parameter. CN varies from 0 (Q=0 for all P) to 100 (Q=P for all P). In Eqn. 8, the 10 and 1000 have inch dimensions. Conversion can be made to the metric system.

Background information on the derivation of the procedure can be found in a paper by Rallison and Cronshey (1979). In the mid-50s when the SCS CN procedure was developed, the only data available were daily precipitation and runoff records from agricultural watersheds and infiltration curves from infiltration studies. Rainfall versus Runoff (P vs Q) data were plotted. A grid of plotted CN for \( I_a = 0.2S \) was then overlaid and the median CN selected. The values in the SCS NEH-4 manual (1971) represent the averages of median site values for hydrologic soil groups, land cover and hydrologic conditions. The SCS work involved considerable interpolation and extrapolation for different soil types and land cover. The rainfall versus runoff plots were also used to define enveloping CN for each site.

The SCS CN procedure is in widespread use and there has been criticism of the procedure (Hawkins 1978, Altman et al. 1980, Golding 1979) because it is often applied beyond the original conditions and intended use.

Some of the concerns about the procedure are over:

1. why the antecedent moisture range of values is a discrete rather than a continuous relationship,
2. the lack of a clear definition of AMC II, the standard reference moisture condition,
3. use of a 5-day time interval as a basis for classifying antecedent moisture conditions,
4. why is $I_a = 0.2S$,
5. what probability levels are associated with the envelopes in defining AMC I and AMC III.

The SCS CN procedure may severely underestimate the runoff volume, especially for small rainfalls. It was found that the runoff volumes obtained from real measurements on two residential watersheds were greater than those computed using CN = 90 (corresponding to a high degree of imperviousness) (Figure 2).

![Figure 2 Relationship Between Rainfall and Runoff: CN and Real Measurements (Wisner, Gupta, Kassem, 1980)](image)

**FIGURE 2 RELATIONSHIP BETWEEN RAINFALL AND RUNOFF: CN AND REAL MEASUREMENTS (WISNER, GUPTA, KASSEM, 1980)**

A study in Texas (Altman et al. 1980) involving four watersheds found that the optimized CN were greater than the weighted CN for four of the six watershed conditions studied (Table 1). For areas with low CN, the SCS procedure may give significant errors. Golding (1979) utilized the SCS CN procedure to simulate runoff from a gauged urban basin in South Florida (58.3 ac., Group A soil, 36% imperviousness, 18% directly connected imperviousness). He found that the computed $I_a$ amounted to 0.86 inches (CN=70), which was greater than the total recorded rainfall on the basin, which had peak flows of up to 40 cfs in many cases. Reduction of the initial abstraction may give a more realistic runoff volume. Figure 3 compares the runoff volumes obtained for different $I_a$ and $I_a = 0.2S$ for storms of 3 return periods (Rowney, 1982). The SCS CN procedure is still a popular and simple tool, which will be around for some time to come. It is felt that with some improvements in the procedure and proper application, the method is still useful.
The methodology used in OTTHYMO involves determining the initial abstraction $I_a$ from the runoff threshold curve obtained from rainfall and runoff records (Jobin, 1982). A program called SECSER has been written in order to do this. The runoff volumes for the storms are then used to calibrate the CN with this $I_a$. The resulting CN are called $CN^*$. Instead of using 3 discrete AMC classes, the antecedent moisture condition is classified by the API (antecedent precipitation index) which is calculated from the hourly rainfall records. The API for each storm is then plotted against the $CN^*$. The $CN^*$ for other storms can then be determined from this $CN^*$-API relationship once the API for these storms are determined. This relationship would be a continuous one as

**FIGURE 3 RELATION BETWEEN RUNOFF VOLUME: $Q(I_a)$ (USING INITIAL ABS. = $I_a$), $Q(0.2S)$ (USING INITIAL ABS. = 0.2S), (ROWNEY, 1982)**
compared to the 3 discrete classes used in SCS. It also would not require the definition of a standard reference moisture condition.

A small program (Figure 4) has been written to calculate the runoff volumes $Q$ for different rainfalls $P$ using the specified $I_a$. The results can then also be plotted on the $Q-P$ chart (Figure 5). These charts are useful for a quick comparison of $CN$ and $CN^*$. Since $CN^*$ are a function of the $I_a$, different charts will result for different $I_a$.

```
C
PROGRAM FOR CN PROCEDURES
C
DIMENSION CN(20),S(20),AI(30),F(20,20),J(20,20),CV(20,20)
CV(1)=.5,
DO 10 J=1,10
CV(J)=CN(J).5,
S(1)=100./CV(1)-10.,
S(2)=C.2*(J)
10 CONTINUE
AI(1)=C.1C
DO 20 J=1,20
AI(J)=AI(1)*C.1C
DO 30 J=1,10
WRITE(6,11) CN(I)
11 FORMAT(10X,'CN= ',F6.0)
WRITE(6,12) S(I)
12 FORMAT(10X,'S(IP)= ',F6.2,' IN.')
WRITE(6,13) S2(I)
13 FORMAT(10X,'S2(IP)= ',F6.2,' IN.')
WRITE(6,14) AI(I)
14 FORMAT(10X,'AI(IP)= ',F6.2,' IN.')
P(I,J)=1.
DO 60 J=1,12
P(I,J+1)=P(I,J)*.7,
IF(P(I,J+1)>AI(J)) GO TO 81
Q(I,J)=[(P(I,J)-AI(J))/P(I,J)-AI(J)]*S(1)
CT(I,J+2(I,J))=F(I,J)
GO TO 82
81 Q(I,J)=0.
82 CONTINUE
WRITE(6,55)(P(I,J),J=1,12)
55 FORMAT(10X,'P(IP,J)= ',F6.2,' IN.')
WRITE(6,56) C(I,J),J=1,12
56 FORMAT(10X,'C(IP,J)= ',F6.2,' IN.')
WRITE(6,17) CV(2,2),J=1,12
17 FORMAT(10X,'CV(IP,J)= ',F6.2,' IN.')
CONTINUE
STOP
END
```

FIGURE 4 PROGRAM FOR CN PROCEDURES
FIGURE 5 HYDROLOGY OF RUNOFF EQUATION, \( Q = \frac{(P-0.2S)}{(P+0.8S)} \)

4.2 CALIBRATION OF THE MODIFIED SCS CN PROCEDURE

The modified SCS CN procedure was tested first on the Seymaz watershed in a joint study by the University of Ottawa and the Ecole Polytechnique Federale de Lausanne who had previously done extensive monitoring. This watershed is composed of 30.2 km\(^2\) of rural areas and 8 km\(^2\) of urban areas and is located in the suburbs of Geneva, Switzerland.

Using the SECSER program and the rainfall and runoff records, a runoff threshold curve can be plotted. From the curve, the initial abstraction \( I_a \) was found to be 1.5 mm. Figure 6 also shows a comparison between the simulations using both \( I_a = 1.5 \) mm and \( I_a = 0.2S \). In the latter case, the first peak cannot be simulated accurately because of the large initial abstraction. The variation of \( CN^* \) with API for some storms on the Seymaz watershed is shown in Figure 7. \( CN^* \) is the calibrated CN obtained by using \( I_a = 1.5 \) mm as obtained from the runoff threshold curve. (It was not possible to find a similar correlation of this type with \( I_a = 0.2S \)). With the \( I_a = 0.2S \) assumption, peak fitting for small rainfalls is possible if the CN values are increased, without consideration of antecedent conditions, to unrealistic values, e.g., \( CN = 90 \) or higher. Two of the simulated storms obtained with the new \( I_a \) are compared with the observed storms in Figure 8 and 9. The \( CN^* \) values used in the simulations are determined from the curve, Figure 7, once the API for the
storms are obtained. A similar $CN^*$-API relationship has been determined for the Etobicoke Creek watershed (Figure 10) in Metro Toronto. One of the typical comparisons between simulated and observed storms is shown in Figure 11.

**TABLE 1 COMPARISON OF CALCULATED AND OPTIMIZED CN (ALTMAN, ESPEY, FELMAN, 1980)**

<table>
<thead>
<tr>
<th>Watershed</th>
<th>Date</th>
<th>CN (calc.)</th>
<th>CN (opt.)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Austin, Texas Region</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waller Creek (urban)</td>
<td>1957-1959</td>
<td>84</td>
<td>92</td>
</tr>
<tr>
<td></td>
<td>1962-1965</td>
<td>84</td>
<td>79</td>
</tr>
<tr>
<td></td>
<td>1971-1973</td>
<td>84</td>
<td>81</td>
</tr>
<tr>
<td>Wilbarger Creek</td>
<td>1964-1975</td>
<td>83</td>
<td>85</td>
</tr>
<tr>
<td><strong>Dallas, Texas Region</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Turtle Creek (urban)</td>
<td>1967-1976</td>
<td>86</td>
<td>93</td>
</tr>
<tr>
<td>Spanky Branch</td>
<td>1973-1975</td>
<td>84</td>
<td>96</td>
</tr>
</tbody>
</table>

**FIGURE 6 OBSERVED AND SIMULATED RESULTS FOR EVENT OF 77/10/24**
FIGURE 7 RELATION OF CN VERSUS API FOR SEYMAZ WATERSHED
FIGURE 8 OBSERVED AND SIMULATED RESULTS FOR EVENT OF 78/08/07

FIGURE 9 OBSERVED AND SIMULATED RESULTS FOR EVENT OF 77/10/24
FIGURE 10 CN*-API RELATIONSHIP FOR ETOBICOKE CREEK WATERSHED

FIGURE 11 COMPARISON OF SIMULATED AND OBSERVED HYDROGRAPHS FOR ETOBICOKE CREEK WATERSHED
4.3 INFILTRATION PROCEDURES IN STANDHYD

4.3.1 HORTON’S EQUATION

For pervious areas, there are two options for calculating the infiltration losses. The first option is Horton’s equation where the infiltration capacity rate is an exponential function of time, which decays to a constant rate. It is written as follows:

\[ f_t = f_c + (f_o - f_c)e^{-\alpha t} \]  

(9)

where:

- \( f_t \) = the infiltration capacity rate (in/hr or mm/hr) at time \( t \);
- \( f_o \) = the initial infiltration capacity rate (in/hr or mm/hr);
- \( f_c \) = the final infiltration capacity rate (in/hr or mm/hr);
- \( \alpha \) = the decay rate (1/hr).

The equation is only satisfactory for the condition that the rainfall intensity is higher than the infiltration capacity rate. To overcome this problem, the cumulative form of the equation can be used. It has the advantage that the infiltration rate becomes a function of the amount of water accumulated into the soil.

\[ F = \int_{0}^{t} f_t \, dt = f_c + \frac{(f_o - f_c)}{\alpha} (1 - e^{-\alpha t}) \]  

(10)

where \( F \) is the cumulative infiltration volume, at time \( t \).

The average infiltration capacity rate during the next time step is

\[ \bar{f}_t = \frac{F(t + \Delta t) - F(t)}{\Delta t} \]  

(11)

In order to determine the actual infiltration rate \( f \), the average infiltration capacity rate is then compared with the average rainfall intensity \( i \) during the time period \( \Delta t \).

If

\[ f = \begin{cases} \bar{f}_t & \text{if } i > \bar{f}_t \\ i & \text{if } i < \bar{f}_t \end{cases} \]  

(12)

then the calculation proceeds to the next time step with the cumulative infiltration volume at \( F(t + \Delta t) \). If \( f = i \), then the actual cumulative infiltration would be

\[ F_{act.} = F(t) + i\Delta t \]  

(13)

where

\[ F_{act.} < F(t + \Delta t) \]  

(14)
The new time $t_1$, which would correspond to the cumulative infiltration $F_{act}$, is determined by means of an iterative process. The calculation then continues from this point for the next time step.

The antecedent moisture condition can be represented by the water, $F$, accumulated into the soil before the start of the storm. $F$ can be directly specified as input. The other infiltration parameters also need to be specified.

For a decay rate of 4.0 hr$^{-1}$ the infiltration capacity rate declines 98% towards the limiting value $f_c$ after 1 hour (if the rainfall intensity is always higher than the infiltration capacity rate). For $\alpha = 2.0$ hr$^{-1}$, the decline is 76% after 1 hour. This should be considered when selecting the time increment $\Delta t$ for computation.

![Figure 12 CUMULATIVE FORM OF HORTON’S INFILTRATION EQUATION](image)

**4.3.2 MODIFIED CN PROCEDURE**

The second option for infiltration losses in the previous area is the modified CN procedure, which is used in NASHYD.

**4.4 CONSIDERATIONS IN USING THE RAINFALL LOSSES**

For flood control purposes and master drainage planning, there are both rural and urban areas in the watershed. In Visual OTTHYMO, the rainfall losses in the rural areas are computed by means of the CN* procedure. The critical storms for rural conditions are long-duration storms such as the Southern Ontario Regional Storm with a peak intensity of 2.08 in/hr. The modified SCS method (CN*) is used in such conditions. The Horton model may result in underestimating the runoff mainly for low intensity storms, since it generates runoff only if the rainfall intensity is higher than the infiltration capacity rate. In such cases, the rainfall losses in the pervious portion of the urbanized areas should also be computed with the CN* procedure. The ratio of the peak rainfall excess intensity to the peak rainfall intensity is an indicator of the effect of rainfall loss model. This ratio is called $R_i$ and Figure 12 shows $R_i$ against CN* and the maximum infiltration capacity rate for (Horton) for the Regional Storm. $R_i$ for this storm would be sensitive to the $f_c$ (minimum infiltration capacity rate) selected.

If the same storm is used in studying the effects of urbanization (e.g. comparing pre- and post-development flows), the CN* procedure can continue to be used for post-development conditions with STANDHYD.
For design purposes under urban conditions, however, the critical storms are the short-duration, high intensity storms such as the Chicago-type storms. Here Horton’s procedure is preferred because it is more sensitive to the storm intensity and in general results in higher peak flows than the CN* procedure. This is shown in Figure 13 for a residential watershed (30% imperviousness) for three storms, the 5-year, 100-year Chicago and the Regional storms.

A series of numerical experiments have been done to find a range of values in which the Horton and CN* procedures would give the same runoff volumes. The runoff volumetric coefficient $C_v$ was calculated for different combinations of $f_o$, $f_c$ and values (Horton) and CN* values (with $I_a = 0.10$ in). The range of values tested were 1.0 to 5.0 in/hr for $f_o$, 0.10 to 0.50 in/hr for $f_c$ and 2.0/hr and 4.14/hr for $\alpha$ (decay constant). The results are shown in Figure 14 and 15. The peak flows for a 121-acre residential watershed for the values shown in Figure 15 are plotted in Figure 13. It is observed that equivalent $C_v$ does not mean that the corresponding peak flows are equivalent.

It is also found that total runoff for the Regional storm is more sensitive to $f_c$ while for the Chicago storms they are more sensitive to $f_o$. There is no range of values for which the $C_v$ are matched for all three storms. Figure 14 shows that the $C_v$ for the Regional storm can be matched by varying $f_c$ and Figure 15 show that the $C_v$ for the Chicago storms can be matched by varying $f_o$.

