A GEOMORPHIC EVALUATION WITH CALIBRATED HYDRAULIC AND HYDROLOGIC MODELING OF THE HOP BROOK WATERSHED IN MASSACHUSETTS

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INTRODUCTION

The Hop Brook Watershed in Franklin County, Massachusetts was selected by NRCS for survey and analysis as part of a regional geomorphic and hydraulic geometry curve study of New England streams. The study area of 3.39 square miles is located within the Worchester Uplands of the New England Physiographic Province. The Worchester Uplands are characterized by rolling hills and rounded mountains, interrupted by numerous generally narrow valleys. The geology is igneous and meta-sedimentary Paleozoic and Pre-Cambrian crystalline rocks covered by glacial till. Hop Brook drains into the Quabbin Reservoir, a quality drinking water supply for central Massachusetts. Land management practices have limited development and logging to protect the watershed and runoff. The USGS established gage 01174000 on Hop Brook near New Salem in October of 1947 and operated a continuous recording gage until September 1982.

We are not often afforded the luxury of modeling watersheds that are already gaged. Initially, it would seem a waste of human resources to develop a hydrological model and calibrate it, when a flood frequency relationship was established by measurement. The Natural Resources Conservation Service (NRCS) has recently updated two (DOS version) hydrologic computer programs to window versions - WinTR-55 and WinTR-20. These computer programs are nearly always used on ungaged watersheds. Testing WinTR-55 on a gaged watershed would provide an opportunity to see how well the computation engine performs based on six input parameters of drainage area, time of concentration, curve number, rainfall amount, rainfall distribution, and dimensionless unit hydrograph. Comparing WinTR-55 results to measured flows from several storm events over several years would indicate the level of effort required to calibrate the model and the exercise would give indications of hydrologic parameter sensitivity; would the input parameters have to be changed on a storm by storm basis? Is there some natural variability of the input parameters over time? After calibrating the hydrologic model, would the model provide insight into NRCS design rainfall distributions?

CALIBRATION

There are three calibrations involved with this case study – a hydraulic calibration of the HEC-RAS model, a geomorphic calibration of the hydraulic geometry characteristics, and a hydrological calibration of the WinTR-55 model. The hydrologic calibration relies on the geomorphic calibration, which in turn relies on the hydraulic calibration. Each will be discussed, with major emphasis on the hydrologic calibration and results.

Hydraulic Calibration: Hydraulic calibration relies on good definition of the stream geometry that defines the model. Cross sections should be representative of the reach, extend across the floodplain, and be perpendicular to flow. Cross sections should also describe abrupt changes in conveyance; changes in width, depth, slope, and roughness. Good definition of the channel thalweg and water surface throughout the modeled reach facilitates the calibration process. Make note of stream discharge on the days surveying. If working on an ungaged stream, a discharge measurement at the beginning and at the end of the day will facilitate calibration. Once cross section data, reach lengths, and roughness coefficients are entered into HEC-RAS, first runs should try and match a computed water surface on the day(s) of survey with the water surface surveyed. This step usually entails adding in "interpolated" cross sections along the profile, where the bed profile changes, especially in pooled areas. What seems to work well is to take an existing surveyed cross section (either upstream or downstream) from the location of the interpolated cross section and translating the floodplain elevations up or down by the valley slope multiplied by the distance translated. Channel bottom elevations are adjusted up or down depending on the difference between the existing cross section thalweg elevation and the thalweg elevation on the profile for the interpolated location. All channel bottom elevations below bankfull are adjusted by this thalweg difference. Usually a good match between computed and measured water surface profiles can be made by inserting enough interpolated cross sections without altering Manning's n. The second phase of hydraulic calibration is to modify Manning's roughness coefficients starting at the downstream cross section (for sub-critical flows) and working up to the surveyed cross section that represents the USGS rating curve. Try and match the computed rating curve to the USGS rating curve working from low discharges up to higher discharges. However the roughness values are changed in order to create the match, a similar relative adjustment of Manning's n should be made to the cross sections upstream from the rating cross section.

