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Hydrologic modeling for flood control detention basin design and operation

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HYDROLOGIC MODELING FOR FLOOD CONTROL

DETENTION BASIN DESIGN AND OPERATION

by

Mark Andrew Smiley

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As members of the Final Examination Committee, we certify that we have read the dissertation prepared by Mark Andrew Smiley

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Final approval and acceptance of this dissertation is contingent upon the candidate's submission of the final copy of the dissertation to the Graduate College.

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ABSTRACT

This dissertation presents a methodology for hydrologic modeling related to the design and operation of flood control detention basins. Prior to this document, a comprehensive, tractable methodology for detention basin hydrologic modeling did not exist. Furthermore, techniques used in the past have not always taken advantage of computer technology or recent advances in the field of hydrology. New and original methods are presented and are developed from personal experience, recent literature, and relevant courses at The University of Arizona. Chapters in this document include precipitation data analysis, detention basin stormwater inflow, detention basin sediment inflow, stored water losses through evaporation and infiltration, design issues, and operation under competing water use objectives. Engineering constraints and data availability are explicitly addressed throughout the methodology. The goal is to determine hydrologic variables for detention basin design such as active storage volume, spillway capacity, drain outlet capacity, and, additionally for some systems, the bypass channel capacity and side-weir theshold spill flow rate. In addition to providing an increased level of protection from flood damage, detention basins may also accommodate land use and water conservation objectives of urban society.

CHAPTER 1

INTRODUCTION

This dissertation presents a tractable methodology for hydrologic modeling associated with the design and operation of flood control detention basins. In general, these basins are constructed in urban areas with the primary purpose of increasing the level of protection against flood damage to downstream property and perhaps the prevention of loss of life. There is no single standard procedure to perform the hydrologic modeling necessary for detention basin design and operation. Furthermore, techniques used in the past have not always taken advantage of computer technology or recent advances in the field of hydrology. This methodology is based on my personal experience on the subject acquired as a professional civil engineer in California and Arizona, as well as methods learned as a graduate student in the Department of Hydrology and Water Resources and the Department of Atmospheric Science at The University of Arizona.

Combination of rainfall, soil moisture before the rainfall, and, in some cases, snowmelt can create enough runoff above the ground surface to become a flood hazard. Flood waters may either inundate property, cause damage due to the velocity of the flow, or both. In addition to the water hazard, erosion or sedimentation caused by flooding are also destructive.

There are various approaches to the control of floods. Unfortunately, simply not building in flood-prone areas is not often a solution based on human nature. Lands suitable for building, farming, and access to water with mild slopes and thick soil horizons are often flood-prone areas. Civilizations have historically been based on river valleys or coastal areas. A basic approach to the control of floods is flood fighting. During a flood event, sand bags and emergency diversion levees may be built. A more permanent approach is to construct permanent water courses large enough to convey extreme floods as quickly as possible away from the area of concern. This approach may be referred to as an all-river or all-channel plan. Other permanent approaches may include levees, diversion water courses, and land filling. The approach often used in urban areas, which is the subject of this work, is the use of detention basins that reduce peak flow rates downstream and release temporarily stored water in a more timely fashion so as not to cause downstream flooding. The various approaches all have their place in flood control as some may work better than others, depending on the specific site.

Given that a detention basin or series of basins may be a preferred alternative, it is not clear how to size the basin or the associated hydraulic control structures. There are numerous methods for predicting runoff, none of which are universally applicable either by political regulation or by physical science. In most cases, the method to be used is dictated by the regulatory agency which will be in charge of the maintenance of the detention basin. However, the hydrologic method will vary from one jurisdiction to another. This lack of uniformity is not necessarily undesirable. Different regions will have different rainfall patterns, different soil types, and different watershed responses; thus, local methods based on empirical studies may perform quite well. This begs the question of why prepare a tractable methodology for hydrologic modeling for detention basins?

Purposes for developing a tractable methodology for the hydrologic modeling of detention basin systems include possible guidance for regulatory agencies revising or adopting procedures, synthesizing hydrologic techniques directly applicable to detention basins from a variety of sources, incorporation of new and original methods which may be an improvement over existing techniques, and providing a concise treatment of this particular problem which does not exist.

1.1 Problem Statement

This document presents a tractable methodology for designing and operating flood control detention basins which include recent advances in hydrologic modeling, experience with data availability, and experience with application techniques. It addresses the problem that there is no single standard procedure for the hydrologic modeling for flood control detention basin design and operation; and that many existing procedures do not use state-of-the-art hydrologic model components or take advantage of the computational capability of computers.

Recently published books on stormwater detention (Stahre and Urbonas, 1990; Urbonas and Stahre, 1993) do not detail the subject of hydrologic modeling for detention basin design and operation as is done in this dissertation. They are valuable references, however, because their focus complements rather than duplicates what is presented herein. Many types of stormwater detention other than detention basins are discussed, and documentation of their experience is well done. However, their books do not come close to addressing this problem statement.

Recently available software, Pond Pack-Detention Pond Design Made Easy (Haestad Methods, Inc., 1993), is a useful tool for detention basin design. However, its associated literature and on-screen computer menus do not address the aspects of this dissertation. Namely, data availability, conjunctive land uses, conjunctive water uses, optimal operation, mapping techniques, stream transmission losses, evaporation, infiltration, stochastic modeling and confidence limits on discharge values, all of which are integrated into the hydrologic modeling of detention basins herein. Thus, this software package does not adequately address the problem statement.

1.2 Approach

The approach for developing this tractable methodology includes incorporation of personal experience, research and development of hydrologic components, and synthesis of existing published techniques. The goal is to determine hydrologic variables for detention basin design such as active storage volume, spillway capacity, drain outlet capacity, and additionally for flow-by detention basins, the bypass channel capacity and side-weir threshold spill flow rate. The design process incorporated the hydrologic modeling with other aspects, such as regulatory requirements, water-use objectives, right-of-way limitations, utility relocations, access, land-use objectives, hydraulic control design, maintenance, and operation into a set of drawings suitable for construction. The design process is an iterative procedure between the hydrologic modeling and other aspects, with each often serving as a constraint on the other. In essence, the design process is a multi-objective decision-making process. Competing objectives may include level of flood protection, water conservation, construction cost, and land cost. Constraints may include available land space, limited budget, and regulatory requirements. The design process is often not treated as a classical multi-objective decision-making process because of issues such as accounting stance, politics, and changing information. The hydrologic components developed in this methodology may have to be performed several times during a single design process to accommodate these other aspects of the project. The major components, along with a discussion, are presented in Chapters 2 through 7.

The sequential order of chapters (and, thus, hydrologic components) envisions the order in which a thorough hydrologic analysis for the design and operation of a detention basin would take place. However, situations may dictate a different order of analysis, exclusion of some components, or inclusion of other components not covered, such as groundwater discharge or significant snowmelt within the urban watershed. The theme of tractability is not compromised because remaining components with respect to data requirements and temporal and spatial resolution are coordinated between chapters. The tractability of the hydrologic modeling procedure presented here is also based on personal experience with regard to data availability, computer processing capability, and achievable field techniques.

The remaining part of this chapter presents a literature review on detention basins. Chapter 2 discusses precipitation data analysis with the goal of developing a database for continuous simulation of the watershed response and multi-day extreme design storm patterns for testing detention basin performance under extreme flood hazard situations. The runoff generated from rainfall which becomes inflow to the detention basin is presented in Chapter 3. Detention basin sediment inflow prediction is the topic of Chapter 4 and may or may not be an important component depending on the watershed characteristics. Chapter 5, on storage losses, presents improved analytical expression for the accounting of evaporation and seepage losses. Concepts and methods related to the design of the detention basin geometry and hydraulic controls are presented in Chapter 6. Optimal operation of dual use detention basins, that is, basins with conjunctive water supply purposes, using the results of the modeling in the preceeding chapters is the topic of Chapter 7. Specifically, a combined dynamic programming and Gauss elimination algorithm is used. This will require establishing water storage and release targets for the detention basin. This work is summarized and concluded in Chapter 8.

1.3 Literature Review

A literature review associated with this dissertation is presented here. A discussion about a particular well-known reference, namely Design of Small Dams produced by the U.S. Bureau of Reclamation (1987), is presented to begin this section. By its title, this valuable reference may indicate that it includes a comprehensive treatment of the hydrologic modeling for detention basin design. However, it does not. Much of its focus is on geotechnical engineering and material specification for dam construction. Urban detention basins may not be constructed with a dam in the classic sense in many cases. Often, they are in natural depressions or are constructed as below grade structures. Furthermore, they are often smaller in terms of storage capacity than what is required to be regulated as a dam structure. In addition, most of the hydrologic modeling techniques which are scattered through Design of Small Dams do not reflect methods that are used for small urban watersheds. However, the empirical models of sediment transport and capture for detention basins found in Appendix A of Design of Small Dams form the basis for the sedimentation model in this work.

Of the books devoted specifically to detention basins, perhaps the earliest is Practices in Detention of Urban Stormwater Runoff (APWA, 1974). This book actually includes other forms of storm water detention besides basins, such as rooftop storage and hydrobrake control devices. The hydrologic modeling found in the third chapter of that book is relatively simple, focusing on the rational method and the unit hydrograph method. More sophisticated techniques, such as the ones developed in this dissertation, are necessary for a complete evaluation of detention basin performance. The emphasis of the APWA publication is on case studies of detention versus nondetention approaches to flood control, rather than on a detailed tractable methodology for hydrologic modeling.

The book Stormwater Detention Facilities, by DeGroot (1982), includes proceedings of an ASCE conference on the subject. Most of the articles are on case studies, and some are on hydrologic modeling techniques. However, this book does not include a comprehensive tractable methodology for hydrology modeling.

A more recent conference is documented in Stormwater Detention Outlet Control Structures (ASCE, 1985). This is a technical report by the conference rather than a collection of individual articles. The focus is on design concepts related to outlet control structures with a limited qualitative discussion of hydrologic modeling.

Stormwater Detention for Drainage, Water Quality and Combined Sewer Outflow Management (Stahre and Urbonas, 1990) and Stormwater Best Management Practices and Detention for Water Quality, Drainage, and CSO Management (Urbonas and Stahre, 1993) are the only books that comprehensively address the subject of stormwater detention. In fact, my work on this dissertation, which began in 1988, was partially motivated by a lack of a book at that time on the subject. However, the books by Stahre and Urbonas complement, rather than compete, with this treatise. Their recommended hydrologic methodology differs a great deal from the new and original methods presented herein, and their books include aspects of detention basins beyond hydrologic modeling in more detail than I have. Designers and regulators of detention basins would benefit by reading both of Stahre and Urbonas' books as well as this dissertation. However, this dissertation addresses the hydrologic modeling for detention basin design and operation in a much more comprehensive manner because of the discussion of interactions between data availability, the political and regulatory environment, and all aspects of the hydrologic budget. The two books by Urbonas and Stahre do not differ from each other very much. The 1993 version includes additional emphasis on best management practices for enhancing water quality through the use of detention basins. The introduction to Chapter 16, "Precipitation Data Needs for Estimating Storage Volumes", in their 1993 book eloquently discusses the lack of precipitation data availability for realistic hydrologic modeling. I concur with their discussion, but the approach in this document is to explicitly analyze the natural variability of rainfall and runoff more so than the simpler techniques found in the two books by Stahre and Urbonas.

CHAPTER 2

PRECIPITATION DATA ANALYSIS

Designing and developing an operation plan for a detention basin will rely on rainfall-runoff modeling in this methodology. Spatial and temporal rainfall distributions are necessary inputs to the model components presented in the subsequent chapters. Analysis of historical rainfall data is used to derive spatial and temporal distributions which are relatively easy to apply and as realistic as possible. The methodology presented assumes that there is a daily rainfall record proximate to the watershed and a recording hourly rain gauge close enough for assessing regional temporal rainfall characteristics.

The daily rainfall record will be used for continuous simulation of long-term detention basin performance and for the dynamic programming operation algorithm. It will also serve as a means for establishing parameters for the design storm approach. The hourly record will be used to develop the temporal distribution of rainfall values within the multi-day design storm. This is done to assess short-term detention basin performance during extreme storm events and to facilitate design of hydraulic controls, including the spillway.

2.1 Statistics from Historical Record

Based on my experience, there is often a daily rainfall record for a gauge site within or proximate to the watershed of interest. Unfortunately, recording gauges with hourly data are scarce. Usually, the closest hourly data will be from the nearest regional or international airport. Acquisition of these data is discussed in the following subsection. Some stations may have short periods of missing data because, although the cooperative observer system is good, it is not perfect. Techniques for filling in missing data are discussed in the second subsection. At any instant during a storm event, rainfall intensity will vary spatially within the area affected by the storm. This area is also changing in time. Opportunities for considering the spatial and temporal distribution of rainfall are discussed in the third subsection, including the standard depth-area adjustment curves published by NOAA (1973) which is a default method when more sophisticated techniques cannot be used. The dynamic programming algorithm for optimum operation developed in Chapter 7 requires that each year be separated into seasonal periods. The fourth subsection discusses a heuristic method for doing this. Finally, a frequency analysis technique is presented in order to estimate rainfall values for specific durations for any recurrence interval. These statistics will be used to construct the design storm in Section 2.2.

2.1.1 Data Acquisition

Daily rainfall data are published by NOAA in the Climatological Data Series (NOAA, 1992a). Hourly rainfall data are published by NOAA separately in the Hourly Rainfall Series (NOAA, 1992b). Both of these series are published separately for each state. The same data can also be obtained from the National Climatic Data Center in Asheville, North Carolina in printed and electronic media form. Private data brokers such as Hydrodata, a division of U.S. West, provides these data for a subscription rate available on CD-ROM format. Local universities may also have data libraries with the information in various formats. Local agencies, such as flood control districts, may have their own rain and stream gauge network distinct from what one would find using the other aforementioned sources.

Temperature data are also published in the same series with the daily precipitation. The other sources discussed above should also have temperature data.

Daily weather maps can be found in libraries, some university departments, or can be ordered from the National Climatic Data Center.

2.1.2 Missing Data

If the selected gauge record has missing values for temperature or precipitation, the following techniques can be used to estimate the missing values. For either case, it is useful to have a record of at least one nearby gauge.

For missing precipitation data, first check to see if there was any precipitation in the area by examining other nearby records as well as the daily weather map. Also check for low humidity values and low dewpoint temperatures. Upon examining these, a determination can be made that there was zero precipitation at the gauge. If, based on this judgment, there was precipitation at the gauge, the Normal-Ratio Method (Paulhus and Kohler, 1952) with modifications presented in Young (1992) can be performed to estimate the missing value. This procedure produces a more skillful interpolation than multiple linear regression techniques (Young, 1992). The modified Normal-Ratio Method as developed by Young (1992) is as follows.

Correlation coefficients are calculated for each candidate station (typically chosen from precipitation recording stations in the vicinity) having observations corresponding to the missing values to be interpolated (the base station). The three stations with the highest correlation coefficients are selected. A weighting factor, w_i, is computed using the following equation:

$$w_i = \frac{r_i^2(n_i - 2)}{1 - r_i^2} \tag{2.1}$$

where:

w_i = weighting factor for gauge i;

r_i = correlation coefficient for gauge i; and

n_i = number of non-missing pairs of data for gauge i and primary gauge.

The missing value of rainfall for any day t is given by the following equation:

$$P_t = \frac{\sum_j w_j P_{jt}}{\sum_j w_j} \tag{2.2}$$

where:

P_t = estimated rainfall for day t at primary gauge;

j = subset of gauges i with non-missing pairs for day t (j may equal i);

P_{jt} = measured rainfall at gauge j on day t; and

w_j = weighting factor for gauge j.

It should be noted that the weighting factor, w, is the square of the t statistic used to determine the significance of the correlation coefficient. Young (1992) found that it yields a more skillful interpolation than equal weighting.

Missing temperature data can be estimated in an easier manner. A polynomial fit using the temperature data from adjacent measured days from the primary gauge can be used. This method is less computationally intensive than the one described for missing precipitation data, and it can be used because of the persistence of temperature data. However, for long strings of missing data (longer than three days), the normalized anomaly method (Young, 1992) should be considered for use.

The aforementioned techniques can be used to fill in missing data; however, they provide an estimate of a conditioned mean value and are in essence acting as a filter on the data with the disadvantage of not reflecting extreme values which may have occurred. Nevertheless, if a continuous data record is required for continuous simulation, the missing data must be estimated.

2.1.3 Spatial Distribution of Precipitation

At any instant during a storm, rainfall intensity will vary throughout the area of the storm. The area of the storm also changes in time. In addition, storm movement may be oriented in a predominant direction with respect to the orientation of the watershed. Consider an eastward-moving storm and a watershed draining from east to west. It is possible that the west (lowest) end of the watershed near the concentration point will have passed its runoff before it receives runoff from the eastern uplands subject to the storm last. Alternatively, a watershed draining from west to east will have runoff from its uplands combined with directly occurring runoff from its lowlands, causing a larger peak runoff than in the first case. For the smaller watersheds considered for detention basin design, directional orientation will not be considered in this methodology. However, for regional watershed studies with upstream detention basins, directional orientation may be considered. In general, it is not as simple as determining directional orientation from the lower troposphere wind direction. Consider the discussion on mesoscale thunderstorm complexes in Wallace and Hobbs (1977) on page 244. In that discourse, storm cells form at an angle to the prevailing wind direction. Perhaps using a fairly dense network of recording rain gauges or using radar data, one could statistically determine predominant storm directions conditioned on the synoptic scale mechanisms.

As a default procedure, the point precipitation reduction curves for watershed area (NOAA, 1973) can be used to approximate the spatial and temporal variability in rainfall intensity. This is documented in slightly more detail in Section 3.4 on inflow from design storms. This default procedure is typically used because of lack of a dense recording gauge network and lack of a sufficiently long radar data record.

2.1.4 Identification of Seasonal Periods

The dynamic programming algorithm described in Chapter 7 is used for optimal operation of detention basins with variable outflow controls and will require aggregation of streamflow values into seasonal periods in order to determine a transitional probability matrix of detention basin inflow from one season to the next. These seasonal periods can be determined heuristically based on the annual distribution of rainfall, as was done in Buras (1985).

The following is a discussion of aspects to consider when identifying seasonal periods. Upon determining average monthly rainfall for each of the 12 months, identify wet and dry seasons. If the region has substantial snowfall and a semi-persistent snowpack, attempt to identify a snowmelt season during the spring which could be generated by either increasing temperatures or a rainy seasonal period. Local experience is highly recommended. Fortunately, many urban areas where the detention basin design will be performed will not be subject to significant snowmelt. If a rainy season is followed by a very dry season, further partitioning of the rainy season is recommended for changing the storage and release targets for each seasonal period to accommodate both flood control and water conservation objectives. For instance, if the last month of a rainy season is followed by a very dry season, desired storage of water will dictate a shift in targets combined with a reduced flood threat. Setting storage and release targets and optimizing them for each seasonal period are discussed in detail in Chapter 7. The purpose of mentioning seasonal periods here is to be aware of rainfall data requirements and to be aware of the local climatology early in the design phase. Local experience should add to the modeler's confidence in identification of seasonal periods.

2.1.5 Frequency Analysis

Frequency analysis is used to estimate rainfall values for specific durations and specific return periods. These statistics will be used to construct the multi-day design storms for assessing detention basin performance during extreme events.

The first step is to identify the annual maximum rainfall values for each duration (e.g., one day, two days, three days, through seven days) for each year in the period of record. Each set should then be plotted in histogram form. The mean, standard deviation, and skew for each set of annual maximums should be calculated. Three different distributions will be fitted to each set of annual maximums. They are the normal distribution, lognormal distribution, and gamma distribution. Other distributions, including the log-Pearson III distribution or the Gumbel extreme value distribution, can also be used. A Chi-square analysis, similar to what is presented on page 460 of Benjamin and Cornell (1970), can be used to determine which of these distributions best fits each set of annual maximum histogram values.