These results show that for consistency the selection of infiltration parameters should consider the characteristics of the soil and also those of the storm. Tables given in literature in which infiltration parameters like $f_o$, $f_c$ and CN are given in terms of soil groups A, B, C, D alone may not give consistent results.

If data is available and the CN*-API relationship has already been derived during the planning stage, the CN* procedure can also be used for design purposes. The use of the CN* procedure with design storms is discussed in the section on design storms. This will result in compatibility between the planning and the design stages for the watershed.
FIGURE 13 $R_i$ VERSUS CN* AND $f_o$: REGIONAL STORM ($f_c=$0.30 in/hr, $\alpha=2.0$/hr)

FIGURE 14 PEAK FLOWS FOR RESIDENTIAL WATERSHEDS (30% IMPERV.) (($f_c=$0.30 in/hr, $\alpha=2.0$/hr))
FIGURE 15 CV VERSUS CN* AND $f_c$ WITH $f_o=3$ in/hr, $\alpha=2$, $I_a=0.10$ in.

FIGURE 16 CV VERSUS CN* AND $f_c$ WITH $f_o=3$ in/hr, $\alpha=2$, $I_a=0.10$ in.
5  UNIT HYDROGRAPH OPTIONS IN VISUAL OTTHYMO

In Visual OTTHYMO, the response of a watershed to the effective rainfall is obtained by convolution of a short duration unit hydrograph (UH) derived from the theory of conceptual "instantaneous unit hydrographs" or IUH. The characteristics of these unit hydrographs are not dependent on rainfall duration. However, depending on the size of the area being simulated, their use usually requires short computational time steps (1 to 15 minutes).

Visual OTTHYMO has three types of IUH's which have a common parameter, the time to peak, \( t_p \). Another parameter, \( K \), is related to the hydrographs’s recession limb. \( K \) is also called a ‘storage coefficient’ and has different values in each IUH.

Another option in Visual OTTHYMO is the SCS non-dimensional UH, which is a specific NASH IUH defined only by \( T_p \) (see IUH Relations).

For \( t_p \) equal to the computational time step, the STANDARD IUH is identical to the single linear reservoir IUH from the URBHYD command in OTTHYMO 83.
### 5.1 IUH Relations

<table>
<thead>
<tr>
<th>TYPE OF IUH</th>
<th>RELATION</th>
<th>REMARKS</th>
</tr>
</thead>
</table>
| STANDARD    | \[ \frac{q}{q_{\text{peak}}} = \frac{t}{T_p} \quad \text{for } t < T_p \]
|             | \[ \frac{q}{q_{\text{peak}}} = e^{-(t-T_p)/k} \quad \text{for } t > T_p \] | For \( T_p = DT \) (the computational time step) the STANDARD IUH becomes the URBHYD IUH from OTTHYMO 83. |
| NASH        | \[ \frac{q}{q_{\text{peak}}} = \left( \frac{t}{T_p} \right)^{(N-1)} e^{\left(1-N\right)\left(\frac{t}{T_p}\right)} \] | \( N = T_p / k + 1 \)
|             | \( N \) is also the “number of reservoirs” |
| WILLIAMS    | for \( t < t_o \) \[ \frac{q}{q_{\text{peak}}} = \left( \frac{t}{T_p} \right)^{(N-1)} e^{\left(1-N\right)\left(\frac{t}{T_p}\right)} \] \( q_{\text{peak}} = \left[ 1 / \left( K_n \Gamma(N) \right) \right] e^{\left(1-N\right)\left(N - 1\right)^{(N-1)}} \) \( t_o \) is the inflection point after the peak; \( K_n \) is the storage coefficient of each reservoir; \( N \) is the number of reservoirs, and \( \Gamma(N) \) is the gamma function. |
|             | and, \( q_{\text{peak}} = \left[ 1 / \left( K_n \Gamma(N) \right) \right] e^{\left(1-N\right)\left(N - 1\right)^{(N-1)}} \) |
|             | for \( t_o < t < t_1 \) (where \( t_1 = t_o + 2k \)) \[ \frac{q}{q_{o}} = e^{(t_o-t)/k} \] Calibration recommended. |
|             | for \( t_1 < t \) \[ \frac{q}{q_{o}} = e^{(t_1-t)/3k} \] |
| SCS         | Is the NASH IUH with \( N = 5 \) |
5.2 THE STANDARD IUH

The standard IUH is used mainly for urban areas with pervious and impervious contributions calculated separately.

The standard IUH was developed and tested in Germany by Verworn and Harms in 1978. It is used in the model HYSTEM. In Visual OTTHYMO, $T_p > DT$ and therefore, for a given storm, $T_p$ varies with the size of the watershed. (The URBHYD command in OTTHYMO 83 is equivalent to a STANDARD IUH with the time to peak equal to the time step, DT).
A relation derived from overland routing by the kinematic wave method (Peterson and Altera) gives the storage coefficient, $K$. This relation is close to the relation by Neumann used in HYS-TEM.

$$K = C \frac{L^{0.6} \cdot n^{0.6}}{i^{0.4} \cdot s^{0.3}}$$

where:

- $L$ = an equivalent flow length which requires calibration. A default value for impervious areas obtained from $A = 1.5L^2$ where $A$ is the watershed area, was frequently tested with measurements. For pervious areas, the default value is $L = 40$ m, representing an average travel length on inter-spaced green areas.
- $n$ = the roughness coefficient. Testing shows that adequate results are obtained with $n = 0.013$ for impervious areas and $n = 0.25$ for pervious areas.
- $i$ = the dominant rainfall intensity (maximum average intensity during $K$).
- $s$ = the characteristic slope in m/m.
- $C$ = a constant (0.00775 for $L$ in feet, $i$ in inches/hour).

The STANDHYD command is based on analysis of comparisons with measurements and practical applications. In STANDHYD, the dominant rainfall intensity is averaged over the duration of $K$. Since $K$ varies with rainfall intensity this IUH varies from one rainfall to the other, the STANDARD IUH is a quasi-linear model.

For the impervious area, the time to peak, $T_p$, in the STANDARD IUH of Visual OTTHYMO is equal to the storage coefficient, $K$. For the pervious areas, fragmented in backyards and connected to storm sewers, $T_p$ is equal to $K$ pervious + $K$ impervious, at time of convolution, $T_p$ is rounded to the nearest multiple of the time step, $DT$.

In the STANDHYD command, the pervious hydrograph and the impervious hydrograph have, in general, different $T_p$ values. There is also a lag between the peak discharge of the total hydrograph and the end of the peak rainfall intensity.

For watersheds with large estate lots and semi-urban areas with relatively large pervious components, it is recommended to simulate two component hydrographs:

- a) The first, an equivalent smaller urban area can be simulated with STANDHYD.
- b) The remaining area which is only (or mostly) pervious, can be simulated with NASHYD.

The equivalent urban area and the imperviousness of this area (e.g., say 30 %) satisfy the following rule of thumb:

$$Equivalent A_{urban} * 0.30 = A_{total} * (Real \ imperviousness)$$

For very large urban areas (> 200 hectares), STANDHYD requires calibration.
5.3 THE NASH IUH (NASHYD)

This linear IUH is used mainly for rural areas. With Nash, the peak discharge increases with N and decreases with Tp. Measurements in Ontario and in Switzerland indicate that an average of 3 number of linear reservoirs may be appropriate.

The time to peak, Tp, is obtained from the time of concentration, Tc:

\[ Tp = \frac{(N-1)}{N} Tc \]

where, N, is the number of linear reservoirs

\[ Tp = 0.67 Tc \]

In general, the time of concentration, Tc, can be determined using one of three methods:

1. **Empirical formulas** (only recommended if they are based on regional verifications).
2. **Velocity methods**, \( Tc = 3 \frac{(L_i)}{V_i} \). The overland velocities are determined with an SCS graph, and channel velocities can be determined from Manning’s equation.
3. **Kinematic wave method** (which accounts for the rainfall intensity).

The NASHYD command is used for non-homogeneous areas, and in the case of SCS abstraction methods, uses a weighted average of CN. Comparisons with measurements show a better performance if the response from the pervious and impervious areas are simulated separately.

For very large urban areas (> 200 hectares), NASHYD requires calibration.

Furthermore, if the response time of an urban watershed is increased by significant channel storage, this effect must be simulated by channel routing (unless Tp is calibrated).

5.4 THE SCS IUH (SCSHEYD)

The shape of the SCS UH is obtained from the NASH relation, with N=5. This value is greater than the one determined from studies in Ontario, Switzerland, and the United Kingdom. It is, however, conservative if the time to peak is correct.

The SCS non-dimensional unit hydrograph is used by SCS abstraction methods for both rural and urban areas. Comparisons with measurements show that even if Tp, Ia, and CN* are calibrated, the proper shape of the hydrograph is not always generated.

The 1986 SCS TR-55 publication indicates the following limitations:

1. Hydrographs obtained by this method are not developed for comparisons with measurements.
2. The method should only be used in cases where runoff is greater than 12.5 mm.
3. The lag formula given in previous SCS publications is no longer recommended (it may underestimate the peak flow).

The SCS methods apply the non-dimensional UH in conjunction the SCS CN method with the assumption that Ia = 0.2 x S. Although this may overestimate the rainfall losses, it was maintained in the SCS command for special agency requests.
It is recommended the to determine \( T_c \) with the velocity method:

\[
T_c = 3 \left( \frac{L}{V_i} \right)
\]

### 5.5 The Williams IUH

The method is recommended for rural watersheds where observations indicate a long hydrograph recession limbs. The Williams formula for \( T_p \) is not recommended in Ontario as it has been shown to give significant errors.

**FIGURE 17 WILLIAMS IUH**

As is predecessor (INTERHYMO / OTTHYMO.89), Visual OTTHYMO does not recommend default values for \( K \) and \( T_p \) in the Williams command, since it is considered that this IUH requires calibration.

**FIGURE 18 COMPARISON OF WILLIAMS AND NASH IUH**

[Image of graphs showing hydrographs]
5.6 USE OF IUH’S FOR I/I SIMULATION AND BASEFLOW (DWF)

Visual OTTHYMO can be used to simulate the Infiltration/Inflow into sanitary sewers or combined sewers. The four types or rainfall-induced infiltration/inflow are:

1. Fast responses from directly connected impervious areas.
2. Rapid responses from grassed areas in combined sewers systems.
3. Semi-rapid responses from weeping tiles.
4. Slow responses from cracked pipes and leaking joints in the sewers.

Visual OTTHYMO can simulate these responses during a single event by adding individual response hydrographs from each type of contributions within the same area. The first three responses can be simulated with the quasi-linear instantaneous Unit Hydrograph (STANDHYD) while the fourth, slow response, can be simulated with the NASH unit hydrograph.

Baseflow can be super-imposed to account for the domestic sewage contributions during wet conditions.

5.7 UNIT HYDROGRAPH OPTIONS FOR RURAL AREAS

For computation of flows from rural watersheds, the subroutines NASHYD, WILHYD or SCSHYD (NASHYD with N=5) can be used. The rainfall excess distribution is obtained by means of a modified CN procedure, which is then convoluted with the unit hydrograph obtained by means of the Nash model (NASHYD) or the Williams and Hann unit hydrograph (WILHYD).

5.7.1 INSTANTANEOUS UNIT HYDROGRAPH

Many ways of deriving synthetic unit hydrographs or IUH have been proposed since the early studies of Snyder in 1938. One frequently used way is by means of a conceptual model made up of a cascade of equal, linear reservoirs, first proposed by Nash in 1957 (Figure 19). The IUH for Nash’s model can be written as:

\[ q(0,t) = \frac{1}{K_n \Gamma(n)} e^{-t/K_n} \left(\frac{t}{K_n}\right)^{n-1} \]  \hspace{1cm} (15)

where:

\[ \Gamma(n) = \text{the gamma function}; \]
\[ n = \text{the number of reservoirs}; \]
\[ K_n = \text{the storage coefficient of each reservoir}. \]

By differentiating Equation 15 with respect to \( t/K_n \) and equating to zero, the time to peak \( t_p \) in terms of \( n \) and \( K_n \) is obtained.

\[ t_p = (n - 1)K_n \]  \hspace{1cm} (16)
The peak flow then becomes

\[ q_p = \frac{1}{K_n \Gamma(n)} e^{1-n} (n - 1)^{n-1} \]  

(17)

By substituting Equations 16 and 17 in Equation 15, the 2-parameter gamma equation is obtained

\[ q = q_p \left( \frac{t}{t_p} \right)^{(n-1)} e^{(1-n)(\frac{t}{t_p} - 1)} \]  

(18)

Williams and Hann (1973) use this equation from the time of rise to the inflection point for the IUH in WILHYD. Figure 19 shows the variation of the outflow hydrograph from NASHYD, with the number of reservoirs, \( n \), for a fixed time to peak.

As shown in Figure 19, for the same time to peak, the peak flow is sensitive to \( n \) in the range 2 to 6. The parameter \( n \), can be a non-integer. The calibration of watersheds with areas of less than 15 km\(^2\) on the Seymaz and Etobicoke studies presented in the previous section has shown that a first estimate for \( N = 3 \) can be used if data is unavailable. For consistency, the various subwatersheds should use the same \( n \) unless data is available for each subwatershed.
5.7.2 ESTIMATION OF TIME TO PEAK ($t_p$) IN NASHYD

It is, of course, best to obtain $t_p$ by calibration with measurements. If data is available, the following procedure may be utilized to estimate $t_p$.

FIGURE 20 DEFINITION OF TIME LAG

The first step involves determining the time lag $t_L$, which is defined as the time difference between the centroids of the rainfall excess hyetograph and the direct runoff hydrograph (after subtracting baseflow). $t_L$ is related to $n$ and $K_n$ in the Nash conceptual model by

$$t_L = nK_n$$

Once $T_L$ is determined and $n$ is estimated by 3 for example, then $t_p$ can be obtained by equation 16.

$$t_p = (n - 1)K_n$$

If $n = 3$, $t_p = 0.667t_L$

Since measurements are usually available only at the outlet of a watershed, the $t_p$ values would still have to be determined for each subwatershed after discretization. The main parameters that affect $t_p$ are the slope and the area. Since in small watersheds the slope does not vary too much, an approximate relation $t_p = m(\text{area})^n$ can be utilized. With the calibrated $t_p$ at the outlet, constants $m$ and $n$ can be obtained by trial and error.

In the Seymaz and Etobicoke studies, the Williams and Hann equation for $t_p$ was found adequate. For smaller watersheds, the $t_p$ values obtained can be checked by using the velocity charts in the SCS TR-55 tables (1975) for overland flow and swale flow.

Several relations for $t_p$ or $t_L$ can be found in the literature such as Chow (1962), Kibler et al (1982), Boyd (1978) and Nash (1960).