Geomorphic Calibration: During the profile survey, there are usually three rodmen; one shooting channel bed features and one on each bank shooting left and right bankfull indicators respectively, this way there are up to three independent opinions on bankfull stage along the profile. Secondly, the reach is modeled in HEC-RAS. The hydraulic model is "calibrated" when two conditions are met; first HEC-RAS must produce a water surface profile that matches the water surface on the day(s) of survey and HEC-RAS must produce a rating curve for a cross section representing the USGS gage that matches the USGS rating curve. After calibration, the water discharge that produces a water surface profile that matches a best fit of bankfull indicators all along the reach is chosen as the bankfull discharge. The water surface elevations at bankfull discharge are examined more closely in each of the surveyed cross sections (usually in middle of riffles or at the downstream end of pools in the glides). The bankfull cross section dimensions are averaged for three or more cross sections. This gives an average value of hydraulic geometry dimensions more representative of the reach. Thirdly, mean shear stress is calculated at bankfull stage (using hydraulic radius and energy gradient at bankfull discharge), when dealing with gravel sizes, multiplying the shear stress calculation by a factor of two gives a likely particle size moved in inches, this size is converted to millimeters and this particle size is compared to the Wolman Pebble count. Usually, the particle size moved based on calculated shear stress and the D_{50} are within $^{+/-}$5 mm. Fourth and finally, the reach averaged hydraulic geometry at bankfull discharge are compared to the hydraulic geometry of stratified regime curves for the same stream classification. Eighty cubic feet per second produced a water surface that matched many of the bankfull indicators. Average channel topwidth is 20.4 feet and mean depth is 1.18 feet, mean channel velocity is 3.3 feet per second, the stream classified as a C4 in Rosgen's Classification System.

Hydrologic Calibration: The strategy to calibrate the WinTR-55 hydrologic model was to "nail down" as many of the input variables as possible, starting with the most stable of the parameters (with respect to time). For instance drainage area is one input parameter that should not change over time, can be planimetered from a USGS topographic map and verified through the USGS gaging records. However road construction and culverts can alter flow paths and change drainage area size so even drainage area is not a guaranteed stable time invariant parameter.

Time of Concentration (Tc): Tc is a relatively stable hydrologic parameter, but it can vary due to changes in the flow path, development of roads, or debris in the conveyance system. The velocity method outlined in TR-55 was used to calculate Tc. The longest flow path from basin divide to the gage was delineated on a 7.5 minute USGS topographic map. The flow path was segmented into sheet flow, shallow concentrated flow, and channel flow. Channel flow was segmented into four reaches based on slope breaks. The hydraulics of Hop Brook were already modeled and calibrated during the geomorphic analysis, so estimates of channel velocities near the gage were taken from the HEC-RAS model at bankfull discharge. Channel velocities for upper reaches with smaller drainage areas were estimated from the New England Physiographic Regional Curves. Travel times are computed by dividing the lengths of the flow paths by the average velocity within the flow path, travel times are then summed and the time was converted to hours. Time of Concentration for the Hop Brook basin was computed to be 1.56 hours; the longest flow path from basin divide to gage is 16,100 feet, the average velocity throughout the basin is 2.5 feet per second, channel velocities of reaches varied from 2.9 to 4.1 feet per second.

Curve Number (CN): CN is probably the second most unstable hydrologic parameter with respect to time. Curve Number changes between growing and dormant seasons. CN can change due to antecedent rainfall, land use/management, land development or fire. The Hop Brook was a continuous recording gage, so mean daily discharge as well as peak discharge records are available from the USGS. Eight historical storms were modeled; six storm events occurred during the growing season and two storm events occurred during the winter. Each storm produced the peak annual discharge for their respective water year. Curve Number was calculated from gaged data – precipitation records and runoff measurements. For the six storms that occurred during the growing season, the CN varied between 71 and 72, the antecedent runoff condition was in either a II or III condition. Calculating CN from hydrologic soils groups, vegetation and land use would have resulted in a CN of 70. For the two storms that occurred in the winter, it is assumed that the ground was frozen (January and February). Curve Number changed drastically to 93 and 89 respectively. Peaks were matched for the two winter storms, but runoff volumes are a guess because it is not known how much snow or equivalent water was on the ground prior to the precipitation events.

Rainfall: twenty-four hour rainfall totals for the eight storms were taken from the New Salem Precipitation Station (195306). New Salem is approximately 1.5 miles from the USGS gage and is located just outside of the watershed boundary. With daily recording stations, possible discrepancies in time series data may exist because any precipitation that falls after the observation time (in this case 8 am) is recorded under the next days rainfall total.