The normal distribution is given by the following equation:

$$f(u) = (2\pi s_x^2)^{-0.5} \, e^{-0.5u^2} \tag{2.3}$$

where:

$u = (x - \bar{x})/s_x$;

x = annual maximum rainfall value;

$\bar{x}$ = mean of observed annual maximum rainfall values;

s_x = standard deviation of observed annual maximum rainfall values;

e = base of natural logarithm; and

f(u) = value of probability density function for a particular value of u.

It is convenient to use the cumulative distribution function (CDF) to calculate corresponding histogram values by taking successive differences of the CDF. The CDF of the normal distribution is given by the following equation:

$$F(u) = (2\pi)^{-0.5} \int_{-\infty}^{u} e^{-0.5v^2} \, dv \tag{2.4}$$

where:

v = dummy variable of integration; and

F(u) = value of CDF for a particular u.

Because integrating from minus infinity is not numerically convenient, the CDF can be evaluated from either side of its midpoint (F(0) = 0.5). The lower limit of integration becomes zero for positive values of u. For negative values of u, the integration is from u to zero.

The lognormal distribution is also compared to the data. This is accomplished by calculating the natural logarithms of the data and applying the standard normal distribution to the logged data.

Finally, the gamma distribution is often a likely candidate because the annual maximum set is always non-negative and often positively skewed. The gamma distribution is given as follows:

$$f(x) = \frac{\lambda(\lambda x)^{k-1} e^{-\lambda x}}{\Gamma(k)}; \; x \geq 0 \tag{2.5}$$

where:

x = annual maximum rainfall amount;

$\Gamma(k) = \int_0^\infty e^{-u} \; u^{k-1} \, du;$

λ and k are parameters to fit the data using the method of moments shown below; and

f(x) = value of probability density function for a particular x.

The two parameters, λ and k, are estimated from the mean and variance of the annual maximum rainfall values by the following equations (Benjamin and Cornell, 1970):

$$\lambda = \bar{x}/s_x^2 \tag{2.6}$$

$$k = \bar{x}^2/s_x^2 \tag{2.7}$$

It is also convenient to use the CDF of the gamma distribution to calculate the corresponding histrogram value by taking successive differences of the CDF. The CDF of the gamma distribution is given as follows:

$$F(x) = \int_o^x f(x)\,dx = \frac{\Gamma(k, \lambda x)}{\Gamma(k)} \tag{2.8}$$

where:

$\Gamma(k, \lambda x)$ is the incomplete gamma function;

$\Gamma(k, \lambda x) = \int_o^{\lambda x} e^{-u}\, u^{k-1}\, du.$

To test which distribution fits the data better, the Chi-square test is used. From each histogram, the normalized square deviations, χ^2, from the assumed distributions (normal, lognormal, or gamma) are computed by the following equation:

$$X^2 = \sum_{i=1}^{j} \frac{(O_i - P_i)^2}{P_i} \tag{2.9}$$

where:

O_i = observed number of occurrences in the data in the i^{th} histogram interval;

P_i = predicted number of occurrences by the distribution in the i^{th} histogram interval; and

j = number of histogram intervals.

The histogram intervals on the tails of the distribution may be combined on each side as recommended by Benjamin and Cornell (1970) so that the expected number of occurrences is not less than five. This is done so that the normalized square deviation for discrete analysis is Chi-square distributed. However, other authors are more lenient on this issue. For instance, Lindley (1965) stated that this test may be safely used for histogram intervals with as low as two expected occurrences, and Fisz (1963) stated that, for an $\alpha = 0.05$, the observed number of occurrences in one histogram interval can be as low as one, without "fear of making a big mistake."

The distribution with the lowest value of χ^2 fits the data best, provided that the number of histogram intervals and the degrees of freedom, f, are identical. Otherwise, the distribution with the maximum p-value is used. The p-value is a measure of robustness. For the purpose of fitting distributions in this frequency analysis, the calculation of the p-value will be shown. The degrees of freedom reflect the notion that, if the histogram data are high in one interval, they will be compensated by a low value in another interval due to a fixed number of samples. The degrees of freedom, f, are also reduced for the number of parameters used to fit the distribution to the data. For the three distributions tested, all use two parameters (mean and standard deviation for the normal; mean and standard deviation of the logged data for the lognormal; and λ and k for the gamma). The number of degrees of freedom, f, is given as follows:

$$f = j - r - 1 \tag{2.10}$$

where:

j = number of histogram intervals; and

r = number of parameters for the distribution.

The Chi-square distribution is used to determine the p-value for each of the fitted distributions. It has been shown (Benjamin and Cornell, 1970) that the CDF for the Chi-square distribution for this type of histogram analysis is distributed gamma ($\nu/2$, 1/2), where ν equals degrees of freedom. While programming this procedure, I transformed the relationship between p-value and the squared deviation, χ^2, as follows:

$$p = 1 - \frac{\int_0^{0.5X^2} e^{-u}\, u^{\nu/2-1}\, du}{\int_0^{\infty} e^{-u}\, u^{\nu/2-1}\, du} \tag{2.11}$$

where:

χ^2 = normalized square deviation; and

p = p-value for the fitted distribution being tested.

For the fitted distribution to significantly describe the data, the p-value should be greater than a specified level of Type 1 error. The Type 1 error is the probability that the fitted distribution is rejected even if it, in fact, describes the data. By convention, the Type 1 error can be set at 0.05. For computational purposes, the value of $\chi_{.05,\nu}{}^{2}$ can be determined using the equation below:

$$0.05 = 1 - \frac{\int_{o}^{0.5\chi^2_{.05,\upsilon}} e^{-u}\, u^{\upsilon/2-1}\, du}{\int_{o}^{\infty} e^{-u}\, u^{\upsilon/2-1}\, du} \tag{2.12}$$

There are two conditions to be met in order to select which fitted distribution should be used:

Condition 1: p-value of distribution > 0.05

Condition 2: p-value of distribution > p-value of other distributions tested.

If Condition 1 is not met for any of the distributions, either other distributions should be tried or the frequency analysis discontinued. If so, consultation with the local flood control agency or the U.S. Army Corps of Engineers should be made.

Upon determining which distribution performs best in fitting the annual maximum rainfall values, rainfall values for specific frequencies are easily determined by evaluating the CDF of the fitted distribution.

$$F(x) = 1 - 1/T \tag{2.13}$$

where:

T = recurrence interval (frequency) in years; and

F(x) = CDF value for rainfall amount x.

The rainfall value associated with that CDF value can be computed using the equation describing the CDF for the best fit distribution. The use of these rainfall values for design storm construction is illustrated in the next section.

2.2 Design Storm Analysis

An event-based rainfall-runoff model is used to design and test the performance of the detention basin under extreme runoff events. The term "event-based" refers to a modeled storm, either historical or synthetic, with an established duration. The range of the duration may be from a few hours to several days. For detention basin modeling, a multi-day storm is often required because of the prolonged drain out time in some of these systems. On the other hand, a continuous simulation approach uses a rainfall input of several weeks, months, or even years, if a rain gauge record is used as direct input. Continuous simulation is discussed in Section 3.3. An event-based, rather than continuous simulation, approach is used here because small time steps are required to model the instantaneous peak in the detention basin hydrograph. Continuous simulation with such small time steps (typically five minutes) is not feasible either from a computation standpoint or from rainfall data availability. The event-based approach can be used with a multi-day design storm as input, which is usually sufficiently long enough to model the detention basin prior to the instantaneous peak. Two approaches which could be used include directly using a historical storm or using a synthetic design storm as input to a rainfall-runoff model. Based on my experience, direct use of historical storms may not provide an adequate test for a particular watershed due to the relationship between average rainfall intensity and area. For example, a historical storm may have extreme rainfall totals for certain durations within the storm, but moderate or even low rainfall totals for other durations. Hence, any given historical storm will not provide a good test for a range of watershed sizes. A type of synthetic storm which provides a good test for a range of watershed sizes is a nested duration storm. Extreme rainfall values for any time duration are nested within successively longer time periods. For instance, the peak five-minute rainfall occurs within the peak 30-minute rainfall which occurs within the peak hour rainfall. A nested duration synthetic design storm (McCuen, 1982) can be developed to adequately test any common urban watershed size, and it can be based on historical storms as shown below.

This section presents a new and original method for constructing a multi-day design storm using hourly rainfall data from historical storms from a gauge near the watershed of interest. Extreme historical storms are initially identified as candidate storms by selecting a series of precipitation amount thresholds. Autocorrelation analysis of the hourly rainfall data for each storm is then performed. The storm will be classified based on its interpeak time; that is, the number of hours between intense rainfall episodes during the multi-day period. Interpeak times will be reflected as peaks in an autocorrelation of the hourly data. The most significant peak in the autocorrelogram is determined for each storm, and that storm is then classified based on its corresponding interpeak lag time. Predictor variables are evaluated for each storm using multiple discriminant analysis (Miller, 1962). Predictor variables will be discussed in Section 2.2.4. They are needed to distinguish different storm generation mechanisms which influence interpeak times for multi-day storms. Upon identifying the significant predictor variables, discretized lag categories are re-evaluated based on predictor variable values and heuristically based on visual observation of the daily weather maps for each storm. The goal is to estimate statistically probable interpeak rainfall times for different classes of storms. A multi-day design storm is then constructed for at least one of the storm class/lag class categories. Additional design storms may be constructed for other categories. The design storm then will be used as input to a rainfall-runoff model presented in Chapter 3.

A hypothetical example of the usefulness of this approach is as follows. In the month of October in the southwestern United States, there are three potential extreme storm generation mechanisms. Type I is the monsoon mechanism caused by synoptic scale pressure patterns bringing in moisture from the south and statistically likely interpeak times of 24 hours from daily afternoon thunderstorms. High 850-millibar dewpoint temperatures serve as the predominant predictor variable based on multiple discriminant analysis. Type II is the tropical cyclone mechanism caused by a combination of mesoscale low pressure and warm sea surface temperature in the Sea of Cortez. Statistically likely interpeak times may be 18 hours based on this hypothetical example. Type III is the general winter storm (with an early onset because it's October) caused by a series of cold fronts or rain bands associated with a cutoff low from the northwest brought by a very slow-moving 500-millibar low-pressure system. Statistically likely interpeak times are 36 hours based on analysis. Now, many extreme rainfall episodes may not fall neatly into one of these categories based on a cursory examination of the daily weather maps. Multiple disciminant analysis provides an automated way of storm classification for determining statistically likely interpeak times for a class of storm generation mechanisms. Different multi-day design storms with different interpeak times for these mechanisms could be modeled to test the detention basin under these different conditions. The relationships between rainfall volumes for various durations and interpeak time can be more realistically determined because the storm generation mechanisms have been distinguished. The effect on detention basin performance may not be obvious until the methodology presented in this document has been performed.

2.2.1 Extreme Storm Identification

A series of precipitation amount thresholds is used to narrow the field of candidate storms. The initial threshold is monthly rainfall in excess of one-half of the average annual rainfall in the historical rainfall record. The number of months in the record exceeding this threshold will be on the order of the number of years in the record, based on experience. It is possible that some storms may be missed if they occurred during a change in month and split almost evenly between the two calendar months; however, the number of such storms should be small, and it is not necessary to have an exhaustive list of candidate storms. If it is desired to further reduce the list of candidate storms, a second threshold can be set based on total storm rainfall within each of the candidate months. Additional thresholds may be based on maximum daily rainfall within each storm or on maximum hourly rainfall. Selection of thresholds should be done with consideration of the size of the watershed. Smaller watersheds will be more sensitive to peak hourly rainfall and, even though the storm volume and its temporal distribution will influence the design, storms with extreme hourly values should not be eliminated from consideration hastily.

2.2.2 Autocorrelation

Autocorrelation is the correlation of a time series with itself at a lag. For an hourly rainfall time series, high values of autocorrelation for a particular lag time indicate statistically likely times from an ongoing peak precipitation event to another peak event at a future time equal to that lag time. Based on experience, the autocorrelation of lag equal to one hour will be high because of the persistence of a storm episode to last into the next hour. A high correlation at 24 hours may occur due to the diurnal characteristics of thunderstorms likely to occur in the late afternoon for consecutive days in some areas. Winter storms associated with synoptic scale low-pressure systems tend to have a high autocorrelation at 34 to 36 hours in southern California (Smiley et al., 1993), perhaps due to short-wave perturbations traveling around the long-wave synoptic scale low.

The equation for computing the autocorrelation for a particular lag time is shown below and is taken from Matalas (1963):

$$r_k = \frac{\sum_{i=1}^{n-k} X_i X_{i+k} - \frac{1}{(N-k)}\left(\sum_{i=1}^{N-k} X_i\right)\left(\sum_{i=1}^{N-k} X_{i+k}\right)}{\left[\sum_{i=1}^{N-k} X_i^2 - \frac{1}{N-k}\left(\sum_{i=1}^{N-k} X_i\right)^2\right]^{0.5}\left[\sum_{i=1}^{N-k} X_{i+k}^2 - \frac{1}{N-k}\left(\sum_{i=1}^{N-k} X_{i+k}\right)^2\right]^{0.5}} \tag{2.14}$$

where:

r_k = autocorrelation at lag k;

k = lag (hours);

X_i = hourly rainfall during hour i; and

N = time series duration (candidate multi-day storm duration) in hours.

A plot of autocorrelation versus lag is called an autocorrelogram. High autocorrelations can be noted visually on the autocorrelogram. However, even a random or independent time series will have deviations from zero correlation for small sample sizes. A test for autocorrelation significantly different from zero is required to help identify significant lag times (Matalas, 1963). For a confidence level α (usually equal to .05), the minimum autocorrelation value for significance is given by the following equation (Matalas, 1963):

$$\hat{r}_k = \frac{-1 + t_\alpha \sqrt{N-k-1}}{N-k} \tag{2.15}$$

where:

$\hat{r}_k$ = minimum positive autocorrelation for significance for lag k; and

t_α = t-statistic at level α.

Tables of the t-statistic are presented in many texts on statistics, including Benjamin and Cornell (1970).

The significance level can be plotted with the autocorrelation values onto the correlogram.

After performing the autocorrelation and examining the correlogram, a characteristic lag time should be selected for each storm. Lags less than 12 hours should be excluded because the goal is to determine interpeak times for multi-day storms. Lags with autocorrelations less than the minimum value for significance should be excluded. Ideally, there will be a single peak in the correlogram which will serve as the characteristic lag time. If there are multiple peaks, the peak with the largest correlation value should be selected, as that would be statistically more typical interpeak time. If there are no peaks above the significance line, the storm could be removed from further consideration for aiding in the design storm construction process. An example of a correlogram is shown in Figure 1. Values above the open circles are statistically significant at $\alpha = 0.05$.

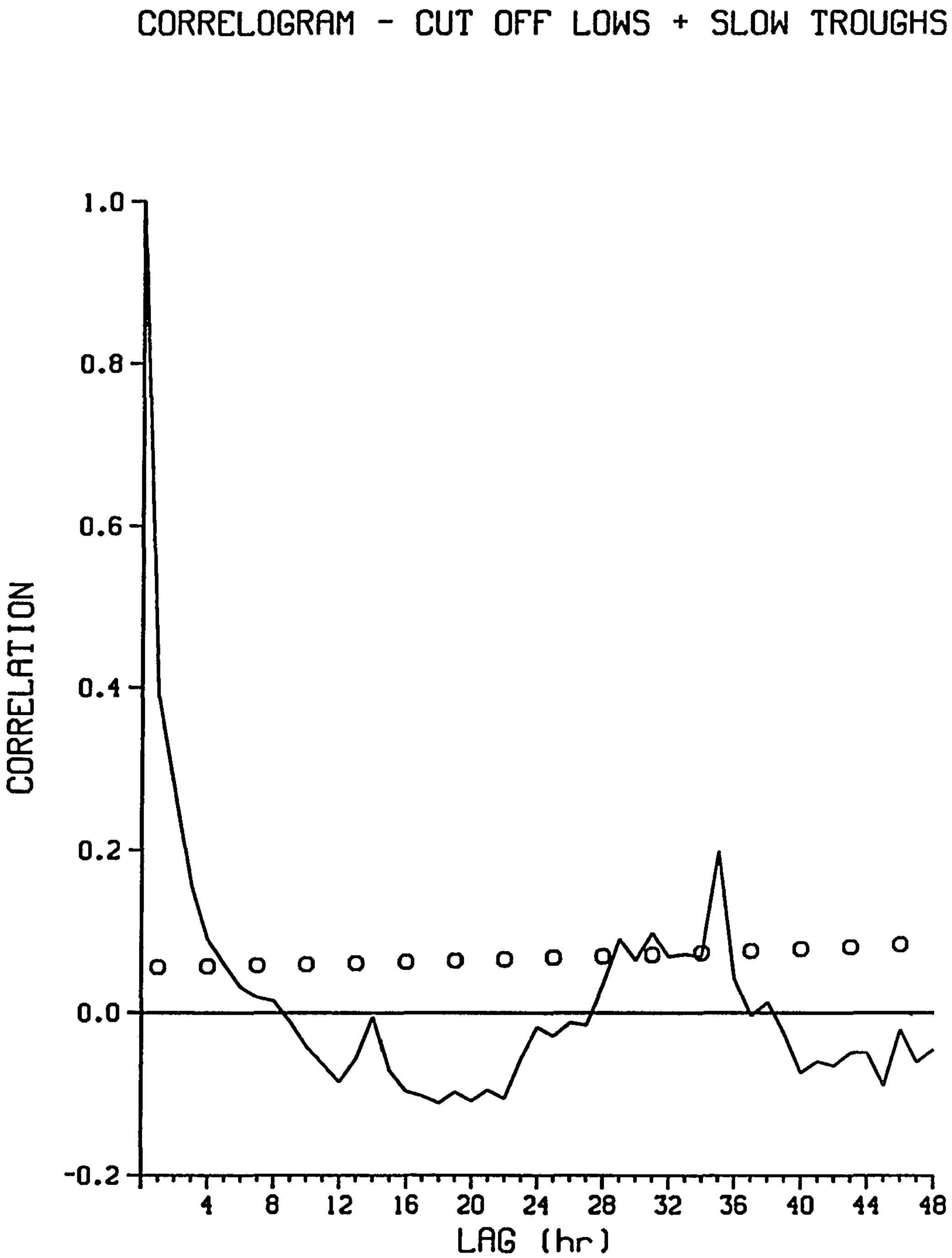


Figure 1. Example of correlogram.

2.2.3 Interpeak Time Classification

In order to distinguish storm-generation mechanisms which may have different characteristic interpeak times, multiple discriminant analysis is used. The multiple discriminant analysis technique will require mutually exclusive and exhaustive groups for the variable being predicted, which in this case is lag time (Miller, 1962). The range of characteristic lag times will be from the shortest to the longest characteristic lag time for the ensemble of storms. Categories can be formed heuristically by searching for groups of characteristic lag times. The number of categories should be around four, as a rule of thumb.

2.2.4 Predictor Variables

The purpose of using multiple discriminant analysis in this case is to identify the most important predictor variables for storm classification. The predictor variables are what distinguish storm-generation mechanisms for each candidate storm. In many cases, a distinct storm-generation mechanism is not always obvious from a cursory examination of the daily weather maps. In addition to meteorological variables near the gauge site, other variables such as month of the year, southern oscillation index (El Nino phenomenon), and meteorological variables upwind from the gauge site may be important. Meteorological variables could include temperature, wind direction, and humidity at levels from the surface up through the troposphere. Some of these data can be estimated from the daily weather map. Weather Bureau stations, often located at airports, usually collect data twice per day from weather balloons. These data are archived through the National Climatic Data Center and may also be available through data brokers as was discussed in Section 2.1.