5.7.3 WILLIAM'S UNIT HYDROGRAPH

WILHYD is the subroutine that uses the unit hydrograph proposed by Williams and Hann (1973). The unit hydrograph is divided into three parts for computation. The first part, from the beginning of rise to the inflection point, $t_o$, is computed by the 2-parameter gamma distribution equation
(Equation 18). The second part from the inflection point, \( t_o \) to \( t_1 \) where \( t_1 = t_o + 2K \), is computed by

\[
q = q_o e^{\frac{t_o-t}{K}}
\]  

(20)

The third part from \( t_1 \) onwards is computed by

\[
q = q_1 e^{\frac{t_1-t}{3K}}
\]  

(21)

\( n \) is computed as a function of \( K/t_p \) and \( q_p \) is a function of \( n \) and \( t_p \). Therefore only 2 parameters, \( K \) and \( t_p \) are necessary to compute the entire unit hydrograph. Empirical relations have been derived for \( K \) and \( t_p \) (Williams 1977) based on Southern U.S. watersheds. These relations may not be applicable in other areas.

\[
K = 16.1A^{0.24}S^{-0.84}
\]  

(22)

\[
t_p = 6.54A^{0.39}S^{-0.50}
\]  

(23)

where:

- \( K \) = the recession constant (hr);
- \( t_p \) = the time to peak (hr);
- \( A \) = the watershed area (sq.miles); and
- \( S \) = the difference in elevation in feet, divided by flood plain distance in miles, between watershed outlet and most distant point on the watershed.

The unit hydrograph in WILHYD has a longer recession tail than that in NASHYD and a smaller peak. It can therefore be used in those watersheds where the recession limb is longer.

A comparison of the two unit hydrographs is shown in Figure 21.
FIGURE 21 COMPARISON OF UNIT HYDROGRAPH BY: (I) WILLIAM'S AND HANN'S METHOD (II) NASH'S METHOD
6 Routing Options In Visual OTTHYMO

<table>
<thead>
<tr>
<th>METHOD</th>
<th>BRIEF DESCRIPTION</th>
<th>COMPLEXITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>SHIFT HYDROGRAPH</td>
<td>A simple translation of the hydrograph. Does not attenuate the peak discharge.</td>
<td>LOW</td>
</tr>
<tr>
<td></td>
<td>Combines the three routing commands of HYMO into a single command, based on the</td>
<td></td>
</tr>
<tr>
<td></td>
<td>hydrologic method VSC (Variable Storage Coefficient).</td>
<td></td>
</tr>
<tr>
<td>ROUTE CHANNEL</td>
<td>Applies the Muskingum-Cunge method of routing, which is based on the continuity</td>
<td>HIGH</td>
</tr>
<tr>
<td></td>
<td>equation and the storage-discharge relation.</td>
<td></td>
</tr>
<tr>
<td>ROUTE MUSKCUNGE</td>
<td>Applies the VSC method for conduits, and gives the minimum size to avoid surcharge.</td>
<td>MEDIUM</td>
</tr>
<tr>
<td>ROUTE PIPE</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.1 Simulation Time Steps

The storm time step is determined by the format of meteorological data. For synthetic storms it is usually five to ten minutes. The hydrograph computational time step, DT, is determined from the watershed characteristics. For example:

Convolution with NASHYD requires DT < Tp (time to peak - preferably DT about 1/5 Tp)

Visual OTTHYMO will transform automatically for each sub-watershed, new storm input with the time step DT.

In routing with the VSC method, it is recommended to maintain a small time step. Although this is not required for mathematical stability, Ponce and others recommend short time steps and the use of the Courant criterion for hydrologic routing.

\[
DT = \frac{LENGTH}{CELERITY}
\]

The celerity is given by:

\[
CELERITY = \frac{Q}{\sqrt{g \times Avg. Depth}}
\]

Celerity ranges from 1.1 to 1.6 times the average velocity. Using the above criterion, it is found that, for time steps used in convolution (hydrograph commands) the length cannot be very short.

For short reaches, the hydrograph should be simply ‘shifted’ in time. In comparison, routing with EXTRAN is usually conducted with time steps of 2 to 10 seconds, and gives an error message if the courant criterion is not met.
6.2 TIME SHIFT ROUTING

For discharges close to critical or supercritical flow, and for very short reaches (with time step constraints), SHIFT HYD can be used. Comparisons with the kinematic wave method show that, for a circular conduit, the time lag can be selected with the relation

\[ \text{Time lag} = \frac{\text{reach length}}{(\alpha - \text{full pipe velocity})} \]

Where \( \alpha \) is given by the following table:

<table>
<thead>
<tr>
<th>( \frac{Q_{\text{peak flow}}}{Q_{\text{full}}} )</th>
<th>ALPHA</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.40</td>
<td>1.10</td>
</tr>
<tr>
<td>0.60</td>
<td>1.17</td>
</tr>
<tr>
<td>0.80</td>
<td>1.19</td>
</tr>
<tr>
<td>1.00</td>
<td>1.11</td>
</tr>
</tbody>
</table>

6.3 VARIABLE STORAGE COEFFICIENT ROUTING IN VISUAL OTTHYMO

Like other hydrologic routing methods, the variable storage coefficient (VSC) is based on the continuity relation. It does not apply empirical or calibrated parameters. It calculates channel storage based on average channel characteristics, and travel time based on Manning's relation. It can be used for artificial and natural channels with three roughness coefficients in the same cross-section.

In Visual OTTHYMO, the three routing commands of the original HYMO model are lumped in a single command ‘Route Channel’. The VSC routing cannot be used when backwater effects are significant. In such cases, a fully dynamic model (e.g. EXTRAN should be used).

For circular or rectangular pipes, ROUTE PIPE command should be used. The command sizes the pipe to the minimum diameter necessary to avoid surcharging. For design, the user should increase the size to the next standard diameter.

6.4 MUSKINGUM-CUNGE CHANNEL ROUTING

The Muskingum method is based on the continuity equation and the storage-discharge relation. Cunge (1969) extended the method into a finite-difference scheme. The Muskingum-Cunge channel routing technique is a non-linear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflow hydrograph. The advantages of this method over other hydrologic techniques are:

- it is very simple conceptually, and can be readily applied by desk calculation, and is much cheaper than the other methods when applied by computer,
- it would be advantageous to use the Muskingum-Cunge method for rivers that have major tributaries and are not well gauged,
- this method can include a tributary as a discrete lateral inflow, which the other methods cannot do in a simple way,
• the hydrologic approach greatly improves computational efficiency and speed, and reduces the amount and detail of field data traditionally needed for hydraulic routing,
• the parameters of the model are physical based, the scheme is stable with properly selected coefficients,
• the method has been shown to compare well against the full unsteady flow equations over a wide range of flow situations,
• it produces consistent results in that the results are reproducible with varying grid solution,
• it is comparable to the diffusion wave routing,
• it is largely independent of the time and space intervals when these are selected within the spatial and temporal resolution criteria,

The major limitations are:
• it cannot account for backwater effects,
• the method begins to diverge from the full unsteady flow solution when very rapidly rising hydrographs are routed through flat channel sections,
• a disadvantage with the Muskingum-Cunge method arises when there is a disturbance such as a tide affecting the flow in the river upstream of the downstream boundary,
• it does not accurately predict the shape of the discharge hydrograph at the downstream boundary when there are large variations in the kinematic wave speed, such as due to the inundation of a large flood plain.

6.4.1 Basic Flow Equations
The outflow hydrograph at the downstream end is calculated using the following formula.

\[ Q_{j+1} = C_1 Q_j + C_2 Q_{j+1} + C_3 Q_{j+1} + C_4 \]  \hspace{1cm} (24)

where:

\[ C_1 = \frac{Kx + \frac{\Delta t}{2}}{D} \]  \hspace{1cm} (25)

\[ C_2 = \frac{\frac{\Delta t}{2} - Kx}{D} \]  \hspace{1cm} (26)

\[ C_3 = \frac{K(1 - x) - \frac{\Delta t}{2}}{D} \]  \hspace{1cm} (27)

\[ C_4 = \frac{q\Delta t\Delta x}{D} \]  \hspace{1cm} (28)

\[ D = K(1 - x) + \frac{\Delta t}{2} \]  \hspace{1cm} (29)
Where:

\[ Q = \text{discharge} \]
\[ K = \text{travel time in seconds} \]
\[ x = \text{weighting factor, } 0 \leq x \leq 0.5 \]
\[ \Delta x = \text{subreach length} \]
\[ \Delta t = \text{time interval} \]
\[ q = \text{lateral flow} \]
\[ c = \text{wave celerity} \]

The parameters of \( K \) and \( x \) are expressed as follows (Cunge, 1969 and Ponce, 1978):

\[
K = \frac{\Delta x}{c} \quad (30)
\]

\[
x = \frac{1}{2} \left( 1 - \frac{Q}{cBS\Delta x} \right) \quad (31)
\]

where:

\[ B = \text{top width} \]
\[ S = \text{the channel slope} \]

6.4.2 Solution of Flow Equations

The outflow hydrograph is iterative and is calculated based on equation 24, the routing coefficients \((C_1, C_2, C_3, C_4)\) are re-calculated for every distance step \(\Delta x\) and calculation time step \(\Delta t\).

**Numerical Stability**

\(\Delta t\) and \(\Delta x\) are chosen internally by the model for accuracy and stability.

\(\Delta t\) is selected as the smallest of the following 3 rules:

1. the user defined computation interval, DT,
2. the time of rise of the hydrograph divided by 20,
3. the travel time of the channel reach.

The model checks the difference between the computational time interval (DT) and the time increment of the inflow hydrograph (SDT). If DT is less than SDT, the inflow hydrograph will be interpolated. The calculation time step must be equal or less than the inflow hydrograph SDT.

A computational space increment \(\Delta x\) can be equal to the length of the entire routing reach or to a fraction of that length. It is initially selected as the entire reach length. If the size of this space increment does not meet the accuracy criteria for flow routing given by Ponce and Theurer (1982), it is re-evaluated by subdividing the length of the routing reach into even subreaches that produce \(\Delta x\)’s that satisfy the accuracy criteria.
where,

\[ \Delta x = \frac{1}{2} \left( c \Delta t + \frac{Q}{B SC} \right) \]  \hspace{1cm} (32)

where,

\[ Q = Q_B + 0.5 \times (Q_p - Q_B) \]  \hspace{1cm} (33)

\( Q_B \) = baseflow from the inflow hydrograph
\( Q_p \) = peak flow from the inflow hydrograph

The Courant (C) number can be defined as:

\[ C = \frac{c \Delta t}{\Delta x} \]  \hspace{1cm} (34)

Main and overbank channel portions are separated and modelled as two independent channels. Right and left overbanks are combined into a single overbank channel.

Momentum at the flow interface between the two channel portions is neglected, and the hydraulic flow characteristics are determined separately, for each channel portion. At the upstream end of a space increment, the total inflow discharge is divided into main channel and overbank flow components. Each are then routed independently, using the previously described routing scheme. The flow redistribution between the main and overbank channels is based on Manning’s equation.

6.4.3 DATA REQUIREMENTS

Data required for the Muskingum-Cunge method are as followings:

- channel length
- main channel bed slope
- floodplain bed slope
- beta parameter (a function of the storage-discharge curve)
- channel cross section data
- number of cross section segments
- Manning roughness coefficient

6.4.4 SIMULATION RESULTS

The channel routing in Visual OTTHYMO was tested using a natural channel, 5200 m long, main channel bed slope is 0.001, Manning's n is 0.03, floodplain bed slope is 0.001, Manning’s n is 0.05, no lateral flow, the cross section parameters are shown in Figure 22.
FIGURE 22 NATURAL CHANNEL

The simulation results from Visual OTTHYMO-MC are compared with the complete unsteady flow equation (SWMM-EXTRAN) and Visual OTTHYMO-VSC and are shown below in Figure 23.

FIGURE 23 COMPARISON OF TEST RESULTS

The results show that the Muskingum-Cunge (MC) routing method compares very well with the complete unsteady flow equations of EXTRAN. The peak discharge is attenuated slightly more from EXTRAN than that from the MC method; however, the time to peak for both methods is the same. The difference in peak discharges could be due to the fact that the inertial terms in the complete unsteady flow equations are becoming more dominant when rapidly rising hydrographs are routed through the flat channel, compared to the bed slope, as the channel slope is decreased. The Muskingum-Cunge routing method does not account for the inertial effects, and consequently the method tends to show more diffusion than what may actually occur.
7 Design Storms for Stormwater Management Studies

Flow simulation for urban drainage studies is mostly done with one-event simulation models. The single event models determine flows produced by a single storm event. Continuous simulation models require rainfall data over a continuous period for the desired length of analysis. A frequency analysis is then conducted on the peak flows so that a flow of a desired return period may be found.

The flow with a single event model may be found by using a series of selected historical events or by using a ‘design storm’. The historical storm series may be selected using a continuous simulation program or by analyzing a rainfall record using a selection criteria. Each event in the selected series is then run through the event simulation model. The generated peak flows are then analyzed to determine their return period.

Design storms or model storms are single event rainfalls that are assumed to produce flows of a desired return period. They are of two types; synthetic design storms and historic design storms. Synthetic design storms are storms developed from intensity-duration-frequency (IDF) curves. Historic design storms are large single storm events; usually containing the maximum precipitation on record. In southern Ontario, hurricane Hazel is used as an historic design storm. In this text only synthetic design storms are examined.

Each design storm has a unique temporal variation of intensity. Two general methods are used to determine the hyetograph shape. The first method derives the storm pattern based on an IDF curve. The design storms using only an IDF curve are the Uniform design storm, the Composite design storm and the Chicago design storm. The second method obtains the temporal structure of the design storm from an analysis of historic storm events. These are the U.S. Soil Conservation Service (SCS) 24-hour design storm, the SCS 6-hour design storm, the Illinois State Water Survey (ISWS) design storm, the Atmospheric Environment Service (AES) design storm, the Flood Studies Report (FSR) design storm, the Pilgrim and Cordery design storm and the Yen and Chow design storm. Design storms that are not discussed are the Sifalda design storm, the Hamburg design storm and the Desordes (French) design storm. A more detailed description of each design storm is contained in the Design Storm Profiles section of this document. Table 2 summarizes the main characteristics of these design storms.

