FDM 13-10-1 Design Criteria

August 8, 1997

1.1 Introduction

To the highway engineer, hydrology includes the analysis of precipitation and runoff, and the determination of a flood flow rate for a given stream or channel. It also addresses the frequency of flood occurrence.

1.2 Flood Frequency

Flood frequency or recurrence interval is defined as the average interval in years between the actual occurrence of a hydrological event of a given or greater magnitude. For example, a flood frequency of 50 years means that a storm of that magnitude or greater would be expected to occur on the average of once every 50 years. It also can be stated that a 50-year flood would have a 2% chance of occurring in any one year.

Flood frequencies for various classes of highways and types of drainage structures have been selected to produce a balance between the cost of a drainage facility and the cost of potential flood damage - including risk to the traveling public. These selected frequencies are referred to as design flood frequencies or design frequencies, and are used in determining the magnitude of the design flood - which the drainage structure must accommodate with low probability of risk to the traveling public, minimum damage to the roadway, and minimum flood damage to adjacent property. By common definition, the design flood does not inundate the roadway. In many instances, the design flood will not approach overtopping of the roadway, but will be limited to a maximum backwater elevation so as not to create unreasonable flood damage to either the roadway or adjacent property.

1.3 Design Frequency

The hydraulic design of drainage structures shall use the flood design frequencies given in Attachment 1.1 of this procedure. Design frequencies for bridges and box culverts are not included in this attachment, but the procedure for their sizing is discussed in the Bridge Manual Chapter 8.

1.3.1 Major Drainage Structures

Watercourses of sufficient magnitude to potentially produce significant flood damage (to the roadway, drainage structure, or abutting property) are most frequently crossed using a major drainage structure (a bridge, box culvert, or their replacement with large drainage conduits). Therefore, when a major drainage structure is required, the process of selecting a design frequency which best produces a balance between structure costs and the cost of potential flood related damages or risks, requires a detailed analysis of each situation. It also requires that the designer be knowledgeable of FAPG Part 650A, "Location and Hydraulic Design of Encroachments on Flood Plains;" NR 116, "Wisconsin's Floodplain Management Program;" NR 320, "Bridges an Culverts in or Over Navigable Waterways;" and the "Cooperative Agreement Between the Wisconsin Department of Transportation and Department of Natural Resources" (refer to https://wisconsindot.gov/Pages/doing-bus/eng-consultants/cnslt-rsrces/environment/formsandtools.aspx).

Therefore, the following method should be used when designing a major drainage structure:

The hydraulic design of major drainage structures is to be addressed in terms of either a replacement structure condition, or a structure associated with a highway on new location.

Replacement structures should typically be sized to develop headwater elevations not greater than that experienced with the existing structure in place. This presumes that extensive experience at the existing structure site has indicated acceptable backwater elevations, permissible stream velocities, and adequate protection for the roadway and motorist. When this is the case, the headwater elevation for the regional flood (100 year-flood) with the existing structure in place should be computed and used as a controlling hydraulic factor in the design of the replacement structure.

Occasionally a reasonable increase in headwater depth would lead to a material savings in structure costs that would obviously outweigh backwater related impacts or risks. In these situations, the acceptable headwater elevation under regional flood conditions should be determined and then used as a controlling hydraulic factor in the structure sizing. The "acceptable" headwater elevation must also take into consideration the floodplain management standards of NR 116, relevant local floodplain zoning ordinances, and the potential need for drainage easements.

Upon completion of the structure design, predicted water surface elevations shall be made available to the applicable local zoning authorities. When a structure is located on a stream that has an established water surface profile for the regional flood incorporated into the local zoning ordinance, the region shall provide the local agency with the predicted water surface elevations. It is then incumbent upon the local agency to amend their zoning ordinance, as outlined in NR 116, whenever the headwater elevation would be increased over that contained in the zoning ordinance.

Structures for highways on new locations should generally be designed to accommodate the regional flood without increasing the backwater (0.01') over that of existing conditions. However, if reduced structure costs significantly outweigh any backwater related impacts, the procedures required for its accomplishment are the same as previously described for replacement structures.

Requirements for documentation of structure sizing are contained in FDM 13-1-10, "Documentation of Hydrologic/Hydraulic Design."

Plan survey datum must conform to datum in use by local zoning authorities. The datum in almost all cases are USGS or USC and GS datum.

1.4 Freeboard Considerations

The provisions for freeboard in the design of bridges is desirable and should be achieved whenever practicable. While sound engineering judgment must be used in this determination, experience has shown that 2 ft of freeboard for the 100-year flood provides a reasonable allowance for the passage of debris, ice flow, etc. under extreme flood conditions. If other factors outweigh the achievement of a 2 ft freeboard (e.g. high cost, undesirable profile, etc.), this should be documented in the "Discussion of Structure Sizing" which is addressed in FDM 13-1-10.

Freeboard may also be necessary to provide reasonable clearances for navigation purposes. Section NR 320 of the Wisconsin Administrative Code makes reference to a 5-foot clearance over navigable waterways, which is measured from a waterway's "ordinary high-water mark" as would be evident from observation of the stream bank. The need to provide freeboard for this purpose should be investigated whenever existing usage of the waterway would indicate that this is a relevant consideration.

A discussion on the hydraulic design of culverts and associated freeboard considerations is given in FDM 13-15-5.

1.5 Use and Design of Overflow Sections

Normally, hydraulic structures on arterials should be designed to convey an appropriate frequency of flood without inundation of the highway. However, under special circumstances on collectors and local roads, a specified flood (i.e. overtopping flood) may be conveyed by the structure and an overflow section, both acting together as a hydraulic system. This type of design should be undertaken only after considering an incremental analysis of estimated construction costs; probable property damage, including damage to the highway; traffic volumes and the cost of traffic delays; duration and depth of inundation; frequency of occurrence; length of roadway to be flooded; availability of alternate routes, emergency supply, and evacuation routes; and considering the potential for loss of life and budgetary constraints.

Where possible, the roadway approach embankments for an overflow section should be constructed slightly above the design flood elevation while the low point of the superstructure should be constructed with an appropriate amount of freeboard. With this type of design, the structure would convey the design flood while the overflow section would convey the "super flood" (or unusually large flood). Thus, large floods would cause minimal damage to the structure itself. If during flood stage the overflow section operates as a weir having no downstream tail water, the downstream roadway embankment may erode. Under these circumstances the downstream roadway embankment of the overflow section should be protected with riprap or some other erosion-resistant material if significant damage is likely to occur.

References detailing the hydraulic design of overflow sections are contained in FDM 13-20-1.

For projects on collectors or local roads that are being designed in anticipation of roadway overflow, the designer should consider specifying "HIGH-WATER" advance warning signs. In general, this sign should be used when all three of the following conditions exist:

1. The current ADT is greater than 300 AND
2. The operating speed exceeds 35 mph AND
3. The expected overflow frequency is more often than once every ten years, i.e. the ten-year storm is expected to cause overflow.

1.6 Probability of Flood Occurrence

The probability (P) that an event with a recurrence interval t_p will be equaled or exceeded in any one year is:

$$P = 1/t_p$$

For example, floods with recurrence intervals of 10 years, 50 years, and 100 years are also called a "10 percent flood," a "two percent flood," and a "one percent flood," respectively. In other words, in any one-year period the probability of getting a 10-year, 50-year, or 100-year flood is 10 percent, two percent, and one percent, respectively.