Rainfall Distribution: Historic storm distributions are probably the most unstable hydrologic parameter with respect to time. Storm distributions vary temporally and spatially, and need not repeat themselves. Good calibration requires using representative historical rainfall distributions. Erroneous rainfall distributions may affect CN and Tc calibration. Historical rainfall distributions were re-created from “nearby” hourly recording precipitation stations. NOAA’s U.S. Hourly Precipitation Dataset (TD3240) CD and website were used to find stations with available data on the days of storm events. When one procedure doesn’t always provide sufficient answers, we try another and then call the technique “art”. There are four hourly recording stations within twenty miles from the New Salem precipitation gage; Petersham 3N – station 196322 is 9.6 miles east of New Salem, Barre Falls Dam – station 190408 is 15.3 miles away, Birch Hill Dam – station 190666 is 16.8 miles and West Brimfield – station 199093 is 19.9 miles south-southeast of New Salem. For each storm event, the precipitation recorded from New Salem was always used. The rainfall distribution was always made dimensionless by dividing each cumulative hour by the total amount recorded. To choose which station to use at New Salem was an art. The rainfall distribution either came from the station whose total was closest to New Salem’s total or two rainfall distributions were made dimensionless, averaged together and then applied uniformly over the Hop Brook watershed.

Dimensionless Unit Hydrograph (DUH): The dimensionless unit hydrograph may be the second most stable hydrologic parameter with respect to time next to drainage area. The shape of the unit hydrograph is in some ways a measure of the watershed’s storage characteristics and its’ response time of shedding water once rainfall has hit the ground. It is not a parameter easily measured. The five other hydrologic parameters were measured or calculated, so in order to find a suitable DUH for the Hop Brook watershed, it was a matter of trial and error of running WinTR-55 and trying to match peak discharge and runoff volume without having to significantly change any of the other input parameters.

The first storm modeled was the June 6th, 1982 storm in which 2.72 inches of rain fell on the day the peak stream discharge was recorded, 0.56 inches on the day prior to the peak. The five day antecedent rainfall was 2.75 inches, ARC III was assumed. Peak discharge on June 6, 1982 was 168 cubic feet per second (cfs). The runoff was approximately 0.66 watershed inches. From daily mean discharge records, the three day runoff was approximately 1.04 watershed inches. From Figure 10.2 of NEH Part 630, the runoff curve number is approximately 70.

The hydrologic parameters used to model the 6/6/82 storm in WinTR-55 are as follows; drainage area (DA) is 3.39 square miles, Time of Concentration (Tc) is 1.56 hours, Curve Number (CN) is 70, the rainfall distribution was derived from Petersham 3N, an hourly recording gage 9.6 miles to the east of the watershed. The 24-hour precipitation total of 2.72 inches recorded from the New Salem gage on the edge of the watershed was applied over the one basin model. The DELMARVA Unit Hydrograph was selected. Computed peak discharge was 202 cfs and runoff was 0.564 inches, reducing CN lowered the peak but also reduced runoff volume.

A unit hydrograph with a peak rate factor (PRF) of 200 was selected from NEH Part 630 Chapter 16 – Hydrographs. Tc was increased to 1.6 hours, all other parameters remained the same; computed peak discharge was reduced to 181 cfs, but the runoff remained at 0.564 watershed inches.

A unit hydrograph with a PRF of 150 was selected, again from Chapter 16. Tc remained at 1.6 hours, CN was increased to 72, all other parameters remained the same; computed peak discharge was reduced to 171 cfs, and runoff increased to 0.646 watershed inches. Given the uncertainty of the average rainfall depth, the actual rainfall distribution, no further parameter adjustments were made. It was time to test these new parameters against a second storm event.

The second storm modeled was August 5th, 1969; 2.95 inches of rain fell in approximately 24 hours, 0.84 inches fell on the previous day and 2.20 inches in the previous five days. August being in the growing season, again the ARC III condition was assumed. The peak discharge recorded was 147 cfs and the runoff volume measured was approximately 0.78 watershed inches. Using a PRF of 150, a Tc of 1.6 hrs, CN of 72, Rainfall of 2.95 inches, and a rainfall distribution based on both the West Brimfield and the Petersham 3N stations, WinTR-55 computed a peak discharge of 141 cfs and a runoff volume of 0.778 watershed inches. Decreasing Tc to 1.5 hours and running WinTR-55 with all other parameters the same gave a computed peak discharge of 145 cfs. Changing Tc does not affect runoff volume. In a hydrologic sense, there are too many variables and uncertainties to try and refine the answer any closer. It appears that PRF, CN, and Tc are consistent enough to be “calibrated”, at least for the ARC III condition, however testing a third storm is advisable.