Each of the design storms has a different hyetograph shape. Storm hyetographs were constructed and compared for some of the design storms. A five year return period was selected and the storm volumes were obtained from the Bloor Street station (Toronto) IDF curve. The duration of the storms are not all the same, for this reason the storm volumes are different. The storm hyetographs for the Uniform, Composite, Chicago, SCS 24-hr., ISWS, AES, FSR, and Yen and Chow design storms are shown in Figures 24 and 25.
### TABLE 2 SUMMARY OF DESIGN STORM CHARACTERISTICS

<table>
<thead>
<tr>
<th>Design Storms</th>
<th>Design Return Period</th>
<th>Storm Duration</th>
<th>Total Rainfall Depth</th>
<th>Temporal Distribution</th>
<th>Antecedent Moisture Conditions</th>
<th>Intended Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform</td>
<td>User spec.</td>
<td>t_c</td>
<td>i t_c</td>
<td>No variation in intensity</td>
<td>No</td>
<td>Sewer sizing</td>
</tr>
<tr>
<td>Composite</td>
<td>User spec.</td>
<td>t_d</td>
<td>i t_d</td>
<td>User selected</td>
<td>No</td>
<td>Sewer sizing</td>
</tr>
<tr>
<td>Chicago</td>
<td>User spec.</td>
<td>Usually between 2-6 hrs. T</td>
<td>i T</td>
<td>Based on an IDF curve</td>
<td>No</td>
<td>Sewer sizing</td>
</tr>
<tr>
<td>SCS 24-hr.</td>
<td>User spec.</td>
<td>Long duration usually 12-24 hrs. T</td>
<td>i T</td>
<td>Tabulated type 1 &amp; 2 distributions</td>
<td>Yes</td>
<td>Rural watersheds</td>
</tr>
<tr>
<td>SCS 6-hr.</td>
<td>User spec.</td>
<td>6 hrs.</td>
<td>Given in maps</td>
<td>Tabulated</td>
<td>Yes</td>
<td>Design of small dams</td>
</tr>
<tr>
<td>ISWS</td>
<td>User spec.</td>
<td>1 hr.</td>
<td>i T</td>
<td>Huff 1st quartile 50% distribution</td>
<td>No</td>
<td>Sewer sizing</td>
</tr>
<tr>
<td>AES</td>
<td>User spec.</td>
<td>1 or 12 hrs. T</td>
<td>i T</td>
<td>Regional charts for the 1 and 12 hr. durations</td>
<td>No</td>
<td>Not specified</td>
</tr>
<tr>
<td>FSR</td>
<td>User spec.</td>
<td>12, 30, 60, 120 min. t_d</td>
<td>i t_d</td>
<td>50% summer profile</td>
<td>Yes</td>
<td>Non urban studies</td>
</tr>
<tr>
<td>Pilgrim &amp; Cordery</td>
<td>User spec.</td>
<td>User spec. T</td>
<td>i T</td>
<td>Local analysis of storm events</td>
<td>Yes</td>
<td>Urban and rural areas</td>
</tr>
<tr>
<td>Yen &amp; Chow</td>
<td>User spec.</td>
<td>t_c</td>
<td>i t_c</td>
<td>Triangular</td>
<td>No</td>
<td>Drainage facilities in small areas</td>
</tr>
</tbody>
</table>

- $t_c$ – time of concentration
- $T$ – user selected storm duration
- $t_d$ – storm duration selected using an iterative procedure. The design storm is tested using different durations. The one with the largest peak flow is selected.
- $i$ – average intensity for the return period and selected duration.

All of the design storms are different. The peak intensities, storm profiles, durations and volumes vary even though they all have the same return period. The Uniform design storm has the lowest intensity. It has a constant intensity and is not recommended for use with an event simulation model. The Chicago design storm has a high peak intensity. The peak intensity of this storm depends on the time step one selects. In Figure 25 the time step was increased from 5 to 10 minutes this reduced the peak intensity by 29% from 168 mm/hr to 120 mm/hr. The FSR and Composite design storms also have high peak intensities, but their shapes are not similar. The ISWS, SCS 24-hour, AES and Yen and Chow design storms have peak intensities that are in the same range. The peak rainfall for the SCS 24-hour and the Yen and Chow storms that were computed are the same.

The wide variety of hyetograph profiles is why design storms of the same return period will not produce the same peak flows. Studies are therefore required to determine if design storms can be used to predict flows of a desired return period.
A review of previous studies showed that there is contradictory opinion regarding the use of design storms. Marsalek (1978) does not recommend the use of design storms while the results of Arnell (1982) and Watson (1981) suggest that design storms should be used. Other researchers have concluded that further studies are required.

Those who recommend the use of design storms consider that their advantages outweigh the shortcomings. The advantages of using design storms are that:

1. They are an inexpensive procedure for obtaining flows of a desired return period.
2. If properly selected they give conservative results for peak flows and volumes.
3. They are widely used in current engineering practice.

Some of the disadvantages of design storms are that:

1. The runoff frequency is assumed to be the same as the rainfall frequency. This equivalence of return period has not been shown to be true.
2. The rainfall volume is not the rainfall volume of real storm events.
3. Using IDF relationships to obtain a design storm hyetograph may be incorrect.

A study was conducted using IMPSWM procedures to test two design storms commonly used by Canadian engineers. The uniform design storm was not tested because of its low, unrealistic intensity. The Chicago and the SCS 24-hr design storms were selected for the study. The AES design storm could have been used but the 30% profile with a 1-hour duration gives peak flow results close the Chicago storm and historical storm flows (Wisner and Gupta, 1980). With a 12-hour duration the AES 50% profile gives results similar to the SCS 24-hr design storm. The third chapter contains the results that compare the Chicago and the SCS 24-hr design storms. The methodology can be used for any other design storm by a municipality.

FIGURE 24 COMPARISON OF DESIGN STORMS
7.1 METHODOLOGY OF DESIGN STORMS

The researchers comparing peak flows from design storms and historical storms used different catchments and different simulation programs. A comparison of the peak flow frequency results for the Chicago design storm is presented first in this chapter. The results for this storm are summarized in Table 3.

A study conducted in the IMPSWM program is also presented here. The methodology used to compare the Chicago and SCS 24-hr design storms with the historical storms is given in the second section of the chapter.

7.1.1 RESULTS FOR THE CHICAGO DESIGN STORM

J.F. McLarens Ltd. (1978) has conducted studies on catchments in Edmonton and Winnipeg. They found that the ratio of the Chicago storm peak flow to the flows from an historic storm series ranged between 1.0 and 1.2. It was recommended that the Chicago design storm be used for urban drainage design.

Marsalek (1979) developed a Chicago design storm for the Burlington area. He found that the peak flows produced from the Chicago design storm are 80% larger than those produced from historical storm events. He also found that the peak flow was attenuated as the catchment size increased. The peak flow increased as the catchment imperviousness increased but the peak flow overestimation remains at approximately 80%. These results were analyzed in the IMPSWM program by Wisner and Gupta (1980). They concluded that discrepancies can be reduced if the
peak intensity of the design storms are reduced to values in agreement with measured peak intensities.

Watson (1980) compared the peak flows obtained from the Chicago design storm and historical storm events. A 2-hr. duration and a non-dimensional time to peak of 0.28 is used to develop the Chicago storm. The rainfall data is discretized at 5 min. intervals.

**TABLE 3 SUMMARY OF RESULTS FROM PERVERVIOUS STUDIES (AVERAGE PERCENTAGE DIFFERENCE BETWEEN THE CHICAGO DESIGN STORM AND HISTORICAL STORM FLOWS)**

<table>
<thead>
<tr>
<th>Study</th>
<th>Catchment</th>
<th>Chicago Design Storm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Arnell (1980)</td>
<td>Bergsjon</td>
<td>-2.2%</td>
</tr>
<tr>
<td></td>
<td>Linkoping 1</td>
<td>10.3%</td>
</tr>
<tr>
<td></td>
<td>Linkoping 2</td>
<td>6.0%</td>
</tr>
<tr>
<td>Marsalek (1979)</td>
<td>Burlington (area 26 ha, imp. 30%)</td>
<td>80.0%</td>
</tr>
<tr>
<td>Watson (1980)</td>
<td>Pinetown</td>
<td>2.0%</td>
</tr>
<tr>
<td></td>
<td>Kew</td>
<td>-5.0%</td>
</tr>
</tbody>
</table>

Watson found that on the Pinetown catchment the peak flows from the Chicago storm agreed closely with those from the historical storms. The agreement for the Kew catchment was not quite as good. The peak flow is slightly underestimated. It is within 95% confidence interval bands of the historical storms, though. The Kew catchment is less impervious than the Pinetown catchment; therefore, it is more sensitive to antecedent moisture conditions.

Arnell (1982) used a Chicago design storm with a 4-hr duration. The non-dimensional time to peak, \( r \), is 0.43 if the return period is less than 1 year. If the return period is greater than 1 year, \( r \) is 0.35. The Chicago storm is developed with a step size of one minute.

Arnell found that the Chicago design storm overestimated the peak flow by approximately 5%. On the Bergsjon catchment, the peak flow is underestimated by 2.2%. On the Linkoping 1 and Linkoping 2 catchments the flow is overestimated by 10.3% and 6% respectively. The Bergsjon catchment was the smallest of the three catchments. The Chicago storm produces peak flows almost identical to the historical storms on this catchment.

With the exception of Marsalek (1979), the estimation of peak flow produced by the Chicago storm gave acceptable results compared with that produced by historical storms. Differences range from a 2% underestimation to an 10% overestimation of peak flow.

Watson recommended that the Chicago design storm be used for peak flow design. Arnell also found that the deviation of the Chicago Storm peak flow values from the historical storm peak flow values are not large. He concludes that the Chicago Storm should overestimate peak flows because of the way it is developed. He does not recommend the use of the Chicago design storm because of the large overestimation of peak flow Marsalek found.
7.2 Methodology for Comparing Design Storms and a Historical Storm Series

7.2.1 Rainfall Input

The rainfall inputs used with the event simulation models were a historical storm series and two design storms. The historical storm series was selected from the Bloor Street station rainfall record. A criteria was selected based on the storm volume and intensity so that approximately one storm event for each year in the record was chosen. This results in some years having more than one event and other years having no events. The selected events were then discretized to ten minute time intervals. A summary of the storm events and their characteristics is given in Table 4.

The SCS 24-hour and Chicago design storms were compared with the historical storm series. The design storms were developed from the Bloor Street station IDF curves for return periods of 5, 10 and 25 years. The Chicago storm was 4 hours in duration and was discretized at 10 minute intervals. The SCS storm was 12 hours in duration and was discretized at 12 min. interval. The peak intensity and antecedent moisture conditions should be adjusted so that the design storm resembles real storm conditions.

The adjustment is necessary on urban catchments because the peak flows are dependent on the peak intensities. The scattergram in Figure 26 shows that the correlation between peak flows and peak intensity is close to 1 in urban areas.

The choice of the time step is important in obtaining a peak intensity close to the peak intensity of real storms. An analysis was conducted to demonstrate the importance of the time step used with a design storm. The 5 and 10 min. intensities were extracted for the highest recorded storms in Toronto (Hogg,1980). These were plotted along with the peak intensities of the Chicago design storm peak intensities discretized at 5 and 10 minute intervals (Figure 27a, 27b). For the 5 minute intensities, the Chicago design storm intensities are higher than the real storm intensities, while for the 10 minute intensities they are slightly lower. Wisner and Gupta (1979) show that there can be a large variation in flows depending on the step size chosen for the design storm. Time steps between 10 and 20 minutes are recommended for use with the Chicago design storm. If the design storm peak intensity is still larger than that of real storms it should be adjusted so that the two peak intensities are similar.

The antecedent moisture conditions are usually not considered as being important when a design storm is used with an event simulation model. Some studies, though, have been conducted to investigate this. Wenzel and Voorhees (1979) tested design storms using both wet and dry antecedent moisture conditions, but they do not recommend a procedure for determining what conditions should be used with a design storm. The Flood Studies Report (NERC,1975) present a procedure for determining the antecedent moisture conditions. Using a relationship between the Urban Catchment Wetness Index and the Standard Average Annual Rainfall the antecedent moisture conditions can be determined for the FSR design storm in any area in the U.K. In the present study the modified curve number is used to represent the antecedent moisture conditions. With the OTTHYMO model an average modified curve number, for a watershed, is used with the design storms.
TABLE 4 HISTORICAL STORM CHARACTERISTICS

<table>
<thead>
<tr>
<th>Date</th>
<th>Duration (hrs.)</th>
<th>Volume (mm.)</th>
<th>Time to Peak (hrs.)</th>
<th>Peak Intensity (in./hr)</th>
<th>Average Intensity (in./hr)</th>
<th>API (mm)</th>
<th>CN*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sept. 15/57</td>
<td>6.67</td>
<td>47.84</td>
<td>4.167</td>
<td>59.18</td>
<td>7.19</td>
<td>19.8</td>
<td>37.5</td>
</tr>
<tr>
<td>July 9/60</td>
<td>5.50</td>
<td>62.33</td>
<td>5.000</td>
<td>82.37</td>
<td>11.33</td>
<td>12.3</td>
<td>24.0</td>
</tr>
<tr>
<td>June 19/61</td>
<td>6.17</td>
<td>37.12</td>
<td>2.867</td>
<td>49.28</td>
<td>6.02</td>
<td>24.5</td>
<td>45.0</td>
</tr>
<tr>
<td>Sept. 13/62</td>
<td>2.00</td>
<td>42.62</td>
<td>0.167</td>
<td>159.26</td>
<td>30.28</td>
<td>13.2</td>
<td>25.0</td>
</tr>
<tr>
<td>Nov. 9-10/62</td>
<td>12.00</td>
<td>58.03</td>
<td>5.867</td>
<td>17.83</td>
<td>4.88</td>
<td>14.0</td>
<td>27.0</td>
</tr>
<tr>
<td>Aug. 11/64</td>
<td>6.50</td>
<td>40.61</td>
<td>4.167</td>
<td>39.62</td>
<td>6.25</td>
<td>12.7</td>
<td>24.5</td>
</tr>
<tr>
<td>Aug. 5/68</td>
<td>4.67</td>
<td>42.38</td>
<td>4.167</td>
<td>70.64</td>
<td>9.09</td>
<td>10.5</td>
<td>19.0</td>
</tr>
<tr>
<td>Aug. 22/68</td>
<td>9.00</td>
<td>72.90</td>
<td>3.167</td>
<td>58.62</td>
<td>8.10</td>
<td>31.6</td>
<td>53.5</td>
</tr>
<tr>
<td>Aug. 29-30/70</td>
<td>3.50</td>
<td>67.60</td>
<td>11.867</td>
<td>92.25</td>
<td>14.15</td>
<td>8.1</td>
<td>15.0</td>
</tr>
<tr>
<td>May 16/74</td>
<td>12.00</td>
<td>58.32</td>
<td>8.500</td>
<td>56.34</td>
<td>4.85</td>
<td>49.4</td>
<td>74.0</td>
</tr>
<tr>
<td>Aug. 23/74</td>
<td>0.67</td>
<td>51.20</td>
<td>0.333</td>
<td>153.62</td>
<td>76.81</td>
<td>5.5</td>
<td>8.0</td>
</tr>
<tr>
<td>Aug. 23/75</td>
<td>9.17</td>
<td>57.22</td>
<td>2.667</td>
<td>77.72</td>
<td>6.25</td>
<td>7.7</td>
<td>14.0</td>
</tr>
<tr>
<td>July 6/77</td>
<td>7.17</td>
<td>51.10</td>
<td>7.167</td>
<td>50.29</td>
<td>7.41</td>
<td>28.0</td>
<td>49.0</td>
</tr>
<tr>
<td>July 31/77</td>
<td>0.87</td>
<td>45.19</td>
<td>0.333</td>
<td>156.77</td>
<td>54.23</td>
<td>16.2</td>
<td>31.0</td>
</tr>
<tr>
<td>Sept. 24/77</td>
<td>10.67</td>
<td>60.96</td>
<td>8.333</td>
<td>26.24</td>
<td>5.99</td>
<td>24.8</td>
<td>45.1</td>
</tr>
</tbody>
</table>

FIGURE 26 CORRELATION BETWEEN PEAK RAINFALL AND FLOWS (URBAN AREA 50% IMP).
7.2.2 Watersheds Studied

Three different types of watersheds were examined in this study; a rural watershed, an urban watershed and a mixed land use watershed. Two rural watersheds in southern Ontario were tested, one was large and had an area of 6540 ha, the other was small having an area of 44 ha.
Simulation runs were also conducted on three southern Ontario urban watersheds. The catchment characteristics are summarized in Table 5. On two of the catchments the urban area routine URBHYD of OTTHYMO was used. Impervious conditions of 35% and 50% were used on urban catchment No.1 to observe the change in difference between the design storm peak flows. On the third urban catchment the SWMM simulation program was used. A schematic of this catchment is shown in Figure 28.