When communicating with the public about specific floods, it is probably more effective to talk about a percent flood instead of recurrence interval. The use of recurrence interval may give the false impression that a specific flood will occur only at those intervals, whereas in fact there is a specific constant probability that it will be equaled or exceeded in any one year.

The risk of flooding is the probability that a flood with a given probability will be equaled or exceeded at least once in a specified number of years. Attachment 1.2, which lists risks of flooding for various design periods and recurrence intervals, shows that there is a 64 percent chance that the 50-year flood (or greater) will occur in any 50-year period, and even a 40 percent chance that the 100-year flood (or greater) might occur in the 50-year period.

The risk of floodland occupancy can be determined from Attachment 1.3. For example, if the "100-year flood stage" is coincident with the first floor of a building, the probability of first floor (or more) damage before the 25-year mortgage is paid is 22 percent.

For values not listed in Attachment 1.2, the risk of flooding R that at least one event that equals or exceeds the t_p year event will occur in any series of N years is:

$$R = 1-(1-P)^{N1.}$$

1.7 Future Development Effects

Future land development and urbanization will greatly affect the anticipated runoff peaks and volumes for some drainage structures and ditches. In most cases it is very difficult to predict the type and extent of future urbanization. Despite this, it is suggested that the following methods be used as guides in this regard:

1. Areas of 200 acres or less should have a runoff coefficient C for the rational formula determined on the basis of future anticipated conditions. If the majority of the drainage area will be urbanized, the Rational Method may be used on areas up to five square miles.

2. For drainage basins less than five square miles with scattered urban development and for urban drainage basins over five square miles, comprehensive studies of the watershed must be undertaken. These comprehensive studies entail using synthetic hydrographs, which are combined and routed through the drainage basin to the design structure and/or drainage channel.

   Use a runoff figure based on land development expected in the watershed 20 years in the future. Data on existing and future land use can be obtained from regional planning commissions. In addition, these regional planning commissions have published comprehensive plans for various watersheds, which give flood flows for present and/or future (20 years hence) land-use conditions.

1.8 Hydraulic Information on Plans

The hydraulic data that must be shown on structure plans is given in WisDOT's Bridge Manual. This includes providing the flood magnitude and water surface elevation (headwater) associated with the 100-year flood. If the roadway will be overtopped by a flood of lesser magnitude than the 100-year flood, the recurrence interval of the overtopping flood and its magnitude should also be given. When the overtopping flood is greater than the 100-year flood a note should be included with the hydraulic data that states, "Overtopping Road Not Applicable."

Whenever it is determined to use large drainage conduit to replace a major drainage structure, the hydraulic data noted above shall be provided on an appropriate roadway plan sheet.

LIST OF ATTACHMENTS

Attachment 1.1	Flood Design Frequency Selection Chart
Attachment 1.2	Probability of Flood Occurrence (Table)
Attachment 1.3	Probability of Flood Damage Before Payment of 25-Year Mortgage

FDM 13-10-5 Methods of Determining Peak Runoff

November 15, 2019

5.1 Design Discharge

The first step in designing a hydraulic structure is to determine the amount of water to be carried - the design discharge. The problem is particularly difficult for small watersheds, say, under five square miles, because the smaller the area, the more sensitive the design discharge is to conditions that affect runoff and the less likely there are runoff records for the area.

The design discharge is related to the effective rainfall, which is that portion of precipitation that produces direct runoff. Losses or abstractions are that portion of precipitation that is removed from direct runoff through detention, infiltration, evapotranspiration, etc. The best method of determining a design discharge is to use site specific runoff records; but, since these are often non-existent, estimates of runoff must be based on frequency of rainfall by assuming the runoff to have the same frequency as the rainfall of the design storm or on flood-frequency equations developed from regional gauging stations.

There are many methods used to determine discharge values. The methods presented in this chapter may be classified as being based on rainfall frequency (first two methods), runoff records (next two methods), a combination of rainfall frequency and runoff records (next method), and historic data (the last method).

The runoff methods presented in this chapter are:

1. Rational method
2. Hydrology for small watersheds, NRCS - Urban Hydrology for Small Watersheds, (TR-55)
3. USGS flood frequency equations for Wisconsin
    - Flood Frequency Characteristics of Wisconsin Streams, 1992
    - Techniques for Estimating Magnitude and Frequency of Floods in Wisconsin - 1981
    - Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams - 1986
4. Gauging station
    - Log-Pearson Type III distribution
    - Transferring gaged discharges
5. Published watershed studies
6. Field review notes, interviews, and historic data

Due to inherent differences in the methods, it is recommended that the designer compute runoff by at least two of these methods. The results serve as a comparison check and may be averaged or weighted according to the most applicable method to arrive at a design discharge. Attachment 5.1 is a guideline for area limits of various methods.

5.2 Discharge Frequency Graph

A discharge-frequency graph should be constructed for each of the runoff methods presented in this chapter. For an example of the construction and use of a discharge-frequency graph, the designer is referred to the design methods entitled "Flood Frequency Characteristics of Wisconsin Streams" and "Techniques for Estimating Magnitude and Frequency of Floods for Wisconsin," which are contained in this procedure.

5.3 Rational Method

The Rational Method has been the most common approach used to design storm sewers since the publication of a paper by Kuichling in 1889 [1]. The rational formula has the advantage that its physical meaning is reasonably clear. However, it should be used with caution, because it can overestimate peak flows for large drainage basins. As stated previously, comparing multiple methods of determining peak flow is always advised.

The Rational Method is recommended for use in estimating design discharges for urban areas or potential urban areas of five square miles or less. In addition, it may also be used for small rural basins 200 acres or less having similar or non-similar ground cover, e.g., combinations of woodlands, pastureland, and cropland.

The basic assumptions for the Rational Method are:

1. Peak flow occurs when the entire watershed is contributing to the flow.
2. Rainfall intensity is the same over the entire drainage area.
3. Rainfall intensity is uniform over time duration equal to the time of concentration, t_c. The time of concentration is the time required for water to travel from the hydraulically most remote point of the

basin to the point of interest.

4. Frequency of the computed peak flow is the same as that of the rainfall intensity, i.e., the 10-year rainfall intensity is assumed to produce the 10-year peak flow.
5. Coefficient of runoff is the same for all storms of all recurrence probabilities.

The rational formula is:

$$Q = CIA$$

Where:

Q = peak runoff rate in cubic feet per second (cfs)

C = runoff coefficient, which is the ratio of the peak runoff rate to the average rainfall rate for a duration equal to the time of concentration

I = intensity of rainfall for a duration equal to the time of concentration in inches per hour

A = drainage area in acres

Note that the rational formula is not dimensionally correct, but 1.008 cfs = 1 ac-in/hr

Runoff Coefficient: A matrix of runoff coefficients (C) for various types of land use, hydrologic soil groups, and land slopes is shown in Attachment 5.2, Details A and B. FHWA policy is to use a consistent value for the runoff coefficient, C, over all storm recurrence intervals. The composite runoff coefficient is the weighted average C value of the various surface types.

Time of Concentration: The time of concentration t_c is defined as the flow time from the most remote point (point from which the time of flow is greatest) of the drainage area to the design point. In practice, it is considered to be composed of an overland flow time (called inlet time in urban areas) plus a channel flow time. The time of concentration for small drainage basins can be obtained from the nomograph in Attachment 5.3. The channel flow time may also be determined by dividing the longest channel by the average velocity of flow in the channel at about bank-full stage. For most basins WisDOT's preferred method to compute the time of concentration is using TR-55 methodology, which is detailed in Reference 5: "Urban Hydrology for Small Watersheds, TR-55 by NRCS. A computation tool to determine time of concentration can be found in FHWA's Hydraulic Toolbox software.