2.2.5 Multiple Discriminant Analysis

Multiple discriminant analysis is a technique for identifying significant predictor variables and for estimating a probability distribution for the variable being predicted, given values of the predictor variables (Miller, 1962). Identification of predictor variables will be used for storm classification. The forecasting application is not used in this methodology; however, it could be used for real-time operation of detention basins which is beyond the scope of this dissertation.

After classifying the characteristic lag time for each storm into the mutually exclusive and exhaustive categories previously discussed in Section 2.2.3, and after tabulation of potential predictor variable values for each of the storms, each individual predictor variable is analyzed. This analysis includes calculating the mean for each group (lag category) and the mean value of the predictor variable for all groups combined. The predictive power for each variable is given by the following equations:

$$D_1^2 = (n-1-G)\,\frac{SSB(x_p)}{SSW(x_p)} \tag{2.16}$$

where:

D_1^2 = predictive power of predictor variable p;

n = number of storms;

G = number of lag groups;

$$SSB\ (x_p) = \sum_{g=1}^{G} n_g\ (\bar{x}_{pg.} - \bar{x}_{p..})^2;$$

g = index number of lag group;

n_g = number of storms in group g;

$\bar{x}_{pg.}$ = mean of predictor variable p for group g;

$\bar{x}_{p..}$ = grand mean of predictor variable p;

$$SSW\ (x_p) = \sum_{g=1}^{G} \sum_{k=1}^{n_g} (x_{pgk} - \bar{x}_{pg.})^2; \text{ and}$$

x_{pgk} = value of predictor variable p for storm k in group g.

The predictor variable with the largest predictive power is selected. The remaining variables will be evaluated in conjunction with this selected variable. For each remaining variable, the following two matrices should be computed:

$$W = \begin{bmatrix} SSW(x_1) & SPW(x_1,x_2) \\ SPW(x_1,x_2) & SSW(x_2) \end{bmatrix} \tag{2.17}$$

where:

x_1 = selected variable with largest predictive power;

x_2 = candidate variable; and

$$SPW\ (x_1,x_2) = \sum_{g=1}^{G} \sum_{k=1}^{n_g} (x_{1gk}-\bar{x}_{1g.})\ (x_{2gk}-\bar{x}_{2g.})$$

$$B = \begin{bmatrix} SSB(x_1) & SPB(x_1,x_2) \\ SPB(x_1,x_2) & SSB(x_2) \end{bmatrix} \tag{2.18}$$

where:

$$SPB\ (x_1,x_2) = \sum_{g=1}^{G} n_g\ (\bar{x}_{1g.}-\bar{x}_{1..})\ (\bar{x}_{2g.}-\bar{x}_{2..}). \tag{2.19}$$

The trace of the matrix, $W^{-1}B$,, needs to be calculated. The trace is the sum of the diagonal terms.

The predictive power of the candidate variable in conjunction with the selected variable is given as follows:

$$D_2^2 = (n-1-2G)\ \text{trace}\ (W^{-1}B) \tag{2.20}$$

The candidate variable with the largest value of $D_2{}^2$ is chosen. To test if the second variable adds a significant amount of information, the Chi-square statistic with G-1 degrees of freedom is used, as shown below:

$$D_2^2 - D_1^2 > X^2_{G-1,\alpha} \tag{2.21}$$

If the above inequality holds, there are now two selected variables; otherwise, only the initial predictor variable is used. The process is repeated for the remaining candidate variables until the increase in predictive power no longer is greater than the Chi-square statistic.

The remaining procedures in multiple discriminant analysis are for forecasting. The selected predictors are orthogonalized using eigenvalues, and the sum of the product of the eigenvectors with value of the predictor variables is used to calculate probabilities of each lag category occurring (see Miller, 1962, for the detailed forecasting procedures). This is beyond the scope of this dissertation and is not necessary for developing the design storm.

2.2.6 Storm Classification

Interpeak rainfall times for classes of storms are determined by evaluation of the selected predictor variables and by observation of the daily weather maps associated with every selected storm. It is possible that there is only one predominant severe storm generation mechanism, in which case it would be expected that one of the lag classification groups would contain many more storms than the other groups. Bimodal distribution of the characteristic lags may indicate two predominant storm generation mechanisms, such as summer afternoon thunderstorms and winter frontal precipitation. Evaluation of predictor variables for each storm and examination of the daily weather maps will aid in classifying storms. The purpose of storm classification is to eliminate anomalous storm episodes and to re-evaluate the characteristic lag time for a specific storm generation mechanism. As discussed in the hypothetical example at the beginning of Section 2.2, identifying storm generation mechanisms and evaluating their statistically likely interpeak times yields more realistic multi-day design storms.

2.2.7 Multi-Day Design Storm Patterns

One-day, two-day, three-day, etc. storm rainfall totals for a particular return period have been determined using frequency analysis, as discussed in Section 2.1. Determining a realistic hourly distribution of this rainfall is the task at hand.

Nested duration design storms have the feature of having shorter duration rainfall with a specific return period nested into successively longer durations with the same return period (McCuen, 1982). This feature is attractive because it allows the same storm to serve as input for various watershed sizes and various times of concentration (McCuen, 1982). For example, a historical storm may have extreme rainfall totals for certain durations within the storm, but moderate or even low rainfall totals for other durations. On the other hand, a nested duration multi-day design storm has a constant recurrence interval at a variety of durations, thus testing the detention basin performance at the stated recurrence interval. An example of this type of storm will be introduced at the end of this chapter with the example values tabulated in Table 1 of Appendix A. Multi-day design storms have been developed for detention basin design (OCEMA, 1986); however, based on personal knowledge, the interpeak times were determined without a thorough analysis of hourly precipitation patterns, as is presented in this methodology.

The multi-day design storm will have peak hourly rainfall spaced apart at the characteristic lag time. The one-day total rainfall will be preceded by the two-day total less the one-day total. The two-day total rainfall will be preceded by the three-day total less the two-day total. Precipitation values for durations less than a day can be estimated using a regression procedure similar to what is outlined in NOAA (1973) but with 24-hour rainfall constrained by the aforementioned daily nesting totals and one-hour rainfall computed using frequency analysis of the hourly maximums of the selected storms. A partial duration analysis for the one-hour rainfall would be appropriate due to the possibility of more than one historical storm occurring in a year. The six-hour rainfall total could be based on the percentage of 24-hour rainfall that is found in the rainfall maps in NOAA (1973). All of these guidelines should be considered for use. Because the characteristic lag time will likely be different than 24 hours, some manipulation of the design storm pattern will be required to preserve rainfall totals. An example of a three-peak design storm for a 50-year return period with a characteristic lag time of 30 hours, with one-, two-, and three-day totals of 5.47, 8.60, and 11.12 inches, with 6-hour rainfall equal to 66% of 24-hour rainfall, and with a partial duration frequency analysis of selected storms yielding one-hour rainfall of 2.25 inches for a 50-year return period is presented in Table 1 of Appendix A.

A multi-day design storm developed from the methods presented in this chapter can now be used as input to the rainfall-runoff model (stochastic unit hydrograph model) discussed in Chapter 3.

CHAPTER 3

DETENTION BASIN INFLOW

Methods for estimating detention basin inflow are presented in this chapter. The inflow will be mathematically "routed" through the detention basin in Chapter 6. Detention basin geometry, spillway size, and other hydraulic controls will be designed based on the results of the mathematical model. The first two sections of this chapter are related to watershed characterization; the third section, on continuous simulation of inflow, is useful for modeling typical and long-range conditions that the detention basin will be exposed to. Section 3.4, in which design storms are discussed, is for the extreme runoff events which are especially important for spillway design. Storm transposition, Probable Maximum Flood (PMF), and standard project storm methods (U.S. Bureau of Reclamation, 1987) are other methods that can be used; however, they are not intended for more frequent, extreme storms (e.g., 25-year storms) which may be a more appropriate level of design for smaller detention structures. The specific recurrence interval which the detention basin will be designed to is often dictated by policy. Removal of land from the National Flood Insurance Program would result from a detention basin system designed to accommodate a 100-year return period. If this is not an issue, a more frequent event, such as a 25-year recurrence interval, may be the design standard. It is conceivable that an economic analysis may result in setting the recurrence interval, due to the tradeoff between construction costs and flood damage costs. For situations when streamflow data exist for the watershed, analytical techniques are provided for assessing and using the data (Section 5). Streamflow data can augment modeled inflow, and they are particularly useful for developing the transitional probability matrix used in the dynamic programming algorithm described in Chapter 7.

This chapter on detention basin inflow begins with a discussion on mapping the drainage area geometry in Section 3.1, as the inflow to the detention basin will depend on geometric characteristics of the watershed. Land use and soil types also affect the timing and amount of storm runoff that will inflow to the detention basin (Section 3.2). A continuous simulation of detention basin performance is useful for optimizing a design which accommodates both flood control and water conservation objectives through the application of the dynamic programming algorithm (presented in Chapter 7) using the continuous input developed in Section 3.3. In order to estimate instantaneous peak flow rates for hydraulic design of spillways and other hydraulic controls, a design storm method can be used with a rainfall-runoff model using a small time step to estimate the instantaneous peak flow rate, as will be shown in Section 3.4. Measured streamflow data, if available, can also be used for the hydrologic modeling under some circumstances, as discussed in Section 3.5. The last section in this chapter deals with aggregation of modeled (or measured) streamflow into seasonal periods for input to the dynamic programming algorithm for optimal detention basin performance.

3.1 Drainage Area Geometry

The inflow to the detention basin will depend on geometric characteristics of the watershed. The important geometric characteristics of the watershed and how they are obtained are discussed in this section.

The area of the watershed is the most important geometric characteristic. However, for very large watersheds, additional area may not increase peak detention basin inflow due to the depth-area reduction of rainfall, as discussed in Section 3.4

The profile of the major water course is also an important geometric characteristic. A useful definition of the major water course is the water course delivering flow to the point of concentration (location of the detention basin) and starting from the remotest point in the watershed. There are occasional situations where this definition should not be used. If the remotest point is a long "finger" which does not contribute much area, a parameter such as time of concentration based on the water course length will result in low peak inflow estimates. Alternative major water courses should be identified and considered. For distributed modeling where channel routing is necessary, the profile and cross-sectional geometry of the water course will be used for velocity estimates.

Mapping the watershed is essential for determining the drainage area geometry. Suitable topographic maps may already exist for an initial determination of the drainage area and stream profile. If not, a new survey of the area is required. A field investigation is recommended to confirm the validity of the initial mapwork and to visualize and document other watershed characteristics, such as land use and soil type. Sometimes, anthropogenic changes to the watershed or water course are not reflected in the initial mapwork.

3.1.1 Initial Mapping

For projects within the United States, the U.S. Geological Survey's (USGS) 7.5-minute topographic maps are valuable initial sources. Local governmental agencies may also have useful maps. For example, the public works office may have a series of maps of street flow directions and existing storm drains, helpful for watersheds which may be partially urbanized. They may also have Master Plans of Drainage which, when combined with land use information, provide a means of anticipating the future watershed boundaries that may change as the result of land development.

If there is inadequate topographic information, a new survey should be performed. This often is necessary for small watersheds when the USGS map is not at an appropriate scale or has coarse contour intervals. A rough estimate of the watershed extent is necessary to define the limits of the survey. Contour intervals and the scale of the final map are other items which the surveyor will typically request prior to commencing the work.

If the major water course is an improved facility, as-built plans of it should be obtained from the local public works agency.

3.1.2 Field Mapping

Watershed boundaries, land use, soil type, and drainage features should be verified in the field and noted on a map produced during the initial mapping stage. Such information from site visits often adds realism and constraints to the hydrologic modeling options and parameter values. Opportunities for talking with local residents and workers occasionally provide useful information as well. Such anecdotal information might include the performance of existing hydraulic structures, debris problems, and other drainage problems.

An efficient way of verifying the watershed boundary is to start at the concentration point and proceed along the initial mapped boundary. Check for unmapped conveyance structures that contribute additional drainage areas. In natural terrain with prominent ridge lines, this is not necessary.

The major water course should also be field checked. Improved facilities should be checked against the as-built drawings. Natural creeks should have approximate cross-sectional dimensions noted and whether the profile of the creek bed is relatively smooth or more like a series of steps. The slope of both man-made and natural creeks may be less than what would be calculated from mapping alone due to check dams, drop structures, culverts, geologic controls, or other conditions where there is a change in creek bed elevation over very short distances. For channel routing, the travel time should be based on the shallower actual slope rather than on the gross computed slope from locations far apart.

3.2 Land Use/Soil Cover Complex

Land use and soil types in the watershed will affect the timing and amount of runoff that will flow into the detention basin. In general, urbanization will increase the amount of impervious land cover, thus decreasing the amount of infiltration, as well as increasing the velocity of channeled water, which usually increases the peak flow rate. Soil type will influence infiltration. Coarse-grained soils will have high infiltration rates, while fine-grained soils will tend to have lower infiltration rates.

Some of the more important aspects of a watershed's land use/soil cover complex, with an emphasis on parameters required for the Soil Conservation Service's method of curve number assignment, are presented in this section. The curve number is an empirical parameter used to estimate what fraction of precipitation will appear as watershed runoff.

3.2.1 Land Use Anticipation

While aerial photographs and site visits are ways of determining existing land use, consultation with the local planning department is important for knowing what the future land use plans are and what is the timing of future development. The design of detention basins should accommodate runoff quantities from possible future land use patterns and sedimentation from existing undeveloped or agricultural areas.

In some areas of the United States, land development is conditioned to not increase peak flow rates from existing conditions. A common solution is to build a detention basin near the development's surface runoff discharge point. Hydrologic modeling of both existing and future watershed conditions is often necessary.

3.2.2 Hydrologic Soil Groups

The Soil Conservation Service publishes county soil surveys which identify soils and assign hydrologic soil groups to each type of soil. The four soil groups are as follows (McCuen, 1982):

Group A: Deep sand, deep loess, aggregated silts;

Group B: Shallow loess, sandy loam;

Group C: Clay loams, shallow sandy laom, soils low in organic content, and soils usually high in clay;

Group D: Soils that swell significantly when wet, heavy plastic clays, and certain saline soils.

The hydrologic soil group, which is used to assign a curve number as will be discussed in Section 3.2.4, corresponds to the following minimum infiltration rates (McCuen, 1982):

Group A: 0.30-0.45 in/hr;

Group B: 0.15-0.30 in/hr;

Group C: 0.05-0.15 in/hr;

Group D: 0-0.05 in/hr.

If a county soil survey is not available, soil sampling is another method for determining the hydrologic soil group; this is discussed in the next section.

3.2.3 Soil Sampling

For situations when a county soil survey is unavailable, the soil in the watershed should be analyzed. Sites should be selected throughout the watershed and mapped. Methods for assigning a hydrologic soil group to the sample are discussed in this section.

An easy way to determine if there is substantial clay in the soil is to take a small handful of it, add a small amount of water (or saliva), and roll it between thumb and forefinger. If it is cohesive and rolls easily into a pliable cylindrical shape, there is a substantial amount of clay in it, and it is either Group C or D; otherwise, it is Group A or B.

An infiltrometer test (ASTM, 1990) can also be performed to determine the minimum infiltration rate and refine the assignment of the hydrologic soil group constrained by the clay analysis. Because of possible experimental error, the imposed constraint of the clay analysis is recommended, rather than just assigning the hydrologic soil group based on measured infiltration rate. In addition, the natural spatial variability of minimum infiltration rate may require more samples than what are taken to characterize the soil. Ideally, enough samples would be taken from the beginning, and there would be no experimental error. However, my experience is that measuring this soil property is not always a certain proposition, especially if there is a budget constraint on the soil analysis.

Samples can also be taken with a ring-barreled sampler and sent to a laboratory for analysis. Particle size, plasticity index, and permeability analysis would provide enough information to assign a hydrologic soil group. These are common analyses that any soils laboratory should be able to perform. It is important that the sample be as undisturbed as possible; therefore, ring-barreled sampling is recommended. Although field analysis is generally cheaper, occasionally lab analysis is already required for other purposes, and these results can also be used. Detention basin design for runoff from mining areas (Smiley and Tang, 1991) or from landfills are instances when laboratory analysis may already exist.

3.2.4 Curve Number Assignment

The curve number assigned to the combination of hydrologic soil group, land use, and antecedent moisture condition (AMC) is used to determine what portion of rainfall will run off from the surface. This application is shown in Section 3.3.1. The portion of rainfall which does not run off is removed from the short-term hydrologic budget. These short-term losses include direct infiltration, small-scale detention storage (puddles), and interception by vegetation and buildings. The curve number method, originally developed by the Soil Conservation Service, is widely used for predicting runoff yield, and calibrated curve numbers for various land uses are available. Table 2 in Appendix A for AMC II (moderate antecedent soil moisture) is compiled from several sources (McCuen, 1982; OCEMA, 1986; AHD, 1968; and Hromadka and Whitley, 1989).

Curve numbers for AMC II should be adjusted for non-moderate antecedent soil moisture conditions. AMC I is used for dry conditions prior to the storm. According to McCuen (1982), the total rainfall for the preceding five days should be less than 0.5 inches for a dormant (limited plant growth) season or less than 1.4 inches for the growing season. AMC III is for a wet antecedent moisture condition with at least 1.1 inches of rain during the preceding five days in the dormant season or at least 2.1 inches of rain in the growing season. Upon selecting the AMC, Table 3 in Appendix A is used to adjust the curve number from AMC II, if necessary.

3.3 Continuous Simulation of Inflow

A continuous simulation of the detention basin performance is useful for optimizing a design that accommodates both flood control and water conservation objectives. In this methodology, the term continuous simulation refers to modeling the hydrologic budget every day for several years. Of course, the hydrologic budget includes the amount of water in the detention basin as well as the runoff yield from rainfall on the watershed. The number of years modeled will be dictated by the length of the measured recorded rainfall or streamflow with missing values estimated as done in Section 2.1.2. If the detention basin is intended solely for flood control objectives, an event-based (design storm) approach, as presented in Section 3.4 of this document, is usually sufficient as discussed in Hromadka and Whitley (1989).

Streamflow data for the watershed often do not exist or are insufficient. Streamflow data sufficiency is discussed in Section 3.5. Rainfall-runoff modeling provides a method for synthetically developing a streamflow hydrograph for continuous simulation of detention basin inflow.

There exists a wide range of rainfall-runoff models from simple, empirical models, such as the rational method, to complex multi-parameter, quasi-physically based models, such as KINEROS. In most cases, there is insufficient spatial and temporal rainfall data to justify using the more complex models (Schilling and Fuchs, 1986). The SCS method, as presented in McCuen (1982) is well-documented, easy to apply, and calibrated for a variety of land use-soil combinations. It does have a drawback in that the time step for runoff calculation is daily; however, most rainfall data are available on a daily basis anyway. For performance aspects which require shorter time steps, such as spillway performance, a design storm method is recommended as discussed in the Section 3.4.

3.3.1 Distributed SCS Method

Generally, curve numbers are calibrated for separate land use-soil group combinations. However, most drainage areas have a mixture of land use and soil groups. To account for such mixtures, McCuen (1982) suggested area-weighting curve numbers in order to obtain a composite curve number for the watershed. Another method is to divide the watershed into subareas based on topographic subarea watershed divides and compute the daily runoff separately for each subarea.