### TABLE 5 URBAN WATERSHED CHARACTERISTICS

<table>
<thead>
<tr>
<th>Watershed</th>
<th>Area (ha.)</th>
<th>Imperviousness (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. 1</td>
<td>294.4</td>
<td>35 &amp; 50</td>
</tr>
<tr>
<td>No. 2</td>
<td>290.3</td>
<td>50</td>
</tr>
<tr>
<td>No. 3</td>
<td>150.5</td>
<td>30</td>
</tr>
</tbody>
</table>

A mixed land use watershed in Metropolitan Toronto, having a rural area of 1597 ha. and an urban area of 5536 ha., as also tested. The flows on this watershed were found using the OTTHYMO model. The total area contained 21 urban subwatersheds and 12 rural subwatersheds. They ranged in size from 53 ha. to 778 ha. The urban subwatersheds had an imperviousness of 35%.

#### 7.2.3 SIMULATION MODELS AND THEIR CALIBRATION

The two, single event simulation models used to test the design storms were the SWMM and the OTTHYMO models. The SWMM model was used on urban catchment No.3. The Chicago design storm with a 10 min. time step, was compared with a series of flows generated from real storms. The antecedent moisture conditions are accounted for using values for the Horton equation’s initial infiltration capacity and final infiltration capacity.

The lumped OTTHYMO model was used on the rural, urban and mixed watersheds. The rural watershed in southern Ontario is used as an example to show how the model was calibrated. The catchment data was first obtained and used to compute the time to peak Tp and the storage coefficient K. The initial abstraction Ia was found by examining the rainfall record and the stream flow record and distinguishing between the rainfall events that produce runoff and those that do not produce runoff. The relationship between the modified curve number CN* and the antecedent precipitation index API was found using five storm events for which there were discretized rainfall measurements as well as stream flow records. Simulations were then conducted and the generated hydrographs were compared with the measured hydrographs and found to be similar. The model was therefore properly calibrated.
7.3 RESULTS OF PEAK FLOWS FROM DESIGN STORMS AND HISTORIC STORM EVENTS

7.3.1 RURAL WATERSHEDS

In this study design storm and historic storm flows were generated on large rural watersheds. The results from the historic storm events were examined to determine if the peak intensity or the antecedent moisture conditions influence the flows on a rural catchment. The peak intensity was found to be independent of the peak flows (Figure 29). On the other hand the antecedent moisture conditions, as measured by the API, are correlated with the peak flows. The correlation coefficient between the API and the peak flows is close to 1.

The flow frequency curves for the large rural watershed is shown in Figure 30. The SCS 24-hour design storm flows are greater than those given by the real storm series. On the large rural watershed, in Southern Ontario, the flows are overestimated by 5% to 10%.

Using the SCS 24-hour design storm with an average CN* resulted in flow frequency curves slightly larger than-the historic storm series flows on the rural watersheds.
FIGURE 29 RURAL AREAS
(a) CORRELATION BETWEEN API AND PEAK FLOWS.
(b) CORRELATION BETWEEN PEAK RAINFALL AND FLOWS.
The Chicago design storm was also tested on these catchments. It had a shorter duration than the SCS 24-hour storm but a higher peak intensity. The Chicago design storm gave lower flows than those from the historic storm series. The flows on the southern Ontario watershed were from 2.9% to -27.3% different from the historical storm flows. On the small rural watershed the 100 yr. Chicago and SCS 24-hr. design storms gave almost identical peak flows. The flows were 1.28 cms and 1.37 cms respectively. The Chicago design storm flows produced lower flow frequency curves than the historic storm series flows on the rural watersheds.

This comparison of the design storm flows on the rural watersheds shows that the SCS 24-hour design storm gives a good prediction of the peak flow frequency curves. The importance of antecedent moisture conditions in determining the peak flow was also demonstrated. On rural catchments the OTTHYMO model uses an average CN* with the design storm. This was found to give good predictions of the peak flow when used in conjunction with the SCS 24-hour design storm.
7.3.2 **URBAN WATERSHEDS**

The dependence of the historic storm flows to the peak intensity and antecedent moisture conditions were examined on the urban catchments. The peak intensity was found to be an important factor in determining the peak flows. The correlation between the peak intensity and peak flows was close to 1 (Figure 29). Antecedent moisture conditions do not show any correlation with the peak flows (Figure 31). It was found that runoff from urban catchments was independent of CN*.

The SCS 24-hour design storm flows were compared with the historic storm series flows on urban watersheds No.1 and No.2. The comparison of the flows is shown on Figures 32 and 33. The SCS 24-hour design storm underestimated the flow on both of the watersheds. On watershed No.1 the flow was underestimated by 11% to 22% while on watershed No.2 from 14% to 26%. On these urban watersheds the SCS 24-hour design storm has underestimated the peak flow.

The Chicago design storm was tested on the three urban watersheds. The flow predictions given by this storm are slightly below the historical storm series on urban areas No.1 and No.2 (Figures 32 and 33). The flows were underestimated by approximately 6% on watershed No.1 and 4% on watershed No.2. On urban watershed No.3 the SWMM simulation program was used. The Chicago storm gave results that were similar to the real storm flows (Figure 34). From the tests conducted on these three urban areas the Chicago design storm gave peak flow predictions close to the flows from the historic storm series.

The effect of changing the catchment imperviousness was examined on urban watershed No.1. Flows were generated on this catchment using the design storms and the historic storms for impervious conditions of 35% and 50%. The flow frequency curves for the two impervious conditions are shown in Figure 34. Increasing the catchment imperviousness resulted in the SCS storm giving lower flows with respect to the real storm series. The relative position of the Chicago storm flows did not change. The Chicago storm is less sensitive to changes in the catchment imperviousness than the SCS 24-hour storm.

The comparison of the design storms on the urban catchments has shown that the Chicago storm gives a good prediction of peak flow on urban catchments. Flows in the urban areas were found to be dependent on peak intensity and independent of antecedent moisture conditions. The sensitivity of peak flows to peak intensities showed the importance of having design storm peak intensities similar to real storm peak intensities.

The Chicago storm that was used obtained the peak intensities by having a 10 minute step size. Increasing the imperviousness of urban catchment No.1 demonstrated that the Chicago design storm continued to give a good prediction of the peak flow. For these reasons the Chicago design storm may be used with single event simulation models on urban watersheds.
**FIGURE 31** URBAN AREA (50% IMP.) CORRELATION BETWEEN API AND FLOWS

**FIGURE 32** FLOW FREQUENCY CURVES, URBAN WATERSHED NO. 1, 35% AND 50% IMP.
FIGURE 33 FLOW FREQUENCY CURVES, URBAN WATERSHED NO. 2, 50% IMP.
7.3.3 Mixed Land Use Watershed

The mixed land use watershed tested combines both urban and rural areas. The previous sections have shown that the SCS storm can be used on larger rural areas and the Chicago storm on urban areas. It would not be satisfactory to use the SCS storm on the rural segments and the Chicago storm on the urban areas of the mixed watershed. Both the Chicago and the SCS 24 hr. storm were tested on this watershed.

The flow frequency curves for the design storms and the historic storms are shown in Figure 35. The SCS design storm underestimated the flow by 11% for a 5 yr. return period and overestimated the flow by 18% to 39.9% for return periods of 10 to 100 years. The Chicago storm was found to give better estimates of the peak flow; for a 5 yr return period the flow was, underestimated by 5% and for return periods from 10 to 100 years the flow was overestimated by 0.3% to 28.0%.

The flow predictions made on this watershed with the design storms were not the same as the historical storm series estimates. This shows that the peak flow cannot be obtained by using an
arbitrarily selected single design event. An analysis using a historical storm series should be conducted before a design storm is selected. In general, it is more desirable to use a historical storm series on this type of watershed.

![Flow Frequency Curve](image)

**FIGURE 35 FLOW FREQUENCY CURVES, MIXED LANDUSE WATERSHED**

### 7.4 CONCLUSIONS

Simulations with the Chicago and SCS 24-hr design storms were compared with a series of real storms to determine if the flow frequency results they produce are reliable. The design storms and a historical storm series were tested on rural, urban and mixed land use watersheds using the OTTHYMO model. For the historical storms antecedent moisture conditions were accounted for using the modified curve number CN*. According to the IMPSWM methodology the antecedent precipitation index, API, is determined at the beginning of a storm event. The modified curve number is then found by using the API vs. CN* relationship for the watershed. In addition, the SWMM model was used with the Chicago design storm on an urban area. The conclusions of the study are listed below.
1. In rural areas antecedent moisture conditions were shown to be critical in determining peak flows.

2. A modified curve number was calibrated which in conjunction with the SCS 24-hr. design storm gives adequate results on rural watersheds. This "design curve number" is proposed to be part of the design storm concept.

3. Using the "calibrated design curve number" on large rural watersheds it was found that Chicago design storms underestimate the flow. The SCS 24-hr design storm gave good flow predictions on both large and small rural watersheds. For a small rural watershed the difference between the Chicago and SC-S 24-hr peak flow was found to minimal.

4. For urban areas the SCS 24-hr design storm underestimated the peak flow. The Chicago storm gave consistent peak flow results. It was also demonstrated that the peak intensity is an important factor in determining he peak flows in urban areas. For this reason design storm peak intensities should be selected on the basis of a study of peak intensities of critical real storm events.

5. On the large mixed land use watershed antecedent moisture conditions must be considered if the contribution of the rural watershed is important. For flood control purposes in large mixed areas it is preferable to use a series of real storms. One cannot tell prior to analysis of the watershed which design storm will produce acceptable results.

6. Results found for the homogeneous watershed indicate that for routine SWM studies design storms can be used. It is however desirable to compare the two types of design storms and eventually a critical historic storm.
8  A REVIEW OF DESIGN STORM PROFILES

8.1 INTENSITY DURATION FREQUENCY CURVES

In Canada, the Atmospheric Environment Service (AES) of Environment Canada does the collection of rainfall records. The rainfall amounts are collected using a tipping bucket rain gauge. The rain gauge typically records every 0.01 inches of rainfall. The AES standard procedure locates an Type B Standard Rain Gauge (AES standard) together with the tipping bucket and adjusts the tipping bucket data so that the daily totals of both gauges agree. In the United States the U.S. Weather Service collects data which is then stored and analyzed at the National Climate Centre (NOAA, Environmental Data Service Ashville N.C.). Worldwide meteorological information may be obtained from the World Meteorological Organization.

The AES analyzes the daily records obtained from the tipping bucket rain gauge. In the case of strip charts, these are analyzed to determine the maximum rainfall volume occurring in 5, 10, 15, 30, 60, 120, 360, 720, and 1440 minute intervals during each day of a year. The daily record of maximum volumes is scanned at the end of each year to determine the annual maximum volumes for each of the time periods. This analysis is conducted for every year of the rainfall record to form an annual maximum series. Each set of volumes is divided by the appropriate fixed duration to obtain the annual maximum series for intensities.

The return period of extreme intensities is of special interest to engineers who want to calculate the return period of the rainfall-induced peak flows. The return period can be determined using a plotting position formula or by using an extreme-value distribution. For example, the Atmospheric Environment Service uses the extreme value type 1 distribution. The statistical parameters of mean, standard deviation and skew are computed for each set of annual maximum intensities. The intensity of a particular frequency is computed using the statistical parameters and the frequency factor form of the probability distribution (Chow, 1954). For each duration, the intensities for the desired return periods are computed. The points of intensity and duration for known return periods are plotted. Lines joining all the points having the same return period are drawn to form an IDF curve. An example of the intensity-duration-frequency (IDF) curves developed by the AES for the Bloor Street station in Toronto are shown in Figure 36.

Engineers have also found advantageous to fit empirical equations to the statistically developed IDF curves. Equations 35 and 36 are the general forms of the empirical equations used most frequently.

\[
\begin{align*}
    i &= \frac{a}{(t + c)^b} \\
    i &= \frac{a}{t^b + c}
\end{align*}
\]

(35) (36)

To describe the variation in average intensity with duration for a given frequency, the parameters a, b and c must be determined. This is done by fitting the either of the two equations to the statistically determined values for i and t. The value of b is estimated, then the following equation can be solved for the constants c and \( \ln(a) \).
\[ \ln(i) = \ln(a) - \ln(t + b) \] (37)

The values of a, b and c are unique for each IDF curve. They cannot be used to develop IDF curves of different frequencies or IDF curves at different locations.

The design storms developed from IDF curves are not representative real storm events. This is because the method of analyzing rainfall to obtain the IDF curves is independent of the real storm events. Each annual maximum volume for the 5, 10, 15, 30, 60, 120, 360, 720 and 1440 minute duration may come from a different storm event. This is demonstrated in Figure 37. For the series of storm hyetographs shown, the maximum 5 min. volume occurred in event 4, and the maximum 10 min. volume in event 2. For time periods such as 12 hrs. and 24-hr., the maximum volume may come from more than one storm event.
In a real storm event, the maximum average intensities for fixed duration within the event have different frequencies. Real storm events do not have a single frequency. For example, event 4 has a return period of 2 yrs. for the 5 min. duration, a return period of 1 year for the 10 minute duration, and a 2 year return period for the average intensity of the total event duration. The frequency of average intensity for different duration in some storms may vary from 1 yr. to 50 years. Design storms on the other hand are developed so that the maximum average intensity for each fixed duration in the design storm has the same frequency. Since design storm are developed from IDF curves, they contain the maximum average intensities from different storm events. Design storms are therefore models of real storms and their validity requires testing.

8.3 Uniform Design Storm

The uniform design storm is the oldest and simplest design storm. The storm originates from the use of the rational method. By selecting a duration and return period, the average maximum intensity is found from an intensity-duration frequency curve. The intensity remains constant for the duration of the storm.

The uniform design storm contains only a part of the real storm volumes. The volume of the uniform design storm is obtained from an IDF curve it is not the volume from a real storm event. The uniform hyetograph does not show any variation in intensity with time. Real rainfall events have intensities that are highly variable, this variability affects the peak flow.
8.4 COMPOSITE DESIGN STORM

To develop this design storm, the storm duration is first selected. Average intensities are then found by reading an IDF curve at selected duration shorter than the storm duration. The selected duration should be separated by a constant time step. Next, the accumulated storm volume is computed by multiplying each duration by the average intensity. The incremental rainfall volumes and intensities are computed and then arbitrarily rearranged to form a storm pattern. If the incremental intensities are not rearranged a front-end hyetograph, following the shape of the IDF curve, is obtained.