In rare instances, partial basin contributions may produce higher peak flows than full basin contributions. This usually occurs when the area near the discharge point has runoff coefficients higher than the rest of the basin.

For example, the area could be a parking lot for the small basins or a large subdivision for the large basins. The combination of higher runoff coefficients and higher rainfall intensity caused by the shorter t_c results in higher peak flows.

Rainfall Intensity: The value of rainfall intensity for various rainfall durations (times of concentration) and recurrence intervals is obtained from the intensity-duration-frequency curves in Attachment 5.4 which are derived from NOAA Atlas 14, Vol. 8: Precipitation Frequency Atlas of the United States[2]. NOAA Atlas 14, Vol.8 was released in June 2013 updating rainfall data in TP No. 25[3] and TP No. 40[4].

Drainage Area: The drainage area, A, can be determined using Geographic Information System (GIS) or civil engineering design software. Drainage area maps should be retained as design documentation.

5.3.1 Rational Method - Example Problem

Refer to Attachment 5.5. Note that the area of this example is out of the normal range of 0 to 200 acres of the Rational Method. However, this drainage basin is used in the example problems throughout this procedure. Therefore, a good comparison of the application of different runoff methods to the same drainage basin is produced.

Drainage Area = 1,067 acres

To find the time of concentration, divide length AC into two lengths of different characteristics.

1. Well-defined channel with heavy grass:
   - $Length_{(1)}$ = 2,500 feet
   - $Fall_{(1)}$ = 200 feet
2. Well-defined channel:
   - $Length_{(2)}$ = 8,300 feet
   - $Fall_{(2)}$ = 62 feet

From Attachment 5.3, read:

t_{cl} =8.5 minutes and modify to 10 minutes

t_{c2} =54 minutes

Time of Concentration = 10 + 54 = 64 minutes

3. From Attachment 5.6, the hydrologic soils group is determined to be B-C.

   Design for a 50-year recurrence interval.

   Enter the La Crosse intensity-duration-frequency curve (see Attachment 5.4, 16 of 36) at 64 minutes and 50 years and find the rainfall intensity I as 2.90 inches per hour.

Table 5.1 Composite Runoff Coefficient

Land Use	C *	Percent	Products
Woods	0.25	40	10
Mixed Cover	0.30	60	18
Thus;			
Weighted C = 28/100 = 0.28			
Q = C*I*A = (0.28)(2.90)(1067) = 866 cfs.			

* = refer to Attachment 5.2.

5.4 Urban Hydrology for Small Watersheds (TR-55)

The Natural Resources Conservation Service (NRCS) created TR-55 for estimating the volume and rate of runoff in watersheds that range in size from 1 to 2000 acres. It provides two methods for doing this, the Graphical Peak Discharge method and the Tabular Hydrograph method. Both methods are derived from TR-20 (NRCS 1983) output. For a description of the hydrograph development method used by NRCS, see chapter 16 of the National Engineering Handbook, Section 4 - Hydrology (NEH-4) (NRCS 1985) [6]. The routing method (Modified Alt -Kin) is explained in appendixes G and H of the draft Technical Release 20 (TR-20) (NRCS 1983). TR-55 software can be downloaded from the following site:

http://www.nrcs.usda.gov/wps/portal/nrcs/detailfull/national/water/?&cid=stelprdb1042925

At this time the windows version (WinTR-55) should not be used. Instead, use Version 2.1 of TR-55 (simplified flood peak and hydrograph development for small watersheds). Despite its title, TR-55 is applicable to rural as well as urban drainage basins. The Graphical Peak Discharge method is outlined below.

Basically, the runoff volumes are determined by using the following parameters:

1. Soil type (see Appendix A); TR-55, for definitions of hydrologic soil types A, B, C, and D. Also refer to Attachment 5.6.
2. Cover type.
3. Rainfall depths (24-hour duration) for selected recurrence intervals.

In addition to the above parameters, peak rates of discharge are related to:

1. Rainfall distribution type.
2. Flow length.
3. Land slope, watercourse slope, channel slope.
4. Drainage area.
5. Percent ponding and swampy areas.

All of these parameters may be converted to numerical figures by using the design figures in TR-55. Hydrologic results by this procedure are for a Type II rainfall distribution (standard NRCS design rainfall distribution applicable to Wisconsin). This method, unlike most other methods, does provide a means to include the effects of ponding and swampy areas, thus lowering the peak runoff.

Procedure: The design figures used in this procedure are located in the TR-55 publication. A list of soil names and their hydrologic classification is located in TR-55, Appendix A.

5.4.1 TR-55 - Example Problem

See Attachment 5.5. This drainage basin is in a rural area of Jackson County with no foreseeable urbanization.

- Drainage Area (A_m) = 1,067 acres = 1.67 mi^2
- Composite Runoff Curve Number (CN):

Table 5.2 Composite Runoff Coefficient

Land Use	HGS	CN *	Percent	Products
Woods (good cover)	B-C	62.5	40	2500
Mixed Cover	B-C	75.5	60	4530
(conservative treatment)			Sum =	7030
Composite CN = (7030/100) = 70				

*Refer to TR-55, Table 2-2a - 2-2d or Attachment 5.6

Design for a 50-year recurrence interval.

- From TR-55, Chapter 3, T_c = 1.43 hr
- From TR-55, Chapter 4, table 4-1, Initial abstraction I_a = .857
- From TR-55, Appendix B, page B-8, 50 year, 24 hour rainfall P = 5.3 inches
- Compute I_a/P = .16
- From TR-55, Table 2-1, Runoff Depth Q=2.29 inches (by interpolation)
- From TR-55, exhibit 4-II, q_u = 264
- No ponding or swamp areas, F_p = 1.0
- $Q_{50} = q_p = q_u A_m Q F_p$ = (264)(1.67)(2.29)(1)
- Q_{50} = 1010 cfs

For an urban drainage basin, use the same general procedure as used in the above sample problem. However, the curve numbers must reflect an urban land use.

Refer to Attachment 5.7 for an example using NRCS TR-55 Urban Hydrology for Small Watersheds version 1.11.

5.5 USGS Flood Frequency Equations for Wisconsin

The U.S. Geological Survey and WisDOT have an ongoing cooperative agreement for analyzing gaging station data to develop general flood-frequency relationships for streams with any size drainage basin. To date, the USGS has published seven reports containing methods for estimating specific flood-frequency relationships (Q_2, Q_5, Q_{10}, Q_{25}, Q_{50}, and Q_{100}).

Flood-frequency equations and comparison methods acceptable to WisDOT for the design of culverts, bridges, and flood protection structures are contained in the three reports entitled:

1. Flood-Frequency Characteristics of Wisconsin Streams, 1992 [8]
2. Techniques for Estimating Magnitude and Frequency of Floods for Wisconsin, 1981 [9]
3. Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams, 1986 [11]

Since the 1992 flood-frequency equations were developed from more years of record, they are statistically more accurate than the 1981 flood-frequency equations. This is evident by the decrease in the standard error of prediction in many of the equations. The 1981 publication is still widely used for the method of transferring discharges at gaged sites to ungaged locations using regional drainage-area exponents.

