

### A. Introduction

The runoff discharge hydrograph is a graph of discharge rate versus time, where the discharge is in units of volume/time (i.e.,  $\text{ft}^3/\text{sec}$ ,  $\text{acre-ft}/\text{day}$ ). The runoff hydrograph can also be expressed as might be defined for a stage-discharge relationship for a stream channel of floodway (i.e., the discharge rating curve for a channel cross section). The shape of the storm hydrograph is related to the rainfall hyetograph (intensity vs. time) and the combined effects of the storage in the watershed or channel. The watershed and channel storage effects smooth out much of the variation evident in the storm hyetograph. The runoff hydrograph increases in magnitude shortly after the start of the rainfall event and reaches a peak after the maximum rainfall intensity has occurred.

For the design of storm sewers and culverts, the required pipe or culvert diameter is typically based on conveyance capacity for the peak discharge, and a hydrograph is not typically used for the analysis. As a watershed becomes more urbanized, the impact of more impervious area, decreased potential for infiltration, and loss of natural depression storage will change the response to rainfall and thus the shape (peak and time base) of the resulting runoff hydrograph. When routing runoff through a stream reach or where detention storage will be used, an inflow (upstream) hydrograph must be determined.

### B. Unit hydrographs

The unit hydrograph is defined as the hydrograph of direct runoff that results from 1 inch of excess rainfall generated uniformly over the watershed at a constant rate during a specified time. A T-hour (or T-minute) hydrograph is the hydrograph resulting from a storm with a continuous rainfall excess of 1 inch over the duration to T-hours (or T-minutes). Another common unit hydrograph used in urban hydrology is the dimensionless unit hydrograph. The ordinates of the dimensionless unit hydrograph are expressed as ratios of the peak discharge ( $q/q_p$ ) and the time axis (abscissa) is given as ratios of the time to the time to peak ( $t/t_p$ ), where  $q_p$  is the discharge rate at the time to peak,  $t_p$ .

In hydrograph analysis, the storm hyetograph (rainfall input function) is converted to the direct runoff hydrograph (output) in common design practice using a unit hydrograph (transfer function). In an event-based model such as the NRCS method (WinTR-55 and WinTR-20), the rainfall hyetograph for the design rainfall event is based on the CN method and the application of the NRCS 24-hour rainfall distribution (Chapter 3 - Section 2 Rainfall and Runoff Analysis). The NRCS dimensionless unit hydrograph (1985) is frequently used in practice, is well-documented, and is recommended for design of BMPs included in this manual. In development of the method, a large number of actual watersheds were evaluated and then made dimensionless by dividing all discharge ordinates by the peak discharge and the time values by the time to peak. An average of these dimensionless unit hydrographs (UH) was computed. The time-base of the dimensionless UH is approximately five times the time to peak, and approximately  $\frac{3}{8}$  of the total volume occurred before the time to peak. The inflection point on the recession limb occurs at about 1.7 times the time to peak, and the hydrograph has a curvilinear shape. The curvilinear hydrograph can be approximated by a triangular UH with similar characteristics. The average dimensionless curvilinear UH and triangular UH are illustrated in Figure C3-S7-1. While the time base of the triangular UH is only  $\frac{3}{8}$  of the time to peak (5 for the dimensionless UH), the area under the rising limb of both UH's is the same (37.5%). The discharge ratios for the NRCS dimensionless UH are given in Table C3-S7-1.

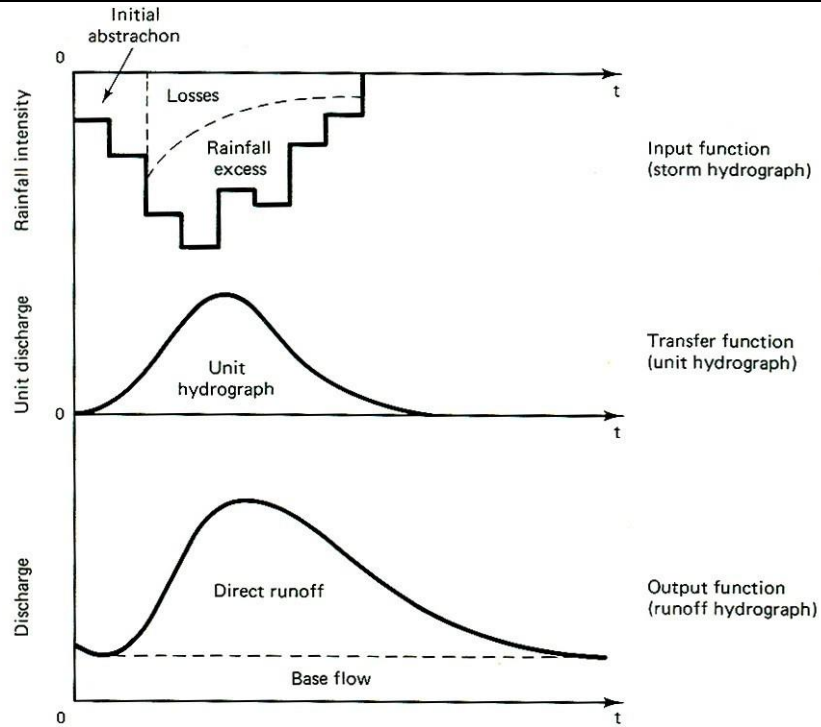


Figure C3-S7-1: Relationship of storm, unit, and direct runoff hydrographs  
Source: McCuen, 1989