8.5 CHICAGO DESIGN STORM

The Chicago design storm is a design storm distribution widely used by practising drainage engineers. This representation of the temporal distribution of rainfall was proposed by Keifer and Chu in 1957. They developed a storm pattern which would preserve the maximum volume of water falling within a specified duration, the average amount of rainfall before the peak intensity and the relative time of the peak intensity.

To determine the time distribution of rainfall and preserve the previously mentioned characteristics, they adopted the empirical IDF curves. By using IDF curves, they stayed with a procedure and concepts engineers were familiar with and is simple to obtain and therefore, it has become widely accepted for use in engineering practice.

8.5.1 DERIVATION OF THE CHICAGO DESIGN STORM

The Chicago design storm is developed from empirical IDF relationships. Keifer and Chu observed that if they had a continuous function representation of the instantaneous intensity and they integrated this function over a given duration, the rainfall volume occurring in that duration is obtained. The rainfall volume divided by the duration gives the average intensity. Therefore, multiplying the average intensity by the storm duration and differentiating with respect to the duration they obtained expressions for the instantaneous intensity:

\[ i = \frac{a((1-c)t + b)}{(t + b)^{c+1}} \]  
(38)

\[ i = \frac{a((1-b)t^b + c)}{(t^b + c)^2} \]  
(39)

These equations for the shape of the storm hyetograph are for a storm pattern where the peak is at the beginning of the storm event. If the peak occurs after the beginning of the storm, the storm duration \( t \) is divided into two time periods, the time before the peak \( t_b \) and the time after the peak \( t_a \). The ratio of the time to peak to the total storm duration is given by the equation:

\[ r = \frac{t_b}{t} \]  
(40)

The storm duration in terms of the time before the peak and the time after the peak is given by equations:
\[
t = \frac{t_b}{r} \\
t = \frac{t_a}{1 - r}
\]

The storm hyetograph shape is given by:

\[
i_b = \frac{a \left( (1 - b) \frac{t_a}{1 - r} \right) + c}{\left( \frac{t_a}{1 - r} + c \right)^{1+b}} \tag{42}
\]

\[
i_a = \frac{a \left( (1 - b) \frac{t_a}{a - r} \right) + c}{\left( \frac{t_a}{1 - r} + c \right)^{1+b}} \tag{43}
\]

A typical shape for the Chicago design storm hyetograph is shown in Figure 38, where the intensity before the peak is given by \(i_b\) and the intensity after the peak is given by \(i_a\).
8.5.2 PARAMETER ESTIMATION

The procedure for determining the constants for the empirical IDF curve was discussed previously in the Intensity Duration Frequency Curves section. The only constant that is required for the Chicago design storm is the ratio of the time before the peak to the storm duration, \( r \). Two procedures may be used to determine the value of \( r \):

1. The ratio of time to peak intensity to the storm duration is computed for a series of events for various duration. For a given duration, the average time to peak is determined from a number of rainfall events of that duration. This is done for a set of duration. The mean value of the time to peak to the storm duration ratio is computed as a weighted average. The following equation is an example of how \( r \) is computed.

\[
r = \frac{t_{d1}(\bar{t}_p1) + t_{d2}(\bar{t}_p2) + t_{d3}(\bar{t}_p2)}{t_{d1} + t_{d2} + t_{d3}}
\]

where, \( t_{d1}, t_{d2}, t_{d3} \) are the durations of the different rainfall occurrences, \( \bar{t}_p1, \bar{t}_p2, \bar{t}_p3 \) are the average time to peak for the different rainfall occurrences and \( r \) is the ratio of time to peak to total storm duration.

2. The ratio can be computed by analyzing local storm distributions to determine the rainfall depth before the peak intensity. The design storm duration is then chosen. The rainfall depth prior to the peak is determined for durations less than the design storm. A weighted average for \( r \) is determined based on the ratio of the antecedent rainfall depth to the total rainfall depth. This is done by using:

\[
d_a = r(t_{d}I_{td} - tI_t)
\]

where:

- \( d_a \) = depth of antecedent rainfall,
- \( t_d \) = design storm duration,
- \( t \) = rainfall duration,
- \( I_{td} \) = maximum average intensity for the total duration,
- \( I_t \) = maximum average intensity for duration \( t \).

Specific applications of each procedure have been documented. For example, McPherson (1958) has criticized the second procedure while both have been demonstrated by Bandyopadhyay (1972). The later obtained a value 0.416 for the first procedure and an \( r \) value of 0.37 for the second. Table 6 presents a list of \( r \) values obtained by different researchers.
TABLE 6 VALUES OF R FOR THE CHICAGO DESIGN STORM

<table>
<thead>
<tr>
<th>Location</th>
<th>R</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baltimore</td>
<td>0.399</td>
<td>McPherson, 1958</td>
</tr>
<tr>
<td>Chicago</td>
<td>0.375</td>
<td>Keifer and Chu, 1957</td>
</tr>
<tr>
<td>Cincinnati</td>
<td>0.325</td>
<td>Preul et al., 1973</td>
</tr>
<tr>
<td>Ontario</td>
<td>0.488</td>
<td>Marsalek, 1978</td>
</tr>
<tr>
<td>Philadelphia</td>
<td>0.414</td>
<td>McPherson, 1958</td>
</tr>
</tbody>
</table>

8.5.3 DETERMINATION OF THE CHICAGO DESIGN STORM HYETOGRAPH

The steps to produce a Chicago design storm Hyetograph are:

1. Select a design storm duration.
2. Select a time step. The time step size should not be less than the minimum duration that was used when determining the IDF curves. For the Canadian IDF curves, step size should be at least 5 minutes. It is recommended that the step size should be about 10 minutes.

If the parameters a, b, c and r for the Chicago design storm are not known they should be determined.

The instantaneous rainfall intensities at the different time intervals from the peak intensity are computed.

8.6 SCS 24-HOUR DESIGN STORM

The U.S. Soil Conservation Service (SCS) has developed the ‘Type 1 and 2’ design storm. These have and are used both, in the U.S.A. and Canada. The SCS determined the mass curve for percent of accumulated rainfall depth over a duration of 24-hr. To obtain the mass curve, rainfall was analyzed across the U.S.. SCS characterized the rainfall using the 2 types of storm patterns: The Type 1 rainfall distribution is applicable to Hawaii, Alaska, the Coastal Sierra Nevada, the Cascade Mountains in California, Oregon and Washington. The type 2 rainfall distribution applies to the remainder of the United States, Puerto Rico and the Virgin Islands. Figure 39 illustrates the SCS 24-hour rainfall distribution. In Canada, the type 2 curve applies in most areas, however, there are some regions in British Columbia where the type 1 curve is used.

To create a SCS design storm, a duration and return period is first selected. The corresponding volume is then distributed over the steepest portion of the SCS-24 hour curve. The incremental rainfall volumes and intensities are then obtained based on the volume distribution over the selected duration.

It’s worth noting that the Composite Design Storm can be rearranged to give a storm pattern very similar to the SCS design storm. This occurs because the SCS type 1 and type 2 curves have
been produced so that, for a selected 24-hour rainfall depth, the depth-duration curve derived from a SCS distribution would be very similar to the curve produced by the U.S. Weather Bureau.

**FIGURE 39 SCS II – 24 DESIGN STORM**

Cronshey (1980) developed regional rainfall distributions to replace the SCS type II distribution in the 37 eastern and central states. The distributions consider variations due to the rainfall return period as well as regional differences. The ratio of rainfall volume in a fixed duration to the 24-hour rainfall volume was determined for many stations in the eastern United States. It was found that the rainfall ratios increase with distance from the coast and that the ratios are lower for a 100-year return period than for a 2-year period.

Four regions were identified with similar rainfall ratios and a special distribution was developed for each. Maps are available within each area corresponding to fixed durations and return period. The maps are for 5, 15 and 60 minute durations and return periods of 2 and 100 years. There are also locations in the eastern U.S. where 2 or 3 maps could be used depending on the duration and return period being considered. However, these new distributions are not significantly different from the SCS Type 2 distribution and they do not apply to Canadian conditions.

**8.7 SCS 6 HOUR DESIGN STORM**

This is a second type of design storm developed by the U.S. Soil Conservation Service. The SCS 6 hour design storm was developed for designing small dams. The duration is selected as 6 hours or the time of concentration which ever is larger. The rainfall depth is determined from maps of probable maximum precipitation or from the 6 hour precipitation depth for a 100 year return
period. The hyetograph profile is determined using a 6 hour design storm distribution, an example is shown in Figure 40.

This storm was developed for use in conjunction with the SCS method of runoff computation. With the SCS method the rainfall depth and duration must be known. The watershed soil type and antecedent moisture conditions are also important. These catchment characteristics are reflected in the SCS curve number. The amount of direct runoff is computed using the total precipitation and the curve number. The hyetograph shape is not important in determining the direct runoff with the SCS method. It is important, though, when the runoff is computed using a single event simulation model.

The rainfall depth for the SCS 6 hour design storm has not been determined using Canadian meteorological data. It therefore should not be used on Canadian catchments unless local data is used.

The return period is much larger than is typically used in urban drainage design. The design storm was developed for rural conditions and should therefore not be used in urban areas.

![Figure 40 6-Hour Design Storm Distribution](image)

**FIGURE 40 6-HOUR DESIGN STORM DISTRIBUTION**

### 8.8 Illinois State Water Survey Design Storm

The Illinois State Water Survey (ISWS) design storm is based on research conducted by Huff (1967). Huff examined storm events in central Illinois having durations between 3 hours to 48 hours. He divided the storms into 4 groups depending on the time period in which the majority of the rain occurred. The storms with the most of the rain occurring in the first quarter of the event duration are termed first quartile. All of the storm events that examined were placed in one of the four quartile groups. The rainfall mass curves in each quartile group were determined for various probability levels (Figure 41). It was found that short duration storm events dominate the first and second quartile. For this reason, the median distribution in the first quartile is commonly used for...
design. Terstriep and Stall (1974) recommended that the first quartile storm with the 50% probability level be used as the design storm with the Illinois runoff model.

The ISWS design storm is determined by selecting a design storm duration. The maximum depth for the duration and the given frequency are derived from local data or from an IDF curve. The rainfall depth is then distributed according to the Huff quartile median distribution.

The ISWS design storm is derived in a similar way to the SCS design storm. When it is used in Illinois it has the advantage of being developed from an analysis of local data. An analysis of rainfall similar to Huff’s analysis is required if the rainfall structure is to be known for a region other than Illinois.
FIGURE 41 CUMULATIVE DISTRIBUTION OF RAINFALL (HUFF, 1967)
8.9 **ATMOSPHERIC ENVIRONMENT SERVICE DESIGN STORM**

The Canadian Atmospheric Environment Service (AES) examined the temporal variation of rainfall using almost 2000 extreme events in Canada. Hogg (1980) conducted the analysis of the time distribution of rainfall in short duration events. Rainfall duration of 1 and 12 hours were selected for the analysis and to develop Time-Probability curves. The events were chosen to have samples from both thunderstorms and large-scale cyclonic origin. The selected events did not have to be individual storm entities, they could also be part of a larger storm sequence. Fixed rainfall durations were selected so that the same definition would be used to identify rainfall that is used in obtaining depth-duration-frequency curves.

In analyzing the rainfall, the one hour events were divided into twelve 5 min. increments and the 12 hour events into 1 hour increments. Rainfall for each event was expressed as a cumulative percentage of, total event rainfall for the twelve equal increments through the storm. All the events for a particular duration were analyzed and the cumulative rainfall distributions for different probability levels were computed (Figure 42).

The temporal distribution patterns were found to vary in the different regions of Canada. The temporal distributions for the coastal regions were quite different from the distributions for the continental regions. A comparison was made between Huffs 50% distribution for the second and third quartile storms and those Hogg developed for southern Ontario. The second quartile distribution was up to 25% different in the cumulative rainfall. The third quartile distribution does not resemble any of the distributions Hogg computed.
FIGURE 42 AES RAINFALL DISTRIBUTION
8.10 Flood Studies Report Design Storm

The Flood Studies Report (FSR) design storm was developed in the United Kingdom. Rainstorm profiles of historical events were examined, and the cumulative percentage of rainfall for different probabilities of storm peakness was developed. The peakness of a storm was defined as the ratio of maximum to mean intensity. A study of summer and winter storm profiles showed that summer storms were more peaked than winter storms. The curves of storm peakedness were published by the National Environment Research Council (NERC, vol.2, 1975).

The FSR design storm duration is selected so that the largest peak flow calculated at each point in the system is taken as the design discharge. The developers of the FSR design storm found that the peak discharge is not very sensitive to the storm duration, a doubling of duration caused a change of less than 10% peak discharge. Therefore, a fairly coarse series of rainfall duration may be used; values of 15, 30, 60, and 120 minutes are recommended. The return period is selected to be the same as that of the required discharge. This is usually established by the local government agency.

The rainfall depth is determined using a procedure developed for all of the U.K. Using maps and a formula, the depth for a particular return period is computed. The depth can also be computed from a depth-duration-frequency curve.

The antecedent moisture conditions were examined and incorporated into the model developed by the FSR. The antecedent conditions are expressed by the Urban Catchment Wetness Index (UCWI). Runoff simulations were conducted on a catchment using a variety of UCWI's for return periods between 1 and 10 years. An optimum UCWI was found for the catchment that resulted in good predictions of the peak flow. Similar studies were conducted on other catchments in the U.K. An analysis of the optimum UCWI’s and rainfall data for the catchments led to a relationship between the UCWI and the Standard Average Annual Rainfall (SAAR). This relationship can be used to determine the UCWI for any location in the U.K.

The FSR design storm recommended for use in the U.K. has the following properties:

1. The storm duration should be that which gives the maximum discharge.
2. The return period of rainfall should equal that of the required discharge.
3. The storm profile should be the 50% summer profile.
4. The UCWI varies as the average annual rainfall and should be determined from the Figure shown below.

Some things to consider before applying this type of storm: A) the FSR design storm peak intensity always occurs at the centre of the storm. B) A local analysis of historic events is not conducted to determine the time-to-peak. C) The FSR design storm was developed for use in the U.K. only. A similar analysis would have to be conducted if it was to be used outside the U.K.
8.11 PILGRIM AND CORDERY DESIGN STORM

The Pilgrim and Cordery design storm was developed to provide an approach which would produce storm patterns consistent with the storm patterns of historical events. To develop the Pilgrim and Cordery Design Storm, a duration is selected and a set of events with a large rainfall for the specified duration are selected. The duration is divided into a number of time periods. The rainfall volume in each of the periods is ranked and an average ranking for each period is computed. The percentage rainfall in each period is computed and is ranked from the largest to the smallest. An average percentage rainfall is then computed for each rank in the rain period. The average percentage rainfall is then assigned to the average ranking in each period. The percentage rainfall in each period determines the hyetograph shape. It is recommended that 50 events be used in the analysis.

The storm hyetograph is determined from an analysis of local rainfall data. If the analysis has not been conducted, it would be inconvenient for an engineer to use this design storm.

The hyetograph shape that is obtained using this procedure depends on the number of internal divisions that are selected for the design storm duration. The number of rainstorms that are used in the analysis is also important. Yen and Chow (1980) show that real storm events have a very
uneven temporal distribution of rainfall. For this reason the value of the average ranking of intervals tend to a single number when a large number of rainstorms are used.

