Design and Maintenance of Forest Road Drainage

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PEAK FLOW ESTIMATION AND STREAMFLOW SIMULATION FOR SMALL FORESTED WATERSHEDS

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INTRODUCTION

Flow prediction or the relationship between precipitation and streamflow is at the very core of the profession of hydrology. In our contemporary society there is considerable need for flow prediction from the development and design of water resource systems for municipal and industrial water supply, crop irrigation, flood protection, to instream flow needs, and many other uses and demands. The different uses of streamflow requires that many different streamflow characteristics be estimated. These characteristics include peak-flows, annual water yields, low-flows, and stormflow volumes among many others. Forest road drainage is not different in that there is a need to use the principles of statistical estimation to predict streamflow characteristics for small forested watersheds. For live stream crossings, there are two streamflow characteristics for which quantitative flow predictions are needed and these are:

1) The maximum instantaneous discharge or peak flow for an installation for a given design return period flow.

2) A range of discharges which represent the flows during which there is the highest likelihood that fish will be moving through the installation.

For small forested watersheds the task of flow prediction is made more difficult by two factors. First of all, unlike large watersheds which have the inertia of large amounts of water, small watersheds tend to be quite "flashy" and respond rapidly to changes in precipitation. Small forested watersheds tend to be the most "flashy" due to the high efficiency of the streamflow delivery system. As a result, the flow from small forested watersheds tends to be highly variable in time as well as in space. Also, the database for statistical estimation of flows from small forested watersheds is woefully inadequate.

FLOW PREDICTION MODELS

In the discipline of engineering hydrology there are, in general, three types of models used to predict flows. These are physical models, deterministic models, and empirical models. A physical model is a scaled physical replica of a project which is subjected to rainfall or streamflow conditions representative of desired design conditions and then the response of the model is observed. Physical models are predominantly used for large civil projects such as dams, spillways, and bridges and they are only rarely used in forest hydrology. An example of the use of physical models in forested streams is some recent flume studies to quantify the influence of different orientations and sizes of large woody debris on local streambed scour.

In recent years, deterministic models have come to mean complex mathematical descriptions of the hydrologic cycle. These models attempt to simulate hydrologic events by mathematically simulating watershed processes. Currently, their solution is only possible using computers. Most deterministic models represent different processes in the hydrologic cycle with formal mathematical expressions, often differential equations. However, the hydrologic cycle is not so well understood that every process can be mathematically represented. Therefore, in large, conceptual, watershed models many processes are represented by empirical expressions. An example of a large, conceptual, watershed model is the Stanford Watershed Model (SWM). A schematic of the Stanford Watershed Model is shown in Figure 1. Large, watershed models such as the SWM are seldom used for anything but research on forested watersheds. They are not used for routine management because of their intense data requirements. First of all, there is not an adequate research database to mathematically represent all the hydrologic processes in a forested watershed. An example of a current information gap is the role of macropore flow in stormflow generation. Secondly, most deterministic models need prohibitive amounts and quality of data to be initiated and run. This level of data has generally not been available for forested watersheds on a management level.

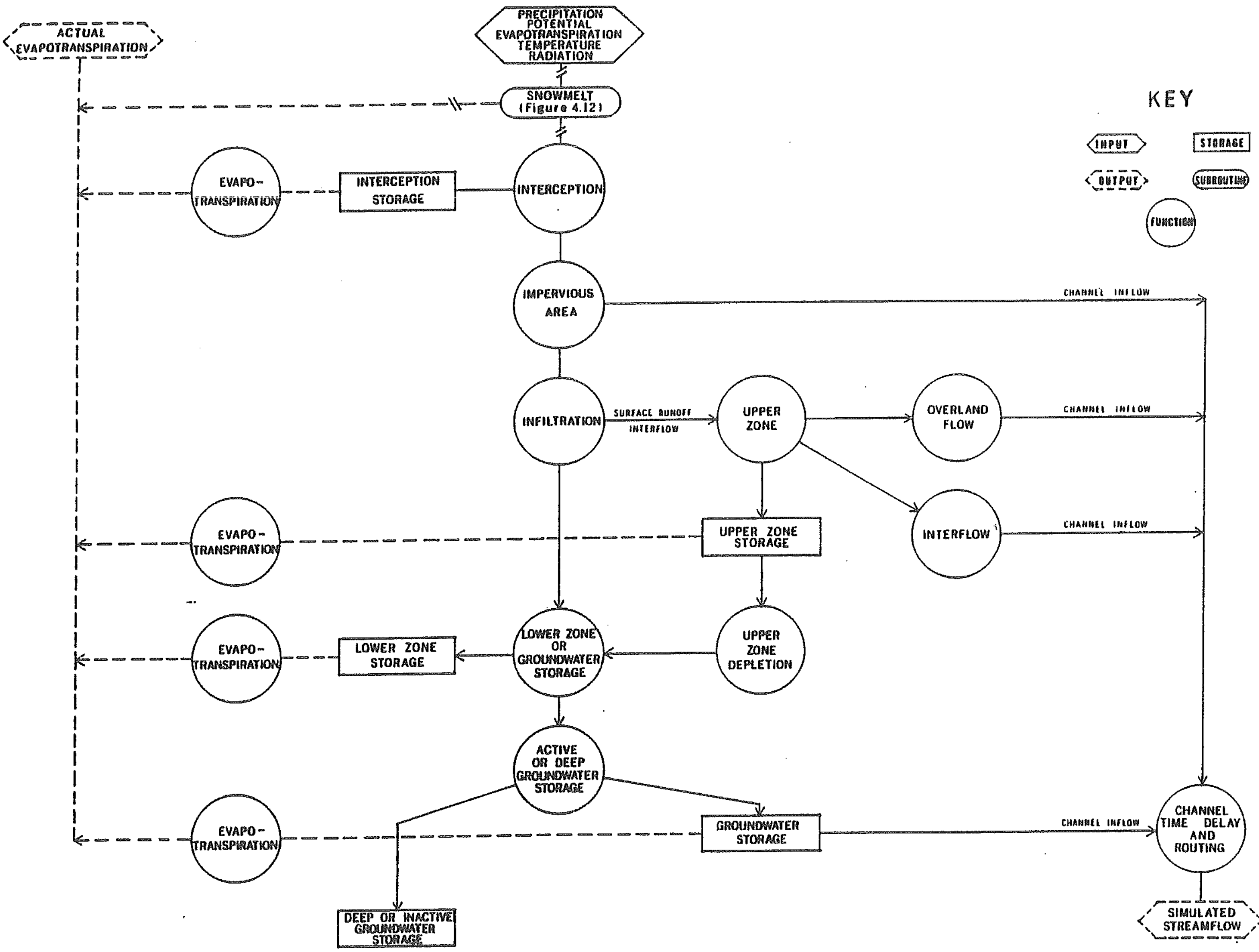

Figure 1. A schematic showing the flowchart of the Stanford Watershed Model IV.