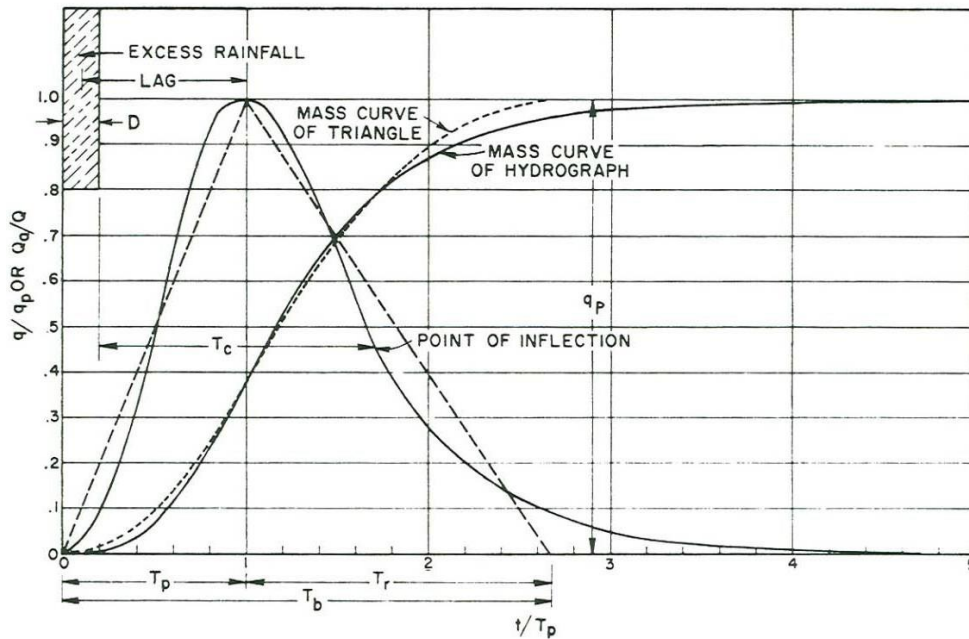


Figure C3-S7-2: NRCS dimensionless unit hydrograph  
Source: NRCS, 1985

**Table C3-S7-1: Ratios for the NRCS dimensionless UH and mass curve**

<b>Time ratios <math>t/t_p</math></b>	<b>Discharge ratios <math>q/q_p</math></b>	<b>Mass curve ratios <math>Q_a/Q</math></b>
0	0.000	0.000
0.1	0.030	0.001
0.2	0.100	0.006
0.3	0.190	0.012
0.4	0.310	0.035
0.5	0.470	0.065
0.6	0.660	0.107
0.7	0.820	0.163
0.8	0.930	0.228
0.9	0.990	0.300
1.0	1.000	0.375
1.1	0.990	0.450
1.2	0.930	0.522
1.3	0.860	0.589
1.4	0.780	0.650
1.5	0.680	0.700
1.6	0.560	0.751
1.7	0.460	0.790
1.8	0.390	0.822
1.9	0.330	0.849
2.0	0.280	0.871
2.2	0.207	0.908
2.4	0.147	0.934
2.6	0.107	0.953
2.8	0.077	0.967
3.0	0.055	0.977
3.2	0.040	0.984
3.4	0.029	0.989
3.6	0.021	0.993
3.8	0.015	0.995
4.0	0.011	0.997
4.5	0.005	0.999
5.0	0.000	1.000

Source: NRCS, 1985

The area under a hydrograph equals the volume of direct runoff (Q), which for a UH is 1 inch. The runoff volume is related to the geometry of the triangular UH by:

**Equation C3-S7-1**

$$Q = 1/2 q_p(t_p + t_r)$$

Where:

$t_p$  and  $t_r$  = time to peak and recession time, respectively

$q_p$  = peak discharge

(Note that in the definition of the unit hydrograph, the time of concentration is defined as the time from beginning of excess rainfall to the inflection point on the recession limb of the hydrograph.)

The term for  $q_p$  then becomes:

$$q_p = Q/t_p \left[ \frac{2}{1 + \frac{t_r}{t_p}} \right]$$

and

**Equation C3-S7-2**

$$q_p = \frac{KQ}{t_p}$$

(K replaces term in brackets)

For units of  $q_p$  in  $\text{ft}^3/\text{sec}$ ,  $t_p$  in hours, and  $Q$  in inches, it is necessary to divide  $q_p$  by the area,  $A$ , in  $\text{mi}^2$  and multiply the righthand side of Equation C3-S7-2 by the constant 645.3; since  $t_r = 1.67 t_p$ , the following equation for  $q_p$  is:

**Equation C3-S7-3**

$$Q_p = \frac{484AQ}{t_p}$$

The constant 484, or peak rate constant, defines a unit hydrograph with  $\frac{2}{3}$  of its area under the rising limb. As the watershed slope becomes very steep (mountainous), the constant in Equation C3-S7-2 can approach a value of  $\sim 600$ . For flat, swampy areas, the constant may decrease to a value  $\sim 300$ . For applications in Iowa, use of the constant 484 is recommended unless specific watershed runoff data indicates a different value is warranted. The default dimensionless UH in WinTR-55 uses a value of 484; however, there is an option in WinTR-55 for a custom UH defined by the designer or the jurisdiction.

The time to peak in Equation C3-S7-3 can be expressed in terms of the duration of unit rainfall excess and the time of concentration. In Figure C3-S7-2:

**Equation C3-S7-4**

$$T_c + D = 1.7t_p$$

and:

**Equation C3-S7-5**

$$\frac{D}{2} + 0.6T_c = t_p$$

then:

$$D = 0.133t_c$$

and:

**Equation C3-S7-6**

$$t_p = \frac{D}{2} + 0.6T_c = \frac{2}{3}T_c$$

Expressing Equation C3-S7-3 in terms of  $T_c$  instead of  $t_p$  gives:

**Equation C3-S7-7**

$$q_p = \frac{726AQ}{T_c}$$

**Example 1:**

Determine the triangular UH for a 240-acre watershed that has been developed into residential land use with  $\frac{1}{4}$ -acre parcels. The post-developed CN is 80 for the predominantly HSG-C soils. The estimated  $T_c$  is 1.12 hours. For 1 inch of rainfall excess, Equation C3-S7-7 provides a discharge of:

$$q_p = \frac{726(240ac)(1in)}{(640 ac/mi^2)(1.12hr)} = 243 ft^3/sec$$

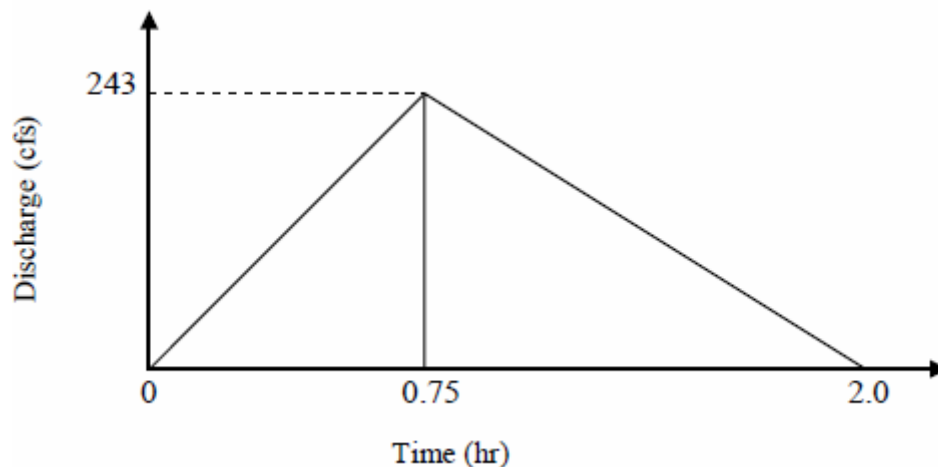
The time to peak is:

$$t_p = \frac{2}{3}T_c = 0.75hr$$

and the time base of the UH is:

$$t_b = \frac{8}{3}t_p = 2hr$$

The resulting triangular UH is shown in Figure C3-S7-3.

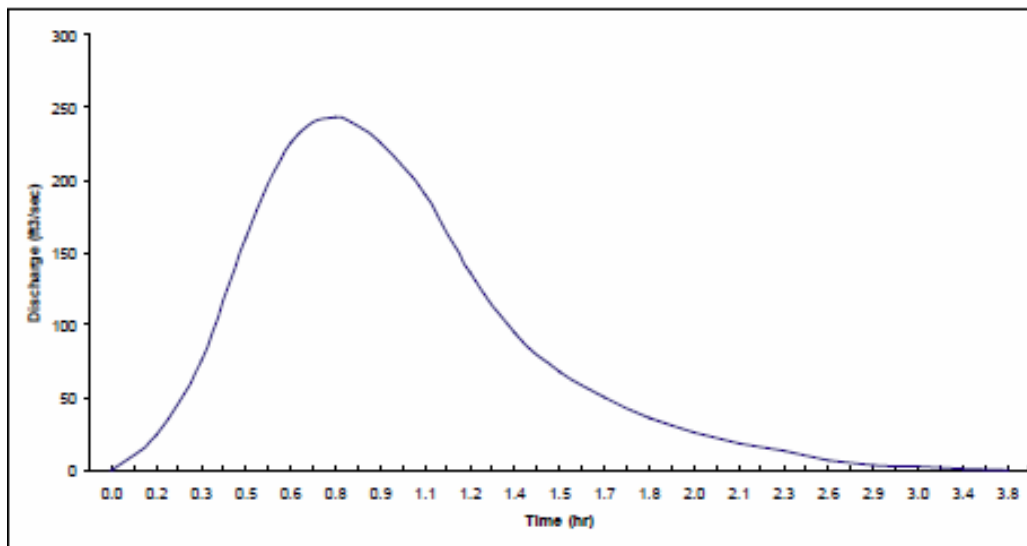


**Figure C3-S7-3: Triangular UH for Example 1**

The NRCS curvilinear UH can be determined for the Example 1 using the values in Table C3-S7-2. The curvilinear UH will be approximated for selected values of  $t/t_p$ ; the WinTR-55 and WinTR-20 computer programs use all of the values shown in Table C3-S7-1. For selected values of  $t/t_p$ , the curvilinear UH is computed in Table C3-S7-2 and shown in Figure C3-S7-4.

**Table C3-S7-2: Calculation of NRCS dimensionless UH for Example 1**

$t/t_p$	$q/q_p$	$t$ (hr)	$q$ (ft <sup>3</sup> /sec)
0.0	0.000	0.000	0.0
0.2	0.100	0.150	24.3
0.4	0.310	0.300	75.3
0.6	0.660	0.450	160.4
0.8	0.930	0.600	226.0
1.0	1.000	0.750	243.0
1.2	0.930	0.900	226.0
1.4	0.780	1.050	189.5
1.6	0.560	1.200	136.1
1.8	0.390	1.350	94.8
2.0	0.280	1.500	68.0
2.2	0.207	1.650	50.3
2.4	0.147	1.800	35.7
2.6	0.107	1.950	26.0
2.8	0.077	2.100	18.7
3.0	0.055	2.250	13.4
3.4	0.029	2.550	7.0
3.8	0.015	2.850	3.6
4.0	0.011	3.000	2.7
4.5	0.005	3.375	1.2
5.0	0.000	3.750	0.0

**Figure C3-S7-4: NRCS curvilinear UH for Example 1**

Although the triangular and curvilinear hydrographs have the same  $q_p$  and  $t_p$ , the time bases are different and the portion of the curvilinear hydrograph near the peak is higher than that of the triangular hydrograph. Both the triangular and curvilinear hydrographs can be considered D-hour, with D computed by Equation C3-S7-5:

$$D = 0.133T_c = 0.149hr = 9min$$

The unit hydrograph would be reported on an interval of 0.15 hours (9 min) and all computations performed at that

interval. The time step,  $d$ , for the UH would be 9 min for this example (0.15 hr) - the ratio of  $d/t_p$  should be in the range of 0.2-0.25. WinTR-55 and WinTR-20 allow computation down to 0.1 hr.

The procedures above can be performed manually using an Excel spreadsheet for preparation of the unit hydrograph table, as well as the graphical form of the unit hydrograph. The computations in WinTR-55 are completed using the TR-20 model using the NRCS 24-hour Type II rainfall distribution and the standard NRCS dimensionless UH as described above. Upon completion of the WinTR-55 analysis, the final runoff hydrograph data is accessed in WinTR-55 under the TR-20 report menu.

### C. Application of the unit hydrograph method

To complete the total direct runoff hydrograph (DRH) for a specified storm event (i.e. 10-year, 24-hour duration) the hyetograph for the design rainfall frequency and duration is computed using the NRCS Type II rainfall distribution, then used with the unit hydrograph to prepare the final total runoff hydrograph for the specified storm event for the watershed. This process in which the design storm hyetograph is combined with the transfer function (dimensionless UH) is called convolution.