Some area-weighting will still be required for determining the subarea's curve number; however, reducing the instances of area-weighting has the following advantages. If it is possible to identify subareas with homogeneous land use-soil cover, that runoff calculation will be based on a calibrated curve number. For lower levels of daily precipitation, some subareas will contribute runoff, while others will not; this allows for partial area runoff production. Using such a distributed SCS method will produce runoff volumes slightly different from the single composite curve number method. The SCS equation for daily runoff is presented below:

$$Q = (P - 0.2S)^2/(P + 0.8S) \tag{3.1}$$

where:

Q = daily runoff (inches);

P = daily precipitation (inches);

S = (1,000/CN) - 10; and

CN = curve number.

It should be noted that this method is a distributed model in that the curve number parameter varies spatially. This is contrary to the definition of distributed models used by Beven (1985), who defined distributed models as models of catchment hydrology that are physically based. The distributed SCS model is both empirical and distributed, despite Beven's nomenclature.

No reduction in the daily precipitation from point estimates to areal estimates for watershed area is used because the recommended reduction for 24-hour rainfall is less than 5% for watersheds smaller than 50 square miles (NOAA, 1973). For situations when more than one rain gauge record is proximate to the watershed, distributed rainfall input is recommended rather than averaging the daily values due to the thresholds and nonlinearity of the distributed SCS equation. There are various schemes for assigning portions of the watershed to each rain gauge. It can be done heuristically by assigning each subarea to its closest rain gauge. It can also be done heuristically by assigning subareas based on elevation if there is enough relief in the watershed and the rain gauges are at different elevations. More objective methods can be used involving geometric analysis (e.g., perpendicular bisectors, polygons). However, averaging the values from each gauge and assigning it to the entire watershed is not recommended because extreme values are artificially filtered and because the runoff yield is not a linear relationship.

3.3.2 Channel Routing for the Distributed SCS Method

Some judgment is required to decide if it is necessary to include channel routing as part of the hydrologic modeling for detention basin inflow. Two situations when it may not be necessary, as well as discussion on when and how to apply it for the distributed SCS method, are described in this section.

For a small watershed with a time of concentration less than a few hours, almost all of the runoff should arrive at the detention basin within the same day as the occurrence of the rainfall event. Thus, for continuous simulation analysis using the distributed SCS method and daily time-step, it is reasonable to assume that the daily runoff volume is the sum of all of the subarea daily runoff volumes. Because channel infiltration losses are neglected, application of the distributed SCS model may overpredict detention basin inflow volume. If channel infiltration may be a significant part of the hydrologic budget, as in the case of desert washes, routing with an infiltration component should be included.

Continuous simulation of inflow is required for application of the dynamic programming algorithm for situations when a stream gauge does not exist or is insufficient. The dynamic programming algorithm for optimal detention basin operation is discussed in Chapter 7, and aggregation of streamflow records in order to use it is presented in Section 3.6. Because the seasonal periods for aggregation may be on the order of several weeks, the daily runoff volumes from the distributed SCS method can be used without the temporal redistribution of channel routing. However, if significant channel infiltration or other groundwater recharge is a hydrologic feature, the distributed SCS method may overpredict detention basin inflow volume, if channel routing is not used.

The daily time step used in the distributed SCS method produces a square-wave shape runoff hydrograph. Although it may be sufficient for the seasonal dynamic program optimal operation model, this is a very crude representation of what a real inflow hydrograph would look like. More realistic runoff hydrographs are required for spillway/outlet design, and methods to produce them are discussed in the next section. Simple hydrograph translation is a logical channel routing mechanism (when it is necessary) for the distributed SCS method. The translation time should be computed based on the routing length divided by the average velocity. The average velocity should be based on application of the Manning equation using the average flow rate of the subarea hydrograph. The resulting translated square wave will be partitioned between the daily time-steps. The detention basin inflow will be the sum of the subarea routed hydrographs.

If it is known that there are significant transmission losses to groundwater through channel bottoms, a reduction in flow rate along the channel routing segments can be approximated using a procedure described in Lane (1983). The following equations summarize the procedure for the case of no lateral inflow along the routing segment. Lateral inflow can be added at either end of the routing segment:

$$Q = \begin{cases} 0 & P \leq P_o \\ a + bP & P > P_o \end{cases} \tag{3.2}$$

where:

Q = outflow volume (acre-feet);

P = inflow volume (acre-feet);

P_o = threshold inflow volume (acre-feet);

a = coefficient; and

b = coefficient.

P is the daily routing segment inflow volume from the distributed SCS model. The threshold volume is given by the following equation:

$$P_o = -\frac{a}{b} \tag{3.3}$$

The coefficients a and b are functions of channel geometry and bed material characteristics. The bed material characteristics are categorized in Table 4 of Appendix A.

The coefficient, b, is given by the following equation:

$$b = e^{-kxw} \tag{3.4}$$

where:

e = base of natural logarithms;

x = length of reach (miles);

w = average width of flow (feet); and

k = decay factor tabulated (see Table 5 in Appendix A).

The coefficient, a, is given by the following equation:

$$a = \frac{a_o}{1-e^{-k}}[1 - b] \tag{3.5}$$

where:

a_o = is tabulated as shown in Table 6 of Appendix A.

3.4 Inflow From Design Storms

In order to estimate instantaneous peak flow rates for hydraulic design of spillways and other hydraulic controls, a design storm method can be used in conjunction with a rainfall-runoff model. The rainfall-runoff model must have a small time step to estimate the peak flow rate. The stochastic unit hydrograph method described in Hromadka and Whitley (1989) is a model which has many advantages. In general, unit hydrograph methods are easy to apply and perform as well as other more complex models (Hromadka and Whitley, 1989). Readily available software, including HEC1, produces consistent results using the unit hydrograph method (Swanson and Smiley, 1987; U.S. Army Corps of Engineers, 1985). The stochastic unit hydrograph method provides a confidence level on the range of peak flow rates. For this methodology, the stochastic variable will be the unit hydrograph ultimate discharge quantity. This is an original technique among stochastic unit hydrograph methods. The reason for developing this technique arises from the natural variability in runoff production from homogeneous soil cover complexes, as will be discussed in Section 3.4.1. The major purpose in using a stochastic model is that some regulatory agencies require the detention basin to protect against flooding from a specified recurrence interval (e.g., 100-year flood) with a certain confidence level (i.e., 85% confidence) (OCEMA, 1986).

The application of the stochastic unit hydrograph method, critical storm area analysis, and channel routing for multi-subarea drainage network models are described in the following subsections.

3.4.1 Stochastic Unit Hydrograph Method

The unit hydrograph method is a widely used procedure for transforming rainfall values into runoff values. It was originally presented by Sherman (1932) and modified for ungauged catchments by Snyder (1938). Graphical depictions of the basic concepts of the unit hydrograph method are found in Figure 1.18 of Hromadka and Whitley (1989), which is presented as Figure 2 in this dissertation. The novel approach developed later in this section will use total rainfall instead of effective rainfall, and the ultimate discharge quantity will be a stochastic value reflecting the natural variability of runoff yield. The unit hydrograph can be retrieved by offsetting the ultimate discharge graph by one time unit and calculating the difference.

Concepts associated with using stochastic integral equations for hydrologic modeling are presented in depth in Hromadka and Whitley (1989). The term stochastic is used to distinguish this approach from traditional deterministic unit hydrograph procedures which ignore the natural variability of runoff production. Pitfalls associated with stochastic modeling are also discussed. A common misapplication is assuming the independence of stochastic parameters which artificially reduces the output variance. The stochastic unit hydrograph method presented in this subsection is an original application not found in Hromadka and Whitley, but it relies on many of their concepts. The method described below is for an ungauged catchment with a multi-day design storm from Chapter 2 used as model input.

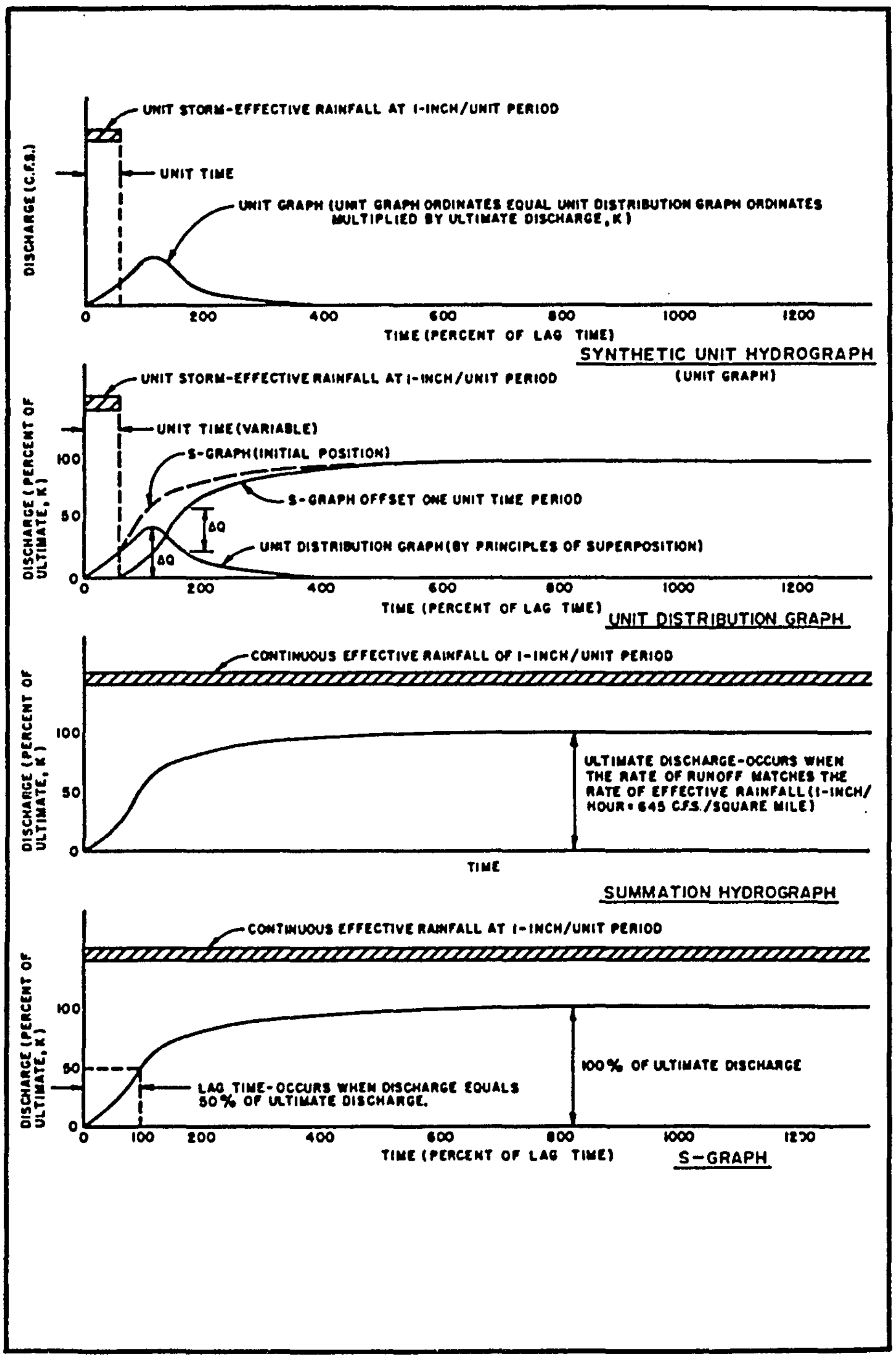


Figure 2. Definitions used in the Unit Hydrograph Concept.

The design storm point rainfall values should be reduced for the area of the watershed because very intense rainfall is typically limited to a smaller area at any given time, and the unit hydrograph method uses area-averaged rainfall. Average point precipitation reduction graphs found in Figure 14 of NOAA (1973) can be used. These reduction curves have been checked against rainfall data from many different areas of the United States and are consistent for many storm generation mechanisms (NOAA, 1973). In some cases, point precipitation reduction curves have been prepared by local regulatory agencies based on local rainfall data. In such cases, the local information should be considered for use. The procedure is to reduce the design storm rainfall values by the corresponding reduction percentages for watershed area, starting with the shortest duration-highest intensity values. Longer duration reductions, which will already include the shorter duration reductions, need to be consistent with the reduction curves for total rainfall reductions. In other words, if the total rainfall reduction for the peak one-hour rainfall exceeds the total required reduction for peak three-hour rainfall, the remaining two hours of rainfall values need to be increased to be consistent with both reduction curves.

Selection of an S-graph is the next step. It should be noted that the unit hydrograph based on the S-graph is for a free-draining catchment. Watershed subdivision and routing are required for any situation with upstream detention or choked flow at culverts. For existing undersized upstream culverts, consideration should be given to future alleviation of upstream flooding problems by modeling both existing and improved upstream conveyances. Subdivision of the watershed may also be used for different types of terrain within the catchment. For instance, the upper portion of the watershed may be in the foothills, while the lower part may be urbanized valley. Judgment should be used with the consideration that increasing model complexity typically does not improve performance and increases parameter uncertainty (Hromadka and Whitley, 1989).

Five S-graphs are presented in Table 7 of Appendix A. The first is the dimensionless S-graph developed by the Soil Conservation Service and is taken from McCuen (1982). The time axis is represented in terms of percentage of time of concentration. The ultimate discharge axis will be discussed later as a stochastic parameter. The other four S-graphs are modified from southern California hydrology methods (OCEMA, 1986). For these four, the time axis has also been represented as a percentage of time of concentration.

In general, the distinction between foothill and mountain watershed characteristics are as follows. Foothill watersheds are characterized by incised channels in relatively steep canyons without plunging reaches in the stream bed. Mountain-type watersheds include plunging reaches (waterfalls) along the stream bed. The valley-developed S-graph was calibrated for urban watersheds in southern California (Hromadka, 1985). Valley-undeveloped watersheds are characterized by floodplains and unlined channels or natural streams. The SCS S-graph is the average of various watersheds (McCuen, 1982).

Two parameters need to be estimated in order to proceed with the application of the stochastic unit hydrograph method. These are the time of concentration and the ultimate discharge. The standard definition of ultimate discharge in a unit hydrograph analysis is the theoretical flow rate observed at the point of concentration if a constant effective rainfall of one inch per time step fell on the watershed. The time step is typically assigned a value no greater than 1/4 of the time of concentration. This standard procedure assumes a deterministic rainfall loss subtracted from storm rainfall to obtain an effective rainfall amount. However, in this new method proposed here, runoff yield is a stochastic parameter, with the design storm used as input without subtracting rainfall losses.

As shown in Hjelmfelt and Burwell (1984), the runoff yield from identical plots experiencing almost identical rainfall due to their close spacing is quite variable. Extending this fact to a watershed leads to treating the ultimate discharge as a stochastic variable. The mean yield could be estimated using the SCS runoff equation presented in Section 3.3 with the maximum 24-hour rainfall, reduced for watershed area, from the design storm. Mean yield would be the ratio of runoff to rainfall. For larger rainfall events, Hjelmfelt and Burwell (1984) found that the spatial coefficient of variation of runoff yield was 0.17. A distribution is needed to represent the stochastic nature of the runoff yield fraction. The β distribution, as presented in Benjamin and Cornell (1970), has the required feature of having a range bounded by zero and one. This is required because ultimate discharge cannot be negative and should not exceed the deterministic design storm total rainfall volume.

The β distribution has been shown to arise from consideration of various underlying mechanisms and is often used to describe empirical data (Benjamin and Cornell, 1970).

The PDF of the β distribution is given by the following equations:

$$f_x(x) = \frac{1}{B} x^{r-1} (1-x)^{t-r-1} \qquad 0 \le x \le 1 \tag{3.6}$$

$$B = \frac{\Gamma(r)\Gamma(t-r)}{\Gamma(t)} \tag{3.7}$$

where:

$$\Gamma(a) = \int_0^\infty e^{-u} u^{a-1} du \quad \text{with } a \ge 1 \tag{3.8}$$

By using a coefficient of variation of runoff yield equal to 0.17 (Hjelmfelt and Burwell, 1984), the values for r and t (the parameters of the β distribution) are given by the following equations:

$$M_x = \frac{Q}{P} = \frac{r}{t} \tag{3.9}$$

$$\sigma_x^2 = r(t-r)/t^2(t+1) \tag{3.10}$$

$$\frac{\sigma_x^2}{M_x^2} = 0.17^2 \tag{3.11}$$

$$t = 34.602 \frac{P}{Q} - 35.602 \tag{3.12}$$

$$r = \frac{Q}{P}\,t \tag{3.13}$$

where:

P = area-reduced 24-hour rainfall (inches); and

Q = runoff (inches) using SCS equation found in Section 3.3.

The stochastic variable, x, will be used to scale the ultimate discharge. Caution is required in applying the β distribution. For average yields greater than 0.97, which would result from a large daily rainfall over a paved surface, the calculated value of t would be negative, which is nonacceptable, or $r < 1$, which is also not acceptable. These are unacceptable parameter values because they result in a U-shaped PDF. Upon determining the PDF for the particular mean yield, a CDF can be numerically calculated using a computer, and a uniform random number between zero and one can be selected. The uniform random number maps into a value of x through the CDF. Repeating this procedure numerous times results in a β-distributed set of values for x.

For each realization of x, the corresponding ultimate discharge is given by the following equation derived from conversion of units:

$$Q_{ult} = 645\,\frac{Ax}{\Delta t} \tag{3.14}$$

where:

Q_{ult} = ultimate discharge (cfs/inch of rainfall);

A = watershed area (square miles);

x = β-distributed parameter; and

Δt = unit hydrograph time step (hours).

This value of Q_{ult}, which is a stochastic value, is multiplied by the appropriate S-graph values tabulated in Table 7 of Appendix A. The unit hydrograph (UH) is the successive difference of the resulting cumulative discharge graph with the time step, Δt, being consistent between this graph and the time step of discretized design storm values. Thus, the unit hydrograph will be stochastic because Q_{ult} is stochastic. The time step should be no greater than one-fourth of the time of concentration, t_c. Some iteration may be necessary because t_c will depend on the peak runoff calculated from this unit hydrograph. The runoff hydrograph at any point in the catchment can now be estimated by convoluting the reduced design storm with the unit hydrograph. For a single realization, this is done using the following equation:

$$Q(t) = \int_0^{\infty} DS(s)\ UH(t-s)\,ds \tag{3.15}$$

where:

Q(t) = runoff hydrograph value at time t (cfs);

DS(s) = design storm rainfall during s^{th} interval (inches); and

UH(t-s) = unit hydrograph value for $t\text{-}s^{th}$ interval (cfs).

The time of concentration estimate can be improved by calculating runoff rates at various points along the major water course and using normal depth travel times. For steep slopes in mountainous terrain, the critical depth velocity should be used. In urbanized catchments, the channel slope between major road crossings should be obtained from as-built plans with exclusion of sharp elevation drops at culverts, rather than use the gross channel slope.

Upon obtaining a better estimate of time of concentration, the detention basin inflow hydrograph for each stochastic realization using this unit hydrograph method can be performed. The entire process for each stochastic realization can be accomplished on a computer. The final product of this process is a set of runoff hydrographs which are stochastic.

3.4.2 Critical Storm Area Analysis

The standard application of a unit hydrograph procedure uses the entire drainage area in the ultimate discharge calculation; however, there are cases when the critical storm area that produces larger runoff peaks and/or volumes is smaller than the entire catchment. Generally oval- or leaf-shaped watersheds will have standard S-graphs associated with them. Hourglass-shaped watersheds or watersheds with attached finger shapes require special consideration. In those cases, the design storm should be applied to a smaller area that more approximates the standard shape. This will increase the amount of design storm rainfall because of the increase in depth-area reduction factor values, and it will reduce the time of concentration which will increase peak runoff rates through application of the unit hydrograph procedure.

