DESIGN MANUAL AND USER'S GUIDE

Greenville County's Calibrate Storm Water Design Procedure
Phase 1: Simplified Detention Study
GREENVILLE COUNTY, SOUTH CAROLINA

November 14, 2002

The Project Scope of Work

The project scope is to modify existing methods used for detention calculations and generate a new calibrated storm water design procedure for Greenville County. In order to accomplish this work, this Design Manual and User's Guide includes the following components.

1. Summary of the methodology for creation of the spreadsheets with explanation of assumptions, etc.
2. Tables for land uses and curves for design aids in an order consistent with the selection and design sequences.
3. Example problems based on scenarios that represent typical design techniques.

Part 1. Need for a Regionalized Unit Hydrograph Procedure.

Background. The two methods most widely used to develop runoff hydrographs are based on: 1) determining rainfall excess and solving the equations of motion using the kinematic wave approximation and 2) use of unit hydrograph theory applied to rainfall excess (Haan et al., 1994). The kinematic wave approximation is available in many models, but is not easily applied in a simplified design procedure. The unit hydrograph theory is well established, dating back to 1938, and is widely used. Where watersheds are gauged and long records are available, it is possible to develop a watershed specific unit hydrograph from rainfall- runoff records and apply this unit hydrograph to future storms. For watersheds that are ungauged, which includes most cases of interest, it is necessary to develop a synthetic unit hydrograph. Techniques for doing this have been established and are widely used. However, there are problems with some of these techniques.

Prior to discussing some of the problems associated with existing unit-hydrograph procedures, a brief overview of synthetic unit hydrographs is appropriate. The most widely used synthetic unit hydrograph is that of the NRCS developed by Mockus (SCS, 1972) where he assumed a triangular shape with a time base that is a multiple of the time to peak discharge. Peak discharge was defined by:

$$\mathbf{q_p} = \frac{\mathbf{PRF * A * Q}}{\mathbf{t_p}} \qquad \textbf{(1)}$$

where A is area in square miles, t_p is time to peak in hours, Q is runoff volume (1 inch for a unit hydrograph) and PRF is the peak rate factor given by:

$$\mathbf{PRF} = \mathbf{643.3 * K} \tag{2}$$

K is a coefficient defined by the time base and given by:

$$\mathbf{K} = \mathbf{2} / \left(\mathbf{1} + \left\{ \mathbf{t_r / t_p} \right\}\right) \tag{3}$$

where t_r is the hydrograph's recession time. Using data from Midwestern agricultural watersheds, Mockus determined that

$$\frac{\mathbf{t_r}}{\mathbf{t_p}} = \mathbf{1.67} \tag{4}$$

Hence

$$\mathbf{q_p} = \frac{\mathbf{484 * A}}{\mathbf{t_p}} \tag{5}$$

Equation 5 represents the current NRCS relationship. A curvilinear approximation to the triangle is also available. Although the Natural Resource Conservation Service's (formerly SCS) National Engineering Handbook 4 (NEH-4) (SCS, 1972) indicates that PRF may range from 300 to 500, based on terrain, no guidance is given on values selection.

Other Studies on PRFs. As a result of criticism of the 484 value in the NRCS unit hydrograph procedure when applied to flat poorly channelized watersheds, a study was conducted by Welle et al. (1980) on four Delmarva watersheds. A mean PRF of 284 worked best for these watersheds; hence a triangular unit hydrograph with a PRF of 284 has been designated as the alternate NRCS unit hydrograph. Of course, since the peak discharge is decreased, the time base must be increased as well to have a runoff volume of one inch.

Some computer models developed in the late 70's and early 80's implicitly include PRF in their unit hydrographs. Examples include the TENN V model (Overton, 1989a, 1989b), the TVA model (Betson et al., 1980, Bales, 1979), the SEDIMOT II model (Wilson et al., 1982) and the SEDIMOT III model (Barfield, et al., 1996). In each case, the PRF is not mentioned explicitly, but the ordinates of the unit hydrograph reflect the concept.

Gray (1973) includes a two-parameter gamma function to describe the synthetic unit hydrograph, and regional equations to estimate the gamma function shape and scale parameters. As will be demonstrated in the Meadows and Ramsey (1991) study, PRF varies directly with the gamma function shape parameter. Thus, PRF is part of the procedure.

Recent research has demonstrated that hydrologic and geomorphologic approaches to defining watershed hydrologic response functions are convergent and may be expressed similarly through the gamma function. These results are significant in that they offer an explanation for observed variations in UH shape. Basically, for a watershed in constant land use, the UH will be the same for all storms of the same pattern when the antecedent conditions are the same. The variation among watersheds is due to variations in basin geomorphology and land use. This line of research has grown to a world-wide effort to relate gamma distribution parameters, notably the shape parameter (and thereby, the UH PRF) to geomorphic measures. Notable recent efforts include Lee and Yen (1997), Rosso (1984), Rodriques-Iturbe and Valdes (1979).

A USDA-ARS coastal study (Sheridan, et al., 1999) evaluated the PRF in eight southeast coastal watersheds ranging from 1.0 to 19.3 square miles. Values for the PRF ranged from 174 to 476, depending on watershed slope and area. The PRF was predicted by:

$$\mathbf{PRF = 631.7 * DA^{0.264} S^{0.882}} \qquad \mathbf{(6)}$$

where *DA* is drainage area in square miles and *S* is watershed channel slope in percent.

Meadows and Ramsey (1991) evaluated the PRF as function of watershed variables in 24 South Carolina watersheds. The unit hydrograph was represented by the gamma function, or:

$$\frac{q(t)}{q_p} = \left[\frac{t}{t_p} e^{(1 - t/t_p)} \right]^{n-1} \qquad \mathbf{(7)}$$

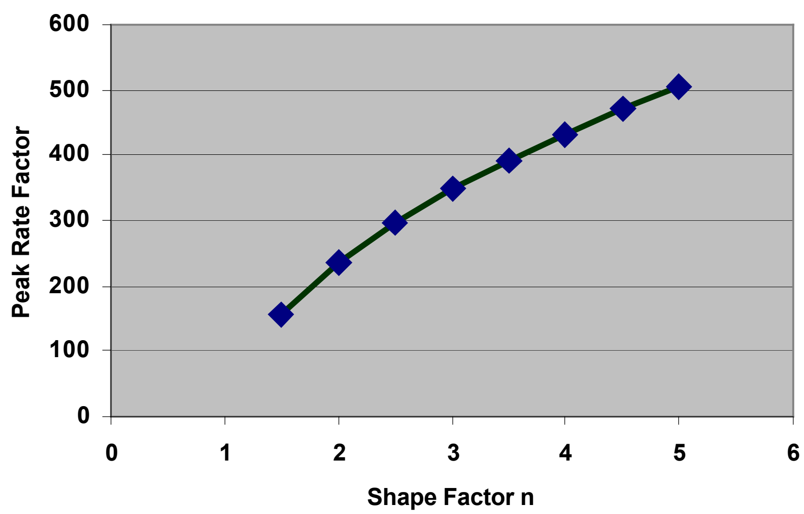


Figure 1. Relationship between PRF and gamma function shape factor n (Meadows and Ramsey, !991).

Table 1. Watershed Characteristics for Meadows and Ramsey (1991) Study.

Province	*No. Watrshds*	*Areas (sq mi)*	*Slopes (%)*	*% Imp (%)*	*Basin Dev. Factor*
Piedmont	8	0.52-3.90	0.52-2.88	14-47	5-12
Upper Coastal Plain	8	0.28-5.49	0.60-3.71	23-51	7-12
Lower Coastal Plain	8	0.09-2.06	0.24-1.03	13-60	2-11

where q is the discharge at time t, q_p is the peak discharge at t_p, and n is the gamma function shape factor. Values of n were correlated to PRF as shown in Figure 1. Watershed characteristics for the Meadows and Ramsey (1991) study are given in Table 1.

Using the values determined for each of the watersheds, Meadows and Ramsey (1991) developed prediction equations for all physiographic regions of SC using the equation form:

$$\mathbf{PRF; t_p = f(\%\,Imp, DA, L, S, BDF)} \tag{8}$$

where DA is drainage area, L is watershed length, S is slope, and BDF is the basin development factor. Meadows and Ramsey (1991) describe the BDF as:

> "An urbanization index which provides a measure of the efficiency of the drainage system. This parameter is defined from drainage maps and field inspection of the drainage basin. The basin is first divided into thirds and within each third, four aspects of the drainage system are evaluated and assigned a code. The four aspects are: (1) channel improvements, (2) channel linings, (3) storm sewers, and (4) curb and gutter streets. The code is assigned one if the aspect was present in at least 50 percent of that third of the basin and zero if less than 50 percent. The codes sum to a maximum possible value of 12."