However, there is still a large and growing need for streamflow information from ungauged watersheds and this demand has lead to the development of empirical models. Empirical models correlate streamflow characteristics with watershed characteristics or precipitation. Most empirical models have a probabilistic base because they use probabilistic data, such as precipitation, for input. This type of flow prediction model is by far the most widely used in the forested environment and the types of empirical models available and the use of the models is increasing. Empirical models have the potential to be even more widely used because the data they require both to develop and use is consistent with the types of data that are, for the most part, available in a forested setting. The two primary tools that will be presented for estimating peak flows and synthesizing streamflow records are empirical models based on watershed characteristics and precipitation.

Recommended Flow Prediction Methods

For the session today, we will present tools for both estimating peak flows and synthesizing streamflow records. For the situation where streamflow records are available, peak flows can be estimated using flow-frequency analysis. Even though this situation is very rare and the actual techniques of frequency analysis may never be used, this technique is the basis for the estimation of many precipitation and streamflow characteristics. Therefore, the rational and underpinnings of the technique need to be understood. In the situation where a watershed of interest is in close proximity to a gauged watershed, the streamflow record or other hydrologic data can be directly transferred from a gauged to an ungauged watershed and streamflow estimates can be determined in that manner. The direct transfer of hydrologic data between hydrologically similar watersheds will be covered.

Finally, there is the situation that is encountered most frequently which is the need for streamflow information on an ungauged, forested watersheds. Two models will be presented for estimating flows for that situation. Peak flows can be estimated using Campbell's equations which correlate peak flows for a given design return period with watershed characteristics. Also, streamflow can be synthesized using the Antecedent Precipitation Index (API) model which requires a record of precipitation intensity and watershed area as input.

Flow Prediction Methods Not Recommended

There are several flow prediction techniques that are not being included in this discussion and by their omission are <u>not</u> recommended for use in small, forested watersheds. There are three methods, Talbot's formula, the Rational Rule, and the SCS Method that are conspicuous by their absence because they have long been the standard, accepted methods for flow prediction in these types of watersheds. All three methods will be presented briefly followed by a short discussion of why they are not considered appropriate for small, forest watersheds.

Talbot's formula is an empirical expression which comes from the midwestern United States and dates back to the late 1880's when virtually nothing was known about hydrology or hydraulic design. The formula takes the form:

$$a = C(A_d)^{\frac{3}{4}}$$

where a = culvert opening in ft^2; C = a coefficient, ranging from 1.0 for steep, rocky ground through 0.6 for hilly country of moderate slopes, to 0.2 for level terrain not affected by snow; and, A_d = drainage area in acres. The formula assumes a unique flow for a given end area opening, an assumption which will be treated in some detail later in this session. Further, the formula gives no opportunity for probabilistic assessment of peak flows except through the coefficient and to my knowledge that treatment has never been undertaken. The Talbot formula has little scientific verification and its widespread use is attributed to its simplicity and, for a long time, the lack of anything better. It is not recommended for peak flow estimation in small, forested watersheds.

The Rational Rule is an empirical relationship that is still widely used in engineering hydrology for, among other uses, highway and sewer design. The method assumes that rainfall of a uniform intensity covers the contributing area and runoff increases until the whole area is contributing equally. At this point the runoff is the proportion of rainfall not infiltrating into the soil. The expression takes the form:

$$Q = C\,I\,A$$

where Q = peak runoff rate in cfs; C = a coefficient ranging from 0.9 for concrete and pavement to 0.1 for woodlands; I = rainfall intensity in inches/hour; and, A = drainage area in acres. The model is an infiltration limited model, therefore it is not consistent with the processes occurring in forested watersheds. A probabilistic assessment of peak flows can be realized if the proper design storm intensity is used however, the method requires that the precipitation intensity used represent the design return period intensity for the time of concentration of the contributing area. Therefore, the method requires knowledge of time of concentration values and precipitation intensity-duration curves for small, forested watersheds. Time of concentration has been studied, somewhat, for agricultural watersheds but the concept has only been borrowed for forested watersheds and has never been rigorously investigated. The precipitation intensity-duration database for forested areas is predominantly for cities at low elevations and does not represent higher elevations in forested watersheds. With knowledge of these shortcomings, the model can be made to preform satisfactorily for forested watersheds if properly calibrated. However, without the knowledge of how to adjust the input variables, it is not recommended for peak flow estimation for small, forested watersheds by this audience.

The Soil Conservation Service (SCS) Method is a precipitation driven streamflow simulator which was developed and calibrated for small, agricultural watersheds. The method uses a simplified infiltration limited model of runoff coupled with a empirical ranking of soils into curve numbers due to hydrologic response, antecedent moisture, cover, and land use practices. The curve numbers are used to generate synthetic unit hydrographs which are summed over time to yield a simulated storm hydrograph. The method has many of the problems that the previous two methods have that invalidate them for use in forested watersheds. First of all, the infiltration limited runoff mechanism is at odds with the known processes in a forested watershed. Secondly, forest soils have never been properly and adequately calibrated for SCS curve numbers. Finally, the shape of the synthetic unit hydrograph has to be adjusted to account for runoff conditions from forested watersheds. In general, these problems invalidate the method for use by the casual user. However, like the Rational Rule, it was believed that the proper individual, who understood forest watershed runoff processes and the SCS method, could make it work for forested watersheds by adjusting soil curve numbers, watershed lag time, and unit hydrograph shape. Fedora (1987) in his Master's Thesis rigorously evaluated the SCS method for small, forested watersheds in the Oregon Coast Range. He found that even when the curve numbers, watershed lag, and unit hydrograph shape were adjusted or fitted, the performance of the method was very poor. Adjusting the coefficients can not make up for the fact that the basic assumption of the model, that runoff is a constant function of precipitation, make the method terribly ill-suited for the complex storm patterns of coastal Oregon. For this reason the SCS curve number method is not recommended for peak flow estimation or streamflow simulation for small, forested watersheds.

FLOW FREQUENCY ANALYSIS

Flow frequency analysis is the technique of using the principles of statistical estimation to make inferences about the total population of streamflows from some finite streamflow record. Frequency analysis techniques can be used for any streamflow parameter of interest. We have already discussed using the technique for the analysis of precipitation intensity data. The most common use of the technique is for the analysis of annual peak flows which will be discussed today. But please realize that this technique is a basic statistical estimation technique that can be used for droughts, low flows, stormflow volumes, annual precipitation, and almost any streamflow or precipitation parameter of interest.