The third storm modeled was October 6th, 1962; New Salem precipitation gage recorded 1.60 inches on the day of peak (8 am observation time) and 2.90 inches of rain recorded 24 hours later. The antecedent rainfall is either 0.11 inches or 1.71 inches. The growing season is coming to an end and the ARC is in either a I or II condition. The peak discharge recorded at the gage was 199 cfs, 1-day runoff was approximately 0.42 inches, 2-day runoff was approximately 0.64 inches and the 3-day runoff volume measured was approximately 0.746 watershed inches. Using a PRF of 150, a Tc of 1.5 hrs, CN of 72, and a rainfall distribution based solely on the Birch Hill Dam (Station 190666), WinTR-55 computed a peak discharge of 181 cfs (-9 % off recorded peak) and a runoff volume of 0.749 watershed inches (+0.4% difference on 3-day runoff volume). A fourth storm was modeled to check for further variance in the hydrologic parameters.

After modeling three storms with good results and little change in hydrologic parameters, it was time to test the model against different climatic conditions. The January 23rd, 1973 storm was modeled because a low 1.28 inches of precipitation produced a high discharge of 246 cfs. Either this rain fell intensely over a short period (unlikely during winter) or the runoff condition may have changed (due to frozen ground?). The 24-hour rainfall value at New Salem was applied to a dimensionless rainfall distribution constructed from the Petersham 3N hourly measurements. Only Curve Number was varied to match peak discharge, a CN of 93 produced a peak discharge of 243 cfs and a runoff value of 0.677 watershed inches. The USGS gage recorded a peak of 246 cfs and 82 cfs-days mean daily discharge (runoff volume = 0.90 watershed inches. This event may have involved rain on snow, since there is evidence of runoff not accounted for due to rain.

The February 3rd, 1970 storm was modeled to test the dramatic change in Curve Number due to frozen ground conditions. The USGS recorded a peak discharge of 223 cfs and a mean daily discharge of 86 cfs (0.943 watershed inches) and a two-day runoff total, discounting base flow, of 1.224 watershed inches. Using a PRF = 150, Tc = 1.6 hrs, CN = 89 and 2.27 inches of rain applied to a dimensionless rainfall distribution based on Petersham 3N and Barre Falls Dam, WinTR-55 computed a peak discharge of 220 cfs and a runoff volume of 1.255 watershed inches.

Modeling the August 19th, 1955 storm shed light on major discrepancies between computed and recorded discharges; The New Salem precipitation station recorded 6.50 inches of rainfall. West Brimfield recorded 11.94 inches within a 24-hour period. The previous 5-day rainfall total was 6 inches (ARC III). The USGS gaging station recorded a peak discharge of 275 cfs. Modeling the storm event using a PRF = 150, Tc = 1.5 hrs, CN = 72, and 6.5 inches of rainfall applied to a dimensionless rainfall distribution constructed from the West Brimfield hourly recording station, WinTR-55 computed a peak discharge of 1,143 cfs. The runoff volume calculated (from the WinTR-20 computation engine) was 3.40 watershed inches however the actual 8-day runoff total recorded after the storm was 2.88 watershed inches. CN was reduced to 66 to better match runoff (2.82 watershed inches of runoff calculated). The corresponding computed peak discharge was 909 cfs, still a far ways off from 275 cfs published by the USGS. A new rainfall distribution was input into the model, in which the 6.5 inches of rain fell uniformly over 24 hours – this distribution minimizes intensities and therefore would minimize the computed peak discharge, given that the 6.50 inches of rainfall recorded was correct. The 24-hour uniform distribution with a CN = 66 and all other hydrologic parameters the same produced a peak of 421 cfs. The model has proven consistent up to this storm, so the USGS published value of 275 cfs seems doubtful. Sometimes even measured and published data are suspect to error.

The peak discharges generated from the August 19, 1955 storm from WinTR-55 were input into the HEC-RAS steady flow model. Computed water surface elevations indicates that a peak discharge of 915 cfs would overtop the road and bridge that are just upstream from the USGS gage by 1.4 feet. Ground observation in the vicinity of the gage also indicates that flows on the far right in the cross section would bypass the USGS gage before returning to the stream. Therefore it is possible that the gage did not record the true peak or volume for a storm of this magnitude. To resolve this discrepancy, the proper methodology would be to model the bridge and road embankment as a structure and run the WinTR-55 hydrographs in the HEC-RAS unsteady flow model.