Table 2. Prediction Equations for *K* and *t_p.*

Parameter	Piedmont	Upper Coastal Plain	Lower Coastal Plain
PRF	$324\dfrac{S^{0.37}}{L_c^{0.38}}$ (9)	$80\dfrac{Imp^{0.31}}{A^{0.15}}$ (11)	$136\dfrac{A^{0.12}\,Imp^{0.328}}{(1+BDF)^{0.33}}$ (13)
t_p	$\dfrac{6.74}{S_c^{0.78}\,Imp^{0.86}}$ (10)	$\dfrac{1.53}{(1+BDF)^{0.98}}\left(\dfrac{L_c}{\sqrt{S_c}}\right)^{0.54}$ (12)	$\dfrac{0.51}{S_c^{1.29}\left(1+BDF\right))^{0.0.75}}$ (14)

Goodness of fit statistics for the PRF and time to peak equations in Table 2 are given in Table 3 The goodness of fit was very good, except for the Piedmont region. This could be a result of the highly varied topography of the Piedmont.

Table 3 Goodness of Fit Statistics for Regional Equations.				
Region	Variable	r	SEE	AER
Piedmont	*PRF*	0.56	45%	31%
	t_p	0.95	20%	13%
Upper Coastal	*PRF*	0.94	7%	5%
Plain	t_p	0.93	19%	9%
Lower Coastal	*PRF*	0.88	14%	9%
Plain	t_p	0.94	17%	9%

r - correlation coefficient
SEE - standard error of estimate
AER – error of regression in percent

In the first approach to defining PRFs, Meadows and Ramsey (1991) developed paired prediction equations for PRF and t_p as given in Table 2. Later in application studies, such as those described in Meadows et al. (1992a, 1992b, 1992c), the advice of the SCS and South Carolina engineers was heeded to have a method that provided a single PRF value for each unique land use. Using the equations in Table 2, typical watershed measures from these studies, and local calibration results, the values listed in Table 4 were developed for urban and agricultural land use. These values were tested during calibration studies using data collected at the watersheds during the course of the studies and "passed the test" according to Meadows (personal communication).

Table 4. Summary of Peak Rate Factors Based on Land Use.

Land Use	Peak Rate Factor
Urban	
Single family	325
Multi-family	375
Commercial	550
Industrial	550
Open Spaces	250
Agricultural	
Forest	180
Pasture	200
Row crop	300

Extending work with the geomorphic unit hydrograph, Helms and Meadows (2000) tested the model on 13 events at 7 small urban watersheds ranging in size from 0.16 to 1.52 sq. mi. and having zero, first and second order streams. UH shape and PRF values were determined for each event using a UH optimization program and compared to values estimated from geomorphic data. They concluded that the predicted shape and PRF values closely matched the optimal values and that the validation results confirm the underlying UH shape varies among watersheds and with storms at a particular watershed. They used the two parameter gamma distribution to describe the UH.

In addition to developing the PRFs for watersheds, Meadows developed a number of storm water procedures which account for PRF, including
:

1. An analytical function for peak discharge which accounts for PRF,
2. Tabular hydrographs similar to the TR55 values, but which account for PRF,
3. A graphical solution for peak discharge as shown in Figure 2, and
4. Simplified relationships for sizing storm water detention structures.

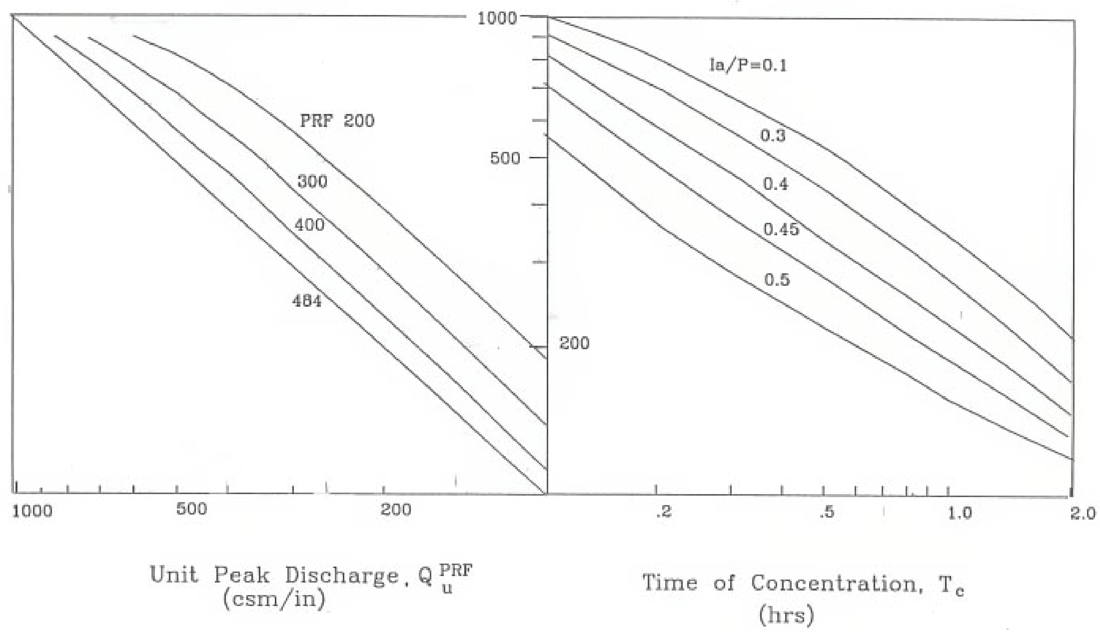


Figure 2. Peak discharge graphs of Meadows (1991)

FEMA Study For Charlotte. A study of flood flows in Charlotte and Macklenberg County was conducted by the consulting firm of Hayes, Seay, Matern and Matern (HSMM) (1999). The original objectives included the development of peak rate factors and regionalized unit hydrographs. In conducting the analysis, they compared TR20 model predictions with observed values using standard UH (PRF = 484) and Delmarva UH (PRF=284) on 11-gauged watersheds. The model did not have the option of using historical rainfall, but only allowed the use of the standard SCS distribution (assumed to be Type II for Greenville), which could be a serious limitation. They found that the model using the Delmarva unit hydrograph produced “encouraging results” in Mallard and McAlpine watersheds. When used on a countywide basis, the SCS standard hydrograph produced results much closer to that of the gages, but the Delmarva unit hydrograph” underestimated the discharge for almost every gage in Mecklenburg county.” HSMM did not attempt to range the PRF factors to get a better fit, which may have resulted in regionalized unit hydrograph models that would have benefited the project proposed in this report.

In addition to the work above, HSMM compared TR20 predictions to HEC1 predictions and found that the comparisons were favorable. Consequently, they chose the HEC1 for final analysis due to its widespread acceptability.

USGS Study The US Geological Survey is conducting a study of peak flows and dimensionless unit hydrographs for Mecklenburg County streams with the following specific objectives:

1. Develop procedures for estimating peak flows in Mecklenburg County.
2. Develop a dimensionless unit hydrograph(s) that is applicable to Mecklenburg County streams.
3. Regionalize the unit hydrograph(s) by developing procedures for predicting unit hydrograph parameters from basin characteristics.

Watersheds to be evaluated in the study have been selected, and information on watershed areas is available. A summary of the areas is given in Table 5 based on data provided by J. Bales at the Raleigh office of the USGS.