In general, the technique is no different from any other statistical estimation technique. The first step is to define the population of interest. Then a subset of the population is sampled and the parameter of interest is measured. A frequency distribution is fitted to the sample data and then the sample data is used to calculate the parameters that describe the population such as mean and standard deviation. Once the population parameters have been estimated, inferences about the population can be made with varying and known degrees of certainty. This is the basic process taught in introductory statistics classes. The big difference for flow frequency analysis is the frequency distribution that is used to describe the shape of the population. In introductory statistics, the only frequency distribution that is discussed at any length is the normal distribution. Hydrologic data in general and flow data in particular never have a normal distribution but are skewed and have what is called an extreme value distribution.

The technique of flow frequency analysis will be illustrated by working through an example from Flynn Creek. The example will be for an annual series of peak flows. The data that will be used is shown in Table 1. Please note that the data is discontinuous and comes from three different time periods. The first time period is the 15 years of record from 1958 to 1973 during the Alsea Watershed Study. The second time period is four years of record from 1977 to 1980 when Dr. Beschta was doing in-stream bedload research at Flynn Creek. The final data point is from 1990 after the gauging stations had been reopened to do subsequent long term water yield monitoring on the watersheds from the Alsea Watershed Study. The point is that a streamflow record does not need to be continuous to be of value. The annual series of peak flow data is simply a sample and the 19 data points that have been captured represent a sample of the total population. Their value as data points is in no way compromised by the fact that the streamflow record is discontinuous.

Once the sample data has been collected, the next step is to divide the data into appropriate flow classes and plot the frequency of peak flows in the flow classes versus the magnitude of the classes. The resulting shape of the distribution should govern the selection of a theoretical frequency distribution which approximates the observed frequency distribution of the sample. There are several frequency distributions that can be used. Four frequency distributions will be mentioned. They are:

1. Normal distribution
2. Log-normal distribution
3. The Gumbel extreme-value distribution, and
4. The Log Pearson Type III distribution

The normal distribution will not ever be used for flow frequency analysis because it is not an extreme value distribution. However, the example data set will be worked both graphically and analytically using the normal distribution so that the concept can be illustrated with a familiar frequency distribution. Hopefully, the addition of the normal distribution to this exercise will make the concept easier to grasp. The other three distributions are all extreme value distributions and are routinely used for flow frequency analysis. The only criteria for choosing any one distribution over the other two is convenience or goodness-of-fit.

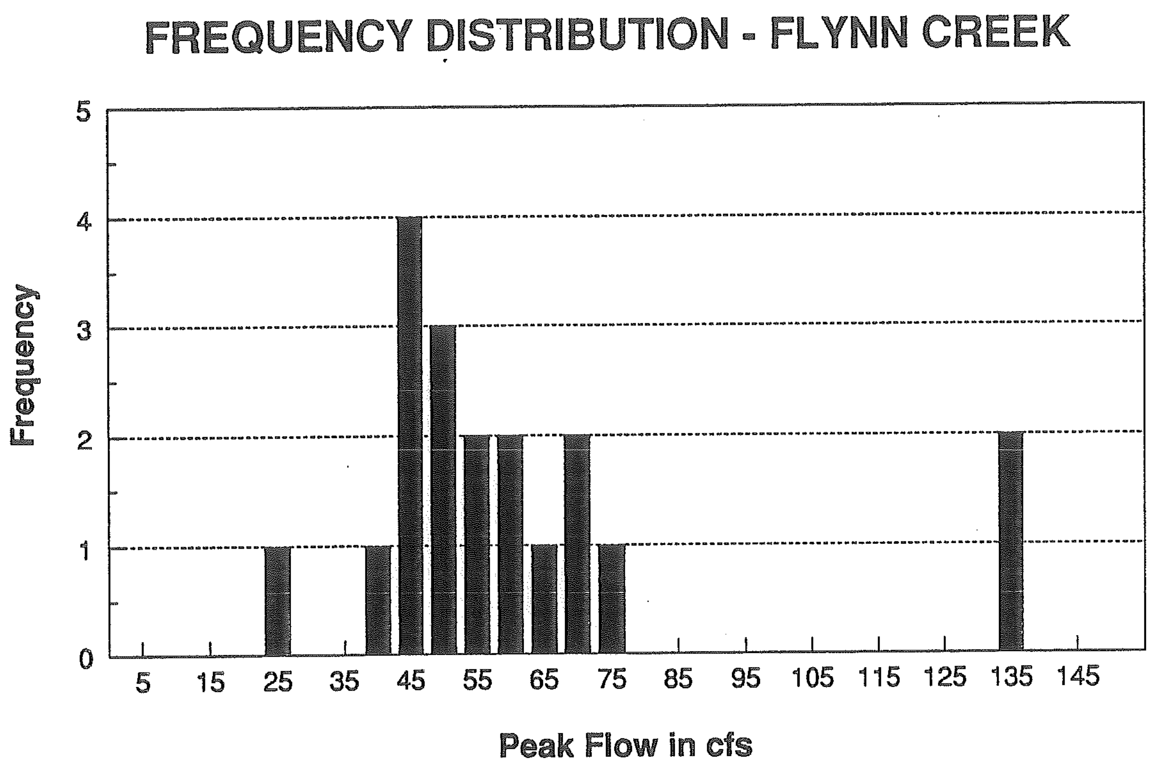

Figure 2. A graph showing the frequency distribution of annual peak flows for Flynn Creek in the Alsea Basin, Oregon.

A frequency distribution is shown for the Flynn Creek data in Figure 2. However, this is not the form that most people expect to see flow frequency data in. The form that an annual series of peak flow data is usually shown in is a cumulative frequency distribution. A cumulative frequency distribution of the same data is shown in Figure 3. There are two ways to estimate the magnitude of different return period floods from frequency distribution data. One way, the graphical method, is relatively easy but is neither terribly accurate nor precise and the other method, the analytical method, is much more cumbersome but more precise and probably more accurate.

Graphical Method

The graphical method involves simply plotting the magnitude of the peak flow against either the probability and/or return period of that flow on the appropriate probability graph paper. If the theoretical frequency distribution chosen for the sample data approximates the correct frequency distribution, then the data will form a straight line on the graph paper. The straight line is the cumulative frequency distribution and the appropriate peak flow estimates can be read directly off the graph paper. Before the data can be plotted however, some initial data preparation must be carried out to determine plotting positions. The first step is to rank the data with highest magnitude flow being given a rank of m=1 and the lowest magnitude flow being given a rank of m=n, where n is the number of records.

Then the probability and/or return period associated with each peak flow is calculated using the relationship,

Table 1. Flynn Creek annual peak flow series and example calculations for plotting positions.