8.12 Yen and Chow Design Storm

All of the previously discussed design storms were developed from rainfall frequency-duration relationships or from observations of rain gauge records, they did not use a statistical procedure to analyze the historical rainfall record. Yen and Chow (1980) use the method of moments to statistically determine the geometry of hyetograph. Using a triangular hyetograph representation, only the first moment is required to determine the location of the hyetograph peak.

Rainstorms are analyzed to determine the average depth per time interval

\[ d_{avg} = \frac{1}{n} \sum_{j=1}^{n} d_j = \frac{D}{n} \]  

(46)

and the first moment arm of the hyetograph

\[ t_{avg} = \frac{\Delta t \cdot (\sum_{j=1}^{n} (j - 0.5)d_j)}{D} \]  

(47)

where:

- \( n \) = number of intervals in the storm,
- \( d_j \) = rainfall depth in interval \( j \),
- \( D \) = total storm depth,
- \( d_{avg} \) = average storm depth,
- \( t \) = time step of each interval,
- \( t_{avg} \) = average storm duration.

To describe the hyetograph in more general terms, the hyetograph is non-dimensionalized using \( D \), the storm depth, and \( t_d \), the storm duration. The non-dimensionalized form of these two equations can be expressed in the following two forms:

\[ d_{avg}^0 = \frac{1}{n} \]  

(48)

\[ t_{avg}^0 = \frac{\bar{t}}{t_d} \]  

(49)

where:

- \( d_{avg}^0 \) = non-dimensional average storm depth,
- \( t_{avg}^0 \) = non-dimensional average storm duration.
The hyetograph is described with geometric variables a, b and h. In a non-dimensionalized form, the variables are expressed by:

\[ a^0 = 3t^0 \]  \hspace{1cm} (50a)

\[ b^0 = 2 - 3\bar{t}^0 \]  \hspace{1cm} (50b)

\[ h^0 = 2 \]  \hspace{1cm} (50c)

The historical rainfall record is then analyzed to determine values for a, b, h for 6 particular locality. Yen and Chow have only done this for a rainfall record from Illinois and from Boston. Once the design storm non-dimensionalized variables are determined, the design storm can be developed. After selecting the duration and return period, the rainfall depth is found from an IDF curve. Using the non-dimensionalized variables for the area the design storm is constructed.

The Yen and Chow design storm requires a rainfall data analysis before it can be used in a particular locality. The non-dimensionalized variable a, can only be found by analyzing a local rainfall record. It should, not be transported from one locality to another.
9 WATER BALANCE PROCESSES IN CONTINUOUS SIMULATION

The water balance in snow pack, depression storage and active soil zone is simulated to extend the single-event simulation to continuous. This chapter describes the equations used in these processes.

9.1 CLIMATE DATA

The climate data used in continuous simulation include precipitation, temperature and evaporation.

9.1.1 PRECIPITATION DATA

The precipitation data is mandatory for continuous simulation. It could be the daily data download from Environment Canada website or the 5-min data from a local rain gauge. To analyze snow-melt, the snowfall should also be included.

9.1.2 TEMPERATURE DATA

Visual OTTHYMO uses the daily maximum and minimum temperature. When the simulation time step is smaller than daily, the temperature of each time step is interpolated assuming the temperature during the day follows a sinusoidal time-varying function. Therefore, the temperature of each time step is calculated as:

\[ T = \left( \frac{T_{MAX} - T_{MIN}}{2} \right) \cdot \sin \left[ \left( \frac{t - t_{T_{MIN}}}{12} - \frac{1}{2} \right) \pi \right] + \left( \frac{T_{MAX} + T_{MIN}}{2} \right) \]

where:

- \( T \) = temperature for current time step (°C),
- \( T_{MAX} \) = daily maximum daily temperature (°C),
- \( T_{MIN} \) = daily maximum daily temperature (°C),
- \( t \) = current time (hours),
- \( t_{T_{MIN}} \) = time when minimum temperature occurs (hours).

Usually, the maximum daily temperature is assumed to occur at 3:00PM and the minimum daily temperature at 3:00AM, i.e. \( t_{T_{MIN}} = 3 \). A sample curve is shown in Figure 44.
9.1.3 **Evaporation Data**

The evaporation data is used to estimate the evaporation from the depression storage and the evapotranspiration from the active soil zone. It could be time-series data of pan evaporation and calculated potential evapotranspiration (PET) or the monthly evaporation.

**Monthly Evaporation**

Monthly evaporation data is usually available from local agencies. The default monthly evaporation data is shown in Table 7.

<table>
<thead>
<tr>
<th>Month</th>
<th>Evaporation (mm/month)</th>
<th>Month</th>
<th>Evaporation (mm/month)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>0</td>
<td>July</td>
<td>150</td>
</tr>
<tr>
<td>Feb</td>
<td>1.1</td>
<td>Aug</td>
<td>99.6</td>
</tr>
<tr>
<td>Mar</td>
<td>2.5</td>
<td>Sep</td>
<td>77.2</td>
</tr>
<tr>
<td>Apr</td>
<td>19.1</td>
<td>Oct</td>
<td>55.4</td>
</tr>
<tr>
<td>May</td>
<td>66.2</td>
<td>Nov</td>
<td>22.7</td>
</tr>
<tr>
<td>Jun</td>
<td>106.9</td>
<td>Dev</td>
<td>4.3</td>
</tr>
</tbody>
</table>

**Calculated PET**

Empirical equations are available to calculate PET based on other climate data, e.g. Penman-Monteith method, Priestley-Taylor method and Hargreaves method. The calculated PET time-series could be read into the model as the upper limit of evapotranspiration.
Pan Evaporation

When pan evaporation time series data is used, PET is calculated using a modified version of the U.S. Agricultural Research Service (ARS) equations (Warren Viessman, 1977). It accounts for the vegetation characteristics and amount of soil water available in the active soil zone. It’s given as

\[ E_p = GI \cdot k \cdot E_{pan} \cdot \left( \frac{S - SA}{AWC} \right)^x \]

where:
- \( E_p \) = potential evapotranspiration (mm),
- \( GI \) = the growth index of crop in % of maturity,
- \( k \) = the ratio of \( GI \) to pan evaporation,
- \( E_{pan} \) = pan evaporation (mm),
- \( S \) = total porosity,
- \( SA \) = the available soil porosity (unfilled by water),
- \( AWC \) = porosity drainable only by evapotranspiration,
- \( x \) = the exponent and could be calculated as

\[ x = AWC/G \]

where \( G \) is the moisture freely drained by gravity.

Monthly growth index \( GI \) is used and the default growth index is given in Table 8.

<table>
<thead>
<tr>
<th>Month</th>
<th>Growth Index</th>
<th>Month</th>
<th>Growth Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>0</td>
<td>July</td>
<td>0.98</td>
</tr>
<tr>
<td>Feb</td>
<td>0.08</td>
<td>Aug</td>
<td>0.83</td>
</tr>
<tr>
<td>Mar</td>
<td>0.29</td>
<td>Sep</td>
<td>0.57</td>
</tr>
<tr>
<td>Apr</td>
<td>0.57</td>
<td>Oct</td>
<td>0.29</td>
</tr>
<tr>
<td>May</td>
<td>0.83</td>
<td>Nov</td>
<td>0.08</td>
</tr>
<tr>
<td>Jun</td>
<td>0.98</td>
<td>Dev</td>
<td>0</td>
</tr>
</tbody>
</table>

The ratio of \( GI \) to pan evaporation, \( k \), is usually 1.0-1.2 for short grasses, 1.2-1.6 for crops up to shoulder height, and 1.6-2.0 for forest.

The pan evaporation \( E_{pan} \) is calculated with lake evaporation \( E_{lake} \) with

\[ E_{pan} = E_{lake}/ELKSPAN \]

where \( ELKSPAN \) is the lake evaporation to pan evaporation ratio with value between 0.6 and 0.8 (Bruce Withers, 1974) and the lake evaporation could be using the monthly lake evaporation as shown in Error! Reference source not found. or the measured data.

\( S \) and \( G \) is the physical soil parameter. Representative values are given in Table 9.
### TABLE 9 HYDROLOGIC CAPACITIES OF SOIL TEXTURE CLASSES (WARREN VIESSMAN, 1977)

<table>
<thead>
<tr>
<th>Texture Class</th>
<th>S (%)</th>
<th>G (%)</th>
<th>AWC (%)</th>
<th>x (AWC/G)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse sand</td>
<td>24.4</td>
<td>17.7</td>
<td>6.7</td>
<td>0.38</td>
</tr>
<tr>
<td>Coarse sandy loam</td>
<td>24.5</td>
<td>15.8</td>
<td>8.7</td>
<td>0.55</td>
</tr>
<tr>
<td>Sand</td>
<td>32.3</td>
<td>19.0</td>
<td>13.3</td>
<td>0.70</td>
</tr>
<tr>
<td>Loamy sand</td>
<td>37.0</td>
<td>26.9</td>
<td>10.1</td>
<td>0.38</td>
</tr>
<tr>
<td>Loamy fine sand</td>
<td>32.3</td>
<td>27.2</td>
<td>5.4</td>
<td>0.20</td>
</tr>
<tr>
<td>Sandy loam</td>
<td>30.9</td>
<td>18.6</td>
<td>12.3</td>
<td>0.66</td>
</tr>
<tr>
<td>Fine sandy loam</td>
<td>36.6</td>
<td>23.5</td>
<td>13.1</td>
<td>0.56</td>
</tr>
<tr>
<td>Very fine sandy loam</td>
<td>32.7</td>
<td>21.0</td>
<td>11.7</td>
<td>0.56</td>
</tr>
<tr>
<td>Loam</td>
<td>30.0</td>
<td>14.4</td>
<td>15.6</td>
<td>1.08</td>
</tr>
<tr>
<td>Silt loam</td>
<td>31.3</td>
<td>11.4</td>
<td>19.9</td>
<td>1.74</td>
</tr>
<tr>
<td>Sandy clay loam</td>
<td>25.3</td>
<td>13.4</td>
<td>11.9</td>
<td>0.89</td>
</tr>
<tr>
<td>Clay loam</td>
<td>25.7</td>
<td>13.0</td>
<td>12.7</td>
<td>0.98</td>
</tr>
<tr>
<td>Silty clay loam</td>
<td>23.3</td>
<td>8.4</td>
<td>14.9</td>
<td>1.77</td>
</tr>
<tr>
<td>Sandy clay</td>
<td>19.4</td>
<td>11.6</td>
<td>7.8</td>
<td>0.67</td>
</tr>
<tr>
<td>Silty clay</td>
<td>21.4</td>
<td>9.1</td>
<td>12.3</td>
<td>1.34</td>
</tr>
<tr>
<td>Clay</td>
<td>18.8</td>
<td>7.3</td>
<td>11.5</td>
<td>1.58</td>
</tr>
</tbody>
</table>

$S$ and $G$ could be calculated using soil total porosity, field capacity and wilting point, where $S$ equals to porosity minus wilting point and $G$ equals to porosity minus field capacity as shown in Figure 45 Relation between Soil Moisture Limits and Soil Texture Class (Rossman & Huber, 2016). Their values are given for different soil textures in Table 11.

![Figure 45: Relation between Soil Moisture Limits and Soil Texture Class](image)

**FIGURE 45 RELATION BETWEEN SOIL MOISTURE LIMITS AND SOIL TEXTURE CLASS**

(ROSSMAN & HUBER, 2016)
9.2 Snow Pack Water Balance

The snowmelt model in GAWSER (Guelph All-Weather Sequential-Events Runoff Model) (Schroeter & Associates, 1996) was used to model the snow pack water balance. Six processes (refreeze, compaction, new snow deposition, rain deposition, snowmelt and release of liquid water water) are considered in the model as show in Figure 46.

![Figure 46 Snow Pack Water Balance](image)

**FIGURE 46 SNOW PACK WATER BALANCE**

It’s assumed that the snow pack is consist of two parts: solid water content (SWC) and liquid water content (LWC). Snow pack is first formed with new snow adding to solid water content. With varying temperature, snow may be melt and added to liquid water content. The liquid water in liquid water content may also be compacted and added back to solid water content. When the amount of liquid water exceeds the holding capacity of the snow pack, it’s then released to generate runoff.

9.2.1 Initial Condition

It’s assumed there is no snow on the ground at the start of simulation.

9.2.2 New Snow Additions

New snow occurs in each time step is calculated using

\[
SNOWE = \begin{cases} 
P & \text{for } T > T_{snow} \\
0 & \text{for } T > T_{snow}
\end{cases}
\]

where:

- \(SNOWE\) = the solid water content of new snow, i.e. snow water equivalent (mm),
- \(P\) = the precipitation in the time step (mm);
- \(T_{snow}\) = the snow fall temperature which is usually assumed to be same as the snowmelt temperature.

The new snow is added to the solid water content in the snow pack.

\[
SDEP_f = SDEP_o + SNOW
\]

\[
SWC_f = SWC_o + SNOWE
\]

where:

- \(SDEP\) = the snowpack depth (mm),
$SWC$ = the solid (ice) water content (mm), the subscripts $o$ and $f$ denote the initial and final values of certain quantities (e.g. $SWC$) at the time of the calculation, $SNOW$ = the depth of new snowfall calculated using

$$SNOW = SNOWE/NEWDEN$$

where $NEWDEN$ is the relative density of new snow (vol/vol) with default value of 0.1 and range from 0.02 to 0.15.

As a result of any new snowfall, the relative dry density of the pack, $RHO$ (vol/vol) is recalculated using

$$RHO = SWC/SDEP$$

And the porosity, $POR$ (vol/vol) is computed from

$$POR = 1 - RHO/RHOICE$$

where $RHOICE$ is the relative density of ice, usually taken as 0.92.

9.2.3 SNOWMELT

A temperature index method is employed to find the potential amount of snowmelt, $MELTP$ (mm) during the time step, where $MELTP$ is the liquid water depth created by ice crystal melt.

$$MELTP = \frac{KM}{24} \times DH \quad for \quad T > TBAS$$

$$MELTP = 0 \quad for \quad T \leq TBAS$$

Where:

$KM$ = the melt factor (mm/d-°C),
$DH$ = degree hour (hr-°C),
$TBAS$ = snowmelt temperature (°C).

The melt factor $KM$ varies by season and it’s described by

$$KM = \left(\frac{KM_{MAX} - KM_{MIN}}{2}\right) \cdot \sin \left[\left(\frac{Mon - Mon_{KM_{min}}}{6} - \frac{1}{2}\right) \pi\right] + \left(\frac{KM_{MAX} + KM_{MIN}}{2}\right)$$

Where:

$KM_{MAX}$ = the maximum melt factor which usually occurs in June (mm/d-°C),
$KM_{MIN}$ = the minimum melt factor which usually occurs in December (mm/d-°C),
$Mon$ = the month plus the ratio of current day in the month,
$Mon_{KM_{min}}$ = the month when the minimum melt factor occur.