1. The standardized NRCS Type II storm, 24-hour storm distribution is selected.
2. The design storm depth is determined from the Iowa rainfall tables (Bulletin 71), based on the return period being modeled. Combined with the rainfall distribution, this specifies the cumulative rainfall depth at all times during the storm.
3. The unit hydrograph has a linear relationship. Two important assumptions are:
  - a. The base time of the direct runoff hydrograph resulting from a rainfall excess of a given duration is constant regardless of the amount of the rainfall excess.
  - b. The ordinates of the direct runoff hydrograph resulting from a rainfall excess of a given duration are directly proportional to the total amount of rainfall excess.
4. Based on the time of concentration, the storm is divided into bursts of equal duration. For each burst, the NRCS runoff equation and the average curve number are used to determine the portion of that burst that will appear as runoff. The duration can be calculated from the  $T_c$  or the  $t_p$  as indicated above. By definition, a rainfall excess with duration of 2 hours and a constant intensity of  $\frac{1}{2}$  in/hr will produce a direct runoff hydrograph (DRH) that is equivalent to  $UH_2$ . With the linearity principle of the UH method, a rainfall excess with a duration of 2 hours and a constant intensity of  $\frac{3}{4}$  in/hr will produce a  $DRH = 1.5UH_2$ . The coefficient 1.5 in front of  $UH_2$  is equal to the depth of rainfall excess in inches ( $\frac{3}{4}$  in/hr for 2 hrs = 1.5 in). A rainfall excess with a duration of 3 hours and constant intensity of  $\frac{2}{3}$  in/hr will produce a  $DRH = 2UH_3$ .
5. The NRCS UH, in conjunction with the time of concentration, is used to determine how the runoff from a single burst is distributed over time. The first burst of rainfall excess of duration  $D$  is multiplied by the ordinates of the unit hydrograph (UH), the UH is then translated a time length of  $D$ , and the next burst of rainfall excess by the UH. The principle of superposition is used to determine the DRH resulting from the composite rainfall excess hyetographs. After the UH has been translated for all bursts of rainfall excess of duration  $D$ , the results of the multiplications are summed for each time interval. The process of multiplication, translation, and addition is the means of deriving a design runoff hydrograph from the rainfall excess and UH. The result is a complete runoff hydrograph for a single burst. The individual hydrographs are added together for all bursts in the storm, yielding the complete runoff hydrograph for the storm.

**Example:** A rainfall excess with constant intensity of 1 in/hr for the first 2 hours and a constant intensity of  $\frac{3}{4}$  in for the next 2 hours. The depth rainfall produces during the first 2 hours is  $(1 \text{ in/hr})(2 \text{ hr}) = 2$  inches. The direct runoff is expressed as  $2UH_2$ . The depth of rainfall excess for the second 2-hour period (burst) is  $(\frac{3}{4} \text{ in/hr})(2 \text{ hr}) = 1.5$  inches, and will result in a direct runoff of  $1.5UH_2$ . However, since the second burst of rainfall excess is delayed 2 hours with respect to time zero, the resulting direct runoff is also delayed. The composite direct runoff resulting from the composite rainfall excess is:

$$DRH = 2UH_2 + 2\text{hours lagged } 1.5UH_2$$

**Example:** The ordinates of a 3-hour unit hydrograph,  $UH_3$ , for a watershed are listed in column 2 of Table C3-S7-3. The

rainfall hyetograph is summarized in Figure C3-S7-6.

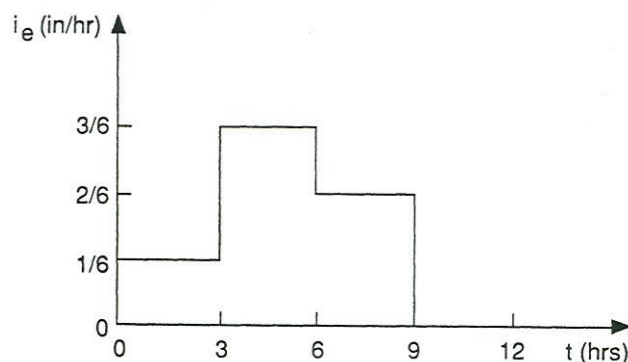
- Determine the depth of the rainfall excess produced during the first, second, and third 3-hour periods (3 bursts at 3-hour duration). For the first time interval (0-3 hours), the depth is  $(\frac{1}{6} \text{ in/hr})(3 \text{ hrs}) = 0.5 \text{ inch}$ . For the second time interval (3-6 hours) the depth is  $(\frac{3}{6} \text{ in/hr})(3 \text{ hr}) = 1.5 \text{ inches}$ . Between hours 6 to 9, the depth is  $(\frac{2}{6} \text{ in/hr})(3 \text{ hr}) = 1 \text{ inch}$ .

$$DRH = 0.5UH_3 + 3\text{hour lagged } 1.5UH_3 + 6\text{hours lagged } 1.0UH_3$$

- Summary calculations are in Table C3-S7-3. The values for the  $UH_3$  are in Column 2. Column 3 is determined by multiplying column 2 by 0.5 inch. Column 4 is determined by multiplying column 2 by 1.5 in. The values in column 5 are determined by lagging the entries of column 4 by 3 hours. Column 6 is obtained by lagging the entries of column 2 ( $1 UH_3$ ) by 6 hours. The final ordinate values for the DRH in Column 7 are the sum of Columns 3, 5, and 6.

**Table C3-S7-3: Example UH to DRH translation**

(1) t (hrs)	(2) $UH_3$ (cfs/in)	(3) $0.5UH_3$ (cfs)	(4) $1.5UH_3$ (cfs)	(5) 3-hrs Lagged $1.5UH_3$ (cfs)	(6) 6-hrs Lagged $1.5UH_3$ (cfs)	(7) DRH (cfs)
0	0	0.0	0.0	0.0	0.0	<b>0.0</b>
1	40	20.0	60.0	0.0	0.0	<b>20.0</b>
2	80	40.0	120.0	0.0	0.0	<b>40.0</b>
3	120	60.0	180.0	0.0	0.0	<b>60.0</b>
4	160	80.0	240.0	60.0	0.0	<b>140.0</b>
5	200	100.0	300.0	120.0	0.0	<b>220.0</b>
6	175	87.5	262.5	180.0	0.0	<b>267.5</b>
7	150	75.0	225.0	240.0	40.0	<b>355.0</b>
8	125	62.5	187.5	300.0	80.0	<b>442.5</b>
9	100	50.0	150.0	262.5	120.0	<b>432.5</b>
10	75	37.5	112.5	225.0	160.0	<b>422.5</b>
11	50	25.0	75.0	187.5	200.0	<b>412.5</b>
12	25	12.5	37.5	150.0	175.0	<b>337.5</b>
13	0	0.0	0.0	112.5	150.0	<b>262.5</b>
14	0	0.0	0.0	75.0	125.0	<b>200.0</b>
15	0	0.0	0.0	37.5	100.0	<b>137.5</b>
16	0	0.0	0.0	0.0	75.0	<b>75.0</b>
17	0	0.0	0.0	0.0	50.0	<b>50.0</b>
18	0	0.0	0.0	0.0	25.0	<b>25.0</b>