			Plotting Position	
Water Year	Peak Flow cfs	Rank m	Probability of Exceedance, %	Recurrence Interval, yrs
1959	53	11		
1960	43	18		
1961	78	3		
1962	46	15		
1963	65	6		
1964	63	7		
1965	137	2		
1966	73	4		
1967	70	5		
1968	45	16		
1969	45	17		
1970	50	13		
1971	58	10		
1972	139	1		
1977	25	19		
1978	61	8		
1979	59	9		
1980	51	12		
1990	48	14		
	$\overline{Q}$ = 64 σ_Q = 29			

n = 19

$$p = \frac{m}{n+1} \qquad or, \qquad T_r = \frac{1}{p} = \frac{n+1}{m}$$

where, p is the probability of occurrence, T_r is the return period, m is the rank of the peak flow, and n is the number of years of record. Please note that the probability of a peak flow occurring is the inverse of its return period and vice versa. The calculation of plotting position for the Flynn Creek data will be left as a class exercise.

Once the plotting positions have been calculated, the peak flows can be plotted on the appropriate graph paper. The magnitude of the peak flow is plotted versus its probability of occurrence or return period which ever is appropriate for the graph paper. Three types of graph paper are provided for plotting the Flynn Creek data. The first is normal probability paper which will plot the data as a normal distribution. This is provided as an opportunity to view how the data looks with an inappropriate frequency distribution. Also, provided for plotting the Flynn

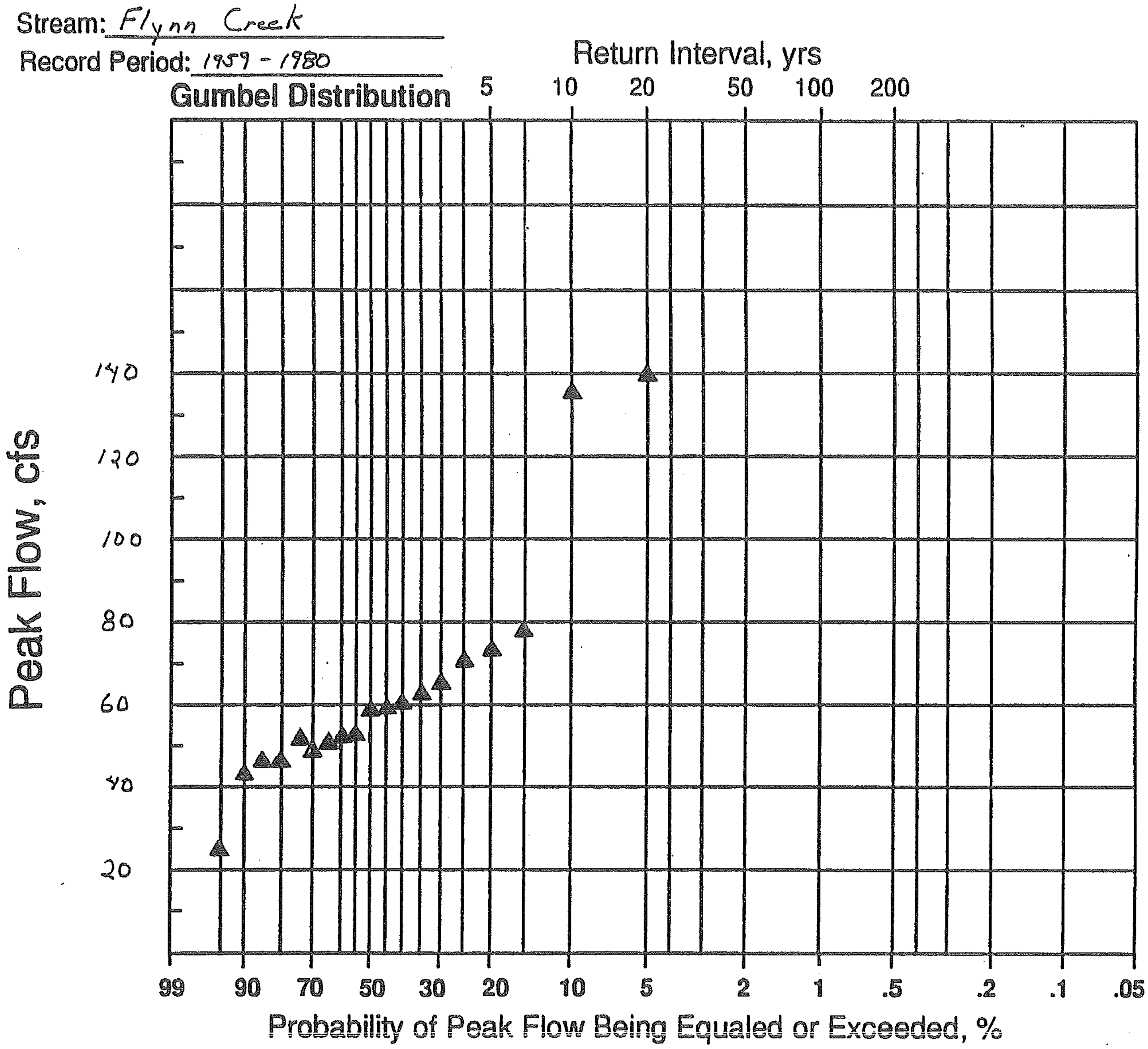

Figure 3. Cumulative frequency distribution for the annual peak flow data from Flynn Creek in the Alsea Basin, Oregon.

Creek data is a sheet of log-normal probability paper and a sheet of Gumbel probability paper. The Gumbel distribution is a commonly used frequency distribution for extreme value populations. It is so popular and commonly used for this purpose that its own graph paper has been developed. The log-normal distribution is also a common way to treat extreme value distributions. Once the data has been plotted, a linear "best fit" can be put in by eye on both of the extreme value plots. At this point the mean of the population, the standard deviation, and estimates of the magnitude of different return period peak flow events can be read right off the graph paper. Operationally, flow frequency analysis can be done at this level for estimates of different return period peak flows.

Analytical Method

If graph paper is not available or if you wish to use a frequency distribution that doesn't have a graphical solution, the estimates of peak flows for designated return periods can be solved analytically. The solution follows the form,

$$Q_p = \overline{Q} + \sigma_Q \cdot y$$

where, Q_p is a peak flow of specified probability, p, $\overline{Q}$ is the mean of the peak flow series, σ_Q is the standard deviation of the peak flow series, and y is a frequency factor for the probability p for the particular frequency distribution being used.