The USGS gage is located on the downstream end of the bridge abutments. Modeling results indicate that the bridge will pass up to the 10-year flows without altering the peak, but the bridge significantly affects peak discharges of storms with longer recurrence intervals.

DESIGN STORMS

A new Weibull Distribution was plotted, based on changing the peak annual discharges for the October 24, 1959 storm (289 cfs measured changed to 537 cfs computed) and the August 19, 1955 storm (275 cfs measured changed to 915 cfs computed). Assuming these are the peak discharges just upstream of the bridge and gage. WinTR-55 was run using the Type I, Type IA, Type II, and Type III design rainfall distributions and using the calibrated hydrologic parameters of CN = 72, Tc = 1.6 hrs and PRF = 150. The 2-year, 5-, 10-, 25-, 50-, and 100-year rainfall amounts were accepted from WinTR-55 for Franklin County, Massachusetts. Results are plotted in Figure 1. Discharges resulting from the Type I design rainfall distribution come closest to the new Weibull Distribution. In Massachusetts, the Type III design rainfall distribution is used. The Type I and the Type III rainfall distributions were then run with the standard design DUH, (which gives a peak rate factor of 484), CN was reduced to 71 and 70 respectively. Results are also plotted in the graphic below. Finally, WinTR-55 was run using a Type I rainfall distribution, a CN = 93 (for frozen ground conditions), a Tc = 1.6 hours, and a PRF = 150. Results were also plotted on the graphic below.

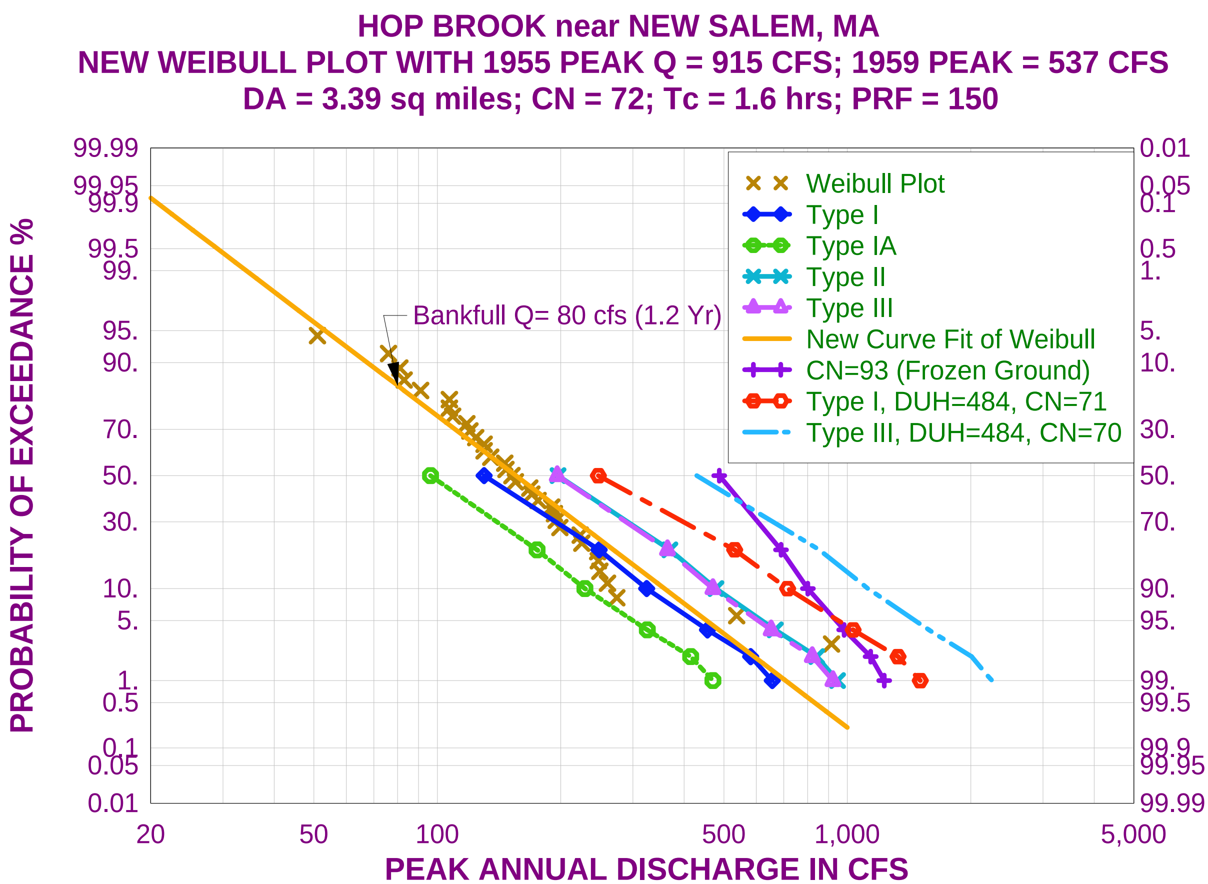

Figure 1 New Weibull distribution plot – Hop Brook near New Salem, MA.