TABLE 8. Watersheds in the USGS Study for Charlotte, NC

Station No.	Site Name and Location	Drainage Area	
		sq. mi.	ac.
02146750	MCALPINE CR BELOW MCMULLEN CR NR PINEVILLE, NC	92.4	59136
02146381	SUGAR CREEK AT NC 51 NEAR PINEVILLE, NC	65.3	41792
02146530	LITTLE SUGAR CREEK AT PINEVILLE, NC	49.2	31488
02146507	LITTLE SUGAR C AT ARCHDALE DR AT CHARLOTTE, NC	42.6	27264
02146600	MCALPINE CR AT SARDIS ROAD NEAR CHARLOTTE, NC	39.6	25344
0212414900	MALLARD CR BL STONY CR NR HARRISBURG, NC	34.6	22144
02146300	IRWIN CREEK NEAR CHARLOTTE, NC	30.7	19648
0214266000	MCDOWELL CREEK NR CHARLOTTE, NC (CSW10)	26.3	16832
0214645022	BRIAR CR ABOVE COLONY RD AT CHARLOTTE, NC	19	12160
02146670	FOUR MILE CREEK NEAR PINEVILLE, NC	17.8	11392
02142900	LONG CREEK NEAR PAW CREEK, NC	16.4	10496
02146409	LTL SUGAR CR AT MEDICAL CENTER DR AT CHARLOTTE, NC	11.8	7552
0214295600	PAW CR AT WILKINSON BLVD NR CHARLOTTE, NC	10.8	6912
02146348	COFFEY CREEK NR CHARLOTTE, NC	9.14	5849.6
02146700	MCMULLEN CR AT SHARON VIEW RD NEAR CHARLOTTE, NC	6.95	4448
0214678175	STEELE CREEK AT SR 1441 NR PINEVILLE, NC	6.73	4307.2
0214642825	BRIAR CREEK NEAR CHARLOTTE, NC	5.9	3776
0214630800	TAGGART CREEK AT WEST BOULEVARD NR CHARLOTTE, N(	5.38	3443.2
0214677974	STEELE CREEK NR SHOPTON, NC	3.57	2284.8
0214266075	GAR CR AT SR2120 (MCKOY RD) NR OAKDALE (C SW08)	2.67	1708.8
02146470	LITTLE HOPE CR AT SENECA PLACE AT CHARLOTTE, NC	2.63	1683.2
02142651	MCDWELL CR AT W ESTMRLND RD NR CORNELIUS (CSW09)	2.35	1504
0214666925	FOUR MILE CR TRIB NR PROVIDENCE, (I-485, CSW07)	0.266	170.24
0214669980	MCMULLEN CR TR AT SILO LN NR CHARLOTTE (CSW04)	0.126	80.64
0214650690	LITTLE SUGAR CREEK TR AT ROSE VALLEY DR (CSW02)	0.123	78.72
	**** REJECTED SITES THAT COULD BE USED****		
0214635212	UNNAMED TR TO SUGAR CR AT CROMPTON ST (CSW06)	0.063	40.32
0214643840	EDWARDS BR TR STORM DRN AT HIDDEN BR DR (CSW03)	0.023	14.72
0214620805	IRWIN CR TRIB BL STARITA RD AT CHAR., NC (CSW05)	0.022	14.08

The largest watershed in the Meadows and Ramsey (1991) study was 3.9 sq. mi. as shown in Table 1. The next smaller size was 3.5 sq. mi. Only eight of the USGS watersheds are in the range of sizes of the Meadows and Ramsey study. Typical watershed sizes draining into a detention basin are in the range of 5 to 100 acres. Clearly, the majority of the watersheds in the USGS study are much larger than those of the Meadows and Ramsey study and outside the range of those appropriate for on-site detention.

Three watersheds were evaluated for the USGS study and rejected as being smaller than the 0.08 sq. mi. minimum, which they imposed. These three watersheds could add valuable information to the database. Because of the paucity of smaller watersheds in the USGS study, it will likely be necessary to supplement the USGS data with information from the Meadows and Ramsey (1991) study and the ARS study (Sheridan et al., 1999) to develop a reliable model for small watersheds. In addition, it may be possible to reanalyze some of the Hayes Seay, Mattern and Mattern (1999) data to get supplemental information.

Part 2. Simplified Detention Procedures

A simplified design procedure, as used in this context, is a design procedure for small catchments where hydrologic computational procedures can be greatly simplified using regionalized constants. The computations should be based on selected simple parameters such as watershed area and percent impervious. The design that results will be right sized in some cases and conservative in all others. The procedure that is to be developed will be referred to as a simplified detention design aid.

A concern expressed in discussions with both design engineers and regulatory personnel is that the current “detention design procedure” is too complex and time consuming. Also, as was found in this project, the procedures used are questionable in some cases.

Simplified detention design procedures are desirable for a number of reasons. First, computations are simple making for reduced design time. Second, there will be more uniformity in design which should make the review process simpler, less time consuming, and more consistent across reviewers. Third, very little data is required for some simplified design procedures. Finally, designs are right sized in some cases and conservative in most cases. Thus, if a developer is willing to use a conservative design, he or she can trade engineering design cost for construction cost. In all cases, the option of using a more complex model such as HEC 1 for design should be available.

Comments on Existing Procedures. An evaluation was made of existing procedures. The following summary comments are made about the procedures.

Table 5. Review of Current Procedures Available for Greenville County.

HEC 1 Modeling	Hec 1 is a well-established and widely used model. The new version should allow a user input UH and consideration of PRF.	
Manual Hydrograph Procedure	NRCS hydrograph generation	Well established procedure, should allow PRF factor
	NRCS Step function	Assumes that shape of unit hydrograph and runoff hydrograph is the same, distorts timing parameters which can cause problems for small reservoirs with large inflows.
Peak Discharge Procedure	NRCS simplified procedure	Widely used procedure. Needs to allow for PRF.
	Rational Equation	Peak flow only, not consistent with other methods and should not be integrated with such procedures as the NRCS method.
Reservoir Routing	Puls Method	Graphical procedure that is tedious and time consuming, and a better procedure is available for spreadsheet computation.
	Chain Saw Method	Timing is off, there are stability problems with routing unless small times steps are used, and simpler alternatives are available with spreadsheets.
	Simplified Procedure	Assumes that the shape of the total hydrograph is the same as the unit hydrograph, thus the timing is off. The assumption of triangular hydrographs is not accurate, thus corrections are needed.

Options for Simplified Detention. A number of options for developing a simplified detention design aid could be presented. Three options were chosen for consideration. A brief discussion is given for each method.

Option 1. Design based on watershed area only with all other parameters conservative.
Option 2. Design based on watershed area and percent impervious with all other parameters conservative.
Option 3. Design using a spreadsheet that emulates a more complex model such as HEC 1. Where complete emulation is not possible, conservative values are chosen.

Option 3 was selected so it was determined to develop a spreadsheet that emulates a complex computer model such as HEC 1 for small watersheds. Many variables are possible to be input with conservative values added for all other parameters. To explain the development process, the steps that were taken to do a design will be described, and the computation procedures in the spreadsheet that make the calculations will be explained. The equations were developed from data

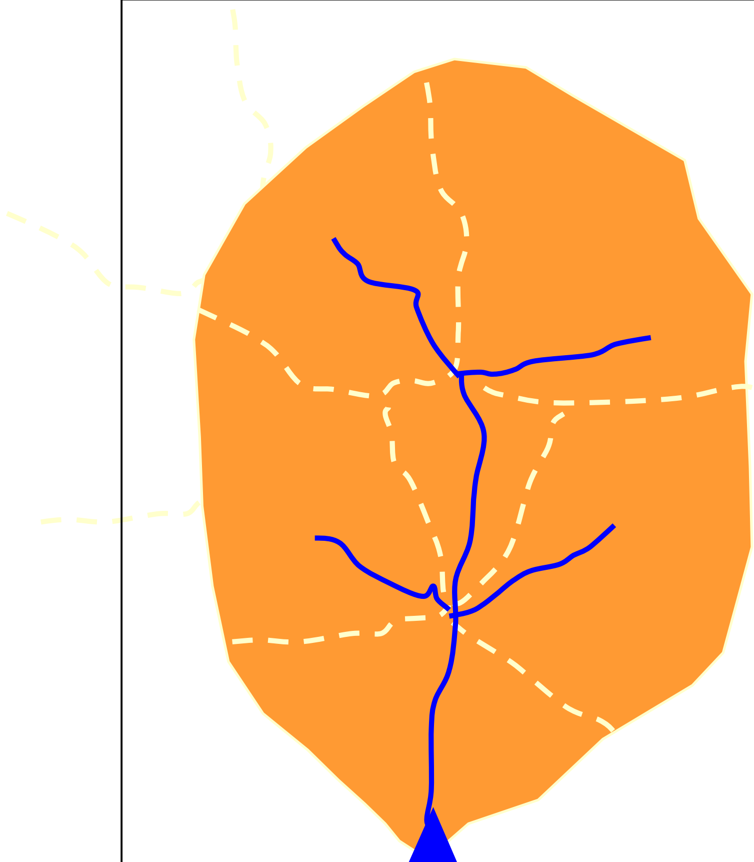


Figure 3. Subdivided watershed.

generated by Meadows and Ramsey (1991). These were modified to fit Greenville conditions.