**Figure C3-S7-5: Rainfall excess hyetograph for UH/DRH example**



For the NRCS dimensionless hydrograph developed in Figure C3-S7-5, and a  $T_c$  of 1.12 hours, the burst duration is 9 minutes, so a 24-hour storm will consist of 160 bursts. If each burst involves a unit hydrograph of 30 coordinates, then 4800 coordinates must be summed to produce the composite hydrograph. While the computations can be completed using a spreadsheet model, a manual convolution can be somewhat time-consuming.

When using the WinTR-55 model, the full runoff hydrograph convolution is calculated using the TR-20 computations as described above, and the hyetograph for each storm frequency is automatically generated using the standard NRCS 24-hour rainfall distribution. A manual shortcut method of hydrograph generation was derived by the NRCS. This simplified tabular method is summarized as follows, and is described in full detail in Technical Release 55 (TR-55). A summary of the NRCS Tabular method follows.

#### D. Watershed routing (hydrograph determination)

Watershed routing is used when the watershed basin has multiple sub-basins and it is desired to add hydrographs together from each of the sub-basins to determine the combined hydrograph at critical points. Common critical points are at control conveyance structures where an inflow hydrograph is required to route the discharges through structures. The most common structure where an inflow hydrograph is required is a stormwater detention basin (Chapter 3 - Section 10 Channel and Storage (Reservoir) Routing for detention basin storage routing procedures). The WinTR-55 model will calculate the full runoff hydrograph for a single watershed or a watershed with up to 10 sub-areas, and complete the watershed routing to provide a complete runoff hydrograph at the watershed outlet or point of design. The NRCS tabular hydrograph method described below is recommended for the manual determination of peak discharge and routing of hydrographs for watersheds with multiple sub-areas.

#### E. Tabular hydrograph method

The TR-55 tabular hydrograph method is used for computing discharges from rural and urban areas, using the time of concentration ( $T_c$ ) and travel time ( $T_t$ ) from a sub-area as inputs. The NRCS TR-55 methodology can determine peak flows from areas of up to 5.5 square miles (2000 acres), provide a hydrograph for times of concentration of up to 2 hours, and estimate the required storage for a specified outflow. The tabular method can develop partial composite flood hydrographs at any point in a watershed by dividing the watershed into homogeneous sub-areas. In this manner, the method can estimate runoff from non-homogeneous watersheds. The method is especially applicable for estimating the effects of land use change in a portion of a watershed. It can also be used to estimate the effects of proposed structures.

1. **Use of the tabular hydrograph method.** The tabular hydrograph method is based upon a series of unit discharge hydrographs that were developed by the NRCS in the late 1970s. The tabular data was developed by computing hydrographs for 1 square mile of drainage area for selected  $T_c$ 's and routing them through stream reaches with the range of  $T_t$ 's indicated. The Modified Att-Kin method for reach routing, formulated by NRCS in the late 1970s, was used to compute the tabular hydrographs. A CN of 75 and rainfall amounts generating appropriate  $I_a/P$  ratios were used. The resulting runoff estimate was used to convert the hydrographs into cubic feet per second per square mile per inch of runoff (csm/in). The full set of tabular data for the Type II rainfall distribution consists of 10 individual tables for the entire range of  $T_c$ ,  $T_t$ , and  $I_a/P$  values. Figure C3-S7-6 is the tabular data for Type II rainfall distribution,  $T_c = 0.75$  hour, travel times of 11.0 to 26.0 hrs, and  $I_a/P$  ratios of 0.1, 0.3, and 0.5. The full set of tables can be obtained at:

<http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?cid=stelprdb1042901>; the tabular data is contained in Chapter 5, of the 1986 TR-55 (version 2.31) user manual.

- a. The input data needed to develop a flood hydrograph include:
  - 1) 24-hour rainfall (in)
  - 2) Appropriate rainfall distribution (I, IA, II, or III)
  - 3) Curve Number (CN)
  - 4) Time of Concentration,  $T_c$  (hr)
  - 5) Travel Time,  $T_t$  (hr)
  - 6) Drainage area,  $A_m$  (mi<sup>2</sup>)

- b. The process for developing a hydrograph using the tabular method is described below. The results of each step are recorded in Worksheet 1 in order to develop a summary of basic watershed data by subarea. The steps are as follows:
- 1) Subdivide the watershed into areas that are relatively homogeneous and have convenient routing reaches. Enter the name of each subarea in the first column.
  - 2) Determine the drainage areas of each subarea in square miles and enter the values in the Drainage Area,  $A_m$  column.
  - 3) Calculate the time of concentration ( $T_c$ ) for each reach in hours. (Chapter 3 - Section 3 Time of Concentration). Enter the values for each subarea in the  $T_c$  column.
  - 4) Determine the travel time ( $T_t$ ) for each reach in hours. (Chapter 3 - Section 3 Time of Concentration). Record the values in the  $T_t$  column.
  - 5) Record the downstream reaches through which the runoff from each upstream area flows in the column labeled "Downstream Sub-area Names."
  - 6) Calculate the cumulative travel times, through each downstream reach, for each sub-area to the point of interest and enter the result in the  $\Sigma T_t$  column.
  - 7) Determine the 24-hour rainfall depth ( $P$ ) for the desired storm event (Chapter 3 - Section 2 Rainfall and Runoff Analysis). Enter the value in column  $P$ .
  - 8) Determine a weighted curve number ( $CN$ ) for each subarea and enter in column "CN."
  - 9) Calculate the total runoff ( $Q$ ) in inches, for each sub-area, computed from  $CN$  and rainfall ( $P$ ) utilizing the NRCS runoff equation:

**Equation C3-S7-8**

$$Q = \frac{(P - 0.2S)^2}{P0.8S}$$

Where:

$Q$  = Runoff (in)

$P$  = Rainfall (in)

$S$  = Potential maximum retention after runoff begins (in)

and:

**Equation C3-S7-9**

$$S = \left( \frac{1000}{CN} \right) - 10$$

- 10) Multiply columns  $A_m$  and  $Q$  to determine the total volume of runoff for each subarea. Enter results in column  $A_m Q$ .
- 11) Determine the initial abstraction ( $I_a$ ). The initial abstraction is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration.  $I_a$  is highly variable, but generally is correlated with soil and cover parameters. Through studies,  $I_a$  was found to correlate with the Curve Number as indicated in Table C3-S7-2.
- 12) Calculate  $I_a/P$  for each subarea and enter in the last column. If the ratio for the rainfall distribution of interest is outside the range shown in Figure C3-S7-6, use the limiting value.
- 13) Now that the drainage basin's sub-area properties have been summarized in Worksheet 1, a composite flood hydrograph may be determined utilizing Worksheet 2 and the appropriate values from Figure C3-S7-6.
- 14) An assumption in development of the tabular hydrographs is that all discharges for a stream reach flow at the same velocity. By this assumption, the sub-area flood hydrographs may be routed separately and added at the reference point.
- 15) Compute the hydrograph coordinates for Worksheet 2 for the selected total reach travel times using the appropriate tabular data sheet for the sub-area  $T_c$  and  $I_a/P$ . The actual  $I_a/P$  values can be rounded to the closest value in the tables (0.1, 0.3, 0.5), or one may use interpolation. The flow at any time is:

**Equation C3-S7-10**

$$q = q_t A_m Q$$

Where:

$q$  = hydrograph coordinate (ft<sup>3</sup>/s) at hydrograph time,  $t$

$q_t$  = tabular hydrograph unit discharge from Figure C3-S7-6 (csm/in)

$A_m$  = drainage area of individual subarea (mi<sup>2</sup>)

$Q$  = runoff (in)

Since the timing of peak discharge changes with  $T_c$  and  $T_t$ , interpolation of peak discharge for  $T_c$  and  $T_t$  values is not recommended. Interpolation may result in an estimate of peak discharge that would be invalid because it would be lower than either of the hydrographs. Therefore, round the actual values of  $T_c$  and  $T_t$  to values presented in the tabular data (Figure C3-S7-6). Perform this rounding so that the sum of the selected table values is close to the sum of actual  $T_c$  and  $T_t$ . An acceptable procedure is to select the results of one of three rounding operations:

- Round  $T_c$  and  $T_t$  separately to the nearest table value and sum
- Round  $T_c$  down and  $T_t$  up to nearest table value and sum
- Round  $T_c$  up and  $T_t$  down to nearest table value and sum.

Transfer the necessary information for each subarea from Worksheet 1 over to Worksheet 2 ( $T_c$ ,  $\Sigma T_t$ ,  $I_a/P$ , and  $A_m Q$ ). Enter a consecutive series of hydrograph times from Figure C2-S7-7. Review the unit discharges in the tables for the various combinations of  $T_c$  and  $T_t$ , and make a rough estimate of when the peak flow for the composite section will occur. This value should be in the middle of the time range entered in the worksheet.

- 16) For each subarea and hydrograph time, identify the table of hydrograph data (i.e. Figure C3-S7-6) for the appropriate values of  $T_c$ ,  $T_t$ , and  $I_a/P$ , and find the corresponding unit discharge in csm/in. Multiply this value by the value shown in the  $A_m Q$  column, and record the result in the worksheet as the discharge for that sub-area at the selected hydrograph time. Repeat this process for each hydrograph time and sub-area. Upon completion, sum each of the time columns to obtain the composite hydrograph at the outlet.
- 17) Review the values for the composite hydrograph. Ensure that the values rise, reach a maximum, and then begin to fall to ensure that the time range selected includes the peak discharge.

2. **Tabular method limitations.** The tabular method is used to determine peak flows and hydrographs within a watershed. However, its accuracy decreases as the complexity of the watershed increases. If you want to compare present and developed conditions of a watershed, use the same procedure for estimating  $T_c$  for both conditions. Use the NRCS WinTR-55 (Chapter 3 - Section 5 NRCS TR-55 Methodology) or the WinTR-20 program models instead of the tabular method, if any of the following conditions applies:
  - $T_t$  is greater than 3 hours (largest  $T_t$  in tabular hydrograph data)
  - $T_c$  is greater than 2 hours (largest  $T_t$  in tabular hydrograph data)
  - Drainage areas of individual subareas differ by a factor of 5 or more
  - The entire composite flood hydrograph or entire runoff volume is required for detailed flood routings. The hydrograph based on extrapolation (tabular method) is only an approximation of the entire hydrograph. Use the full WinTR-55 or WinTR-20 hydrograph routing for determining inflow hydrograph for detention storage routing design.
  - The time of peak discharge must be more accurate than that obtained through the tabular method.

The composite flood hydrograph should be compared with actual stream gage data where possible. The instantaneous peak flow value from the composite flood hydrograph can be compared with data from USGS curves of peak flow versus drainage area.