The solution of this equation is easiest to visualize for the normal frequency distribution. In this case the parameters for the line take the form of the mean and standard distribution that we are all comfortable with.

$$\overline{Q} = \frac{\Sigma Q}{n}$$

$$\sigma_Q = \sqrt{\frac{\Sigma(Q-\overline{Q})^2}{n-1}}$$

The values for y are the area under the normal distribution curve for given probability. These values are listed in Table 2. The predictive equation follows the form shown above.

Table 2. Frequency factor y for the Normal Distribution.

	Recurrence Interval, years				
	5	10	25	50	100
	Probability, %				
	20	10	4	2	1
Frequency Factor, y	0.8418	1.2817	1.7511	2.054	2.3267

The log-normal distribution is the same as the normal distribution except the log of the peak flows are used instead of the actual values. The parameters then become,

$$\overline{\log Q} = \frac{\Sigma \log Q}{n-1}$$

$$\sigma_{\log Q} = \sqrt{\frac{(\log Q - \overline{\log Q})^2}{n-1}}$$

The values of y for this distribution are the same as for the normal distribution and are listed in Table 2. The predictive equation follows the form,

$$\log Q_p = \overline{\log Q} + \sigma_{\log Q} \cdot y$$

The Gumbel extreme value type 1 distribution follows the same form as the normal distribution. The mean, $\overline{Q}$, and standard deviation, σ_Q, of the peak flow data are used in the peak flow equation. The difference is that the standard deviation is multiplied by a frequency factor for the Gumbel distribution which is a function of p. The form of this equation is,

$$Q_p = \overline{Q} + \sigma_Q \cdot K_p$$

The K values for the Gumbel distribution are listed in Table 3.

The final distribution for discussion is the Log Pearson Type III distribution. In the predictive equation for this distribution the logarithm form of the parameters are used for mean, $\overline{\log Q}$, and standard deviation, $\sigma_{\log Q}$. The equations for these terms have been given above. The form of the predictive equation is,

$$\log Q_p = \overline{\log Q} + \sigma_{\log Q} \cdot K_p$$

where K is a function of both p and a skew coefficient, C_s. The equation for the skew coefficient is,

$$C_s = \frac{n \Sigma (\log Q - \overline{\log Q})^3}{(n-1)(n-2)(\sigma_{\log Q})^3}$$

where all the terms have been previously defined. Table 4 shows a list of K frequency factors for the Log Pearson Type III distribution for different probabilities, p, and skew coefficients, C_s. Please note that a Log Pearson Type III distribution with a skew coefficient of 0 is identical to the log-normal distribution.

Table 3. Frequency factor K for the Gumbel Extreme Value Distribution

Recurrence	Probability	Record length, n					
Interval, yrs	%	20	30	40	50	100	200
5	20	0.92	0.87	0.84	0.82	0.78	0.76
10	10	1.62	1.54	1.50	1.47	1.40	1.36
20	5	2.30	2.19	2.13	2.09	2.00	1.94
50	2	3.18	3.03	2.94	2.89	2.77	2.80
100	1	3.84	3.64	3.55	3.49	3.35	3.27
200	0.5	4.49	4.28	4.16	4.08	3.93	3.83

Table 4. Frequency factor K for the Log Pearson Type III Distribution

Skew Coefficient	Recurrence Interval, years				
	5	10	25	50	100
	Probability, %				
	20	10	4	2	1
2.0	.609	1.302	2.219	2.913	3.605
1.0	.758	1.340	2.043	2.542	3.022
.6	.800	1.328	1.939	2.359	2.755
.4	.816	1.317	1.880	2.261	2.615
.2	.830	1.301	1.818	2.159	2.472
0	.842	1.282	1.751	2.054	2.326
-.2	.850	1.258	1.680	1.945	2.178
-.6	.857	1.200	1.528	1.720	1.880
-1.0	.852	1.128	1.366	1.492	1.588
-2.0	.777	.895	.959	.980	.990

It is unlikely that any of you will ever have to do a flow frequency analysis on a streamflow record. Most of you will spend your careers being faced with estimating peak flows for ungauged watersheds. However, a clear understanding of how flow frequency analysis works is critical to understanding the work that has gone into the regression equations for estimating peak flows. There are certain assumptions that are implicit when using principles of statistical estimation for peak flow data. These are that the data are random, independent, and homogeneous. Methods are available for checking the homogeneity of data. They are not presented here and rarely used because our peak flow data bases are so small that no data can ever be discarded. Likewise, we know they are neither random or independent. The data usually cover a short time span and are clumped together making them susceptible to the phenomena of persistence which means that historically wet years follow wet years and dry years follow dry years. Despite these facts we use statistical estimation techniques anyway. However, it becomes evident that terribly sophisticated statistical techniques are not justified where the data base is compromised by ignoring assumptions. This will be the case most of the time for small, forested watersheds. Therefore, for small, forested watersheds, use the simplest techniques available. For the vast majority of gauged forested watersheds there is no reason to get any more complicated than a graphical solution using log-probability paper for peak flow estimates in an operational, management setting.

CAMPBELL'S EQUATIONS

The single piece of hydrologic information that is needed most often for adequate design and maintenance of forest roads is design peak flows estimates for stream crossings. This information need greatly outweighs the need for synthetic streamflow records to design for fish passage because, especially in contemporary forest road layout, the vast majority of stream crossings will not be across fish bearing or even perennial streams. Current forest road layout, especially in steep forest terrain, results in a large number of stream crossings of small, ungauged, forest streams. In the past, the accepted and suggested methods for estimating design peak flows for these small streams were such methods as Talbot's formula, the Rational Rule, or the SCS method. However, as was stated earlier, these methods are no longer recommended for use in these watersheds. They have been replaced by a method, for Oregon, referred to as Campbell's equations. The method is named after the graduate student, Alan Campbell, who developed the equations as a part of his Master's program at the Forest Engineering Department, Oregon State University. The method is a set of regression equations which correlate different design peak flow estimates for small, forested watersheds in six physiographic regions in Oregon with watershed characteristics.

The method was developed by first dividing Oregon into six physiographic regions or regions of hydrologic similarity. The regions are the Coast, Willamette, Rogue-Umqua, Cascades, Blue-Wallowa, and Klamath regions and are shown in Figure 3. An annual series of maximum peak flows for 73 gauging stations in Oregon and seven in Northern California was developed. The area of the watersheds ranged from 0.21 to 10.6 square miles. The watersheds were all predominantly forested and the length of record ranged from 10 to 52 years with the majority of the stations having a record length of between 10 and 20 years. For each gauging station the Log-Pearson Type III frequency distribution was used to determine the magnitude of the 10-, 25-, 50-, and 100-year return period flows. Then the peak flows were related to different watershed characteristics using multiple regression. The watershed characteristics investigated were drainage area, mean basin elevation, gauge elevation, main channel slope, main channel length, percent forest cover, mean annual precipitation, 2-year 24-hour precipitation, mean minimum January temperature, latitude, and longitude. The most important variable influencing peak flow size is drainage area. The only other watershed characteristics that were important for peak flow estimation were mean basin elevation for the Coast Region and mean annual precipitation for the Cascade Region. All of the peak flow estimation equations for the six regions are listed in Table 5 and the range of variables for the equations are listed in Table 6.