Step 1. Subdivide the watershed into subwatersheds as shown in Figure 3. This allows the model to differentiate between smaller developed areas that are dramatically different hydrologically from larger undisturbed areas and not mask out

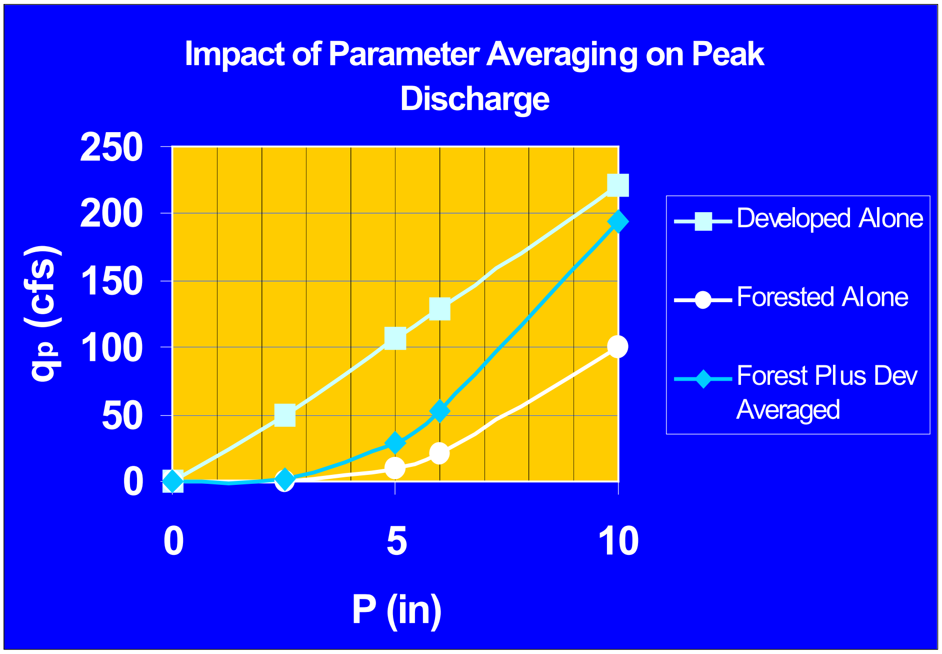


Figure 4. Illustration of the impact of lumping subwatersheds together on predicted peak discharge.

their impacts, as would be the case with lumped parameter models. Figure 4 shows an example of the importance of subdividing the watershed. The watershed is primarily undisturbed with about 20 percent of its area in development. As can be seen, the discharge considering the disturbed area alone is much greater than that computed using a lumped parameter approach based on area weighted parameters, particularly in the range of 2 to 7 inches of precipitation.

Step 2. Determine hydrologic parameters for each subwatershed. These parameters are area, curve number, time of concentration, travel time, PRF, and time to peak. The spreadsheet is set up to simplify these computations.

Step 3. Calculate the peak discharge, $q_{po,i}$, from each subwatershed in Figure 5. This is calculated from the time of concentration, the initial abstraction (dependent on curve number and rainfall), and the PRF. An example computational procedure is that of Meadows (1991), shown earlier in Figure 2. As part of the model development, a new graph was developed specific to Greenville County.

Step 4. Route the subwatershed peak discharge, $q_{po,i}$, to the watershed outlet where it becomes $q_{p,i}$. This requires a functional relationship between the two peaks. An example is the functional form shown in Figure 5 where the ratio of the two peaks is proposed to be a function of time of concentration, travel time, and PRF.

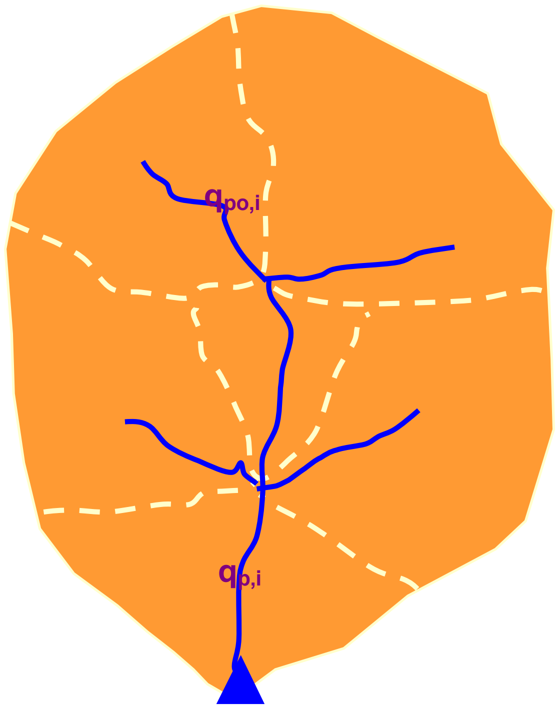


Figure 5. Illustration of peak discharge at subwatershed outlet and the corresponding peak discharge at the watershed

$$\frac{q_{p,i}}{q_{po,i}} = f(t_c, t_t, PRF, etc)$$

An investigation of the functional form of the relationship proposed in Figure 5 was developed from data generated by Meadows (1991) as part of his work on South Carolina. A plot of the ratio of peak discharges as a function of travel time is given in Figure 6 for a variety of times of concentration, ratios of initial abstraction to rainfall, and PRFs. As can be seen, the ratio is obviously a function of more than travel time; however, any one of the lines can be represented

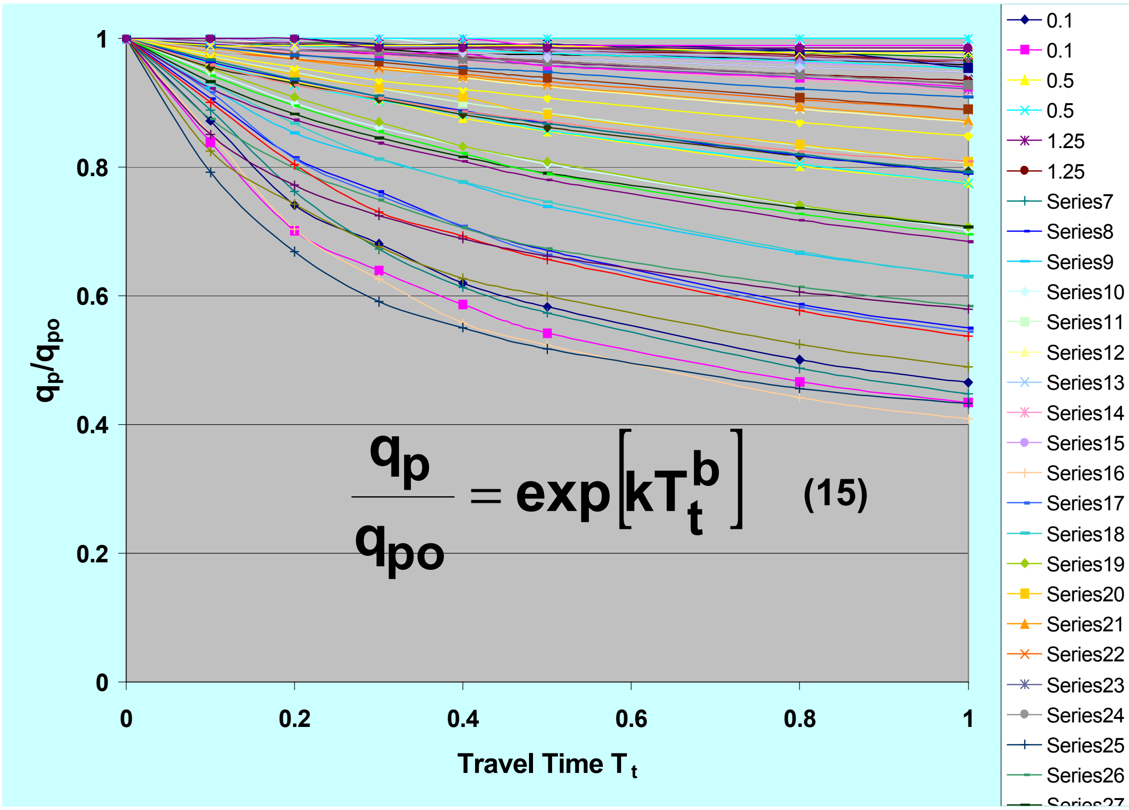


Figure 6. Ratio of subwatershed peak discharge to that routed to the watershed outlet, plotted as a function of travel time. Each line represents a unique combination of time of concentration, initial abstraction to precipitation,

by the exponential function shown. The problem, then, becomes one of developing predictions for the parameters k and b.

The first attempt at finding predictions for the parameters k and b was based on time of concentration dependence, as shown in Figure 7. Each of the lines represents a different PRF. The plots showed that k and b were heavily dependent on time of concentration, and could be defined by exponential functions with parameters a, c, d, and f. It was also apparent that PRF was an important factor in these four parameters.