If there is a detention basin upstream of the one under consideration, application of the unit hydrograph method to the drainage area downstream of that detention basin may also produce increased runoff over that which would be estimated using a model of the entire area with routing.

For complex urban watersheds, multiple trials using different storm areas are sometimes necessary because both fast- and slow-responding subareas may be tributary to the detention basin point of concentration. Although discretization is usually not desirable due to increased parameter uncertainty (Sorooshian and Gupta, 1983), single-area, multiple subarea, and reduced critical storm area analyses may need to be performed to see if significantly larger runoff results from one of these alternatives for the modeled recurrence interval of the design storm. Computerization of the unit hydrograph procedure is essential to perform these alternative analyses expediently. Less than a decade ago, public works regulators were reluctant to require procedures which could not be performed with a hand-held calculator (based on personal knowledge during development of the Orange County Hydrology Manual (OCEMA, 1986)). However, with the affordability of personal computers, all engineers and hydrologists should be able to analyze for critical storm areas. Fortunately, the days of tedious hand calculations have ended.

3.4.3 Channel Routing for the Stochastic Unit Hydrograph Method

The Convex Method of channel routing can be used if routing is required due to discretization of the watershed in the unit hydrograph analysis. This allows the subarea runoff hydrograph to "flow" down to the detention basin. A detailed discussion of various convex routing method alternatives is found in Mockus (1972). Convex routing with a predetermined fixed time step is also presented in Hromadka and Whitley (1989). The governing equation arises from the theory of convex sets (Charnes, 1953). This method can model routing for the small time steps required for spillway and hydraulic control design. Its application to concrete channels in urban watersheds is recommended by some regulatory agencies (OCEMA, 1986). The theory does not preclude its application to soft-bottom channels; however, transmission losses should be considered similar to the discussion on channel routing for the distributed SCS method found in Section 3.3.

A velocity parameter is required to apply the convex method, and it is based on 75% of the peak discharge of the inflow hydrograph. The velocity parameter is set equal to the Manning's equation normal depth velocity for that level of flow rate. The routing coefficient, C, is given by the following empirical relationship developed from measured streamflow data (Mockus, 1972):

$$C = V/(V + 1.7) \tag{3.16}$$

where:

V = velocity parameter (ft/sec).

In order to calculate the convex time step, dT, the following equation should be used (Mockus, 1972):

$$dT = CL/3600V \tag{3.17}$$

where:

dT = convex time step (hr);

C = routing coefficient;

L = reach length (ft); and

V = velocity parameter (ft/sec).

A modified routing coefficient, C^*, for the predetermined fixed time step, dT^* (from unit hydrograph analysis), is calculated as follows:

$$C^* = 1 - (1 - C)^a \tag{3.18}$$

where:

$a = (dT^* + 0.5\ dT) / 1.5\ dT$

The outflow hydrograph is calculated using the following modified convex equation:

$$O_{t+dT} = (1 - C^*)\ O_{t+dT-dT^*} + C^* I_t \tag{3.19}$$

where:

O_{t+dT} = outflow hydrograph value at time t+dT (cfs);

$O_{t+dT-dT^*}$ = outflow hydrograph value at time t+dT-dT* (cfs); and

I_t = inflow hydrograph value at time t (cfs).

Linear extrapolation can be used to obtain each value of $O_{T+dT-dT^*}$ as the outflow hydrograph is determined.

3.5 Inflow from Streamflow Data

It is rare to have streamflow data at or near the proposed detention basin location. If a proximate stream gauge record does exist, it should be analyzed in addition to the aforementioned rainfall-runoff models. Caution is recommended before preferring the streamflow data over the modeling results. Of particular concern is the history of the watershed during the gage record. The length of the stream gauge record is also a concern. These concerns are elaborated on below and in the following subsections.

Frequency analysis of daily runoff volumes can be thought of as a set of realizations of the stochastic runoff generating process. Comparison against the distribution of daily runoff calculated from the stochastic unit hydrograph procedure as outlined in Section 3.4 is recommended. For small time step requirements required in hydraulic control design, frequency analysis of the instantaneous peaks in the record can be performed, provided that the gauge was not destroyed during extreme flow events.

Perhaps the most useful feature of a proximate stream gauge record is in aggregation of total flow for seasonal periods discussed in Section 3.6 and used in the dynamic programming algorithm described in Chapter 7.

3.5.1 Data Sufficiency

The stream gauge record must be long enough to estimate recurrence intervals if it is to be used in lieu of rainfall-runoff modeling. However, even short records can be used in a Bayesian analysis to recompute the basin curve number (Vicens et al., 1974), provided that the stationarity of the watershed land use is not a problem.

For use in developing a transitional probability matrix, enough measured seasonal periods are desired to create a unimodal set of transition probabilities for each storm class. This is discussed in more detail in the next section.

3.5.2 Stationarity

In urbanized watersheds, as is the case for most detention basin projects, stationarity is often a problem. Changes in land use during the period of record may prohibit direct estimation of recurrence interval flow rates using standard frequency analysis techniques. Such changes may also prohibit developing a transitional probability matrix (for use in Chapter 7) from streamflow data alone. Future land-use changes anticipated during the intended use of the proposed detention basin also prohibit use of the record (see Section 3.2) because the watershed response to rainfall will most certainly change. Aggregation of seasonal periods for development of the transitional probability matrix to be used in Chapter 7 is presented in the next section.

3.6 Aggregation for Seasonal Periods

In order for optimal design and operation of a detention basin to accommodate flood control and water conservation objectives, a dynamic programming method is presented (see Chapter 7). This method requires a streamflow transition probability matrix for seasonal periods. In essence, this matrix contains the probability of a streamflow class (a discretized amount of streamflow) occurring in the next seasonal period, given the streamflow class in the preceding period. The need for this arises because hydrologic processes, particularly weather and rainfall, are not time-independent processes.

Identification of seasonal periods was discussed in Section 2.1. The following subsections present details of the aggregation process.

3.6.1 Aggregation

The total flow in units of millions of acre-feet for every seasonal period is summed from the results of the continuous simulation analysis (Section 3.3) or from streamflow data (Section 3.5). Maintaining the time sequence of these aggregated flows is essential for developing the transitional probability matrix. The range of flow for any distinct seasonal period is used to classify the individual flow values (see Buras, 1985). In that example, ten classes of flow were used, and an appropriate level of detail was obtained in the resulting reservoir yield curves. The coupled dynamic programming-Gauss elimination computer program discussed in Chapter 7 will not execute on MS-DOS personal computer systems for too many seasonal periods due to array size limitations. Additional aggregation through a reduced number of streamflow classes should circumvent this problem.

3.6.2 Transition Probability Matrix

An example of the transitional probability matrix is presented in Buras (1985). This matrix contains the probability of a streamflow class occurring in the next seasonal period given the current streamflow class. Each row must sum to unity, and each row should be unimodal, that is, either monotonic or with a single peak. However, a unimodal row is not essential for the transitional probability matrix to be used. An exception to a unimodal row in the streamflow transitional probability matrix may result from climatic phenomena, such as the El Nino oscillation when the typical climatology for the region is not observed, but a "secondary" typical climatology for the El Nino is observed. This could result in a bimodal distribution in a set of rows of the seasonal transitional probability matrix. However, a more likely cause of a non-unimodal distribution would be from insufficient record (or model) length or too many seasonal periods. Thus, a non-unimodal row may indicate these potential deficiencies. The deficiencies could result in a calculated optimal release schedule from the dynamic programming routine, which is sensitive to additional years of record, in Chapter 7. Ideally, the optimal release schedule would be stable with respect to additional information provided by a longer streamflow record.

CHAPTER 4

DETENTION BASIN SEDIMENT INFLOW AND DEBRIS

Sediment from eroded soil and debris from discarded man-made items are likely to fill the detention basin. In most cases, runoff will carry and deposit these things. However, detention basins sometimes are attractive sites for illegal dumping or are convenient places for young people to store items they do not want to bring home. Other examples of urban debris are presented later in this chapter. In any case, a routine inspection and removal of sediment and debris is necessary to preserve the available volume for water storage. Upstream sediment traps can reduce the frequency of inspection and removal. For watersheds with area prone to occasional brush or forest fires, a contingency plan to inspect and remove potential additional sediment and debris is recommended. Watersheds which are close to beaches may require sediment restoration downstream from the detention basin; however, environmental regulations and wetlands protection may have to be addressed as well. All of the aforementioned aspects of detention basin sediment and debris inflow are discussed in this chapter.

4.1 Urban Debris

Based on personal experience, detention basins in urban areas will have a variety of man-made items either directly dumped into them or washed down into them. Items which I have found include shopping carts, wrecked motorcycles, bicycles, other children's toys, clothes, shoes, brief cases, bottles, refuse, yard clippings including pruned branches from trees, and automotive fluid containers. These items reduce the available volume for water storage and may hamper the performance of outlet works. An annual inspection and removal of urban debris should be performed under the direction of the regulatory agency responsible for the detention basin (typically, the local flood control agency).

4.2 Sediment Volume Prediction

Although much work has been done in developing quasi-physically based hydrologic models of sediment transport (Alonso et al., 1981), these models tend to require information on hillslope geometry and channel bed geometry that make them unwieldy for typical complex real-world watersheds. A more empirically based approach is presented in this section because of its ease of applicability. This approach is synthesized from Appendix A on reservoir sedimentation from Design of Small Dams (Bureau of Reclamation, 1987).

The sediment generated by the watershed can be approximated by the following equation modified from Design of Small Dams (Bureau of Reclamation, 1987):

$$q_s = 1.84\ (f_p A)^{-0.24} \qquad (4.1)$$

where:

q_s = annual sediment yield (acre-feet/square-mile);

f_p = pervious fraction of watershed (for urban watersheds $f_p \approx 0.5$); and

A = watershed area (square miles).

This differs from Figure A4 in Design of Small Dams by introducing the pervious fraction of the watershed. Figure A4 is shown on the next page as Figure 3. The original equation was developed from sediment yields in semi-arid climates in the southwestern United States. Its use in more humid areas may overpredict sediment yield because of increased vegetation; however, a conservatively higher estimate of sedimentation provides a higher level of protection.

Some of the sediment will pass through the outlet works of the detention basin. In order to estimate the percentage which will be trapped, the Churchill trap efficiency curve presented in Figure A9 of Design of Small Dams should be used. Figure A9 is shown with Figure A4 in Figure 3. The Churchill trap efficiency curve was developed from data from flood-retarding structures as opposed to the Brune curve for normally ponded reservoirs (Bureau of Reclamation, 1987). Combining the trap efficiency equation with the sediment yield equation produces the following equation:

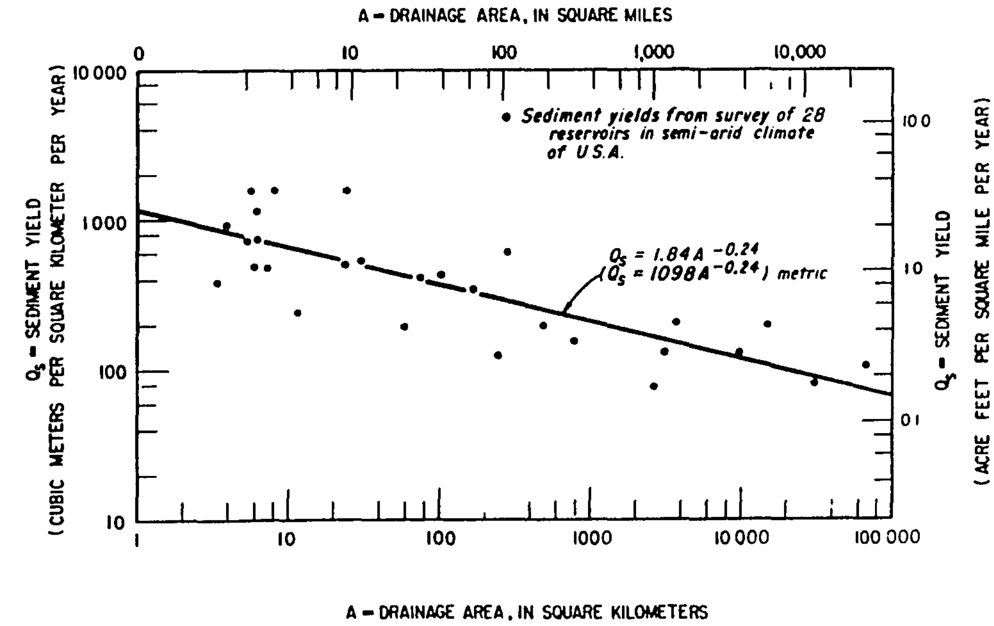
$$Q_s = (2.06 - 0.467\ C^{-0.2}\ Q_{avg}^{+0.2})\ f_p^{-0.24}\ A^{0.76} \qquad (4.2)$$

where:

Q_s = annual sediment trapped (acre-feet);

C = detention basin volume (acre-feet);

Q_{avg} = annual runoff (acre-feet);



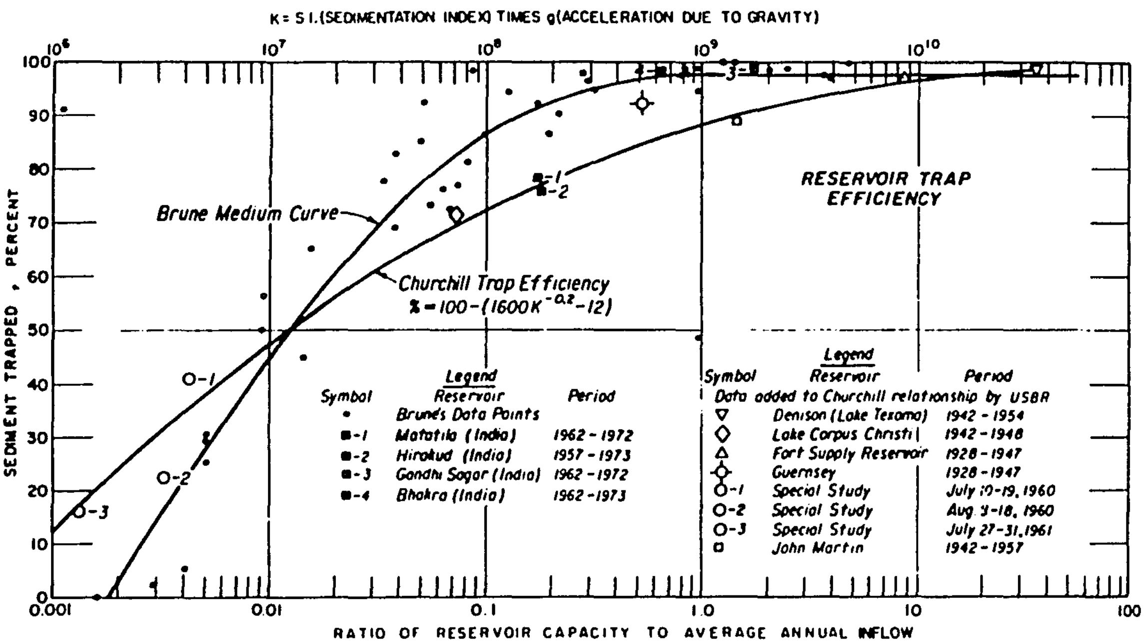


Figure 3. Sedimentation curves.

f_p = pervious fraction of watershed; and

A = watershed area (square miles).

The result of this equation should be checked against the following two standards. The amount of sediment generated from disturbed land is estimated to be 0.125 acre-feet/acre of watershed (EPA, 1976a). This would be a maximum expected value. A typical design value for a combination of disturbed and undisturbed land is 0.042 acre-feet/acre of watershed (EPA, 1976b). These values are from mining areas with high pervious fractions and erosion-prone land-use practices. Urban watershed sediment generation should be lower in magnitude.

4.3 Fire-Contingency Plan

Watersheds with area prone to brush or forest fires will experience increased vegetative debris flow and increased sediment flow during rain storms following a fire incident. Unabated, this additional debris and sediment can reduce detention basin and channel capacity rapidly and can cause flooding, including mudflows. To attempt to prevent this from happening, a fire-contingency plan should be in place, administered by the local regulatory agency responsible for maintenance of the flood control facility. This plan could include placing temporary sediment traps upstream of the detention basin constructed with sandbags. Inspection of the temporary traps and the detention basin should take place after the first rain storm after the fire. Further inspection and removal of the temporary traps, accumulated sediment, and debris should be done, as needed, and the fire-contingency plan should remain active until the watershed restabilizes with new vegetative growth. This inspection and cleaning is above and beyond what is called for during routine conditions. The sediment volume prediction discussed in the previous section does not account for recently burned areas.

4.4 Sediment Traps

It may be advantageous to construct a sediment trap immediately upstream of the detention basin. If this is not done, the detention basin will serve as its own sediment trap, and the low-flow outlet could become clogged. Standard designs for earth outlet and pipe outlet sediment traps can be found in Erosion and Sediment Control (EPA, 1976b). Plywood baffles, which serve to increase residence time and trap efficiency, can be designed and built, as shown in that same guidebook. For larger chanels, side channel sediment traps can be built. An example of such a design is the San Diego Creek facility, which was built by the Orange County Environmental Management Agency in southern California to reduce sediment loads delivered to Upper Newport Bay as part of the wetlands restoration project. Side channel sediment traps work by reducing in-channel flow velocity through reduced streambed profile slope and increased flow width causing finer particles to settle out along side benches along the channel. Periodic dredging of the side channel benches is easily accomplished because of adjacent access roads.

4.5 Right-of-Way Limitations

When retrofitting a detention basin into an urban area already developed, right-of-way limitations may contrain the design of the detention basin and sediment traps. Vehicular access within the right-of-way will also reduce available area. Typically, access roads will be 12-feet wide along the top of the channel bank with additional width taken up by access ramps down to the bottom of the detention basin. There may not be enough room to accommodate the expected design flood volume as well as the expected sediment inflow. If that is the case, reduced sediment volume capacity, combined with a more frequent inspection and cleaning, may be sufficient. If right-of-way is extremely limited, the detention basin should be located elsewhere.

4.6 Downstream Sediment Restoration

Although sediment is often considered as a pollutant (Glymph, 1975), and federal regulations treat it as such (e.g., Clean Water Act and Water Quality Act), and federal agencies monitor and issue permits related to sediment control (e.g., NPDES permits, COE 404 permits), sediment-free water will tend to degrade downstream earthen and natural waterways through increased erosion as the sediment-water system will attempt to achieve equilibrium loading rates. In addition, removal of sediment can hasten beach erosion as the source of silt and sand is cut off. On the other hand, sediment can combine with toxic metals or other organic pollutants, degrade wetlands through filling in of estuaries, and inundate agricultural and urban lands. Each watershed may have unique problems with respect to sediment. If the consensus among the regulatory and private sector actors is that the proposed detention basin will cut off too much sediment flow, downstream sediment restoration should be considered. Restoration could consist of dredging the upstream sediment trap and disposing of the material at a location downstream. An alternative solution is to construct a hydraulic control that allows bedload to proceed downstream unimpeded. Such a device has been built in Handy Creek in Orange County, California. It consists of a low-flow outlet as wide as the channel, with the sill slightly above the channel bottom and a steep drop within the outlet so that it is self-cleaning.

Conflicting objectives of regulatory agencies are common and can delay design and construction of flood control facilities, especially detention basins. The issue of who will pay for the downstream sediment restoration or monitoring and treatment of polluted sediment can be resolved through cooperative meetings and agreements. Public input in the decision-making process by way of public meetings can expedite the entire process in the long run, because political officials with control over funding authorization will be receptive to voters' concerns.