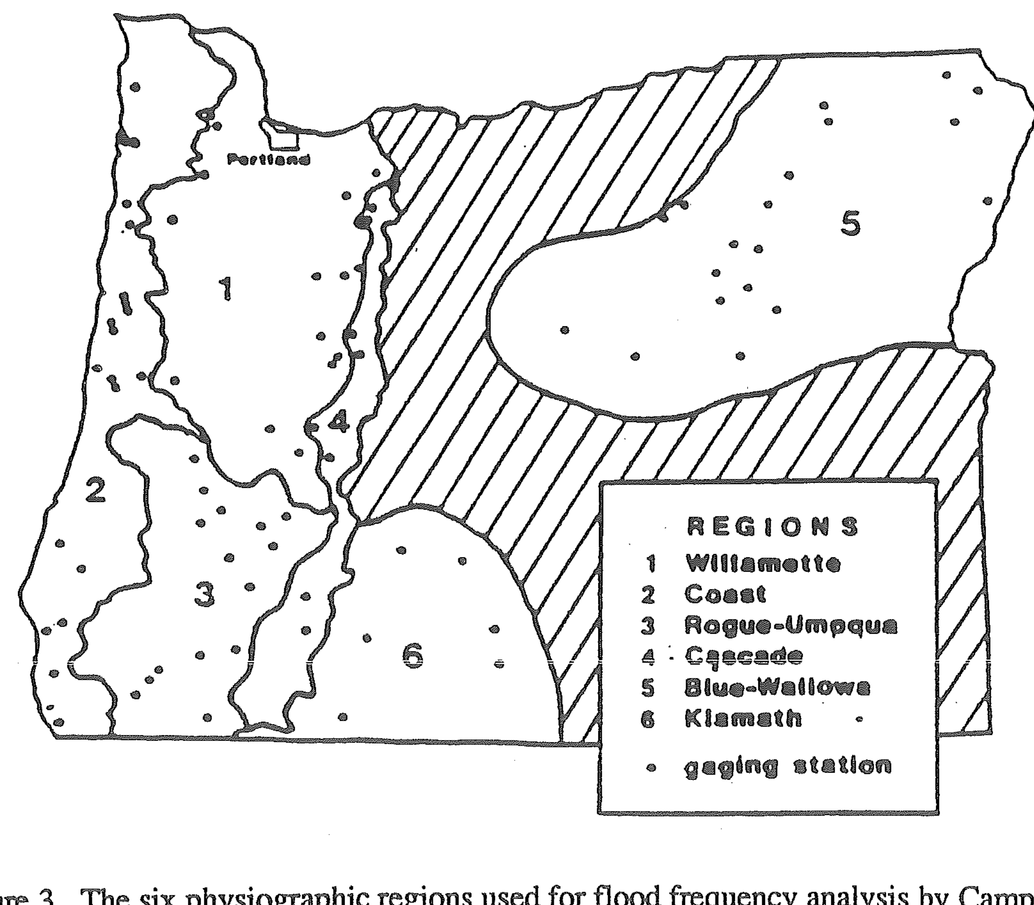

Figure 3. The six physiographic regions used for flood frequency analysis by Campbell. The cross-hatched area is left undefined because of a lack of suitable data base.

Table 5. Recommended prediction equations for peak flows in Oregon.

Drainage Basin Characteristics: A = Drainage basin area (mi^2); E = Mean basin elevation (feet); P = Mean annual precipitation (inches)

Equation	R^2	Average Error (%)	Standard Error (log_{10} units)
	WILLAMETTE REGION		
$Q_{10} = 124\ A^{.79}$	.83	23.3	.129
$Q_{25} = 156\ A^{.80}$	.87	23.9	.127
$Q_{50} = 183\ A^{.80}$	.87	23.9	.127
$Q_{100} = 212\ A^{.80}$	.86	24.1	.129
	COAST REGION		
$Q_{10} = 5.87\ A^{1.04}\ E^{.49}$	.83	25.7	.140
$Q_{25} = 6.31\ A^{1.01}\ E^{.51}$	.79	27.3	.155
$Q_{50} = 7.77\ A^{1.01}\ E^{.50}$	.79	26.1	.155
$Q_{100} = 8.40\ A^{1.00}\ E^{.50}$	.78	26.0	.161
	CASCADE REGION		
$Q_{10} = .010\ A^{.44}\ P^{2.15}$	.80	20.4	.143
$Q_{25} = .023\ A^{.44}\ P^{1.97}$	.86	16.1	.113
$Q_{50} = .063\ A^{.45}\ P^{1.87}$	.81	22.0	.132
$Q_{100} = .111\ A^{.46}\ P^{1.78}$	.71	26.9	.178
	ROGUE-UMPQUA REGION		
$Q_{10} = 125\ A^{.75}$	.37	62.7	.265
$Q_{25} = 163\ A^{.77}$	.46	52.8	.240
$Q_{50} = 191\ A^{.80}$	.50	48.6	.228
$Q_{100} = 221\ A^{.82}$	.53	46.9	.224
	BLUE-WALLOWA REGION		
$Q_{10} = 46.7\ A^{.46}$	.39	62.7	.265
$Q_{25} = 67.6\ A^{.47}$	.46	52.8	.240
$Q_{50} = 85.2\ A^{.48}$	.50	48.6	.228
$Q_{100} = 105\ A^{.50}$	.53	46.9	.224
	KLAMATH REGION		
$Q_{10} = 30.8\ A^{.70}$	.42	62.5	.332
$Q_{25} = 41.9\ A^{.79}$	.56	51.7	.282
$Q_{50} = 54.5\ A^{.77}$	.59	47.2	.257
$Q_{100} = 69.6\ A^{.75}$	.61	64.1	.241

To use the equations, simply determine the physiographic region that your watershed is in then determine its area. If the watershed is in the Coast or Cascades Region, then mean basin elevation or mean annual precipitation, respectively, of the watershed needs to be determined.

Table 6. Range of variables in final prediction equations.

Region	Area (km^2)	Precipitation (cm)	Elevation (m)
Willamette	0.96 - 13.44	---	---
Coast	0.75 - 6.62	---	79 - 860
Cascade	0.54 - 20.72	127 - 224	---
Rogue-Umpqua	1.95 - 16.63	---	---
Klamath	2.51 - 27.45	---	---
Blue-Wallowa	0.67 - 17.95	---	---

Find the appropriate formula for the desired return period flow and simply calculate the estimated peak flow. Be aware that the equations are available in both English and metric forms so make sure the units are consistent and make sense.