Table C3-S7-4:  $I_a$  values for runoff curve numbers

Curve number	$I_a$	Curve number	$I_a$	Curve number	$I_a$	Curve number	$I_a$
40	3.000	55	1.636	70	0.857	85	0.353
41	2.878	56	1.571	71	0.817	86	0.326
42	2.762	57	1.509	72	0.778	87	0.299
43	2.651	58	1.448	73	0.740	88	0.273
44	2.545	59	1.390	74	0.703	89	0.247
45	2.444	60	1.333	75	0.667	90	0.222
46	2.348	61	1.279	76	0.632	91	0.198
47	2.255	62	1.226	77	0.597	92	0.174
48	2.167	63	1.175	78	0.564	93	0.151
49	2.082	64	1.125	79	0.532	94	0.128
50	2.000	65	1.077	80	0.500	95	0.105
51	1.922	66	1.030	81	0.469	96	0.083
52	1.846	67	0.985	82	0.439	97	0.062
53	1.774	68	0.941	83	0.410	98	0.041
54	1.704	69	0.899	84	0.381		

Source: TR-55: Urban Hydrology for Small Watersheds, USDA/NRCS





**Exhibit 5-II: Tabular hydrograph unit discharges (csm/in) for type II rainfall distribution—continued**

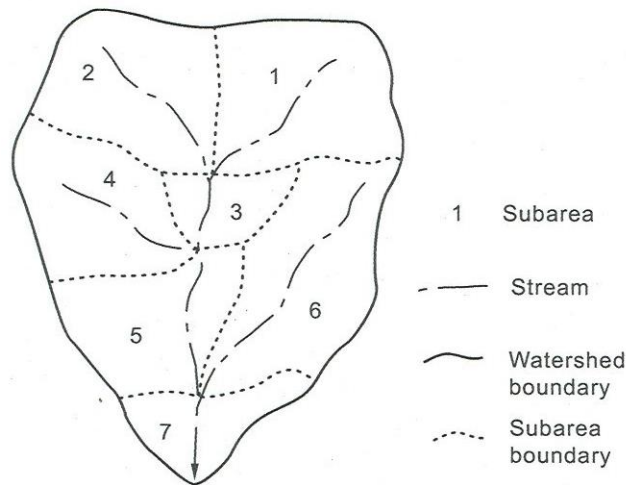
TRWL TIME	HYDROGRAPH TIME (HOURS)																																
	11.0	11.3	11.5	11.9	12.1	12.2	12.3	12.4	12.5	12.6	12.7	13.0	13.2	13.4	13.5	13.8	14.3	14.5	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0				
0.0	13	18	24	36	46	68	115	194	294	380	424	410	369	252	172	123	93	74	61	49	41	35	31	27	24	22	20	19	17	15	12	0	
.10	13	17	23	34	42	59	97	162	250	337	395	405	391	279	191	135	100	77	65	51	42	36	31	28	25	22	21	19	17	15	12	0	
.20	11	15	20	28	32	39	52	82	135	211	295	362	391	351	255	178	127	95	75	57	46	38	32	29	26	23	21	20	17	15	12	0	
.30	11	14	19	26	30	36	47	70	113	179	256	326	379	360	277	196	140	103	80	60	48	38	33	29	26	23	21	20	18	15	12	0	
.40	10	12	16	22	25	28	33	42	61	96	151	221	291	357	336	255	182	131	98	69	54	42	34	30	27	24	22	20	18	16	12	0	
.50	9	12	15	21	24	27	31	39	53	82	128	190	258	358	343	274	200	144	106	74	56	43	35	30	27	24	22	20	18	16	12	0	
.75	8	10	13	17	18	21	23	26	31	39	55	82	122	230	314	329	281	217	161	104	72	51	38	33	29	26	23	21	19	16	12	1	
1.0	6	8	10	13	14	15	17	19	21	23	27	32	42	89	177	272	319	303	249	163	105	66	45	36	31	27	24	22	19	17	13	3	
1.5	4	6	7	9	10	10	11	12	14	15	16	18	20	27	46	90	163	241	295	275	204	119	66	45	35	31	27	24	20	18	13	7	
2.0	3	4	5	6	7	7	8	9	9	10	11	12	13	16	20	28	48	89	151	245	274	213	115	65	44	35	30	27	22	19	14	10	
2.5	1	2	3	4	4	5	5	6	6	7	7	8	10	12	14	17	24	37	86	170	260	219	127	71	47	36	31	24	20	16	11		
3.0	1	1	2	3	3	4	4	4	4	5	5	6	7	8	10	11	14	17	30	64	157	247	205	122	70	46	35	27	22	17	12		
	IA/P = 0.30																IA/P = 0.30																
0.0	0	0	0	0	0	1	6	30	86	174	266	326	348	328	246	181	138	110	92	79	66	57	49	44	40	36	32	31	29	26	23	19	0
.10	0	0	0	0	0	1	4	22	65	137	223	292	329	303	228	170	131	106	89	73	61	52	46	41	37	33	31	29	26	23	19	0	
.20	0	0	0	0	0	0	3	15	48	106	185	256	305	321	245	184	141	112	93	75	63	53	46	42	37	34	31	30	27	23	19	6	
.30	0	0	0	0	0	0	2	11	36	84	151	221	277	308	260	199	152	120	98	78	65	54	47	42	38	34	31	30	27	23	19	0	
.40	0	0	0	0	0	0	0	1	8	27	65	122	188	266	301	243	187	144	114	87	71	57	48	43	39	35	32	30	27	24	19	1	
.50	0	0	0	0	0	0	0	1	6	20	50	98	158	253	292	251	200	155	122	91	74	59	49	44	40	35	32	30	27	24	19	1	
.75	0	0	0	0	0	0	0	0	0	0	2	8	23	51	140	231	269	253	211	167	119	90	68	53	46	42	37	34	31	28	25	19	2
1.0	0	0	0	0	0	0	0	0	0	0	0	1	4	29	96	186	249	261	231	169	120	84	61	50	44	40	36	33	29	26	20	5	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	34	91	163	220	241	197	131	83	61	50	44	40	35	31	27	21	12
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	11	36	85	174	226	206	127	82	60	49	44	39	32	29	22	17	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	6	37	105	196	214	135	87	62	51	44	36	31	24	18		
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	4	24	96	205	189	130	85	62	50	39	32	26	18			
	IA/P = 0.50																IA/P = 0.50																
0.0	0	0	0	0	0	0	2	16	45	92	137	166	185	170	146	125	110	98	89	79	70	63	58	53	48	44	42	41	37	33	28	0	
.10	0	0	0	0	0	0	0	1	11	34	73	115	149	180	163	141	122	107	96	84	74	65	59	54	50	45	43	41	38	33	28	0	
.20	0	0	0	0	0	0	0	1	8	25	57	96	131	173	166	146	126	111	99	86	76	66	59	55	50	46	43	41	38	34	28	0	
.30	0	0	0	0	0	0	0	0	1	5	18	44	79	143	176	160	141	122	108	92	81	69	61	56	52	47	44	42	38	34	28	1	
.40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
.50	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
.75	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	
	RAINFALL TYPE = II																RAINFALL TYPE = II																

Figure C3-S7-6: Tabular hydrograph discharge for  $T_c=0.75$  hr and rainfall distribution Type II  
 Source: NRCS TR-55 Urban Hydrology for Small Watersheds, Version 2.31, 1986

## F. Design example

### Watershed Hydrograph Routing

A new residential development, Hickory Hills, is proposed being proposed for a rural site in Dallas County, IA. A map of the local watershed in shown in Figure C3-S7-7 with the development site located in subareas 5, 6, and 7.