4.7 Maintenance Plan

Every detention basin should have a maintenance plan. This plan includes inspection and removal of sediment and debris. For basins with controlled outlet works, the maintenance plan should ensure that they are in working order. The frequency of activity associated with the maintenance plan should be a function of the design and watershed conditions rather than when funds and time are available. A reasonable maximum time between inspections would be one year. The legal entity owning the detention basin right-of-way should be responsible for implementing the maintenance plan. If private ownership is involved, the regulatory agency may issue conditions of approval associated with site plans or use permits to ensure the maintenance plan is adhered to.

A dredging record should be kept to modify the maintenance schedule as needed. If a smaller quantity of sediment is dredged than initial estimates indicated, a less frequent dredging schedule could be considered. However, the amount of sediment inflow will be a variable quantity, and a long dredging record will be valuable in anticipating the amount of sediment inflow.

CHAPTER 5

STORAGE LOSSES

Storage losses are reductions in stored water resulting from evaporation from the open water surface and infiltration through the bottom and sides of the detention basin. For extreme storm event analysis, storage losses can usually be ignored because they are a small portion of the water budget during the short time period under consideration. However, for continuous simulation of detention basin performance including the dynamic programming algorithm, evaporation and infiltration may account for a significant part of the water budget. For many concrete-lined urban detention basins, infiltration and other seepage are negligible.

5.1 Evaporation

Evaporation is the net change of state of water from liquid to vapor. For detention basins which do not drain rapidly or with dead storage capacity, evaporation could potentially be on the order of one foot per month. Two methods of estimating evaporation are presented in this section. The first is a method combining partitioning the solar energy budget, which provides heat to change the state of water, and advective transfer of moisture in the surface boundary layer of the atmosphere. This combination method was first developed by Penman (1948). The second method discussed here uses pan evaporation data available from many climatological stations and published by NOAA. Other empirical methods such as those discussed in Chow (1964) are not presented in this section because they tend to be site-specific.

The daily estimated evaporation should be subtracted from the stored water, if any, in the detention basin.

5.1.1 Penman Equation

Penman (1948) presented a method for estimating evaporation from an open water surface which combines partitioning of the solar energy budget and advective transfer of water vapor in the surface boundary layer of the atmosphere. The equation which accomplishes this is presented below:

$$E = \frac{H\Delta + E_a\gamma}{\Delta + \gamma} \tag{5.1}$$

where:

E = evaporation from open water surface;

H = radiation term;

E_a = advective term;

Δ = slope of the saturation vapor pressure curve versus temperature; and

γ = psychrometric constant.

The applicability of the original Penman equation was improved by Kohler and Parmele (1967) so that the temperature of the water surface was eliminated from the radiation and advective terms. The only temperature required was that of air, which is more readily available. The revised equation which is still in use today (Jones, 1992) is presented below:

$$E = \frac{(Q_{ir} - \varepsilon\sigma T_a^4)\Delta + E_a\,(\gamma + 4\varepsilon\sigma T_a^3/f(u))}{\Delta + \gamma + 4\varepsilon\sigma T_a^3/f(u)} \tag{5.2}$$

where:

E = evaporation from open water surface (J m^{-2} s^{-1});

Q_{ir} = difference between incident and reflected radiation (J m^{-2} s^{-1});

ε = ratio of molecular weights of water vapor to dry air = 0.622;

σ = Stefan-Boltzmann constant = 5.6696 x 10^{-8} J m^{-2} s^{-1} $^oK^{-4}$;

T_a = air temperature (^{o}K)

Δ = slope of saturation vapor pressure curve at T_a (Pa/^{o}K);

E_a = (e_s - e_a) f(u) in units of (Pa m s^{-1} = J m^{-2} s^{-1});

e_s = saturation vapor pressure at T_a (Pa);

e_a = actual vapor pressure of air (Pa);

f(u) = 0.0382 + 0.0269u;

u = windspeed (m s^{-1});

γ = C_p P/εL = 66.7 Pa/^{o}K;

C_p = specific heat of air at constant pressure = 1004 J $^oK^{-1}$ Kg^{-1};

P = atmospheric pressure (Pa) = 101325 Pa; and

L = latent heat of vaporization ($J\ Kg^{-1}$) = $2.453 \times 10^6\ J\ Kg^{-1}$.

It should be noted that f(u) and γ are expressed in SI units for perhaps the first time. Extensive review of evaporation literature did not show that this has been documented before. The formula for the psychrometric "constant", γ, was taken from Tanner and Pelton (1960). Most of the literature on evaporation does not even provide a value for it, much less an equation. Its value at 20°C and at 1013 mb is 0.667 mb/°K, rather than the value of 0.61 mb/°K which is sometimes used.

The combination method has been shown to perform well for time periods as long as one day (Van Bavel, 1966). In that paper, it was shown that, by using the daily average temperature and daily values of radiation, acceptable estimates of daily evaporation were obtained.

Temperature, windspeed, and humidity can be estimated from daily weather maps or taken directly from meteorological data bases. The radiation term, Q_{ir}, can be estimated by calculating incident radiation and subtracting reflected radiation from clouds and from the open water surface. The incident radiation per day is given in the following equation derived from spherical geometry (Morgan, 1945), which is a result of the integration of the instantaneous surface incident radiation over the duration of daily sunlight:

$$I = \frac{1}{\Delta t}\ \frac{W_o}{r^2}\ (\Delta t \sin\phi \sin\delta + \cos\delta \cos\phi\ (\sin t_2 - \sin t_1)) \qquad (5.3)$$

where:

I = incident radiation (J m^{-2} s^{-1});

$\Delta t = t_2 - t_1$;

t_1 = sunrise time = $-\cos^{-1}(-\sin\phi \sin\delta / \cos\phi \cos\delta)$;

t_2 = sunset time = $\cos^{-1}(-\sin\phi \sin\delta / \cos\phi \cos\delta)$;

ϕ = latitude expressed in radians;

$\delta = (23.45\pi \cos(2\pi(172\text{-}D)/365))/180$ in radians;

D = Julian calendar day;

$r = 1.0 + 0.017 \cos(2\pi(186\text{-}D)/365)$; and

$W_o = 1353$ J m^{-2} s^{-1}.

Some of this radiation will be reflected by the atmosphere and clouds. The amount of radiation incident not reflected by clouds can be estimated by the empirical relationship discussed in Selirio et al. (1971) and shown below:

$$Q_{ir} = I\,(0.23 + 0.57\,n/N) \tag{5.4}$$

where:

n/N is the fraction of cloud-free daylight hours.

The amount of cloud-free daylight hours can be estimated from daily weather maps.

Upon determining the estimated evaporation in units of Joules per square meter per second, conversion to meters per day or feet per day is accomplished by dividing E by the latent heat of vaporization, L, and by the density of water, ρ, and multiplying it by the number of seconds in a day, as shown in the equation below:

$$\hat{E} = 86400\ E/\rho L \tag{5.5}$$

where:

$\hat{E}$ = evaporation in meters per day;

ρ = density of water = 998.2 kg/m^3 at T = 20°C; and

L = latent heat of vaporization = 2.453 x 10^6 J/Kg at T = 20°C.

Therefore, for T = 20°C, the equation reduces to:

$$\hat{E} = 3.53 \times 10^{-5}E \quad \text{in meters per day}$$

$$\hat{E} = 1.16 \times 10^{-4}E \quad \text{in feet per day}$$

$$\hat{E} = 1.39 \times 10^{-3}E \quad \text{in inches per day}$$

5.1.2 Pan Evaporation Data

Pan evaporation data are sometimes available for a station near the watershed and can be found in the published climatological data by NOAA or directly from the National Climatic Data Center. When available, pan evaporation data are much easier to use than the Penman method, which requires daily values of temperature, windspeed, cloud cover, and humidity that would likely have to be interpreted from daily weather maps. The ratio of open water evaporation to pan evaporation is usually near 0.7 (Linsley and Franzini, 1979).

5.2 Infiltration

For soft-bottom detention basins, stored water will infiltrate into the surrounding earth. In some cases, this may be an objective for the detention basin when groundwater recharge is a priority. Whether intentional or not, a method for estimating infiltration is desired for continuous simulation of detention basin performance when infiltration may be a significant part of the total water budget. Infiltration can usually be neglected for short-term severe storm modeling and for concrete-lined (hard-bottom) detention basins.

Three methods will be discussed briefly, and one of these will be developed further. The methods are flow nets, simple infiltration estimates, and quasi-physically based estimates.

Flow nets are a graphical technique which involves drawing equipotential lines and stream lines in a profile view of the dam and its bottom and then estimating seepage using Darcy's law (Sowers, 1979). Although this method is often used for dams built in incised canyons, it has the limitations of two-dimensional flow and steady-state conditions. Detention basin seepage should not be simplified to a two-dimensional analysis because the sides often are not impermeable bedrock. In addition, the short residence time and rapidly fluctuating water levels invalidate the steady-state assumption.

Simple infiltration estimates based on the hydraulic conductivity of the detention basin bottom can be used, but by neglecting the static head and by neglecting the soil moisture budget, this estimate is crude. If this method is used, hydraulic conductivity based on soil sampling of the detention basin bottom at its level of expected compaction should be used. An estimate based solely on the SCS hydrologic soil group may be in substantial error if the bottom has been compacted during construction.

The method which will be developed in this chapter is an extension of the Green-Ampt method for estimating infiltration. It will explicitly include the static water level depth in the detention basin. The presentation of the Green-Ampt method in Koorevaar et al. (1983) serves as a reference. The explicit inclusion of a nonsteady-state static water level depth is a new and original aspect in this methodology and is needed to realistically model the infiltration process. Consider Figure 5.18 in Koorevaar et al. (1983), which is presented as Figure 4 in this text, but with a static water level, say d(t), ponded above the surface. Let $d(o) = d_o$ be the initial water level as a result of runoff into the detention basin. Conservation of mass suggests that the following equation is valid:

$$d_o - d(t) = -(\theta_o - \theta_i)\, s_f(t) \tag{5.6}$$

where:

d_o = initial water level;

$d(t)$ = water level at time t;

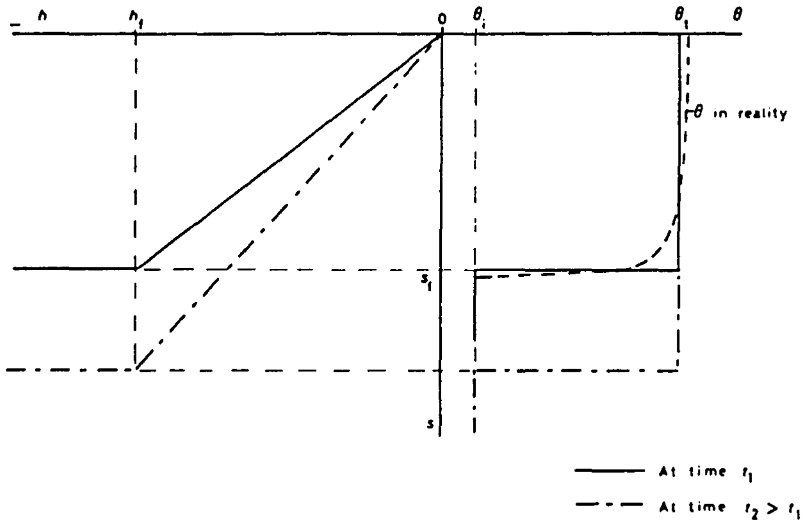


Figure 4. Green and Ampt infiltration model.

θ_o = saturated water content equal to porosity of soil;

θ_i = original soil water content before ponding perhaps equal to the field capacity; and

$s_f(t)$ = distance down to the wetting front which is a negative value with zero elevation at the detention basin bottom.

Rearranging this equation provides the following expression for depth to the wetting front which will be used later:

$$s_f(t) = -\frac{1}{\Delta\theta}(d_o - d(t)) \tag{5.7}$$

where:

$$\Delta\theta = \theta_o - \theta_i.$$

The decrease in ponded elevation will also be equal to the time integral of flux rate given by Darcy's law:

$$d_o - d(t) = -\int_o^t q(t)\,dt = -\int_o^t -k\frac{dH}{ds}\,dt \tag{5.8}$$

where:

$q(t)$ = Darcian flux (a negative quantity because it is downward);

k = saturated hydraulic conductivity;

H = gravitational plus matric head; and

s = distance along vertical axis.

Expanding the derivative of H with respect to s in the equation above yields the following:

$$d_o - d(t) = k\int_o^t \frac{dH}{ds}\,dt \tag{5.9}$$

$$d_o - d(t) = k\int_o^t \frac{H_1 - H_2}{0 - s_f(t)}\,dt \tag{5.10}$$

where:

H_1 = total head at detention basin bottom = d(t); and

H_2 = total head at wetting front = $h_f + s_f(t)$.

$$d_o - d(t) = k\int_o^t \frac{d(t) - h_f - s_f(t)}{-s_f(t)}\,dt \tag{5.11}$$

where:

h_f = initial matric head (a negative value) associated with the initial unsaturated water content θ_i.

Therefore:

$$d_o - d(t) = k\int_o^t \frac{h_f + s_f(t) - d(t)}{s_f(t)}\,dt \tag{5.12}$$

The water level, d(t), can be solved for using numerical integration.

The following subsections discuss acquisition of the necessary soil parameters and application of the above equation for continuous simulation analysis.

5.2.1 Soil Parameters

Continuous simulation of the detention basin storage losses via infiltration will require estimates of values of soil properties. As shown above in the equation for d(t), the parameters include porosity, field capacity, matric head at field capacity, and hydraulic conductivity. The term field capacity refers to the water content in soil when deep percolation has nearly stopped after thorough wetting. All of these parameters should be determined by a soils laboratory using specimens compacted at the level of expected compaction (which may be at natural unconsolidation). Ring-barreled sampling is a method which works well. Tests performed in the soils lab should be done in accordance with ASTM procedures. Consultation with the soils lab before sampling is recommended so that a sufficient quantity of material is obtained.

5.2.2 Continuous Simulation of Soil Water Budget

Application of the ponded infiltration equation developed in Section 5.2 will require tabulation of the depth of water in the detention basin, as well as tabulation of the soil water budget. The method is a very simplified model of reality because only two states of soil moisture level are permitted: saturation and field capacity. Nevertheless, the method should be an improvement over a simple constant infiltration rate and still be relatively easy to apply.

The equation for d(t) which has been developed only explicitly models a falling water surface during infiltration. Because the other components of the continuous simulation of detention basin hydrology use a daily time step, a daily time should also be used for the infiltration component. If runoff enters the detention basin during that time step, the corresponding depth, d(t), should be increased to account for it. If d(t) falls to zero through evaporation and infiltration, it will remain at zero until another runoff event and the evaporation of soil water will begin. The evaporation of soil water can be set at the potential evaporation rate, as developed in Section 5.1, or at some fraction of it. The loss of soil moisture should be accomplished by reduction from saturation to field capacity starting from the top of the soil profile and the drying front, analogous to $s_f(t)$, should be tabulated. This crude model of the drying cycle is for the ease of computation when the detention basin fills again. Recall that the extension of the Green-Ampt method requires a constant initial matric head and sharp wetting front. If the drying front is at a shallow depth when the next runoff event occurs, the wetting front will catch up to it, and the new wetting front at that instant will move down to the previous wetting front. As with all of the components of the hydrologic model of a detention basin, tabulations and calculations are more efficiently done on a computer.

CHAPTER 6

DETENTION BASIN DESIGN

The design process incorporates the hydrologic modeling with other aspects such as regulatory requirements, water use objectives, right-of-way limitations, utility relocations, access, land-use objectives, hydraulic control design, maintenance, and operation into a set of drawings suitable for construction. Many states require that these drawings be stamped by a registered civil engineer. The design process is an iterative procedure between the hydrologic modeling and the other aspects, with each often serving as a constraint on the other. The hydrologic components developed in this methodology may have to be performed several times during a single design process to accommodate these other aspects of the project.

In essence, the design process is a multi-objective decision-making process. Competing objectives may include level of flood protection, water conservation, construction cost, and land cost. Constraints may include available land space, limited budget, and regulatory requirements. The design process is often not treated as a classical multi-objective decision-making process because of issues such as accounting stance, politics, and changing information.

The most important hydrologic parameters that influence the design process are the maximum depth of water in the detention basin, because it directly controls the area of inundation, and the maximum instantaneous outflow from the detention basin because the downstream conveyance system must be able to accommodate it. By using the stochastic inflow hydrographs developed in Chapter 3, confidence estimates on peak outflow and maximum inundation can be made, as the volume requirements will be analyzed in Sections 6.1.3 and 6.2.4.

This chapter presents important items involved with detention basin design for two common types of detention basins. These types are referred to as flow-through and flow-by detention basins.

6.1 Flow-Through Detention Basin

A flow-through detention basin captures all of the runoff from the receiving watercourse into its storage volume. Concepts and geometry of the flow-through detention basin are presented in more detail in the following subsection. Conjunctive land uses, volume requirements, low-flow outlet design, and spillway design are also presented.

During a runoff event, the flow-through detention basin will reduce the flow rate from the rate in upstream receiving watercourse down to a rate which can pass through the low-flow outlet. The primary purpose is flood protection downstream of the detention basin. After the runoff event, the detention basin will continue to discharge from the low-flow outlet until the water level is down to the level of the low-flow outlet's intake. The low-flow outlet may be either controlled or uncontrolled. A controlled low-flow outlet will have a valve or variable gate setting providing a range of discharge rates for a given hydraulic head. The setting may be determined by the optimal release schedule from the dynamic programming algorithm presented in Chapter 7. An uncontrolled low-flow outlet may be an open pipe or an open pipe with flap gate; in either case, there is a single theoretical discharge for a given hydraulic head for uncontrolled low-flow outlets. During a major flood with spillway flow, the detention basin discharge will likely be a combination of spillway flow plus low-flow outlet flow.

If the low-flow outlet has a valve, gate, or other variable control device, an operation plan may be in effect utilizing an optimized operation scheme as presented in Chapter 7. Such a system may be included in the detention basin design to accommodate flood control and water conservation objectives.

6.1.1 Concepts and Geometry

A detailed topographic map of the detention basin site is essential for designing the geometry of the detention basin. A stage-storage curve or tabulation is the quantitative relationship between water depth and volume and is dictated by the basin geometry. In urban areas, right-of-way limits, utility locations, and other aspects may control the geometry of the basin. The stage-storage curve, which should also include area of the water surface at any depth, relates how the detention basin will perform upon receiving inflow, as shown later in this chapter.

Concrete-lined detention basins can have vertical walls as opposed to earthen detention basins which typically have a 2 (horizontal) to 1 (vertical) side slope. Riprap-lined detention basins can have slightly steeper slopes. Grass-covered side slopes should be flatter than compacted earth slopes with a 4 to 1 slope or flatter. In some cases, a natural hillslope may serve as a detention basin side. Railroad embankments, freeway embankments, and arterial highway embankments may also serve as detention basin sides, provided the detention basin drains quickly so that a phreatic surface does not develop.

Determination of the stage-storage curve can be accomplished by calculating the area of inundation at various depths and summing the volume of each "slice" represented by the area. The slices may be assumed to have prismatic or truncated cone shapes. The thickness of each slice is the difference in elevation between consecutive area calculations. The areas should be computed from a topographic map using a planimeter or by a square-grid counting procedure. If the topographic map has been digitized, software is available which can automatically develop the stage-storage curve.