In subsequent research updating Campbell's equations, a sub-region in Southwest Oregon and Northern California was found that has peak flows much greater that in the rest of the Coast Region. In this sub-region, peak flows computed for actual flow data averaged twice the predicted values using Campbell's equations and 25 year return period peak flows averaged twice as large, on a per unit area basis, for the eight gauged streams than for the rest of the region. Campbell's equations are not recommended for this sub-region. In Southwest Oregon and Northern California , peak flow estimates should be determined by direct transference of peak flow data from the closest of the eight gauging stations used. This information is available in WRRI Water Note 1989-1 by Andrus, Froehlich, and Pyles.

The information that has been presented so far concerning Campbell's equations for peak flow estimation is directed specifically at forested regions in Oregon. While the specifics of other equations will not be presented here, this same method for developing peak flow estimation equations has been used for many other regions. Ott Water Engineers (1979) and Waananen and Crippen (1977) have carried out the same analysis and developed equations for estimating peak flows for Northern California and the entire state of California, respectively. The USGS has developed peak flow estimation equations for most, if not all, of the United States for large watersheds and this information is available in USGS Water-Supply Papers. With current PC technology and database availability, this type of project is not an onerous task and there are rumors of individual hydrologists for different agencies developing such equations on an informal basis for local areas. If all else fails and you don't have access to such equations and feel their development would be beneficial for your area, talk to the Forest Engineering Department and fund a graduate student. The work can be done very easily and you will have helped out a student.

FLOW TRANSFERENCE

Streamflow records or estimates of streamflow characteristics can be transferred directly from gauged watersheds to ungauged watersheds if the watersheds are in close proximity and are hydrologically similar. Hydrologic similarity is an ambiguous concept and there are no hard and fast rules regarding it, however, two watersheds can be considered hydrologically similar if they:

1) Are within the same meteorological regime.
2) Have the same physical and biological characteristics such as soils, geology, relief, shape, drainage density, and vegetation.
3) Are approximately the same size, that is within one order of magnitude.

Once the watersheds are determined to be hydrologically similar, the actual transfer of hydrologic information is quite easy. The simplest method of direct transfer is by adjusting streamflow records by differences in watershed area. For example, the process can be as easy a multiplying the streamflow record of the gauged watershed by the ratio of the ungauged to gauged watershed areas. Streamflow records can also be adjusted by differences in watershed elevation as well as mean annual precipitation which are both indicators of, potentially, a higher rainfall regime. Like adjusting for watershed area, the process would be to multiply the streamflow record by the ratio of the ungauged to gauged mean watershed elevation or mean annual precipitation. However, the assumption of hydrologic similarity should not be pushed too far. The method is only good for rough approximations, at best, and the more adjustment that is needed for the transfer of the hydrologic information the less dependable is the information.

Direct flow transference can be used to transfer entire streamflow records, making it a technique for streamflow simulation, or only estimates of streamflow characteristics can be estimated. For example, flow-frequency analysis can be carried out for the original streamflow record and peak flows for the traditional return period floods can be computed. Then only the peak flow estimates can be transferred by adjusting them by area. Or the entire flood-frequency curve for an ungauged watershed can be estimated by adjusting curves developed from a hydrologically similar watershed. An example of direct flow transference is Campbell equation's which are simply a method of direct flow transference of peak flow estimates within a region considered hydrologically similar.

ANTECEDENT PRECIPITATION INDEX (API) STREAMFLOW SIMULATOR

In addition to passing the peak flow of a design return period flood, a stream crossing culvert must also be able to allow for fish passage. There are several problems that arise with this responsibility. One of the problems is determining the limiting flow depth and velocity for the different times of the year for different life stages of different species of fish. We have had a speaker address the complexity of this problem and the nature of the available data. However, given that this data can be ascertained for an installation, the next step is to determine whether these limiting values can be met or exceeded, in other words whether or not fish passage can be assured, for some given flows. The question is what are these flows. Obviously, fish passage will not have to be assured for peak flow conditions, but at some lesser intermediate flows. The problem is determining the magnitude of those intermediate flows for which fish passage will have to be assured for a given installation.

At the time of this writing, we simply don't know how to determine the range of flows for an installation for which fish passage must be assured. We do know however, that once these flows are knows, we will need a fairly complete streamflow record from which the magnitude of the desired flows can be determined. For gauged watersheds or for an ungauged watersheds in close proximity to gauged watersheds for which direct transference can be used, obtaining a fairly complete streamflow record is no problem. In both of these cases a streamflow record exists or can be estimated fairly quickly. The problem that occurs is just like the problem for design peak flows, the majority of the forested watersheds are ungauged and remote from gauged watersheds. What is needed is a tool to synthesize streamflow records for ungauged watersheds. The observation has already been made that for large, deterministic watershed models, the data demands are too excessive to be of use in an operational management setting. For years the recommended tool was the SCS method which, as discussed earlier, is no longer recommended because of its deficiencies in streamflow synthesis for small, forested watersheds. To fill this gap a simple, "black-box" streamflow simulation model has been developed that needs only watershed area and a record of precipitation intensity to be initiated and run. The model is the Antecedent Precipitation Index (API) method and it was developed by Mark Fedora as a part of his Master's program for the Department of Forest Engineering, Oregon State University. The model has been tested for coastal Oregon watersheds (Fedora & Beschta, 1989) and for estimating peak flows and stream level in Hawaii and India, respectively.

The actual streamflow prediction model is a very simple linear regression model between streamflow and the antecedent precipitation index (API). Fedora found that the best "fit" when correlating streamflow to API was to use the square root of the streamflow, therefore the form of the actual regression equation is:

$$\sqrt{Q_t} = b_0 + b_1 \cdot API_t$$

where b_0 and b_1 are the regression coefficients. Solving the above equation for Q_t gives the actual predictive equation

$$Q_t = (b_0 + b_1 \cdot API_t)^2$$

It is important to remember that even though the above equation appears to be quite simple compared to the complexity of the large, deterministic watershed models, the API method is empirically derived. Therefore, the regression coefficients contain the same influence over the streamflow simulation process that the many watershed parameters and subroutines contain for the larger, more complex watershed models. It is also quite evident that critical to this streamflow simulation model is the derivation of API_t the antecedent precipitation index.