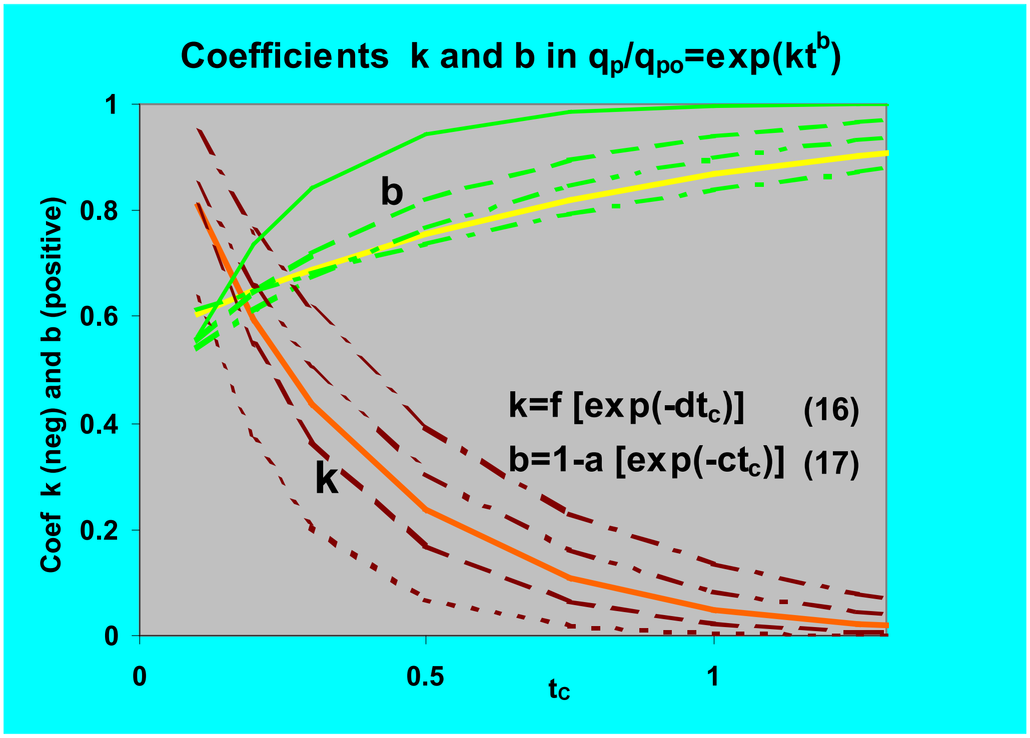


Figure 7. Dependence of k and b on time of concentration.

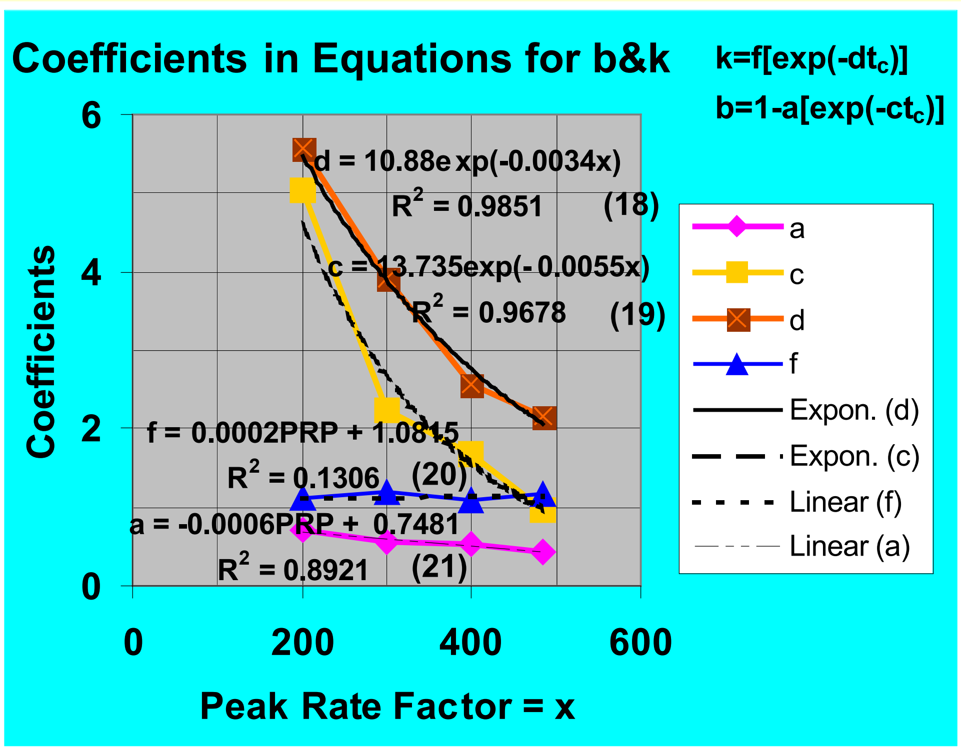


Figure 8. Dependence of the parameters a, c, d, and f on PRF.

The final result was to evaluate dependence of a, c, d, and f on PRF. This is shown in Figure 8, which indicates that the parameters can be predicted by PRF. Thus, a model for the Meadows (1991) data to predict the ratio of subwatershed peak discharge to the corresponding peak discharge at the watershed outlet was developed. It was, of course, necessary to repeat this analysis for the Greenville database, but the analysis presented here shows how it can be done.

Step 6. Sum hydrographs at watershed outlet to get the peak discharge flowing from the watershed into the stormwater detention structure. Since the peak discharges from the routed subwatershed flows will not occur at the same time, it is necessary to predict the hydrograph ordinates from each subwatershed. A number of functions have been tried to fit the hydrograph, and a reasonable fit has been found by using a function similar to that describing the NRCS Type II rainfall distribution (Haan et al., 1994). Further work was done to see if a simpler function can be found that will work as well. It was not actually necessary to sum all points on the hydrograph, but simply those corresponding to the times of peak for the routed subwatershed flows (Barfield et al., 1994). As shown in Figure 9, the total watershed peak discharge should fall under the peak of one of the subwatersheds. Therefore, the hydrograph function can be used to predict the routed discharge for each subwatershed at the time to peak of all the subwatersheds, sums taken at the routed time to peak for each subwatershed, and the spreadsheet can pick the maximum value.

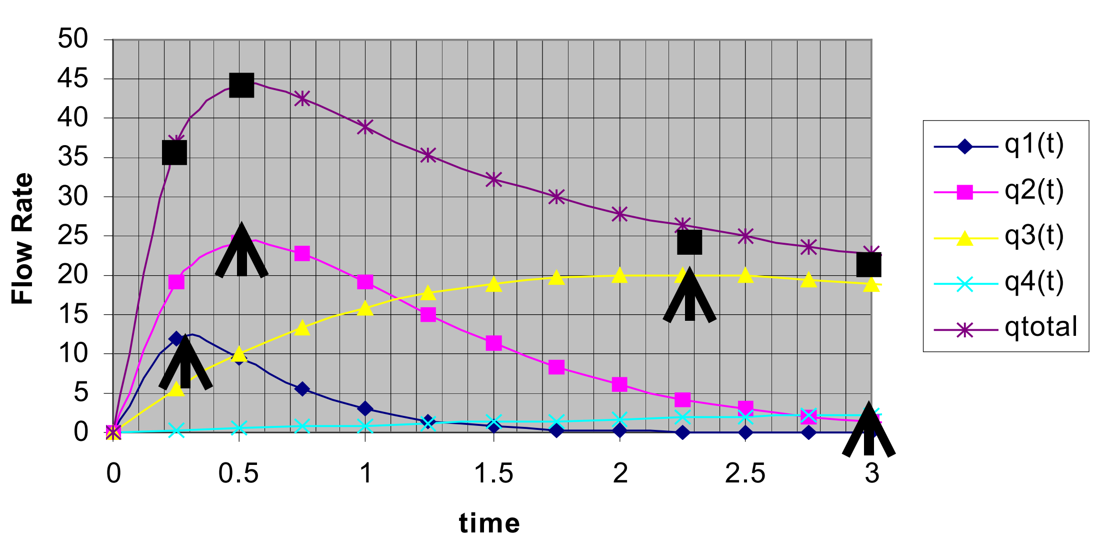


Figure 9. Illustration of the procedure for determining the summing points to predict the peak discharge for the total

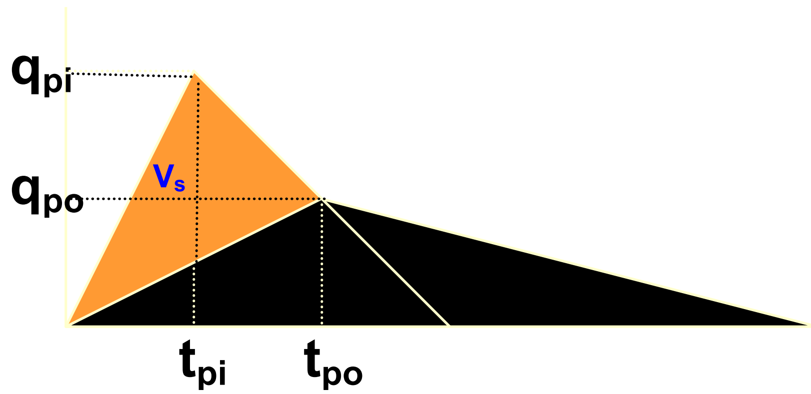


Figure 10. Triangular representation for inflow and outflow hydrographs.