6.1.2 Conjunctive Land Uses

Some urban areas have such high land costs that the detention basin may also serve as an area for other purposes. Land developers who are required to build detention basins may also receive some credits applicable to their open-space requirements, provided that the detention basin plan meets those requirements. Some conjunctive uses of detention basin land include park areas, golf courses, parking lots, ballfields, and natural open space. Of course, this area will be inundated by flood water occasionally. Careful design of the thalweg (very low-flow channel) through the detention basin is necessary. The geometry of the detention basin bottom will control what portions of the detention basin will flood more frequently. The low-flow outlet and the spillway should be located first and other intense conjunctive uses farther away. A fence or some other security may be desirable to separate the outlet works from the intense conjunctive uses. Warning signs indicating the danger of playing near the outlet works, especially during flows, may be placed. Flow-through basins allow for a dead storage pool which may support ducks or other birds sometimes found in parks. Maintenance of a constant dead storage pool volume may be augmented by the municipal water supply.

6.1.3 Volume Requirements

Hydrologic modeling for detention basins includes routing the inflow hydrograph through the detention basin for flow-through basins. Methods for developing detention basin inflow hydrographs are presented in Chapter 3, while this chapter shows how to process the hydrograph through the detention basin outlet works. Storage losses through sediment inflow and volume losses by evaporation and infiltration will likely be occurring as the inflow routes through the detention basin as discussed in Chapters 4 and 5. Modeling this routing produces an outflow hydrograph for downstream modeling and an assessment of detention basin performance with a hydrograph of detention basin stage (and, hence, inundation). Peak stage in excess of detention basin capacity may dictate a change in the design geometry. If so, the model must be performed again as part of the iterative design process as discussed at the beginning of this chapter.

Equations will be presented below for the routing of the inflow hydrograph through the detention basin for both controlled and uncontrolled low-flow outlet designs. Because the inflow hydrograph may be just one realization of the stochastic runoff hydrographs determined in Chapter 3, repeated realizations will result in a set of detention basin discharge hydrographs which are stochastic. From this set of discharge hydrographs, a set of associated instantaneous peaks can be determined, and a confidence on a stated discharge from the detention basin for the recurrence interval of the design storm can be made. Furthermore, a confidence on inundation area at the detention basin can be made. This is an improvement over a standard analysis which ignores the variability of runoff production.

If the detention basin has a variable control on the low-flow outlet, the amount of outflow may be controlled and set in accordance with an operation plan as discussed in Chapter 7. The equation to find the storage level at the end of each time step in the routing process is given by the following equation:

$$\begin{aligned} S_2 = S_1 + \frac{1}{2}(I_1 + I_2)C_1\Delta t - \frac{1}{2}(O_1 + O_2)C_1\Delta t \\ - \hat{E}A_1\Delta t - (d_o - d_1)A_1 \end{aligned} \tag{6.1}$$

where:

S_2 = storage (acre-feet) at end of daily time step;

S_1 = storage (acre-feet) at beginning of daily time step;

I_1 = inflow (cfs) at beginning of daily time step;

I_2 = inflow (cfs) at end of daily time step;

Δt = time step;

C_1 = 1.9835 (AF/cfs x day);

O_1 = outflow (cfs) dictated by operation plan (see Chapter 7) and controlled by variable low-flow outlet at beginning of daily time step;

O_2 = outflow (cfs) dictated by operation plan (see Chapter 7) and controlled by variable low-flow outlet at end of daily time step;

$\hat{E}$ = evaporation (feet/day); see Chapter 5;

A_1 = area of inundation (acres) can be found from stage-storage-area curve developed in Chapter 6.1.1;

d_o = depth (feet) at beginning of time step; and

d_1 = depth (feet) as calculated using infiltration equation in Chapter 5 for one time step.

For uncontrolled low-flow outlets, the modified Pul's method as presented in Chow (1964) can be used but with the addition of evaporation and infiltration terms, as shown below with known values on the right-hand side of the equation:

$$
\begin{aligned}
S_2 + O_2 C_1 \Delta t/2 = S_1 &- O_1 C_1 \Delta t/2 \\
&+ \frac{1}{2}(I_1 + I_2) C_1 \Delta t \qquad (6.2) \\
&- \hat{E} A_1 \Delta t - (d_o - d_1) A_1
\end{aligned}
$$

with all of the variables defined as before, except where:

O_1 = outflow (cfs) at beginning of time step from preceding modified Pul's calculation; and

O_2 = outflow (cfs) at end of daily time step determined by curve of outflow versus $S + OC_1 \Delta t/2$.

The key to using the modified Pul's method is to obtain a graph or tabulation of low-flow discharge (outflow) versus the quantity, $S + OC_1 \Delta t/2$. This relation can be easily developed from the basin geometry already discussed and with the low-flow outlet design to be discussed next.

6.1.4 Low-Flow Outlet Design

The detention basin may have either a controlled or an uncontrolled low-flow outlet. For most storm events, any discharge will be through this outlet. Extreme flood events with spillway flow will be discussed in the next subsection.

In the simple case of an uncontrolled low-flow outlet, a typical design consists of a pipe located near the bottom of the detention basin, placed horizontally through the downstream embankment or wall. A variation on this may include a vertical intake pipe joined with the pipe going through the embankment. In either case, the pipe delivers water from the detention basin to the downstream watercourse. Typical pipe materials include concrete, vitrified clay, corrugated metal, cast iron, polyvinyl-chloride, and polyethylene. A trashrack, which consists of metal bars attached to the intake, is sometimes included in the design to prevent large debris from clogging the outlet.

The discharge rate from the outlet will be governed by the water-level condition downstream. For free-draining outlets, the orifice equation can be used as shown below and adapted from Streeter and Wiley (1979):

$$O(t) = 0.475\ A_o \sqrt{2gd(t)} \tag{6.3}$$

where:

O(t) = outflow rate (cfs) from low-flow outlet under free-draining conditions at time t;

0.475 = coefficient of discharge for outlet pipes extending into detention basin volume;

A_o = pipe inside cross-sectional area (ft^2);

g = gravitational acceleration (= 32.174 ft/sec^2); and

d(t) = depth of water in detention basin above center of pipe (ft.) at time t.

For submerged outlets, as in the case of the downstream watercourse experiencing backwater conditions due to the storm event and mild slopes, the discharge from the outlet can be estimated using the following set of equations adapted from the discussion on closed-conduit flow in Streeter and Wylie (1979):

$$d(t) = H_d + \left(\frac{1}{2} + fL/D + (1 - D/H_d)^2 + K_t\right)\upsilon^2/2g \tag{6.4}$$

where:

$d(t)$ = depth of water in detention basin above center of pipe (ft.) at time t;

H_d = backwater elevation above center of pipe for submerged outlet condition (ft.);

f = friction factor;

L = pipe length (ft.);

D = pipe inside diameter (ft.);

K_t = trashrack loss coefficient;

υ = velocity of water in pipe (ft/sec); and

g = 32.174 ft/sec^2.

The above equation is solved for υ and then the flow rate is given by $O = A_o\upsilon$. The value H_d, which is the backwater elevation above center of pipe at the downstream end, will be a function of the storm event and channel geometry. For extreme storm analysis, a value of H_d may be assumed for the particular frequency of storm from consultation with the regulatory agency in charge of the downstream facility. Typically, they will have design water-surface elevations which are routinely given out for designing connections to their system. For continuous simulation analysis, downstream water-surface elevations should be incorporated into the hydrologic model by including the watershed and drainage features downstream of the detention basin. Open channel flow techniques, not covered in this methodology, will need to be used to model the downstream water-level conditions.

The friction factor, f, can be estimated using the following empirical equation derived from the Darcy-Weisbach equation (Streeter and Wylie, 1979):

$$f = 1070\ C^{-1.852}\ D^{-0.0184}\ Re^{-0.148} \qquad (6.5)$$

where:

f = friction factor in English units;

C = Hazen-Williams roughness coefficient;

D = pipe inside diameter (ft.);

Re = Reynold's number = $\upsilon D/\nu$;

υ = pipeflow velocity (ft/sec); and

ν = kinematic viscosity (ft^2/sec).

For SI units, the coefficient should be 1014 instead of 1070 (Streeter and Wylie, 1979). The Hazen-Williams coefficient is widely tabulated in the fluid mechanics literature. For concrete pipes in good condition, C = 130; for clay pipes and riveted steel pipes in good condition, C = 110. For old pipes in poor condition, C may be as low as 60. PVC and polyethylene pipes will have a high C value, and it is usually published in the catalog used to order these pipes from the manufacturer.

The trashrack loss coefficient, K_t, is given by the following empirical relationship taken from Design of Small Dams (Bureau of Reclamation, 1987):

$$K_t = 1.45 - 0.45 a_n/a_g - (a_n/a_g)^2 \tag{6.6}$$

where:

K_t = trashrack loss coefficient;

a_n = net area through rack bars; and

a_g = gross area of racks and supports.

For modeling a clogged trashrack, a_n should be reduced by 50%.

Controlled low-flow outlets may have a simple flap gate, a valve, or variable control gates as part of the outlet design. For conjunctive use with respect to water conservation, a flow-through detention basin with a controlled low-flow outlet is the appropriate design, as opposed to bypass basins which seldom receive water into their conservation pool. This should become obvious after reading the next section of this chapter on flow-by basins. In order to implement an optimized operation plan as developed in Chapter 7, a valve or variable control gate is required. The flap gate will not suffice because it lacks variable control. The purpose of incorporating a flap gate is to prevent downstream water from backing up into the detention basin. When a flap gate is included in the design, the downstream water-level condition must be included in the hydrologic model to account for the open or closed position of the flap gate. Valves and variable control gates should have their settings calibrated either as part of the original specifications or perhaps after they are installed. These variable control gates facilitate implementation of an optimized operation plan as presented in Chapter 7. Outflow from these devices will not be solely a function of water depth in the basin and the continuity equation in the volume requirements section of this chapter that specifies controlled outlet operation should be used.

6.1.5 Spillway Design

The purpose of a spillway is to direct overflow from the detention basin during extreme runoff events to the downstream watercourse and to protect the structural integrity of the detention basin. Only those aspects of spillway design related to hydrologic modeling are presented here. A thorough presentation of spillway design can be found in Design of Small Dams (Bureau of Reclamation, 1987).

Because detention basins are smaller than typical dams, the type of spillway will usually be of a simple design. Notched concrete weirs, rock-lined spillways, and even broad-crested or ogee spillways are likely candidates. Examples of these types can be found in Design of Small Dams (Bureau of Reclamation, 1987), Water Resources Engineering (Linsley and Franzini, 1979), in the EPA Technology Transfer Seminar Publication (1976), or in most engineering texts related to water resources. For most designs, the weir equation presented below is applicable for relating outflow to depth above spillway crest.

$$O = C_w \, L \, h^{1.5} \qquad (6.7)$$

where:

O = outflow from spillway (cfs);

C_w = weir (spillway) coefficient;

L = width of spillway perpendicular to flow (ft.); and

h = depth of water above spillway crest (ft.).

The weir coefficient, C_w, will vary depending on the type of spillway and may even be a weak function of h. Generally, C_w is around 3.5 in English units and approximately 2.0 in SI units. Calibration and research on the type of spillway should improve confidence and accuracy of the estimate of C_w. For preliminary design and hydrologic modeling, the above equation with the typical value given above will suffice.

As elaborated earlier in this chapter, design is an iterative process, and spillway size will depend on the result of the hydrologic modeling. The performance of the spillway with respect to its maximum discharge and maximum water level should be assessed using the extreme storm analysis (multi-day design storm) and from the continuous simulation analysis as a backup check.

6.1.6 Water-Level Recorder

A water-level recorder should be included in the detention basin design. It will provide data for future hydrologic modeling in the area. Furthermore, it will provide evidnece against false claims of flooding which may be filed against the local flood control agency.

6.2 Flow-By Detention Basin

A flow-by detention basin only receives water when the flow rate in the delivering watercourse exceeds a threshold value. This is typically accomplished by use of a side-weir between the watercourse and the detention basin, as will be discussed in the next subsection. Conjunctive land uses, side-weir design, volume requirements, low-flow outlet design, and spillway design are also presented for the flow-by detention basin in the remaining discussion in this chapter.

During most runoff events, flow will bypass the detention basin entirely; thus, the downstream flow rate will be as if the detention basin is not there. Only during extreme runoff events will the peak flow be "shaved" off as it spills over the side-weir into the detention basin. Once water enters the detention basin, it will either be released from the low-flow outlet back into the bypass channel, evaporate, infiltrate, or during very extreme flooding cascade over the spillway.

The flow-by detention basin design is not typically used when there is a conjunctive water use such as conservation or recharge. However, it is a preferred design for other conjunctive land uses because its bottom is almost always free of standing water and is seldom inundated. Downstream design discharge rates will tend to be based on the bypass channel capacity. These issues will be discussed in the following subsections.

6.2.1 Concepts and Geometry

As with the flow-through detention basin, a detailed topographic map of the site is essential for designing the flow-by detention basin. A stage-storage-area curve of the detention basin can be made from a preliminary plan on the topographic map. A rating curve for the side-weir will also be required for the hydrologic modeling. In urban areas, right-of-way limits, utility locations, and other aspects may control the geometry of the bypass channel and detention basin.

The side slopes of the detention basin depend on the construction material. Concrete-lined basins can have vertical walls, while compacted earth slopes are typically at 2 (horizontal) to 1 (vertical). Riprap (rock-lined) slopes may be steeper than 2 to 1, while grass-covered slopes are typically no steeper than 4 to 1. Natural hillslopes, railroad embankments, freeway embankments, and arterial highway embankments may also serve as detention basin sides, provided the detention basin drains quickly so that a phreatic surface does not develop. The detention basin side with the side-weir will typically be reinforced concrete.

The stage-storage curve can be determined by calculating the area of inundation at various depths and summing the volume of each slice represented by the area. The slices may be assumed to have prismatic or truncated cone shapes. The thickness of each slice is the difference in elevation between consecutive area calculations. The areas should be computed from a topographic map using a planimeter or by a square-grid counting procedure. If the topographic map has been digitized, software is available to perform these calculations automatically.

Development of the side-weir rating curve is discussed later in this chapter.

6.2.2 Conjunctive Land Uses

Flow-by detention basins lend themselves more readily to conjunctive land uses as opposed to flow-through detention basins because they are seldom inundated as most flow events are completely bypassed. Some of the discussion on conjunctive land uses as presented for flow-through detention basins (Section 6.1.2) is reiterated here for the sake of completeness.

Some urban areas have such high land costs that the detention basin may also serve as an area for other purposes. Land developers required to build detention basins may also receive some credits applicable to their open-space requirements. Some conjunctive uses of detention basin land include park areas, golf courses, parking lots, ballfields, and natural open space. As discussed in the preceding paragraph, flow-by detention basins lend themselves more readily to such uses because a threshold amount of flow is needed before any water spills over the side-weir into the basin; thus, the flow-by detention basin is seldom inundated.

In general, the conjunctive uses should be located away from the outlet works of the detention basin. A fence or some other barricade may be installed for security. Warning signs indicating the danger of playing near the outlet works during flow events may be placed.

Flow-by basins do not lend themselves to maintenance of a dead storage pool as in the case of flow-through basins because the low-flows are always bypassed.

6.2.3 Side-Weir Design

In a typical flow-by basin design, a bypass channel conveys most of the inflow around the detention basin. When the level of water in the bypass channel is high enough, it will spill over a side-weir into the detention basin. Usually, the side-weir is a notch in the concrete side-wall of the bypass channel. The hydrologic modeling for these types of detention basins must incorporate a hydraulic analysis of the side-weir performance prior to final design (final construction drawings).

A rating curve which relates upstream flow rate to the flow rate which spills over the side-weir is necessary as part of the hydrologic modeling of the detention basin. A hydraulic analysis of the water surface elevation for various inflow rates and for the channel and side-weir geometry is required to develop an accurate rating curve. A physical, scaled model of the side-weir may be constructed in order to assess weir coefficients for Froude numbers likely to be encountered (Mostafa, 1987).

The equation describing the water surface profile along the bypass channel is given by the following taken from Smith (1973):

$$\frac{dy}{dx} = \frac{S_o - S_f - \dfrac{Q}{gA^2}\dfrac{dQ}{dx} + \dfrac{Q^2y}{gA^3}\dfrac{db}{dx}}{1 - \dfrac{Q^2T}{gA^3}} \tag{6.8}$$

where:

dy/dx = slope of water surface profile in bypass channel along side-weir;

S_o = bed slope of channel;

S_f = slope of energy gradeline ($z + y + \upsilon^2/2g$) along channel axis;

z = elevation of channel bottom;

y = depth of flow in channel;

υ = average velocity in channel;

g = gravitational acceleration;

Q = flow rate in channel;

A = cross-sectional area of flow volume in channel;

b = bottom width of channel;

T = water surface width in channel; and

dQ/dx = change in flow rate in channel due to side-weir spill (a negative quantity).

Smith (1973) provided a flowchart for programming the water surface profile equation. In general, there are four cases to consider. (1) The normal depth (using Manning's equation) is lower than the side-weir height; hence, there is no flow over the side-weir; (2) Subcritical flow on a mild slope with normal depth greater than the weir height which is greater than critical depth (Froude number = $\upsilon / (gy_c)^{1/2} = 1$); (3) Supercritical flow on a mild slope with normal depth greater than critical depth which is above the weir height; and (4) Supercritical flow on a steep slope with critical depth greater than normal depth that is above the weir height.

For prismatic channels, $db/dx = 0$ and for rectangular channels $T = b$ and $A = yb$ so that the water surface profile equation can be simplified as follows:

$$\frac{dy}{dx} = \frac{S_o - S_f - yF\,\frac{dQ}{dx}}{1 - F} \tag{6.9}$$

where:

F = Froude number $= \upsilon/(gy)^{1/2}$.

The term dQ/dx, which is the flow rate over the side-weir, can be approximated by a relatively simple weir equation, such as:

$$\frac{dQ}{dx} = -C\sqrt{2g}\,(y-w)^{3/2} \tag{6.10}$$

where:

w = weir height; and

C = side-weir coefficient.

When using the simple form of the equation, various authors have determined C as a weak function of Froude number and determined empirically. A more rigorous treatment can be found in Hager (1987). For the typical case of a single side-weir constructed as a broad-crested weir using the top of a reinforced concrete wall of thickness B, Hager's equation would be as shown below:

$$\frac{dQ}{dx} = -\frac{3}{5}c\sqrt{gH^3}(Y-W)^{3/2}\left[\frac{1-W}{3-2Y-W}\right]^{1/2}\left[1-S_o\cdot\left[\frac{3(1-Y)}{Y-W}\right]^{1/2}\right] \tag{6.11}$$

where:

$c = 1 - 2/9(1 + E^4)$;

$E = (H - w)/B$;

$H = y + \upsilon^2/2g$;

$W = w/H$; and

$Y = y/H$.

When Hager (1987) applied this complex side-weir discharge equation to an experimental setup, he found that the theoretical versus experimental flow rates were within 5%.

A hydraulic analysis using both the water surface profile equation and the lateral outflow (over the side-weir) equation provides the information necessary to construct the side-weir rating curve. The rating curve should be expressed as flow rate in bypass channel upstream of side-weir versus flow rate over side-weir into detention basin. The inflow hydrograph for the severe design storm model and for the continuous simulation model can be partitioned between the bypass channel and flow into the detention basin using the side-weir rating curve. For preliminary detention basin design, the simpler side-weir equation can be used with approximations of the water surface profile; however, a rigorous hydraulic analysis should be performed later in the design process.