With $KM_{MAX} = 6.2$, $KM_{MIN} = 2.9$ and $Mon_{KM_{min}} = 12$, the variation is shown in Figure 47. The snow melt temperature $TBAS$ is also assumed to follow the similar variation.
The degree hour $DH$ is calculated as the area between the temperatures sinusoidal curve and the line for snow melt temperature $TBAS$. The area is positive when temperature is higher than $TBAS$. Otherwise, it’s negative. For time step $t$, $DH$ is calculated as the difference between the area for $t$ and $t - DT$ where $DT$ is the time interval. The summation of $DH$ in a day equals to $T_{mean} - TBAS$ where $T_{mean}$ is daily average temperature.

The actual melt, $MELT$ is set equal to the lessor of either the $SWC$ or $MELTP$ values and added to the liquid water content as

$$LWC_f = LWC_o + MELT$$

The solid water content if the pack is updated as

$$SWC_f = SWC_o - MELT$$

and the new snow depth becomes

$$SDEP_f = SWC_f / RHO_f$$

9.2.4 RAINFALL
If temperature is higher than snowfall temperature, the precipitation is rainfall and it will be added to liquid water content directly as

$$LWC_f = LWC_o + RAIN$$
9.2.5 Refreeze of Snowpack Liquid Water

During periods of below freezing temperature, liquid water in the pack will refreeze thereby increasing the solid water content. The potential amount of refreeze during a given time interval is calculated using a temperature index method similar to snowmelt.

\[ \text{REFRZP} = \frac{KF}{24} \times \text{DH} \quad \text{for} \quad T \leq TBAS \]

\[ \text{REFRZP} = 0 \quad \text{for} \quad T > TBAS \]

where:

- \( \text{REFRZP} \) = the amount of refreeze (mm);
- \( KF \) = the refreeze factor which is usually set equal to \( KM \).

The actual refreeze, \( \text{REFRZ} \) (mm) is set equal to either \( \text{REFRZP} \) or the current \( \text{LWC} \), whichever is less. Then the water balance of the pack is updated as follows.

\[ \text{LWC}_f = \text{LWC}_o - \text{REFRZ} \]

\[ \text{SWC}_f = \text{SWC}_o + \text{REFRZ} \]

The snow pack dry density \( RHo \) is then updated.

9.2.6 Snowpack Compaction

As snow accumulates, it settles or compacts under its own weight, or by the action of the wind, resulting in an increase in the dry density and a decrease in the porosity. A growth curve approach was chosen to compute compaction effects. The new relative dry density, \( TRHO \) after an interval of \( DT \) hours is determined as

\[ TRHO = \frac{RHO \times MRHO}{RHO + (MRHO - RHO) \times \exp(-DT/KC)} \]

\[ KC = B \times \exp(A \times T) \]

where:

- \( MRHO \) = the maximum specified relative dry density,
- \( KC \) = the compaction time constant (h),
- \( A \) (1/°C) and \( B \) (h) are coefficients.

Similar to \( KM \) and \( TBAS \), the seasonal variation is also applied to \( MRHO \).

The final dry density (due to compaction) for the interval \( DT \) is determined according to

\[ RHOF = RHO_o \quad \text{for} \quad TRHO \geq MRHO \]

\[ RHOF = TRHO_o \quad \text{for} \quad TRHO < MRHO \]
The snowpack depth is also updated with new dry density.

9.2.7 RELEASE OF LIQUID WATER

The maximum amount of liquid water that can be held in the pore spaces of a snowpack by capillary forces is referred to as the liquid water holder capacity, $LWCAP$ and is computed using

$$LWCAP = POR \times SWI \times SDEP$$

Where $SWI$ is the irreducible water saturation and is expressed as a fraction of the total pore volume.

Any liquid water in excess of the holding capacity will be released from the pack, therefore

$$RUNOFF\_SNOW = LWC_0 - LWCAP \quad \text{for } LWC_0 > LWCAP$$

$$RUNOFF\_SNOW = 0 \quad \text{for } LWC_0 \leq LWCAP$$

and $LWC_f$ is then set equal to $LWCAP$.

The values of $RUNOFF\_SNOW$ for each time step provide input to IA routine.

9.3 DEPRESSION STORAGE WATER BALANCE

The liquid water released from snow pack flows into depression storage first. It will first fill the available storage and then flow out. The storage is then restored through evaporation and infiltration (in pervious area).

$$RUNOFF\_IA = STOR_o + RAIN + MELT - ETIA - INFIL - STORMax$$

$$RUNOFF\_IA = 0 \quad \text{for } STOR_o + RAIN + MELT - ETIA - INFIL > STORMax$$

$$RUNOFF\_IA = 0 \quad \text{for } STOR_o + RAIN + MELT - ETIA - INFIL \leq STORMax$$

where:

- $RUNOFF\_IA$ = the amount of water flow out of depression storage (mm),
- $STOR_o$ = the initial amount of water stored in depression storage (mm),
- $ETIA$ = the amount of water lost through evaporation (mm),
- $INFIL$ = the amount of water infiltrates to active soil zone (mm)
- $STORMax$ = the storage capacity (mm).

Then the water held in the storage is updated.

The evaporation $ETIA$ is calculated with upper limit set to PET. The infiltration $INFIL$ is only considered in pervious area and it’s calculated with given infiltration ratio as

$$INFIL = STOR_o \times RATIO$$
9.4  **ACTIVE SOIL ZONE WATER BALANCE**

The water balance in active soil zone (pervious area) is simulated as

\[
S_{STOR_f} = S_{STOR_o} + RUNOFF_{IA} + ICRO - RUNOFF - ET - GWI
\]

where:

- \(S_{STOR_f}\) = the amount of water stored in soil (mm),
- \(S_{STOR_o}\) = the initial amount of water in soil (mm),
- \(RUNOFF_{IA}\) = the runoff from depression storage (mm),
- \(ICRO\) = the effective runoff from indirectly connected impervious land surfaces (mm),
- \(RUNOFF\) = surface runoff (mm),
- \(ET\) = the evapotranspiration (mm)
- \(GW\) = groundwater infiltration (mm).

9.4.1  **WATER CONTRIBUTED FROM INDIRECTLY CONNECTED IMPERVIOUS AREA**

\(ICRO\) is the net contribution of water from impervious surfaces onto pervious surfaces referred to as 'indirectly connected impervious areas'. This net factor accounts for several physical conditions reduce the impacts of indirectly connected impervious areas on pervious areas.

- Unlike precipitation and snowmelt, which are generally uniformly distributed over the pervious areas, runoff distribution from impervious areas are often concentrated in flow channels.
- Indirectly connected impervious areas are not always uniformly distributed over the entire basin and only affect a fraction of the pervious surfaces.

The net effect is that water from indirectly connected areas is somewhat reduced and not available over the entire pervious surfaces. This is modeled with a reduction factor in Continuous OTTHYMO. Evaluation of typical lot layouts and grading suggest that a 0.20 multiplication factor (80% reduction) of the indirectly connected areas is a reasonable value.

9.4.2  **RUNOFF**

The surface runoff \(RUNOFF\) is calculated using modified SCS equation

\[
RUNOFF = \frac{(RUNOFF_{IA} + ICRO)^2}{RUNOFF_{IA} + ICRO + S_{av}}
\]

where \(S_{av}\) is the available storage in active soil zone (mm) calculated as

\[
S_{av} = \begin{cases} 
S_{max} - S_{STOR} & \text{when snowpakc doesn't exist} \\
(S_{max} - S_{STOR}) \times S_{REDUCT} & \text{when snowpakc exists}
\end{cases}
\]

where \(S_{max}\) is the maximum available storage in active soil zone (mm) which is calculated with curve number for antecedent moisture condition I (\(CNI\)) as
\[ S_{\text{max}} = \left( \frac{25400}{CNI} - 254 \right) / 0.85 \]

It’s assumed that soil is saturated (reaches to total porosity) when the amount of soil water equals to \( S_{\text{max}} \). \( CNI \) is calculated from \( CNII \) with

\[ CNI = \frac{CNII}{2.3 - 0.013 \cdot CNII} \]

The effective soil storage is adjusted during winter conditions in the presence of a snowpack with a reduction factor \( SREDUCT \). This factor simulates soil frozen conditions with an apparent reduction in available soil storage.

9.4.3 **Evapotranspiration**

A PET reduction factor, \( PETFACT \), is employed to account for lower evaporation during days with rain. The default value is 0.1.

\[ E_{p,\text{rain}} = E_p \cdot PETFACT \quad \text{for} \quad RAIN > 0 \]

The potential evapotranspiration will first be fulfilled by the evaporation from depression storage and then the active soil zone. The available potential evapotranspiration for active soil zone, \( E_{p,\text{soil}} \), is calculated as

\[ E_{p,\text{soil}} = E_{p,\text{rain}} - ET_{IA} \]

The ET rate from the active soil zone is calculated using the approach employed in Stanford Watershed Model for ET from lower soil zone (Warren Viessman, 1977), which is

\[ ET_{\text{soil}} = E_{p,\text{soil}} - \frac{E_{p,\text{soil}}^2}{2r} \quad \text{for} \quad E_{p,\text{soil}} \leq r \]

\[ ET_{\text{soil}} = r/2 \quad \text{for} \quad E_{p,\text{soil}} > r \]

where \( r \) is the evapotranspiration opportunity representing the maximal amount of water available for evapotranspiration at a particular location. It’s calculated as

\[ r = K^3 \cdot \frac{LZS}{LZSN} \]

where:

- \( LZS \) = the current soil moisture storage (mm),
- \( LZSN \) = a nominal storage level (mm)
- \( K^3 \) = an input parameter that is a function of watershed cover as shown in Table 10.

In continuous OTTHYMO, \( LZS \) is set equal to \( SSTOR \) and \( LZSN \) is set to \( S_{\text{max}} / 2 \).
### TABLE 10 TYPICAL LOWER ZONE EVAPOTRANSPIRATION PARAMETERS
*(WARREN VIESSMAN, 1977)*

<table>
<thead>
<tr>
<th>Watershed Cover</th>
<th>$K_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open land</td>
<td>5.08</td>
</tr>
<tr>
<td>Grassland</td>
<td>5.84</td>
</tr>
<tr>
<td>Light forest</td>
<td>7.11</td>
</tr>
<tr>
<td>Heavy forest</td>
<td>7.62</td>
</tr>
</tbody>
</table>

#### 9.4.4 GROUNDWATER INFILTRATION

The percolation equation used in SWMM *(Rossman & Huber, 2016)* is used to calculate the groundwater infiltration.

$$GW_I = K \cdot e^{-(\phi - \theta)HCO}$$

where:
- $K$ = the saturated hydraulic conductivity,
- $\phi$ = the soil total porosity,
- $\theta$ = the soil moisture
- $HCO$ = the percolation coefficient depending on the soil texture.

The soil parameters are given in Table 11.

### TABLE 11 SOIL PARAMETER FOR DIFFERENT SOIL TYPES

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Porosity (vol/vol)</th>
<th>Field Capacity (vol/vol)</th>
<th>Wilting Point (vol/vol)</th>
<th>Saturated Hydraulic Conductivity (mm/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sand</td>
<td>0.437</td>
<td>0.062</td>
<td>0.024</td>
<td>2890</td>
</tr>
<tr>
<td>Loamy Sand</td>
<td>0.437</td>
<td>0.105</td>
<td>0.047</td>
<td>719</td>
</tr>
<tr>
<td>Sandy Loam</td>
<td>0.453</td>
<td>0.19</td>
<td>0.085</td>
<td>262</td>
</tr>
<tr>
<td>Loam</td>
<td>0.463</td>
<td>0.232</td>
<td>0.116</td>
<td>79</td>
</tr>
<tr>
<td>Silt Loam</td>
<td>0.501</td>
<td>0.284</td>
<td>0.135</td>
<td>158</td>
</tr>
<tr>
<td>Sandy Clay Loam</td>
<td>0.398</td>
<td>0.244</td>
<td>0.136</td>
<td>37</td>
</tr>
<tr>
<td>Clay Loam</td>
<td>0.464</td>
<td>0.31</td>
<td>0.187</td>
<td>24</td>
</tr>
<tr>
<td>Silty Clay Loam</td>
<td>0.471</td>
<td>0.342</td>
<td>0.21</td>
<td>24</td>
</tr>
</tbody>
</table>
### 9.5 Modeling a Wetland

The wetland command in VO is designed to mimic a natural wetland, with a wet storage area surrounded by a dry vegetated area. The storage area and dry area are both dynamic in order to represent the flooding and drying out seen in many natural wetlands. The way this works in the model is that the total area = storage area + dry area. When it rains and the storage area begins to fill, the surface area of the wet portion of the wetland increases and the dry area decreases.

Modeling for the wetland uses the same equations as the NasHyd (dry area) and RouteReservoir (wet area), with the addition of groundwater interaction in the storage area. Groundwater is used as an input to the model and the VO wetland command should not be used to model the impacts of a wetland on the groundwater. The model uses the Darcy equation to calculate the seepage of water into and out of the storage portion of the wetland. Our model can be described in three separate scenarios:

**Scenario 1** – maximum recharge, when the groundwater is below the bottom of the wetland

\[
S_1 = \lambda \left[ \int_0^{A(h^*)} h^* - h_{bed}(A) \, dA \right]
\]

\[
h_{gw} \leq h_{bottom}
\]
Scenario 2 – reduced recharge, when the groundwater is above the bottom of the wetland but below the surface water

\[ h_b < h_{gw} \leq h_{SW} \]

\[
S_2 = \lambda \left[ (h^* - h_{gw})A(h_{gw}) + \left( \int_{A(h_{gw})}^{A(h^*)} h^* - h_{bed}(A) \, dA \right) \right]
\]

Scenario 3 – Discharge, when the groundwater is above the surface water

\[ h_{gw} > h_{SW} \]

\[
S_3 = \lambda \left[ (h^* - h_{gw})A(h^*) + \left( \int_{A(h_{gw})}^{A(h^*)} h_{bed}(A) - h_{gw} \, dA \right) \right]
\]

For the above equations:

- \( h_b \): Elevation of the bottom of the wetland. If the wetland has multiple pools this value is the lowest elevation in the wetland.
- \( h_{gw} \): Elevation of the groundwater, provided as an input file.
- \( h_{sw} \): Elevation of the surface water.
- \( \lambda \): Leakage factor = \( K/L \), where:
  - \( K \) is the hydraulic conductivity of the sediment layer at the bottom of the wetland (m/s)
  - \( L \) is the thickness of the sediment layer (m)
The leakage factor is a calibration parameter for the wetland routine. When calculating the value to use for the leakage factor you should consider the soil within the top meter to meter and half of soil under the wetland. If the soil under the wetland is not uniform there are two scenarios to consider.

If the top soil layer is permeable such as sand, loam or loose organic matter and the lower soil layer is a less permeable layer such as clay use the combined depth of the soils and an average K value.

If the top soil layer is clay or other impermeable soil and the lower soil layer is a more permeable layer such as sand use the depth (L) and K value for the impermeable layer.
Reference


201.  Wright-McLaughlin Engineers (1968), Urban Storm Drainage Criteria Manual, Regional Council of Governments, Denver, Colorado.