These flood-frequency equations are applicable to all drainage areas in Wisconsin, EXCEPT for highly regulated streams, some urban developments, and certain areas of the state, as noted in the reports.

The three methods show the standard error of estimate (SE) for each equation so that the user can evaluate the accuracy of the results. The standard error of estimate is defined as "a range of error such that the value estimated by the regression equation is within this range at about two out of three sites and is within twice this range at about 19 out of 20 sites" (Thomas, C.M., and Benson, M.A. 1969, "Generalization of Stream Flow Characteristics", U.S. Geological Survey, Open-File Report, 45 pp.).

The most recent version of Natural Resource Rule Chapter 116 (NR 116), effective March 1, 1986, states that the current USGS empirical equations (see reference 8) may be used in the estimate of the Regional Flood Discharges [10].

The computed discharge by the USGS empirical equations should be used for design proposes after verification by other methods and/or discharge-frequency curves of stream gaging stations of comparable drainage basins. Methodologies for comparisons are described below.

5.5.1 Flood-Frequency Characteristics of Wisconsin Streams (8)

Flood-Frequency characteristics for gaged sites on Wisconsin Streams are presented for recurrence intervals of 2 to 100 years (Q_2 to Q_{100}). This publication also presents the equations of the relations between flood-frequency and drainage-basin characteristics that were developed by multiple-regression analysis of the gage data. The most significant characteristics considered in this analysis were drainage area, stream slope, storage, forest cover, mean annual snowfall, precipitation intensity, and soil permeability. Flood-Frequency characteristics (Q_2 through Q_{100}) for ungaged sites on unregulated, rural streams can be estimated by use of these equations. This publication divides the state into five regions and lists a set of flood-frequency equations for each area. Each set of equations is correlated with three or more basin characteristics.

5.5.2 Flood-Frequency Characteristics - Example Problem

Using the same example problem data as for the previous examples, determine the Flood-Frequency characteristics for this basin.

1. From Reference Number 8, Figure 3, the basin is in Area 2. From Table 1, the required parameters for the Area 2 equations are Area (A), Soil Permeability (SP), and Main Channel Slope (S).
2. From the USGS quadrangle map in Figure 5, the area = 1067 acres = 1.67 square miles. Drainage area data for Wisconsin streams may also be obtained from Drainage Area Data from Wisconsin Streams [13].
3. From Reference No. 8, Plate 2 the Soil Permeability for this site is 1.65 inches per hour. It is recommended that Plate 2 of reference No.8 be the source for soil permeability for use in the USGS regression equations.
4. The altitude at the 10 percent point (0.20 mile) is 965 feet and the altitude at the 85 percent point (1.74 miles) is 1055 feet. The average slope (S) is:

   $$\frac{1065 - 965}{1.74 - 0.20} = 58.4 \text{ feet per mile}$$

5. Compute the 100 year (Q_{100}) recurrence interval runoff.
   - $Q_{100} = 17.7x(A)^{0.947}x(SP)\ -0.713x(S)^{0.682}$
   - $Q_{100} = 17.7 \times (1.67)^{0.947} \times (1.65)^{-0.713} \times (58.4)^{0.682}$
   - $Q_{100} = 322$ cfs
6. The peak runoffs for the 2-, 5-, 10-, 25-, and 50-year recurrence intervals are computed with the remaining Area 2 regression equations and yield the following results:
   - $Q_2 = 75$ cfs
   - $Q_5 = 134$ cfs
   - $Q_{10} = 178$ cfs
   - $Q_{25} = 236$ cfs
   - $Q_{50} = 279$ cfs
7. The discharge-frequency curve is constructed by plotting the computed discharges against their respective frequencies on log probability paper and fitting a smooth curvilinear line to those points. This discharge-frequency curve is used in picking (interpolating) a new discharge(s) for the selected design frequency(ies) (see Attachment 5.8).

5.5.3 Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams (11)

This report provides a method for estimating the frequencies and magnitudes of floods of ungaged urban streams in Wisconsin. Multiple regression techniques were used to develop flood-frequency equations by relating flood frequency and magnitude characteristics for 32 sites (gages) to basin characteristics, such as drainage area and impervious area. Two sets of equations were developed one set applicable to urban drainage areas in all parts of Wisconsin without significant regulation or diversion and another set applicable only to Milwaukee County. These equations utilize only Drainage Area (A) and Impervious Area (I) and the independent variables. Estimated flood discharges by regression equations should be compared to flood discharges determined from gaged basins with similar types of development whenever possible.

5.5.4 Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams - Example Problem

Use of the Flood-Frequency equation is illustrated by the following problem in which the magnitude of the 100-year flood (Q_{100}) for the urban gaging station 05430403, Fisher Creek Tributary at Janesville, WI, is determined. The applicable equation from Table 2 of Reference 11 is:

$$Q_{100} = 32.8(A)^{0.704} \times (I)^{0.770} \text{ (cfs)}$$

1. Determine the size or the contribution drainage area (A) in square miles from the best available topographic city maps.

   A = 1.88 square miles

2. Compute the percentage of total impervious area (I). See reference 11 pages 9 and 17 for discussion on technique that includes single-family residential, multifamily residential, commercial, industrial, and public facilities.

   I = 19.0%

3. Determine the flood discharge using the selected 100-year flood equation from Table 5.
   - $Q_{100} = 32.8\ (1.88)^{0.704} \times (19.0)^{0.770}$
   - $Q_{100} = 32.8 \times 1.56 \times 9.65$
   - $Q_{100} = 494$ cfs

5.6 Gaging Station Data

In addition to computing discharges by the aforementioned methods, a comparison should be made with steam gaging data from similar drainage basins in the locality. Records of stream flow at gaging stations, partial record stations, and miscellaneous sites are collected as part of the National Water Data System operated by the U.S. Geological Survey and cooperating state and federal agencies in Wisconsin.

Through water year 1960, these records were published in an annual series of U.S. Geological Survey water supply papers entitled "Surface-Water Supply of the United States." Beginning with the 1961 water year, stream flow data have been released in a state boundary basis by the Geological Survey in annual reports entitled "Water Resources data for Wisconsin, Water Year ____."

A search for a stream gaging station must include perusing all available USGS published reports for stream flow data, because the data for some gages are not published every year. Moreover, the data for discontinued gages will only be found in the editions published during the years the gage was operating. If this search fails, the USGS office located in Madison, Wisconsin, may be able to furnish unpublished stream flow data. In any case, they will be able to furnish a complete set of annual flood peak flows for any specific gaging station.

Annual flood peak flows through water year 1988 for most Wisconsin gaging stations, with 10 or more years of records, have been published by the USGS in Flood Frequency Characteristics of Wisconsin Streams [8] Table 6.

Additional annual flood peak flows for years after 1988 may be obtained from the annual Water Resource Data Wisconsin Water Year [14] published yearly by the U.S. Geological Survey.

5.7 Log Pewarson Type III Distribution

This technique is used to construct flood-frequency curves where systematic stream gaging records of sufficient length (at least 10 years) to warrant statistical analysis are available as the basis for the determination. A thorough description of this method is located in Bulletin #17B of the Hydrology Committee, U.S. Water Resources Council entitled "Guidelines for Determining Flood Flow Frequency," September 1981 [12].

One exception to the procedure in Bulletin #17B (update of #17 and #17A) is listed in the Wisconsin Administrative Code, Chapter NR 116.07(1)(a): "When determining skew, a log normal analysis (zero skew) shall be used instead of the generalized skew map found in Bulletin #17" [10].