Step 6. Determine the required storage volume for the 2-year storm. If the inflow and outflow hydrographs are assumed to be triangular, as shown in Figure 9, then the ratio of storage volume to runoff volume is given by

$$\frac{V_s}{Q} = (1 - a) \qquad (22)$$

where V_s is storage volume in the same units as runoff volume Q and α is given by

$$a = \frac{q_p \text{ predisturbed}}{q_p \text{ postdisturbed}} \qquad (23)$$

Hydrographs are not actually triangular in shape; therefore, a correction factor is needed for storage volume, hence:

$$\frac{V_s}{Q} = (1 - a)C_f \qquad (24)$$

where C_f is a correction factor which will be determined from the database developed to produce the design aids will be predicted by:

$$C_f = f(PRF, t_c, t_t, \text{ etc }) \qquad (25)$$

When multiple outlets are used, such as design for 2-, 10- and 50-year storms, the correction factor, C_f, must be modified to account for the discharge from other outlets. For example, assume that:

1. The 10-year storm is to be controlled by a drop inlet,
2. The 2-year storm is to be controlled by an orifice located low on the riser of the drop inlet and the height of the riser crest set at the top of the required storage for the 2-year storm, and
3. The 50-year storm is to be controlled by a weir whose crest is to be located at the maximum stage of the 10-year storm.

Under these assumptions, the following correction factors are needed:

$$
\begin{aligned}
C_{f,2} &= f(PRF, t_c, t_t, q_{p,2}, Q_2) \\
C_{f,10} &= f(C_{f,2}, V_{s,1}, q_{p,10}, Q_{10}) \\
C_{f,50} &= f(C_{f,10}, V_{s,10} + V_{s,2}, q_{p,50}, Q_{50})
\end{aligned}
\qquad \textbf{(26, 27, 28)}
$$

where the subscripts 2, 10, 50 refer to the 2-, 10-, and 50-year storms, respectively. With the determination of storage volume, V_s, the only thing left to do for the 2-year storm is select the reservoir shape, depth, surface area, and size of the outlet. Reservoir shape is fixed by the developer; hence the area, depth, and size of outlet are the parameters the designers can vary, as discussed in the next step.

Step 7. Determine the reservoir surface area, size of the outlet, and stage of the emergency spillway. For illustrative purposes, it is assumed that the reservoir is rectangular and that the outlet is a drop inlet spillway, as shown in Figure 10. It is further assumed that the stage-storage relationship is given by (Lindley, et al., 1997):

$$V_s = a_1 h^{b1} \qquad \textbf{(29)}$$

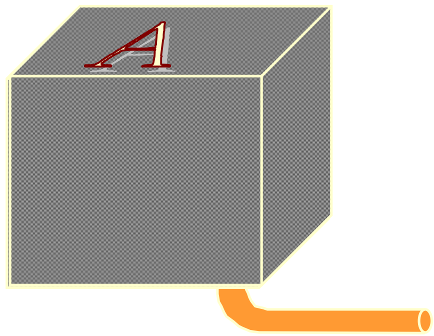
and that the peak discharge is given by:

$$q_{po} = c_1 h^{d1} \qquad \textbf{(30)}$$

where V_s is storage, q_{po} is peak discharge, h is stage, and a_1, b_1, c_1, and d_1 are constants, dependent on the shape of the structure and the type outlet.

Since V_s is known from step 6 and q_{po} is given as the predisturbed peak, equations (26) - (32) can be used to solve for surface area, A, and size of outlet. Note that A_p and K_c for equation (32) depend on pipe diameter.

Other reservoir shapes and outlet structures can be modeled, but the relationships are more complex than can easily be presented here. The spreadsheet model considers the following reservoir and outlet shapes:



$$a_1 = A; \quad b_1 = 1 \qquad (31)$$

$$c = \frac{A_p\sqrt{2g}}{\sqrt{1 + K_e + K_b + K_c L_P}}; \quad d_1 = 0.5 \qquad (32)$$

Figure 10. Illustration of equations that can be used for sizing a rectangular reservoir with a drop inlet spillway. In the equations, A is the reservoir surface area, A_p is the area of the pipe ($pd^2/4$), L_P is the length of the pipe, K_e and K_b are entrance and bend loss coefficients, and K_c is a friction coefficient which is a function of pipe diameter (Haan aet al.,

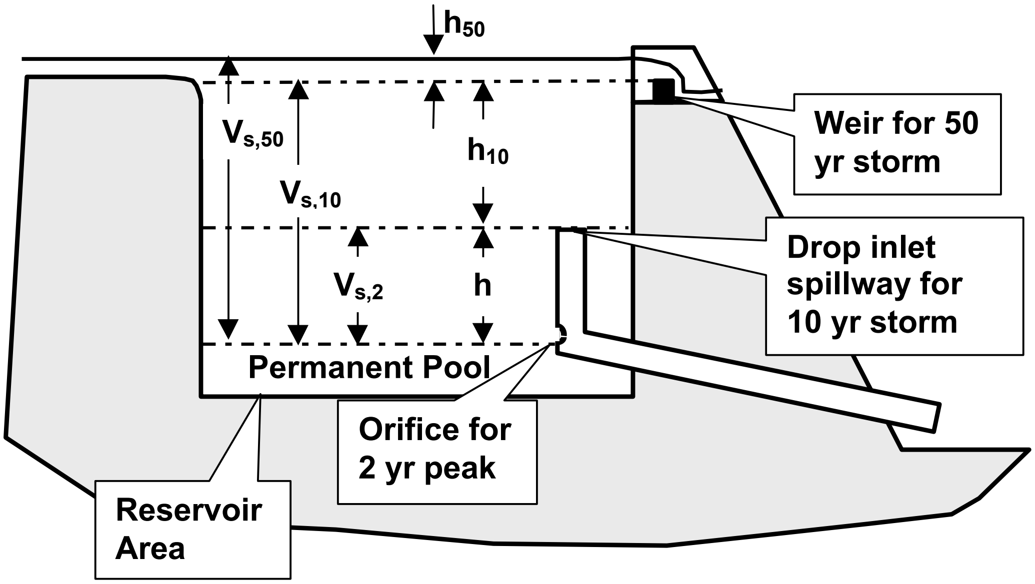
1. Reservoir shapes
   - Rectangular
   - Trapezoidal
   - Triangular

2. Outlet types
   - Orifice
   - Weir
   - Culvert
   - Drop inlet, considering weir, orifice and pipe flow control
   - Open channel outlet, with and without control section

Rock fill outlet procedures could also be developed. However, the variability of discharge resulting from variations in rock characteristics makes this outlet type undesirable for storm water detention structures.

Step 8. Determine storage and outlet size for 10-year and 50-year storm (or other storms as required by the regulatory authority). Steps 6 and 7 are repeated for each return period storm for which a design is required. For the second storm, the discharge is the sum of that from the first outlet and the second outlet, assuming that the outlets are independent. For the third storm, the discharge is the sum of all the outlets, assuming that they are all independent[1]. For example, assume that an orifice is used for the 2-year storm, a drop inlet for the 10-year storm, and a weir type outlet for the emergency spillway (50-year storm). For simplicity, a rectangular basin

[1] In the case where an orifice is located beneath the riser on a drop inlet to control the 2-yr storm, the flow will be the greater of either the orifice or the drop inlet pipe system, not the sum of the two. The flows will only be summed when the outlets are independent.



2 Year Storm

$$q_{po,2} = C_{1,2} h_2^{d_{1,2}}$$

$$V_{s.2} = a_1 h_2^{b_1} = Q_2(1 - a_2)C_{f,2}$$

$$a_2 = \frac{q_{p,2\text{ yr pre-disturbed}}}{q_{p,2\text{ yr post-disturbed}}}$$

Coefficients

$$C_{1,2} = 0.64; \qquad d_{1,2} = 0/5$$

$$C_{1,10} = \frac{A_P\sqrt{2g}}{\sqrt{1 + K_e + K_b + K_c L_P}}; \qquad d_{1,10} = 0.5$$

$$C_{1,50} = 3.5 L_w; \qquad d_{1,50} = 3/2$$

10 Year Storm

$$q_{po,10} = C_{1,10} h_{10}^{d_{1,10}}$$

$$V_{s.10} = a_1(h_2 + h_{10})^{b_1} = Q_{10}(1 - a_{10})C_{f,10}$$

$$a_{10} = \frac{q_{p,10\text{ yr pre-disturbed}}}{q_{p,10\text{ yr post-disturbed}}}$$

50 Year Storm

$$q_{po,50} = C_{1,50} h_{50}^{d_{1,50}} + C_{1,10}(h_{10} + h_{50})^{d_{1,10}}$$

$$V_{s.50} = a_1(h_2 + h_{10} + h_{50})^{b_1} = Q_{50}(1 - a_{50})C_{f,50}$$

$$a_{50} = \frac{q_{p,50\text{ yr pre-disturbed}}}{q_{p,50\text{ yr post-disturbed}}}$$

Figure 11. Illustration of relationship to be used to design a rectangular reservoir for 2, 10, and 50-year storms using an orifice for the 2-year storm, drop inlet (pipe flow control) for the 10-year storm, and a weir

of surface area A is assumed. The equations for sizing the reservoir under this assumption are given in Figure 11. Note that the 2-year flow is controlled by an orifice located on the drop inlet spillway, thus the orifice and drop inlet discharges are not independent. The formulas, therefore, do not sum the orifice flow into flows for the 10- and 50-year storms. The spreadsheet solves for the storage volumes and outlet sizes sequentially for each of the return periods.