What the graphic indicates is that the discharges generated using design parameters from WinTR-55 using the Type III rainfall distribution, along with the standard dimensionless unit hydrograph (PRF = 484) and a CN of 70, which would have been calculated from hydrologic soils groups, vegetation and land use maps, would be sufficient and even overly conservative to estimate discharges based on frozen ground conditions (CN of 93) had all the other parameters been calibrated correctly.

SUMMARY

The Table 1, below, shows the results of the WinTR-55 modeling effort. The storms are listed in the order of analysis and calibration. Comparisons of the recorded peak discharges and measured runoff volumes for the first four storms to computed values indicates that WinTR-55 (and the WinTR-20 computation engine) performs well given that a satisfactory job in calibrating the six hydrologic input parameters; of Drainage Area, Time of Concentration, Curve Number, Rainfall, Rainfall Distribution, and Dimensionless Unit Hydrograph. The largest difference between computed and measured peaks was -9 percent. It wasn't until the 6.5 and the 4.05 inch rains that major discrepancies arose in the computed results. It is the author's belief that the road and bridge above the gage acts as a dam that impounds water and reduces the peak discharge through the eleven-foot wide bridge abutments. There is also a discrepancy in runoff volumes. HEC-RAS modeling indicates that discharges above 900 cfs will overtop the road, and that flows that overtop will seek an alternate route around the gage, to which storm volumes will not be fully accounted for. All storms modeled produced the peak annual discharge for their respective water year. The last two storms modeled occurred in winter, when frozen ground and a dramatic change in Curve Number occurred, CN changes from a 70 during the growing season (72 in ARC III condition) to CN of 89 to 93. The modeling discrepancies in runoff volumes of the winter storms may be due to a melting of accumulated snow. Further detective work on nearby snotel sites may enlighten this theory.

Table 1 Results of the WinTR-55 modeling effort.

USGS Data			WinTR-55 Computed Results and Parameters				
		Volume		Volume			
Date	Peak Q	Watershed	Peak Q	Watershed	CN	Tc	PRF
	cfs	Inches	cfs	Inches			
6/6/1982	168	0.66	171	0.646	72	1.60	150
8/5/1969	147	0.779	145	0.778	72	1.50	150
10/6/1962	199	0.746	181	0.749	72	1.50	150
6/10/1961	247	0.548	244	0.547	71	1.60	150
10/24/1959	289	1.101	513	1.428	71	1.50	150
8/19/1955	275	1.906	915	3.829	72	1.50	150
1/23/1973	246	1.05	243	0.677	93	1.60	150
2/3/1970	223	1.367	220	1.255	89	1.60	150

REFERENCES

Forecast Systems Laboratory, Boulder, Colorado and National Climatic Data Center, Asheville, NC, (1998), US Hourly Precipitation Data. TD 3240 Vol. 1 and 2.

Hydrologic Engineering Center, (2003), HEC-RAS River Analysis System Computer program Version 3.1.1. U.S. Army Corps of Engineers

Rosgen, David L. (1994) "A Stream Classification System," Catena Vol. 22, pp 169-199. Elsevier Science, Amsterdam.

United States Department of Agriculture, Natural Resources Conservation Service (2003 draft), National Engineering Handbook; Part 630 Hydrology, Chapter 16 – Hydrographs. 42 pp

United States Department of Agriculture, Natural Resources Conservation Service (2003), Small Watershed Hydrology (WinTR-55) Computer program.

United States Department of Agriculture, Natural Resources Conservation Service (2004), National Engineering Handbook; Part 630 Hydrology, Chapter 10 – Estimation of Direct Runoff from Storm Rainfall. 79 pp

United States Department of Agriculture, Natural Resources Conservation Service (2004), Watershed Hydrology (WinTR-20) Computer program

Wolman, M.G. (1954), "A Method of Sampling Coarse River Bed Material," American Geophysical Union Transactions Vol. 35 pp 951-956.