USGS Published Solutions: The USGS has performed Log Pearson Type III flood-frequency analyses (Bulletin #17A Procedures) at most Wisconsin gaging stations having 10 or more years of record (through 1978 water year) to determine flood-frequency characteristics. Estimates of the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence interval floods for each of these stations has been published by the USGS in Flood Frequency Characteristics for Wisconsin Steams [8], Table 4.

As more years of data are collected, these published flood-frequency characteristics will become obsolete, and additional gaging stations will meet the 10 or more years of record criterion. Therefore, the published flood-frequency characteristics should only be used for preliminary design. For final design, the designer should collect the additional years of peak data and determine new flood-frequency characteristics with a Log Pearson Type III analysis.

5.8 Transferring Gaged Discharges

In most design problems, there is no gage station located at the project site. The nearest comparison gage or study may be located some distance upstream or downstream. A reasonable comparison gage or study may even be outside of the project basin. The design discharge developed for a gaging station or study site may be transferred to the design site by an equation that relates the discharges and drainage areas of two distinct drainage areas with similar drainage basin characteristics. There are a number of methodologies for transferring gaged and other accepted studied discharges to a project site. The three methods presented here will be the 1992 USGS Adjustment Method, the 1981 USGS transfer method, and Comparison of Similar Drainage Basins at Gaged Sites.

1992 USGS Adjustment Method: This method uses the combination of data for the nearest similar gaging station and data determined by use of the USGS multiple-regression equations [8]. The procedure is applicable for sites that have a drainage area within 50 percent of the drainage area of the gaging station. This procedure was used by Curtis (1987) for streams in Illinois. The procedure is defined on pages 13 and 14 of reference (8) and as follows:

First the regression equation correction or adjustment ratio r is defined by:

$$r = Q_a / Q_r$$

Where:

- Q_a is the accepted (log Pearson III) flood-frequency characteristic at the gaging station.
- Q_r is the flood-frequency characteristic determined for the gaged station by use of the multiple regression equation.

The adjustment for difference in drainage area is determined by r' such that:

$$r' = r - [(A / (0.5 \times A_g)) \times (r - 1.0)]$$

Where:

r = defined above

A = is absolute value of the difference in the drainage area between the ungaged site and the gaged site.

A_g = is the drainage area of the gaged site.

The adjusted flood-frequency characteristic for the project site Q_w is computed by the equation:

$$Q_w = Q_{rug} * r'$$

Where Q_{rug} is the flood-frequency characteristic determined for the ungaged site by the multiple regression equation.

1981 USGS Transfer Method: This method accounts for difference in the drainage area between the gaged site and ungaged upstream or downstream project site. Basically this technique computes a weighted design discharge at the up- or downstream site by weighting the transferred discharge with the flood-frequency (multiple-regression equation) discharge. As the project drainage area approaches that of the gage drainage area the weighted transferred flow at the project site approaches that of the gage. Also, as the difference in drainage area between the gage and the project approaches 50% of the area of the gage, the transferred weighted flow at the project approaches the flow value determined by the regression equation. A thorough discussion of this method is contained in reference (9), pages 11-14.

The transferred discharge, Q_{ud}, is determined by the following formula:

$$Q_{ud} = Q_g \times (A_{ud} / A_g)^n$$

Where:

Q_{ud} = is the discharge at the project site transferred from the gage site by drainage-area ratio.

Q_g = Discharge at the gage site for selected recurrence interval.

N = 1981 USGS regional drainage-area exponent (reference 9, page 12).

- area 1 = 0.59
- area 2 = 0.68
- area 3 = 0.76

- area 4 = 0.60
- area 5 = 0.63

A_{ud} is the drainage area at the project site.

A_g is the drainage area at the gage station.

Then weight this discharge (Q_{ud}) with the discharge (Q_r) determined at the project site by the regression equation with the following equation:

$Q_w = Q_{rud} \times (2A/A_g)$

$= Q_{ud} \times (1-(2A/A_g))$

Where:

Q_{rud} = discharge at project site determined by the regression equation.

Q_w = the weighted discharge for the project site.

A = is the absolute value of the difference between the drainage area at the project site and the gage station.

5.8.1 Transferring Gaged Discharges - Example Problems

This example problem will illustrate both the USGS 1992 and the USGS 1981 Transfer methods.

Problem:

Determine the best estimate of the Q_{100} design discharge for Rowen Creek at Main Street in the Village of Poynette, Columbia County.

Given:

- Drainage Area (A) = 10.6 square miles
- Main-Channel Slope (S) = 30.4 feet per mile
- Storage (ST) = 0.3% + 1.0% = 1.3%
- Precipitation Intensity Index (I24-2) = 2.75
- Intens = (I24-2) - 2.3 = 0.45"
- Soil Permeability (SP) = 1.42 inches per hour.

The Q_{100} flow for the Main Street site by regression equation is determined by Equation #30, Table 1 of Reference (8) as follows:

$Q_{rud}(100) = 64.8\ (A)^{0.863} \times (S)^{0.460} \times (ST)^{-0.299} \times (SP)^{-0.302} \times (Intens)^{0.808}$

$Q_{rud}(100) = 64.8(10.6)^{0.863} \times (30.4)^{0.460} \times (1.3)^{-0.299} \times (1.42)^{-0.302} \times (0.45)^{0.808}$

$Q_{rud}(100) = \underline{1043\ cfs}$

Gage Station 5405600 Rowen Creek at Poynette Wis.:

- Drainage Area (A_g) = 10.4 square miles
- Storage (ST) = 0.0% + 1.0% = 1.0%
- Precipitation Intensity Index (I_{24}-2) = 2.75"
- Intens = (I_{24} -2) - 2.3 = 0.45"
- Soil permeability (SP) = 1.42 inches per hour
- Log Pearson Q_{100} at Gage = 2180 cfs (ref. 10)
- 1992 100-year Regression Equation Q_{100} = 1030 cfs (ref. 8)

1992 USGS Transfer Method:

First, $r = Q_g / Q_r$

$Q_g = \underline{2180\ cfs}$

$Q_r = \underline{1030\ cfs}$

$r = 2180/1030 = \underline{2.12}$

Next, $r' = r - (A /(0.5 \times A_g)) \times (r - 1.0)$

$A = |\ 10.6 - 10.4| = \underline{0.2}$

$r' = 2.12 - (0.2 / (0.5 \times 10.4)) \times (2.12 - 1.0)$

$r' = 2.08$

The transferred flow $Q_w = Q_{ud} \times r'$ (Q_{ud} = 1043 from regression equation at project site)

Transferred Flow = $Q_w = 1043 \times 2.08 =$ 2169 cfs

1981 USGS Transfer Method.

First: $Q_{ud} = Q_g \times (A_{ud} / A_g)^{n}$

(n = 0.63 Area 5)

$Q_{ud} = 2180 \times (10.6 / 10.4)^{0.63}$

Q_{ud} = 2206 cfs

Next: $Q_w = Q_{rud} \times (2 \times A / A_g) + Q_{ud} \times (1 - 2 \times A / A_g)$

$Q_w = 1043 \times (2 \times 0.2/10.4) + 2206 \times (1 - 2 \times 0.2/10.4)$

Transferred Flow Q_w = 2161 cfs

5.9 Comparison of Similar Drainage Basin at Gaged Sites

This method can be used as a check of the regression equations when there are no gaging stations up- or downstream of the project site. This method uses the same drainage area discharge transfer equation as the 1981 USGS Transfer Method to calculate Q_{ud}. However, the calculated transferred flow Q_{ud} may then be further adjusted to account for dissimilar basin parameters between the comparison gage and the project site. The other dissimilar basin parameters are then adjusted in the same manner as the drainage area with the parameters of the project site prorated to the gage site and raised to the appropriate exponent. This factor is then multiplied by Q_{ud}. As many basin parameters can be adjusted as needed, however, the best comparison gages tend to be in the same region with similar basin parameters. Therefore, good comparison gages tend to need few basin parameters adjusted.