Step 9. Determine the reservoir size and shape and print results. Given the shape selected by the user and constraints such as length to width ratio, etc., the spreadsheet selects the final dimensions and prints a summary of reservoir and outlet characteristics along with relevant inflow and outflow discharges.

Recommendations on simplified detention procedures. Based on an evaluation of options for simplified detention, discussions with stakeholders, and evaluation of the current methods, the following decisions were made:

1. Develop Option 3 for use in design.
2. Use spreadsheet format for Option 3.
3. Utilize variable peak rate factors.
4. Have spreadsheets protected so that users can only input values and print output.
5. Have spreadsheets develop final design dimensions based on user input constraints and hydrologic variables.

Summary

Existing methods used for detention calculations were modified and then used to generate a new calibrated storm water design procedure for Greenville County.

An evaluation of the literature was made to determine problems and possibilities associated with current hydrologic computation procedures, particularly synthetic unit hydrographs. The commonly used NRCS triangular unit hydrograph was developed with a time base of $2.67t_p$ based on data from small agricultural watersheds (SCS, 1972) and a prediction equation for the peak discharge of q_p = PRF*A/t_p where A is watershed area in mi^2 and t_p is unit hydrograph time to peak in hrs. This resulted in a fixed PRF of 484 that has become the NRCS standard, although the NRCS NEH Handbook 4 (SCS, 1972) indicates that the PRF may range from 300 to 500, based on terrain. A Delmarva study (Welle, 1980) arising out of criticism of the PRF of 484 resulted in a PRF of 284 that has become the NRCS alternative method. This reduced peak rate factor increases the time base of the unit hydrograph. Some computer models from the late 70's implicitly include variable PRF factors in their unit hydrograph, although PRF is not mentioned explicitly, but the ordinates of the unit hydrograph reflect the concept. Recent research on the use of the gamma function for unit hydrograph shapes has resulted in a number of publications showing variable shapes of the unit hydrograph based on watershed characteristics. Efforts in the last three decades include Lee and Yen (1997), Rosso (1984), Rodriques-Iturbe and Valdes (1979). A recent USDA-ARS coastal study (Sheridan, et al., 1999) evaluated the PRF in eight southeast coastal watersheds ranging from 1.0 to 19.3 square miles. Values for the PRF ranged from 174 to 476, depending on watershed slope and area.

Research in the last decade by Meadows and his colleagues (Meadows and Ramsey, 1991; Meadows, 1991; Meadows et al., 1992a, 1992b, 1992c) has led to the development of procedures for South Carolina that account for PRF as a function of land use and watershed geomorphic characteristics. Using flow data from 24 watersheds, equations pairs were developed for PRF and t_p for the four physiographic regions of South Carolina. In subsequent work, the initial procedures were modified to establishing PRF as a function of watershed land use and t_p calculation based on NRCS overland flow or lag time equations. In addition, peak discharge equations were developed which are similar to the TR55 equations but which also include the impact of PRF. Further, tabular hydrographs similar to the TR55 tabular hydrographs were developed that account for PRF and procedures developed for designing reservoirs that account for PRF. It was recommended that a calibrated unit hydrograph be developed utilizing a variable peak rate factor.

For purposes of this project, a simplified design procedure is defined as a design procedure for small catchments where hydrologic computational procedures can be greatly simplified using regionalized constants. The computations are based on selected simple parameters such as watershed area and percent impervious. The design that results are right sized in a some cases and conservative in all others. The procedure is referred to as a simplified detention design aid. Simplified detention design procedures are desirable for a number of reasons: 1) they result in reduced design time, 2) designs will be more uniform in general which makes the review process simpler, less time consuming, and more consistent across reviewers, 4) very little data is required for some simplified design procedures, and 4) designs are right sized in some cases and conservative in most cases

Before considering options for a simplified procedure, current methods were evaluated. Stakeholders expressed concern that the current "detention design procedure" is too complex and time consuming. Based on a review of the current procedures, it was found that the procedures used are questionable in some cases. In particular, the procedures on timing of peak discharge are based on the assumption that the unit hydrograph and total runoff hydrograph have the same shape, defined by a PRF of 484. This is clearly not the case, resulting in timing distortions that can be a problem on small reservoirs with large flow rates.

Three options were considered for development: 1) Design based on watershed area only with all other parameters conservative, 2) Design based on watershed area and percent impervious with all other parameters conservative, 3) Design using a spreadsheet that emulates a more complex model such as HEC 1. Option 3 was selected based on the data of Meadows (1991). From this data, it was shown that accurate equations can be developed that emulate the more complex models to predict peak discharge from subwatersheds, route peak discharges to storm water basins, sum discharges from subwatersheds to predict total watershed peak discharge, and determine required storage volume and outlet sizes to control the 2, 10, and 50-year storms.

The following recommendations were made for further development:

A. <u>Recommend changes needed to existing Greenville storm water design procedures.</u>

1. A regionalized unit hydrograph be developed using PRF as one of the parameters,
2. Request that the land use and slopes for the < 2 sq. mi. watersheds used by have an appropriate range to give good prediction relationships,
3. Attempt to incorporate all or parts of the Meadows and ARS results into model development data base.

B. Recommendations on Simplified Detention Procedures for Small Site Designs.

1. Use spreadsheet format for Option 3
2. Utilize variable peak rate factors
3. Have spreadsheets protected so that users can only input values and print output.
4. Have spreadsheets develop final design dimensions based on user input constraints and hydrologic variables.

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APPENDIX A

Model for Peak Flow from Subwatershed based on TR-55 Format

Background

The SEDIMOT III model was modified to use peak rate factors (PRF) other than 484. The PRF is a function of the Horton order ratios, based on the material presented in Meadows and Ramsey (1991). The Rosso relationship was used to relate the Horton order ratios to the unit hydrograph shape parameter n:

$$n = 3.29 \frac{R_B}{R_A}^{0.78} R_L^{\ 0.07}$$

A relationship between n and the PRF was developed based on the information from Table 1 in Meadows and Ramsey (1991). Also based on Table 1, 1.5 and 5.0 were set as the lower and upper limits of n. The corresponding limits on PRF were approximately 150 and 500. The equations for computing PRF based on n are:

for n ≤ 2.644 - $PRF = 278.03 \ln(n) + 43.584$

for n > 2.644 - $PRF = 303.44 \ln(n) + 14.08$

The unit hydrograph ordinates are computed as:

$$UHC = q_p \left[\frac{t}{t_p} \exp\left(1 - t/t_p\right) \right]^{(n-1)}$$

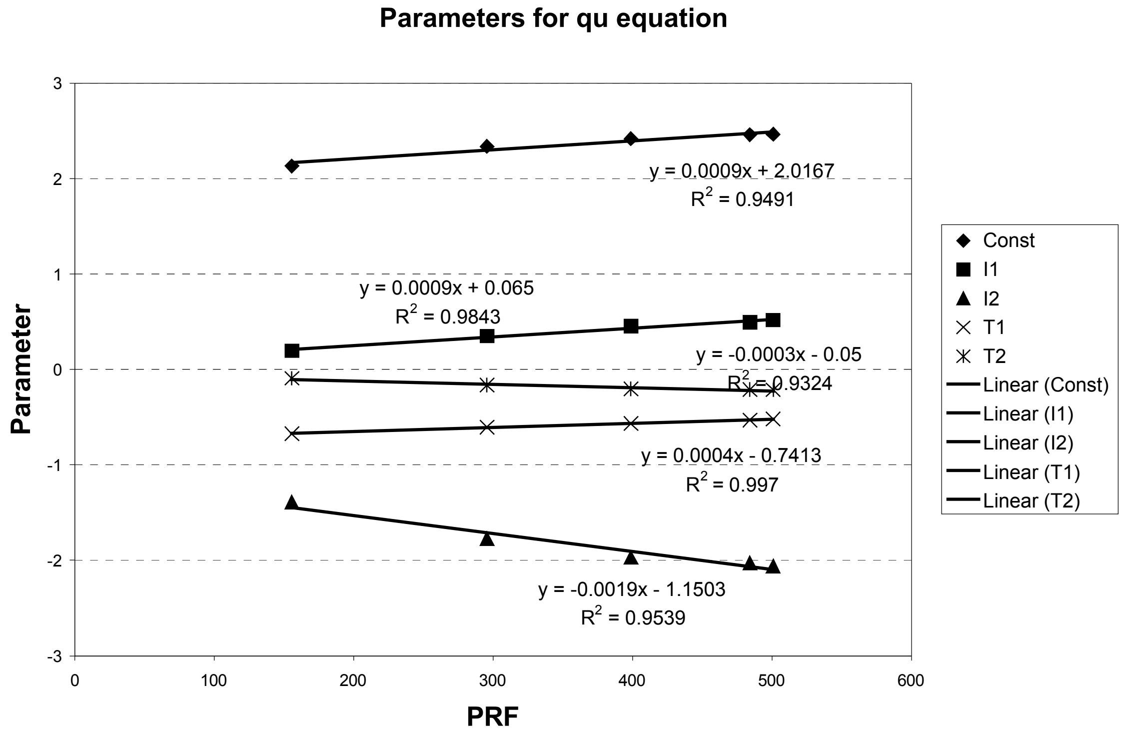
Horton order ratios were selected to produce the desired PRFs to cover the range of values between 150 and 500. The SEDIMOT III model was run with various combinations of rainfall, curve number, area, subwatershed overland length and slope, etc. Due to rounding of the Horton ratios to two or three decimal places, the actual PRFs used were: 155, 295, 398, 483.5, and 500.

Following the NRCS TR-55 model, the watershed parameters Ia/P, time of concentration (T_c) and peak discharge factor, q_u were computed. It was assumed that I_a = 0.2S. The peaking factor was in units of cfs/(mi^2-inches of runoff).

For an individual value of PRF, a model having this form was estimated:

$$\log(q_u) = const + I_1 (I_a/P) + I_2 (I_a / P)^2 + T_1 \log(T_c) + T_2 (\log(T_c))^2$$

The constant, I_1, I_2, T_1, and T_2 were estimated using the data set for each PRF. These values were plotted against the PRF values, and linear equations were developed to predict the parameters based on PRF.



The plots show that the lines are a good fit to the points, and the R^2 values are very good, i.e., greater than 0.93. These models were applied to the data generated using SEDIMOT III to find the parameters, then the log(qu) equation was applied to find the peak rate of runoff from the subwatershed. The root mean square error (RMSE) in qu and in peak runoff were determined for each of the data sets. The data sets included between 450 and 700 records. The table below gives the RMSE.

PRF	RMSE in: qu, cfs/(in-mi^2)	Peak Q, cfs
155.76	64.64	4.89
295.63	95.43	4.06
398.78	151.48	6.14
483.56	220.33	13.40
500.86	235.74	16.96

The errors were evaluated, and it was observed that a relatively small number of predicted values of q_u and Q_{peak} had very large errors. It was determined that these errors were only found when $I_a/P < 0.25$ *and* $\log(Tc) < -1.7679(I_a/P) - 0.0502$. Both conditions had to apply. Screening out those data points improved the RMSE in peak flow considerably:

PRF	RMSE in Peak Q after screening
155.76	2.38
295.63	1.74
398.78	1.39
483.56	5.36
500.86	7.01

However, using the criteria, there were numerous data points screened out that did not have high errors. The recommendation is to use caution in accepting peak flow results when both constraints apply. Note that, due to some differences in the way SEDIMOT III routes flows, etc., we would not expect the results for PRF = 484 to match TR-55 results exactly, but at the same time, they should not be widely different. The figure below shows the TR-55 and SEDIMOT III results.

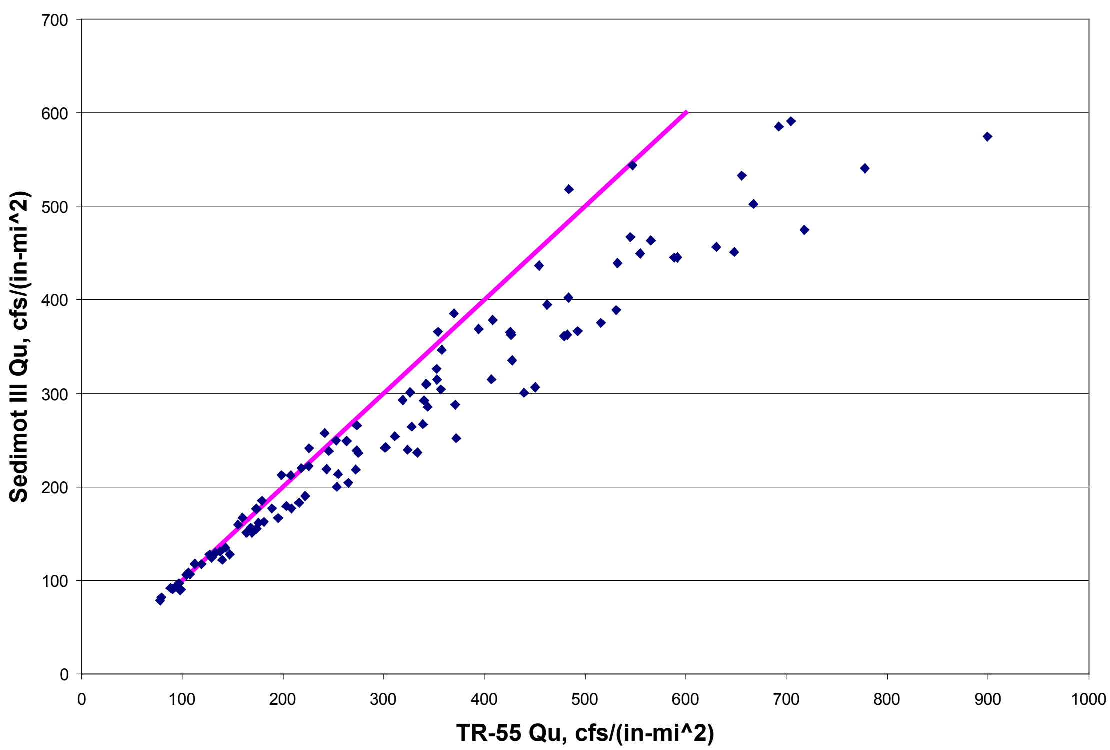


Figure 2 of Meadows Volume III suggests that adjusting Q_u for PRF is a function only of PRF. This was found not to be the case, based on the SEDIMOT III data. The adjustment is also a function of I_a/P. It is possible to produce similar plots, but they have to be for specific values of I_a/P.

Model for Ratio of Peak Flow at Subwatershed Outlet to Peak Flow at Watershed Outlet

A model was developed to predict the ratio Q_{po}/Q_{psw}, where Q_{psw} is the discharge at the outlet of the individual subwatershed, and Q_{po} is that discharge routed to the outlet of the watershed. The data set for developing this model was created by using the combinations of subwatershed data used for the peak runoff model, then having SEDIMOT III route the hydrograph at the subwatershed exit through a channel to the watershed outlet. The channel length, slope, size, and roughness were varied. The SEDIMOT III model was revised to use the Modified Att-Kin routing method.

A general form for the model for the ratio Q_{po}/Q_{psw} was assumed to be

$$Ratio = 1 - e^{-K}$$

The data were examined to determine what constituted the best predictors for K. K was determined to have the strongest relationships with the travel time down the channel (T_t), the peak flow at the subwatershed outlet (Q_{psw}), and I_a/P. For preliminary development to determine the best mathematical form, the data for PRF equal to 484 were used. There was a lot of scatter in the data, and the relationships between K and the identified input parameters were not nearly as strong as the relationships seen for the subwatershed peak flow model.

The maximum ratio found in the data set was 1.0. Points having this ratio had to be excluded from the estimation process, since a K value that would produce a ratio of 1.0 does not exist. However, there were a great many data points with ratios very close to 1.0 (0.9999...), so this was not considered a problem.

A simple power model for K was found to work the best:

$$K = 1.1155 T_t^{-.6199} Q_{psw}^{.0376} (Ia/P)^{.0144}$$

The data for PRF equal to 484 were split in half. One half of the data was used to estimate the model, and the other half was used to test. The RMSE in the ratio (not in K) was 0.135. The model was then applied to the data for other values of PRF and it was determined that PRF was not a factor in this model based on the RMSEs. The table below gives the RMSEs.

PRF	RMSE in Ratio
155.76	0.176
295.63	0.134
398.78	0.129
484	0.135