**Figure C3-S7-7: Watershed sub-areas for tabular hydrograph Example**

Source: Adapted from NRCS TR-55, 1986, version 2.31 user manual, Chapter 5

Compute the peak discharge for the watershed outlet for the developed condition. The basic subarea data for the watershed is entered into Worksheet 2, and is shown in Worksheet 3. The subarea data for  $T_c$ , the summation of travel time to the outlet for each subarea, and  $I_a/P$  values, and the  $A_mQ$  values have been transferred over to Worksheet 2 shown in Worksheet 4.

Entry of the hydrograph discharge values in Worksheet 2 is determined as follows:

1. For subarea 6 and using the tabular data in Figure C3-S7-6 for  $T_c = 0.75$  hr,  $T_t$  of 0, and  $I_a/P$  value of 0.1 (rounded up).
2. The unit peak discharge is 424 csm/in and occurs at the hydrograph time ( $t$ ) of 12.6 hour. The unit peak discharge is multiplied times the  $A_mQ$  value for subarea 6 to complete the calculation of the runoff discharge in  $\text{ft}^3/\text{sec}$  for this time. The final computed value is 411  $\text{ft}^3/\text{sec}$ . The remaining hydrograph values for subarea 6 for the other hydrograph times are completed. In this example, the same data sheet can be used for subarea 4 which has a  $T_c$  of 0.75 hour. However, for this subarea, the travel time is 1.5 hour; the hydrograph peak for subarea 4 is 207  $\text{ft}^3/\text{sec}$ , and occurs at time 14 hours. (For the hydrograph data for the remaining subareas is obtained from the additional tabular data sheets for rainfall distribution Type 11 and additional  $T_c$ 's). The values in this example are extracted from those sheets to provide a complete routing for this example.
3. The individual discharges are summed for each hydrograph time to give the final composite runoff hydrograph for the watershed. The total peak discharge at the outlet is 965  $\text{ft}^3/\text{sec}$ , and occurs at 13 hours.



Project		Hickory Hills Development				Location Dallas County, Iowa				By SEJ		Date 12/12/06	
Check one:		<input type="checkbox"/> Present		<input checked="" type="checkbox"/> Developed		Event Frequency (yr): 25				Checked CJJ		Date 12/19/06	
Subarea name	Drainage Area $A_m$ (mi <sup>2</sup> )	Subarea Time of Concentration $t_c$ (hr)	Travel time through subarea $T_t$ (hr)	Downstream subarea names	Travel time summation to outlet (design point) $\sum T_t$ (hr)	24-hr rainfall P (inches)	Runoff Curve number CN	Runoff Q (inches)	$A_m Q$ (mi <sup>2</sup> -in)	Initial Abstraction $I_a$ (in)	$I_a / P$		
1	0.30	1.50		3,5,7	2.00	5.15	65	2.35	0.71	1.077	0.18		
2	0.20	1.25		3,5,7	2.00	5.15	70	2.80	0.56	0.857	0.14		
3	0.10	0.50	0.50	5,7	1.50	5.15	75	3.28	0.33	0.667	0.11		
4	0.25	0.75		5,7	1.50	5.15	70	2.80	0.70	0.857	0.14		
5	0.20	1.50	1.00	7	0.50	5.15	85	4.31	0.86	0.353	0.06		
6	0.40	1.00		7	0.50	5.15	75	3.28	1.31	0.857	0.14		
7	0.20	0.75	0.50		0.00	5.15	90	4.85	0.97	0.222	0.04		

Project: Hickory Hills Development		Location: Dallas County, Iowa		By: SEJ		Date: 12/12/06																
Check one: <input type="checkbox"/> Present <input checked="" type="checkbox"/> Developed		Event Frequency (yr): 25		Checked: C, JJ		Date: 12/19/06																
Subarea name	Subarea $t_c$ (hr)	$I_a/P$	$A_m Q$ ( $m^2$ -in)	Select and enter hydrograph times in hours from Tabular Hydrograph Tables <sup>2</sup>																		
		$\Sigma T_t$ (hr)		Discharges at selected hydrograph times in hours (From Tabular Hydrograph Tables) <sup>3</sup>																		
1	1.50	2.00	0.18	7	8	11	16	24	39	92	121	137	154	145	136	128	106	89	71	54	40	30
2	1.25	2.00	0.14	7	9	12	20	33	55	108	132	136	132	114	96	81	67	53	42	32	25	17
3	0.50	1.50	0.11	2	2	2	3	3	4	6	7	9	14	29	57	88	106	101	74	46	12	10
4	0.75	1.50	0.14	3	4	4	5	6	7	10	11	14	19	32	63	114	169	207	193	143	83	46
5	1.50	0.50	0.06	117	167	205	215	203	176	122	99	84	70	61	53	46	41	36	32	28	25	22
6	1.00	0.50	0.14	10	12	14	17	21	26	50	102	209	332	408	394	329	256	195	134	97	43	38
7	0.75	0.00	0.04	13	16	19	23	31	45	285	411	358	244	167	119	90	72	59	48	40	34	30
Composite hydrograph at outlet				159	218	269	298	321	352	672	884	947	965	957	918	876	816	741	592	441	263	194

<sup>1</sup> Basic watershed subarea from Worksheet 1. (Note: rounded as needed for use with Tabular Hydrograph unit discharge tables)

<sup>2</sup> Select range of hydrograph times from Tabular hydrograph unit discharge tables to provide overlap of expected peak discharge rates for each subarea based on  $t_c$ , travel times, and

<sup>3</sup> Hydrograph discharge for selected time is  $A_m Q$  multiplied times the tabular discharge (csm/in) from the appropriate table. Use Tables for Type II rainfall distribution for Iowa watersheds.