Each dissimilar basin parameter that is to be adjusted is prorated to the related gage parameter, then this ratio is raised to the 1992 USGS Regression equation exponent for the subject parameter. The basin parameter exponent should correspond to the regression equation used to estimate the discharged at the project site.

The Transfer Equation takes the form:

$$Q_w = Q_{ud} \times (S_s / S_g)^{Ns} \times (ST_s/ST_g)^{Nst} \times (SP_s / SP_g)^{Nsp}$$

Where:

- Q_w = the Transferred Flow
- Q_{ud} = defined in 1981 USGS Transfer Method ref. (9)
- S_s, ST_s, SP_s, ... etc. = basin parameters at the project site.
- S_g, ST_g, SP_g, ... etc. = basin parameters at the comparison gage site.
- N_s, N_{st}, N_{sp}, ... etc. = basin parameter exponents from 1992 regression equation used to estimate Q_{rug}.

5.9.1 Comparision of Similar Drainage Basin at Gaged Sites - Example Problems

This example problem will illustrate the use of the Comparison of Similar Drainage Basins at gages Method.

Problem: Compare or "Transfer" a 100-year flow for a similar gaged basin to McAdam Branch at Morgan Road in Grant County.

Given: McAdam Branch Drain Area (A) = (A_{ud}) = 6.63 sq. mi.

Precipitation Intensity Index (I24-2) = 3.03"

Intens = (I24-2) - 2.3 = 0.73"

Main Channel Slope (S) = 58.0 feet per mile.

From Table 1 Equation 6 of reference (8)

$Q_{rud}(100) = 342 \times (A)^{0.848} \times (Intens)^{4.06} \times (S)^{0.512}$

$Q_{rud}(100) = 342 \times (6.63)^{0.848} \times (0.73)^{4.06} \times (58.0)^{0.512}$

$Q_{rud}(100)$ = 3790 cfs

Gage 05413400, Pigeon Creek near Lancaster Wis.

Drainage Area (A_g) = 6.93 sq. mi.

Precipitation Intensity Index (I24-2) = 3.02"

Intens = (I_{24}-2) - 2.3 = 0.72"

Main Channel Slope (S) = 49.8 feet per mile

$Q_g(100)$ = 3620 cfs (102.5 m^3/s) (Table 4 ref. (8))

$Q_r(100)$ = 3440 cfs (97.4 m^3/s) (Table 5 ref. (8))

First: Basin parameter exponents for transfer method:

Exponent for drainage Area (A) = n = 0.59 (9)

Exponent for Intens (Intens)= N_i = 4.06 (8)

Exponent for Slope (S) = N_s = 0.512 (8)

$Q_{ud} = Q_g \times (A_{ud} / A_g)^{n}$

$Q_{ud} = 3620 \times (6.63/6.93)^{0.59}$

Q_{ud} = 3526 cfs (99.8 m^3/s)

Next: the Transferred Flow Q_w is found by further adjustment of basin parameters,

$Q_w = Q_{ud} \times (INTENS_s / INTENS_g)^{Ni} \times (S_s / S_g)^{Ns}$

$Q_w = 3526 \times (0.73 / 0.72)^{4.06} \times (58.0 / 49.8)^{0.512}$

Q_w = 4031 cfs (114.1 m^3/s)

This transferred flow may indicate that the regression equations are under estimating flows for basins with these characteristics. This also may be evident from a comparison of the regression results (Q_r) and Log Pearson results (Q_g) at the gage.

5.10 Published Watershed Studies

Pertinent hydrologic and hydraulic information for a specific watershed may be obtained from these studies, thus saving many hours of tedious work.

In years past, watershed studies have been prepared and published by many communities because of local flooding problems. Many additional studies have been prompted by the Department of Housing and Urban Development's (HUD) Flood Insurance Program, which was established by the Congress in the National Flood Insurance Act of 1968 and expanded in the Flood Disaster Protection Act of 1973. These studies are now published by the Federal Emergency Management Agency (FEMA).

These watershed studies have been prepared and published by the following agencies:

1. Regional Planning Agencies.
2. U. S. Army Corps of Engineers.
3. U. S. Natural Resources Conservation Service.
4. U. S. Geological Survey.
5. Consulting engineering companies.

A list of these studies, entitled "Floodplain Management Community Status Report," may be obtained from the Wisconsin Department of Natural Resources. This report lists the rivers by county and community with the following information:

1. DNR district.
2. Ordinance dates (adopted and approved).
3. Insurance information (date, type of map).
4. Report publication date.
5. Class of study (flood insurance study, floodplain management, etc.).
6. Source of information (DNR, HUD, NRCS, etc.).
7. Type of district (general, floodplain, etc.).

5.11 Field Review Notes, Interviews, and Historical Data

Field review notes of stream channels and existing structures can indicate high-water elevations that have occurred in the past.

Field interviews of local residents can be very important in determining past flow rates. The high-water elevations pointed out by local residents can be used to compute a flow rate. This can be done by determining a cross-sectional area of the water and an average velocity with Manning's formula and multiplying the two together.

By making a hydraulic analysis of an existing structure with those field-determined headwater depths and tail-water depths, a past flow rate can be determined.

Historic flood information of extreme high-water elevations can often be used to make estimates of peak discharges. The USGS includes some historic flood information in its published reports and computer files. Additional information can sometimes be obtained from the files of other agencies or extracted from newspaper files. If such records are located, a search of the National Weather Service records should be made to determine the corresponding rainfall intensity in the immediate drainage area.

There is one flaw in the above-mentioned flow rates, namely, the lack of knowledge of corresponding recurrence intervals. Therefore, the determined flow rates can only be used as a comparison to confirm and justify the finalized design flow rate. Moreover, it can also be used to show that the design flow rate determined by various mathematical methods is erroneous.

5.12 References

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3. U.S. Department of Commerce, Weather Bureau, "Rainfall Intensity-Duration-Frequency Curves for Selected Stations in the United States and Puerto Rico, Technical Paper No. 25, 1955."

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7. U.S. Department of Transportation, Federal Highway Administration, "Hydrology," Hydraulic Engineering Circular No. 19, McLean, Virginia, 1984, 343 pp.

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10. Wisconsin Administrative Code, Department of Natural Resources, "Wisconsin's Floodplain Management Program," Chapter NR 116, March 1, 1986.

11. Conger, Duane H., "Estimating Magnitude and Frequency of Floods for Wisconsin Urban Streams," U.S. Department of the Interior, Geological Survey Water Resource Investigation Report 86-4005, Madison Wis. 1986.

12. United States Water Resources Council, Hydrology Committee, "Guidelines for Determining Flood Flow Frequency," Bulletin #17, March 1976, Bulletin #17A, June 1977, Bulletin #17B, September 1981.

13.* Water Resource Data Wisconsin Water Year, U.S. Department of Interior, Geological Survey Water Resource Division, open file report WI-89-1 to current year 1994 WI-94-1.