The value of API at any time was derived to give the streamflow at that time a complete "memory" of all the precipitation falling prior to that time. However, the precipitation falling long before the time of interest is not weighted as fully as precipitation falling just prior to the time of interest. In this manner, the model has a complete "memory" of rain falling at the time of interest, a partial "memory" of rain that fell a short time ago, and only a vague "memory" of rain that fell a long time ago. This variable memory is achieved by decaying the importance of rain that fell before the time of interest by a rate identical to the rate of decay of the recession limb of a streamflow hydrograph during a period of no rainfall. For example, Fedora found that for Deer Creek, an experimental watershed during the Alsea Watershed Study, the two-hour decay coefficient, C_{2hr}, was 0.929 meaning that during the recession limb of a hydrograph with no precipitation, the streamflow for any given 2-hour period was 92.9% of the streamflow for the two hours directly preceding the time of interest.

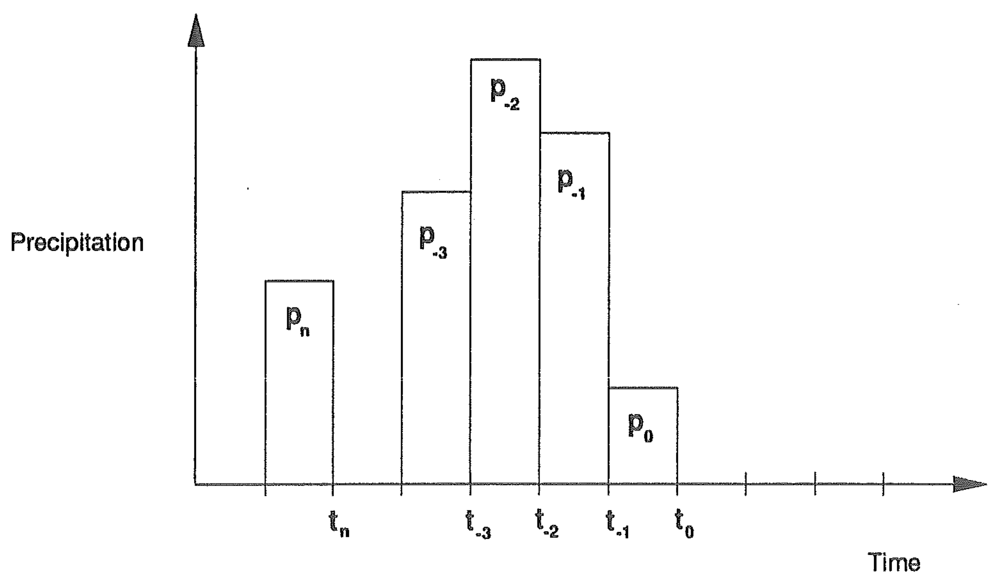

Figure 4. Rainfall record showing rainfall increments and notation.

The equation for determining API at any time is,

$$API_t = p_t + API_{t-1} \cdot C$$

where p is the precipitation just prior to the time of interest and C is the decay coefficient. The nature of this equation can be understood a little better with the help of Figure 4. In this figure there are five times of interest, t_0 to t_n, and five periods of rainfall, p_0 to p_n. We are interested in an expression for the API at t_0. The rainfall during the time period directly preceding t_0, which is p_0, is not adjusted using the recession coefficient and will be related directly to discharge. The rain falling immediately prior to t_{-1} will be adjusted by the recession coefficient and will become,

$$p'_{-1} = p_{-1} \cdot C$$

The rain falling immediately prior to t_{-2} will be adjusted by the recession coefficient raised to the second power, giving,

$$p'_{-2} = p_{-2} \cdot C^2$$

The relationship can easily be discerned and the general form of an adjusted increment of rainfall is,

$$p'_n = p_n \cdot C^{|n|}$$

The API value is then obtained by summing all the adjusted rainfall values from the inception of rainfall.

$$API = p_0 + p'_{-1} + p'_{-2} + p'_{-3} + \ldots + p'_n$$

By substituting the factored form of the adjusted rainfall values the above equation becomes,

$$API = p_0 + p_1 \cdot C + p_2 \cdot C^2 + p_3 \cdot C^3 + \ldots + p_n \cdot C^{|n|}$$

By factoring out the recession coefficient term, C, the expression becomes,

$$API = p_0 + C(p_{-1} + p_{-2} \cdot C + p_{-3} \cdot C^2 + \ldots + p_n \cdot C^{|n|-1})$$

The term inside the parentheses is identical to the expression for API at time t_{-1}. By making this substitution, we are back to the expression we started with.

Therefore, streamflow for an ungauged watershed can be simulated if first a continuous sequence of API values can be calculated for the time period of interest. Then these API values can be multiplied by the appropriate regression coefficients and then squared and the values of streamflow are known for the same time increment as the API values were determined.

Fedora has been undertaken this process for watersheds of the north-central Oregon Coast. Data from Needle Branch, Deer Creek, Flynn Creek, North Yamhill River, and North Fork Siuslaw River were used to determine recession coefficients to calibrate the API model. The recession coefficient C was found to be a function of the watershed area. Regression coefficients b_0 and b_1 were determined for all five calibration watersheds and these regression coefficients were also determined to be indirectly a function of watershed area via the recession coefficient and b_1. These relationships are,

$$C_{\Delta t = 2hrs} = 0.9 + 0.00793 \ln A$$

$$b_1 = 13.6 - 12.8C$$

$$b_0 = 3.95 - 0.545 b_1$$

If all these expressions are put together, an expression is derived that will simulate streamflow for watersheds in the north-central Oregon Coast Range. The final expression is,

$$Q_t = (2.816 + 0.553 \ln A + 2.08 API_t - 0.10105 API_t \ln A)^2$$

This expression, of course, is only good for the range of watershed conditions represented by the database from the calibration watersheds. For this case this means watersheds from the north-central Oregon Coast Range with an area less than 25,000 acres. A source of precipitation-intensity data should be available within eight miles of the centroid of the watershed of interest and the recession coefficient and all API values should be based on a two hour time interval. If all these conditions are met then it is reasonable to expect that the above expression will be an acceptable simulator of the range of streamflows expected for the watershed for given storm magnitudes.

If a problem arises from the precipitation database and a time interval of other than two hours is needed for the calculation of API values, this can be easily obtained by the equation,

$$C_a = C_b^{\left(\frac{\Delta t_a}{\Delta t_b}\right)}$$

where C_a is the recession coefficient based on time interval Δt_a, and C_b is the recession coefficient based on time interval Δt_b.

This form of a streamflow simulation model has been tested for a watershed in the north-central Coast Range and these results have been report in Fedora and Beschta (1989). Also, Beschta (1990) has applied the model to simulate peak flows in Hawaii and river water levels in India. The results show the API method should be able to simulate streamflows at a level of accuracy acceptable to our uses of a streamflow simulator.

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