* *Henrich, E.W., Daniel, D.N., Drainage Area Data for Wisconsin Streams, U.S. Department of Interior, Geological Survey Water Resource Division, Open-file Report 83 - 933.*

NOTE: Documents prepared by the USGS - Water Resources Division can be obtained by calling (608) 828-9901.

LIST OF ATTACHMENTS

Attachment 5.1	Area Limits for Peak Discharge Methods
Attachment 5.2	Runoff Coefficients (C), Rational Formula; and Runoff Coefficients for Specific Land Uses

FDM 13-10-10 Hydrograph Development and Routing

August 8, 1997

10.1 Development

The first step in designing a hydraulic structure is to determine the amount of water to be carried also called the design discharge. The problem is particularly difficult for small watersheds, say, under five square miles, because the smaller the area, the more sensitive the design discharge is to conditions that affect runoff and the less likely there are runoff records for the area.

A hydrograph is defined as the graph of flow (rate versus time) at a stream section. The four basic hydrograph types are:

1. Natural Hydrographs: Obtained directly from the flow records of a gaged stream.
2. Synthetic Hydrographs: Obtained by using watershed parameters and storm characteristics to simulate a natural hydrograph.
3. Unit Hydrographs: A natural or synthetic hydrograph for one inch of direct runoff. The runoff occurs uniformly over the watershed in a specified time.
4. Dimensionless Hydrographs: Made to represent many unit hydrographs by using the time to peak and the peak rates as basic units and plotting the hydrographs in ratios of these units. Also called the "Index Hydrograph."

Hydrographs are used in the planning and design of water control structures, especially detention basins, which are used to minimize downstream flooding by attenuating the peak flows of storms with specific duration frequencies. They are also used to show the hydrologic effects of existing or proposed projects.

The urbanization of rural areas increases peak flows, which has and will continue to overtax existing downstream structures such as highway drainage facilities. Replacing such overtaxed facilities with larger or additional structures is one option, but designers should also investigate adding a detention basin(s) upstream of the problem structure.

For both large and small watersheds, the hydrograph development methods discussed in this section are:

1. HEC-1
2. The Natural Resources Conservation Service (NRCS) Tabular Method, TR-55
3. The Unit Hydrograph Method
4. The NRCS Triangular Dimensionless Unit Hydrograph Method
5. The NRCS Curvilinear Dimensionless Unit Hydrograph Method

These methods can be easily applied through manual computations to small watersheds, but not large watersheds, hence, it is necessary to use a computer program in these cases. The computer program selected for inclusion here is the NRCS TR-55, "Urban Hydrology for Small Watersheds" which makes use of the NRCS curvilinear unit hydrograph.

10.1.1 HEC-1

HEC-1 was developed by the U.S. Army Corps of Engineers, Hydraulic Engineering Center. It is designed to simulate surface runoff from various duration storms over a watershed. The conversion of precipitation to direct runoff can be simulated by HEC-1 for both small and large watersheds. Hydrograph combining, channel and reservoir routing and sub-basin runoff are some of the basic components that HEC-1 uses for a simple or complex watershed study.

The HEC-1 computer package has the following capabilities:

1. Simulates watershed runoff and stream flow from design or historical rainfall.
2. Uses unit hydrograph, loss rate and stream flow routing procedures from measured data.

3. Simulates reservoir and channelization flood controls.

10.1.2 NRCS Tabular Method, TR-55

The Tabular Method is an approximation of the more detailed hydrograph analysis contained in Section 4-Hydrology of the NEH-4 (4). Composite hydrographs can be developed for any point within a watershed by dividing the watershed into subareas, developing simple hydrographs for each subarea, routing the simple hydrographs to the point in question, and adding the routed simple hydrographs. The factors required to determine these hydrographs are:

- 24-hour rainfall amount,
- a given rainfall distribution (Type II in Wisconsin),
- hydrologic soil cover complexes (runoff numbers),
- time of concentration,
- travel time, and
- drainage area.

This method should not be used when the runoff curve numbers of the subareas vary appreciably and when runoff volumes are less than 1.5 inches for curve numbers less than 60. Moreover, for most watershed conditions (urban or rural), this procedure can be used to determine hydrographs for subareas up to approximately 2000 acres.

For a thorough discussion of the Tabular Method, with an accompanying example problem, see routing section.

10.1.3 Unit Hydrograph

The unit hydrograph is a very important tool for estimating runoff amounts for various frequencies that may occur at a specific point of a stream. The use of this method requires continuous records of runoff and precipitation for the specific drainage basin.

Sherman(6) defined the unit hydrograph as a hydrograph with a one-inch volume of runoff from a rainstorm of specified duration, time-intensity pattern, and areal pattern. Increasing the duration of the rainfall increases the unit hydrograph time base and peak, because the unit hydrograph contains only one inch of runoff.

In practice, unit hydrographs are generally based on an assumption of uniform intensity of rainfall. Usually the Unit Hydrograph Method is applied to basins small enough so that the areal pattern is rather uniform. The acceptable drainage basin size is equal to or less than 200 square miles.

Theoretically, a given drainage basin will exhibit an infinite number of unit hydrographs, one for every possible duration of rainfall, every possible time-intensity pattern, and every possible areal pattern. In design practice, only the duration of the rainfall is allowed to vary, while variations in areal patterns are ignored. Moreover, unit hydrographs are developed from rainstorms that exhibit basically a rainfall pattern of uniform intensity. Short-duration unit hydrographs can be used to develop a unit hydrograph resulting from a long rain of varying intensity.

10.2 Procedure

The basic steps in the development of a unit hydrograph are:

1. Analyze the stream-flow hydrograph separating the surface runoff from the groundwater flow.
2. Determine the total volume of direct runoff from the storm that produced the original hydrograph. This volume is equal to the area under the original hydrograph minus the groundwater flow area.
3. Divide each ordinate of the direct runoff hydrograph by the total direct runoff volume in inches. The unit hydrograph is the plot of these answers against time.
4. Finally, determine the effective duration of the rainfall that produced this unit hydrograph. This can be obtained by studying the hyetograph of the rainfall.

Generally, the hydrograph for a given drainage basin for a specified design storm (duration, effective rainfall, or total runoff) may be constructed by multiplying each ordinate of the specified duration unit hydrograph by the total runoff (inches).

10.3 NRCS Triangular and Curvilinear Dimensionless Unit Hydrograph Methods

Basically, the Triangular and Curvilinear Methods are the same, except the Triangular Method, as its name implies, substitutes a dimensionless unit hydrograph for the more accurate curvilinear dimensionless unit hydrograph. This method develops synthetic hydrographs for a specific watershed by using watershed parameters, storm characteristics, and a dimensionless unit hydrograph. The dimensionless unit hydrograph was developed from a large number of natural unit hydrographs from watersheds varying widely in size and geographical location.

The shape of the dimensionless unit hydrograph is determined by the drainage area and time of concentration, hence, the watershed should be divided into hydrologic units of uniformly shaped areas. If possible, these subareas should be less than 20 square miles and exhibit a homogeneous drainage pattern.

The basic data required to develop synthetic hydrographs are:

1. Twenty-four-hour and/or six-hour rainfall amount for a specific rainfall frequency.
2. Rainfall distribution.
3. Hydrologic soil cover complexes (runoff numbers).
4. Times of concentration for the subareas.
5. Travel times through reaches.
6. Drainage areas for each sub-area.

For a thorough discussion of this method, with accompanying example problems, see Chapter 16 of NEH-4 (4). In addition, these synthetic hydrographs can also be generated by computer through the use of version 2.1 of NRCS-TR-55 (5).

10.4 Routing

Hydrograph development and hydrograph routing are closely interrelated. A simple hydrograph for a subarea of a watershed can and is developed without routing, but the downstream, more complex hydrographs must be developed through routing and/or combining the simple upstream hydrographs.

In the American Society of Civil Engineers Manual, "Nomenclature for Hydraulics," flood routing is variously defined as follows:

routing, (hydraulics) (1): The derivation of an outflow hydrograph of a stream from known values of upstream inflow. The procedure utilizes wave velocity and the storage equation; sometimes both (2). Computing the flood at a downstream point from the flood inflow at an upstream point, and taking channel storage into account.

routing, flood: The process of determining progressively the timing and shape of a flood wave at successive points along a river.

routing, stream flow: The procedure used to derive a downstream hydrograph from an upstream hydrograph, or tributary hydrographs, and from considerations of local inflow by solving the storage equation.

The purpose of flood routing is to mathematically determine from the inflow hydrograph the shape of the outflow hydrograph at specific locations in streams or structures during passages of floods. These outflow hydrographs are used in designing a water control structure or project.

Detention and retention basins have been used to control the effects and results of urbanization and urban runoff hydrology.

Urbanization Can Cause:

1. Reduction in natural storage capacity.
2. Increase in impervious area.
3. Greater direction and conveyance of runoff.

Urban Runoff Hydrology Results In:

1. Higher peak discharge (2 to 5 times).
2. Shorter time to peak, as high as 50 percent.
3. Higher velocity of storm runoff.
4. As much as 50 percent increased volume of storm runoff.
5. Reduction of infiltration, inflow and base stream flows.

To help alleviate these problems it may be necessary to design a retention/ detention facility. This facility may be designed as a pond, underground tank or parking lot as well as other types of facilities.

The following steps should be performed to assure a proper design.

1. Determine the purposes for which the basin will be used.
2. Determine the design storm inflow hydrographs before and after development.
3. Estimate the volume of storage needed.
4. Determine the depth-storage curve for the basin.
5. Select the outlet structure types compatible with the uses outlined in step 1 and determine the depth-outflow curve.
6. Determine the routing curve.
7. Perform the routing.
8. Add additional outlet features to ensure that the peak outflow rate is reduced to at least the pre-development rate for the more frequent storms.
9. Perform the routings for these smaller storms to ensure compliance.
10. Check the length of time needed to empty the basin for the various storms to determine if the other uses of the basin will be unduly delayed and/or if water quality detention times are met.

For the example shown in this procedure, a detention pond (122' x 122') will be designed.

Note: The NRCS publication (reference 5) is needed to fully understand the following example.

10.5 Detention Pond Example

(NRCS TR-55 Tabular Hydrograph Method)

Given:

1. Area of Watershed = 10 Acres
2. Curve Number = 75 *
3. 50 Year 24 hour Rainfall = 5"
4. Time of Concentration (t_c)=18 minutes
5. Type II Rainfall Distribution
6. Maximum Post Q £ Pre Q of 8 cfs.

Note: Items 2 - 5 can be determined by using Chapters 2 and 3 of NRCS TR-55(5) and associated exhibits and figures.

Procedure:

The procedure shown below is based on the steps described above.

1. The basin is to be used as a detention pond.
2. Determine storm inflow hydrograph:
   A. Determine runoff from Table 2-1, NRCS TR-55 (5).

      Rainfall = 5"

      CN = 75

      Runoff = 2.45"

   B. Complete work sheet (Attachment 10.1 B)
   C. Complete work sheet (Attachment 10.2 B) using a Type II rainfall distribution to develop a hydrograph. See reference 5. The NRCS TR-55 computer program (version 2.1 non-Windows) may be used instead of manually calculating the results of steps A - C.
   D. Plot the tabulated hydrograph. See Attachment 10.3.
3. Determine volume of storage required to detain a 50 year storm with Q = 8 cfs.

   Required storage can be determined by assuming an outflow curve (see reference 5 and Attachment 10.3 for details) and determining the area between the inflow curve and the outflow curve. For this example, using a planimeter on the area between the curves in Attachment 10.3 yields a required volume of approximately 54,000 ft^3.

4. Depth Storage Relationship:

   We will first evaluate a trapezoidal storage pond with the following dimensions. (see Attachment 10.11)

   Square, L = W = 122 ft. (bottom of pond)

   Side slope (Z) = 4:1

The equation below can be used to find the volume of a trapezoidal pond. Use it to determine the depth needed to provide adequate storage for the detention pond.

Volume = LWD + (L + W) ZD^2 + 4/3 Z^2D^3	
Depth	**Volume**
0.5	7688
1.0	15881
1.5	24594
2.0	333842
2.5	43643
3.0	54011
3.5	64963

From this table a depth of 3.5 ft is chosen to provide ample storage plus some freeboard. See Attachment 10.8 for a plot of this data. This is the depth-storage relationship.

5. Determine outlet pipe size by using Attachment 10.4, with concrete pipe/grooved end with head wall, determine pipe size that can handle 8 cfs w/3.5 ft of head.

   For Attachment 10.4, use a 12" concrete pipe with a grooved end with head wall.

6. The depth/outflow relationship can be determined by multiple applications of Attachment 10.4 with a constant pipe diameter (D) of 1 ft. See the table below.

Depth (ft)	**HW/D (ft)**	**Outflow (cfs)**
.5	.5	.75
1.0	1.0	2.5
1.5	1.5	4.0
2.0	2.0	5.4
2.5	2.5	6.3
3.0	3.0	7.2
3.5	3.5	8.0

   Plot the information as shown on Attachment 10.5.

7. Construct a storage indicator table, Attachment 10.6 B, plot column 2 vs column 6 to create a storage indicator curve as shown on Attachment 10.7. The curve is used to complete Attachment 10.9 B. When the storage indicator number (column 6 of Attachment 10.9 B) reaches a maximum, then peak discharge occurs.

Attachment 10.10 shows the actual inflow and outflow hydrographs. The peak outflow is 5.49 cfs. Since the maximum Q post = 8 cfs, this solution is acceptable. Therefore, steps 8-10 of the process need not be done. If a design with a Q post closer to 8 cfs is desired then the problem should be re-examined.

10.6 References

1. Poertner, Herbert G., "Practices in Detention of Urban Storm Water Runoff," American Public Works Association, Special Report No. 43, 1974.

2. Terstriep, Michael L., and Stall, John B., "Urban Runoff by Road Research Laboratory Method," Journal of the Hydraulics Division-ASCE, November 1969, pp. 1809-1834.

3. Stall, J.B., and Terstriep, M.L., "Storm Sewer Design--An Evaluation of the RRL Method," prepared for the Office of Research and Monitoring of the USEPA, October 1972.

4. U.S. Department of Agriculture, National Resources Conservation Service, National Engineering Handbook, Section 4, Hydrology (NEH-4), August 1972.

5. National Resources Conservation Service, Engineering Division, "Urban Hydrology for Small Watersheds," Technical Release 55, June 1986.

6. Sherman, L.K., "Stream flow from Rainfall by the Unit-Graph Method," Eng. News-Rec., Volume 108, pp. 501-505, 1932.

LIST OF ATTACHMENTS