6.2.4 Volume Requirements

Unlike flow-through detention basins which receive flow every time there is flow in the upstream channel, flow-by basins only receive flow when the depth of flow in the bypass channel exceeds the height of the side-weir. The amount of flow into the detention basin is determined by the side-weir rating curve discussed in the previous section. The maximum volume stored in the detention basin will help determine the volume requirements based on this hydrologic model. If this maximum volume cannot be accommodated due to right-of-way constraints, consideration of redesigning the side-weir geometry to reduce the volume requirements should be made. For example, the height of the side-weir could be raised so that less flow enters the detention basin.

Because flow-by detention basins are to be kept empty except during high flows, the low-level outlet will likely be controlled by a flap gate which would only open when the depth of water in the detention basin exceeds the depth of water in the bypass channel during the receding portion of the hydrograph. The equation to find the storage level at the end of each time step in the routing process is given by the following continuity-based equation:

$$S_2 = S_1 + \frac{1}{2}(I_1 + I_2)C_1\Delta t - \frac{1}{2}(O_1 + O_2)C_1\Delta t - \hat{E}A_1\Delta t - (d_o - d_1)A_1 \tag{6.12}$$

where:

S_2 = storage (acre-feet) at end of time step;

S_1 = storage (acre-feet) at beginning of time step;

I_1 = inflow (cfs) at beginning of time step which equals the spill over the side-weir using the side-weir rating curve;

I_2 = inflow (cfs) at end of time step set equal to spill over the side-weir using the rating curve;

Δt = time step;

C_1 = 1.9835 (AF/cfs x day);

O_1 = outflow (cfs) at beginning of time step throuh flap gate if water level in detention basin is greater than in bypass channel;

O_2 = outflow (cfs) at end of time step through flap gate if water level in detention basin is greater than in bypass channel;

$\hat{E}$ = evaporation (feet/day); see Chapter 5;

A_1 = area of inundation (acres) that can be found from stage-storage-area curve;

d_o = depth (feet) in detention basin at beginning of daily time step; and

d_1 = depth as calculated using infiltration equation in Chapter 5.

The equation presented above, for routing the portion of inflow which spills over the side-weir and enters the detention basin, is for just one realization of the stochastic runoff hydrographs (i.e., inflow hydrographs) modeled in Chapter 3. Repeated realizations will result in a set of detention basin discharge hydrographs that are stochastic. From this set of discharge hydrographs, a set of associated instantaneous peaks can be determined, and a confidence on a stated discharge from the detention basin for the recurrence interval of the design storm can be made. Additionally, a confidence on inundation area at the detention basin can be made. This is an improvement over a standard analysis which ignores the variability of runoff production.

6.2.5 Low-Flow Outlet Design

For most flow-by detention basins, the low-flow outlet will be an open pipe with a flap gate preventing water from backing into the detention basin from the bypass channel. Hence, the detention basin will only drain when its water level is above the level in the bypass channel. A typical low-flow outlet design consists of a pipe through a reinforced concrete wall or earthen embankment with a pre-made cast iron flap gate at the downstream end within the bypass channel.

In most cases, the low-flow outlet will be submerged, and the following set of equations can be used to determine the instantaneous outflow rate which is used in the continuity equation presented in the last section:

$$d(t) = H_d + \left(\frac{1}{2} + fL/D + (1 - D/H_d)^2 + C_{fg} + K_t\right) \upsilon^2/2g \qquad (6.13)$$

where:

$d(t)$ = depth of water in detention basin above center of pipe (ft);

H_d = depth of water in bypass channel above center of pipe (ft);

f = friction factor;

L = pipe length (ft);

D = pipe inside diameter (ft);

υ = velocity of water in pipe (ft/sec);

g = 32.174 ft/sec^2;

C_{fg} = flap gate loss coefficient; and

K_t = trashrack loss coefficient.

The above equation is solved for υ, and then the flow rate is given by $O = A_o\upsilon$ where A_o is the cross-sectional area of the pipe. The value, H_d, can be determined using the same hydraulic model as was used for modeling the side-weir. However, if the low-flow outlet is sufficiently far enough downstream of the side-weir and there are no other hydraulic controls, a normal depth approximation using Manning's equation and the bypassed hydrograph may be used. For extreme storm analysis, a specific value of the water surface elevation in the channel for the given return period may be available from the regulatory agency in charge of the facility. Typically, they will have design water surface elevations for connections to their system.

The friction factor f can be estimated using an adaptation of the Darcy-Weisbach equation taken from Streeter and Wylie (1979):

$$f = 1070\ C^{-1.852}\ D^{-0.0184}\ Re^{-0.148} \tag{6.14}$$

where:

f = friction factor in English units;

C = Hazen-Williams roughness coefficient;

D = pipe inside diameter (ft);

Re = Reynolds number = $\upsilon D/\nu$;

υ = pipeflow velocity (ft/sec); and

ν = kinematic viscosity (ft^2/sec).

For SI units, the coefficient should be 1014 instead of 1070 (Streeter and Wylie, 1979). The Hazen-Williams coefficient is a function of pipe material and is widely published in fluid mechanics texts and in manufacturer's catalogs. For concrete pipes in good condition, C = 130; for clay pipes and riveted steel pipes in good condition, C = 110; for old pipes in poor condition, C may be as low as 60. PVC and polyethylene pipes will have a high C value.

The flap gate loss coefficient C_{fg} should be available from the manufacturer of the flap gate and may be a function of either differential head or velocity. Typical values range from 0.1 for fully open flap gates to 24.0 for gates barely open (Bureau of Reclamation, 1987).

The trashrack loss coefficient K_t is given by the following empirical relationship taken from Design of Small Dams (Bureau of Reclamation, 1987):

$$K_t = 1.45 - 0.45\ a_n/a_g - (a_n/a_g)^2 \tag{6.15}$$

where:

K_t = trashrack loss coefficient;

a_n = net area through rack bars; and

a_g = gross area of racks and supports.

For modeling a clogged trashrack, a_n should be reduced by 50%.

6.2.6 Spillway Design

The reinforced concrete side-weir may also serve as the emergency spillway for many flow-by detention basin designs. An assessment of the detention basin performance using the hydrologic and hydraulic models with the multi-day design storm as input should be performed. Mapping the expected area of inundation during such an event may help in the final design. A typical scenario during a severe runoff event may include the bypass channel and the detention basin completely inundated with the only outflow from the system controlled by conditions further downstream. During a maximum inundation event, the question of where will excess water go should be addressed. If a secondary spillway at a higher level than the side-weir is a design solution, a hydraulic model which may use the standard weir equation could be used. Assuming the primary spillway is the side-weir and, because of complete inundation, no net flow is taking place over it, the only outflow will be the sum of the outflow controlled by downstream conditions and whatever can pass over the secondary emergency spillway.

6.2.7 Water-Level Recorder

A water-level recorder should be included in the detention basin design. It will provide data for future hydrologic modeling in the area. In addition, it will provide evidence against false claims of flooding which may be filed against the local flood control agency.

CHAPTER 7

DETENTION BASIN OPERATION

This chapter presents the methodology for developing detention basin yield curves using a coupled dynamic programming-Gauss elimination algorithm. These curves are an estimate of the cumulative distribution function of detention basin release quantity for each seasonal period in the year. They reflect the performance of predetermined storage and release targets of the operation plan, as well as the optimal release schedule provided by the dynamic programming algorithm. This will assist in managing storm water to meet both flood control and water conservation objectives. The objective function in the dynamic programming algorithm is the sum of squared deviations from storage targets and release targets in water quantity units. The system is constrained by conservation of mass and by transitional probability matrices for detention basin inflow from one seasonal period to the next. Targets can be initially set heuristically by consideration of competing objectives of flood control and water supply (storage or recharge uses). Criteria for the detention basin release or level may be couched in terms of meeting a value by an established percentage of time (e.g., release 100 acre-feet during the seasonal period from May 15 to June 20 four years out of five for recharge purposes). If the detention basin yield curves indicate that the policy will not be achieved, release targets and storage targets may be adjusted, the algorithm reperformed, and yield curves re-evaluated with respect to the criteria.

Urban storm runoff contains pollutants which may exceed drinking water standards or other public health standards. However, it can be used in several beneficial ways, thereby reducing the demand for treated potable water. These beneficial uses include irrigation, municipal, and industrial uses. A vigorous sampling plan combined with flexible delivery to either potable or nonpotable systems can maximize the benefit of this water.

Groundwater recharge may also be a water conservation objective. In arid areas or near the coast, where the fresh water would be wasted by discharging into the ocean, detention basins can either directly recharge the aquifer or, through a controlled low-flow outlet, discharge water to downstream pits or streambeds at a reduced rate for recharge.

During nonflood season, detention basins may also be used to store and perhaps recharge imported potable water. The Miller basin in Orange County, California is one of several examples of this kind of dual use. A controlled low-flow outlet is necessary. Detention basin yield curves developed in Section 7.3 will not include this addition of imported water.

When water conservation is desired in the community where the detention basin is to be built, a gated or valved low-flow outlet can be incorporated into the detention basin design to control discharge and manage storage. To summarize the methodology for low-flow outlet design, Chapter 6 allows for determining the maximum design discharge for the low-flow outlet, while this chapter allows for a reduced discharge, by way of a valve or gate, for water conservation during nonflood seasons.

7.1 Release Targets

Targeted release values for the detention basin for the seasonal periods are required for the dynamic programming algorithm presented in Section 7.3. The number of seasonal periods each year was determined from the meteorological statistics in Chapter 2. For each seasonal period, both flood control and water conservation objectives are considered in the development of release targets. Adjusting the release targets to improve the yield curves developed in Section 7.3.5 can be performed as an iterative procedure. For example, it may be required to achieve a certain release amount or storage volume a percentage of years (e.g., at least 800 acre-feet during the month of April nine out of 10 years for waterfowl habitat). If the initial optimization model fails to achieve this, another simulation with adjusted storage and/or release targets may accomplish it without compromising other constraints.

It should be noted that the release targets are used for the evaluation of the objective function in the dynamic programming algorithm. An unattainable release target does not automatically invalidate this model. The algorithm will provide an optimal release policy constrained by the continuity relationship.

The units for the release targets should be acre-feet to be compatible with the dynamic programming algorithm.

7.1.1 Flood Control Considerations

In general, release targets should be high at the beginning of a rainy season and low the rest of the year. The maximum release target value should be the maximum design discharge for the low-flow outlet continuously discharging at that rate for the entire seasonal period. As determined in Chapter 6, this value would be compatible with the downstream conveyances, as well with the volume capacity of the detention basin, and would not cause flooding.

7.1.2 Water Conservation Considerations

In most cases, the water conservation considerations will be secondary to the flood control considerations for detention basin operation. For nonpeak release seasons, the release targets may be set at downstream water use rates. These use rates may include established water rights downstream and rates based on groundwater recharge. The maximum discharge of the low-flow outlet applied during the entire seasonal period serves as an upper limit on the release target value. The release target may also be zero if there is no use downstream and as much water as possible is to be stored in the detention basin. Any intermediate value between zero and the maximum release target value is permissible.

7.2 Storage Targets

Discretized storage targets for the seasonal periods are required for evaluating the objective function in the dynamic programming algorithm presented in Section

7.3. The number of seasonal periods each year was determined from the meteorological statistics in Chapter 2. For each seasonal period, both flood control and water conservation objectives are considered in the development of storage targets. Adjusting storage targets to improve the yield curves developed in Section 7.4 can be performed as an iterative procedure.

The units for the storage targets should be in acre-feet to be compatible with the dynamic programming algorithm.

7.2.1 Flood Control Considerations

In general, storage targets should be low at the beginning of the rainy season and high the rest of the year. The minimum storage target may be set at zero or at a value corresponding to floating a debris pool above the low-flow outlet sill. This minimum value would be for a seasonal period when flood control concerns predominate.

7.2.2 Water Conservation Considerations

In most cases, the water conservation considerations will be secondary to the flood control considerations for detention basin operation. For seasons when flood control is not imperative, such as the end of the rainy season, the storage target may be as large as the maximum volume in the detention basin at spillway level. Intermediate values may also be storage targets. Emergency water supply, such as for fire fighting, may guide selection of storage targets.

7.3 Optimal Operation

Using the release and storage targets, discretized inflow volumes, and transition probabilities, an optimal operation policy (release schedule) can be developed using a coupled dynamic programming--Gauss elimination algorithm. A computer code, written in FORTRAN, is provided in Appendix B. Detention basin yield curves, which are plots of the cumulative probability versus released volume for each seasonal period, are readily obtainable from the output of the computer code. The optimal operation policy associated with these curves is also obtainable from the output. This optimal operation policy allows for setting the valve or gate controlling the low-flow outlet for each seasonal period based on current storage volume.

7.3.1 Dynamic Programming Algorithm

A recursive backward-moving dynamic programming algorithm for reservoir operation policy is introduced in Chapter 7 of Water Resource Systems Planning and Analysis, written by Loucks et al. (1981). An important feature of Loucks' method is that the objective function is the minimization of departures from storage and release targets in water quantity units, rather than a traditional economic objective function in monetary units. Thus, the problems with the value of water and accounting stance are avoided.

The dynamic programming algorithm is closely related to a modification of the method in Buras (1985). The extension of the dynamic programming algorithm to compute an analytical set of yield curves using Gauss elimination is developed in this dissertation.

The continuity equation for season t is given by the following:

$$R_{ki\ell t} = S_{kt} + Q_{it} - E_{k\ell t} - S_{\ell,t+1} \tag{7.1}$$

where:

R = release (AF);

S = storage (AF);

Q = inflow (AF);

E = evaporation (AF);

k = initial storage index at beginning of seasonal period;

i = inflow index;

ℓ = final storage index at end of seasonal period;

t = season; and

t + 1 = next season.

For the last n seasons, the backward-moving dynamic programming objective function is given by the following equation:

$$f_t^n(k,i) = \min_{\ell} \left[B_{ki\ell t} + \sum_{j=1}^{nlevel} P_{ij}^t \, f_{t+1}^{n-1}(\ell,j) \right] \tag{7.2}$$

for all k,i when ℓ is feasible.

where:

f_t^n = total minimum expected value of system performance with n seasons to go at season t;

$B_{k\ell t}$ = value of system performance when initial storage is at index k, inflow is at index i, final storage is at index ℓ, and season is t;

nlevel = number of discretizations of streamflow;

P_{ij}^t = transitional probability from streamflow at index i in previous season to streamflow at index j in season t; and

$f_{t+1}^{n-1}(\ell, j)$ = total minimum expected value of system performance with n-1 seasons to go in season t + 1 for release in season t at final storage index ℓ and inflow at index j.

The system performance is the sum of squared deviations from storage and release targets.

7.3.2 Optimal Operation Policy

The output from the dynamic programming algorithm is a set of release amounts for all combinations of a seasonal period, initial storage index for that seasonal period, and inflow index for that seasonal period. This optimal set of release values is a subset of the feasible release amounts from continuity considerations. A release schedule independent of current inflow is also produced as output from the dynamic programming algorithm.

7.3.3 Gauss Elimination Algorithm

The Gauss elimination algorithm is needed because the dynamic programming algorithm only produces an optimal release schedule. The optimal release schedule provides the best release value for a set of current conditions (current storage level, current inflow). However, the Gauss elimination algorithm solves for the probability of each combination of conditions in the release schedule. These probabilities are a proper probability mass function and can then be easily transformed into a cumulative distribution function plotted as a detention basin yield curve.

The Gauss elimination algorithm calculates the probability mass function for the optimal release schedule. The probability assigned to each combination of inflow, storage, and optimal release is a result of the transitional probability matrices of seasonal inflow, which are input to the dynamic programming optimization. A sorting algorithm is then used to calculate the cumulative distribution function for direct plotting of the detention basin yield curves.

The Gauss elimination algorithm is adapted from the one found in Chapter 20 of Advanced Engineering Mathematics (Kreyszig, 1983). It is used to solve the system of equations describing the joint probability of initial storage, inflow, and release given in the following equations. As an example, the equations listed below are for a system discretized into ten inflow levels, ten storage levels, and three seasonal periods.

$$\text{Eqn 1:} \quad \sum_{k}\sum_{i} PR_{ki1} = 1 \qquad (7.3)$$

$$\text{Eqns 2-100:} \quad \sum_k \sum_i PR_{ki1}\, P_{ij}^1 - PR_{tj2} = 0 \tag{7.4}$$

$$\text{Eqn 101:} \quad \sum_k \sum_i PR_{ki2} = 1 \tag{7.5}$$

$$\text{Eqns 102-200:} \quad \sum_k \sum_i PR_{ki2}\, P_{ij}^2 - PR_{tj3} = 0 \tag{7.6}$$

$$\text{Eqn 201:} \quad \sum_k \sum_i PR_{ki3} = 1 \tag{7.7}$$

$$\text{Eqns 202-300:} \quad \sum_k \sum_i PR_{ki3}\, P_{ij}^3 - PR_{tj1} = 0 \tag{7.8}$$

where:

PR_{kit} is the joint probability of initial storage at index k and inflow at index i for season t and the corresponding release amount determined by the dynamic programming algorithm; and

P_{ij}^t is the transitional probability from streamflow at index i in season t to the streamflow at index j in the next season.

Equations 1, 101, and 201 in the above example reflect the requirement that the joint probability for each seasonal period must sum to unity. The remaining equations arise from marginal probability being equal to the sum of the probabilities in each row of initial storage volume-seasonal inflow combinations.

The computer program provided in Appendix B will automatically set up and solve this system of equations.

7.3.4 Computer Programs

A series of programs have been written in FORTRAN to perform the dynamic programming, Gauss elimination, and sorting algorithms. They have not been linked through the use of subroutines in order to maintain the size of the executable code below the limit of operational memory available on MS-DOS systems.

Only one input file needs to be created for the execution of the series of programs. It includes parameters on the number of seasons per year, number of discrete inflow levels per season, and number of discrete storage levels in the detention basin. The file also contains the storage targets for each season, release targets for each season, and average evaporation for each season. The values of inflow for each discrete interval for each season, detention basin volume for each discrete level, and surface area for each discrete detention basin volume are also required. Finally, the transition probabilities for streamflow levels from one season to the next are input.

The following is a line-by-line instruction for creating the input file to match the existing format statements.

Line 1	Format 72A1	Description of problem statement for output heading

Line 2	Format 4I3	Number of seasons per year, number of discrete streamflow levels, number of detention basin volume levels, ITRACE (= 1 for full output; = 0 otherwise)
Line 3	Format nF8.2	Storage target values in units of acre-feet for each of n seasons (n = 10 max)
Line 4	Format nF8.2	Release targets in units of acre-feet for each of n seasons (n = 10 max)
Line 5	Format nF8.2	Total potential evaporation in feet for each of n seasons (n = 10 max)
Line 6	Format mF8.2	Discretized streamflow values for season number 1 (m = 10 max discretizations) in units of acre-feet per season
Line 7	Format mF8.2	Discretized detention basin storage values (m = 10 max) in units of acre-feet
Line 8	Format mF8.4	Discretized detention basin surface area in acres for each discrete storage value
Line 9	Format mF8.2	Repeat Line 6 but for season number 2
Line 10	Format mF8.2	Repeat Line 7 identically
Line 11	Format mF8.2	Repeat Line 8 identically

This set of three lines is continued for all of the seasons. For the case when there are three seasons per year, Line 15 is the first line of the transitional probability matrix.

Line 15	Format jF8.2	Transitional probability of streamflow from season 1 to season 2 for level 1 in season 1 to the j levels of season 2
Line 16	Format jF8.2	Same as Line 15 but for level 2 in season 1 to the j levels in season 2

.

Continue this for the remaining levels in season 1.

For the case when there are ten discrete streamflow levels, Line 25 is the first line of transitional probabilities from season 2 to season 3. Lines 35 through 44 are the set of transitional probabilities from season 3 to season 1.

The following list of commands is a set of instructions for executing the series of programs which will create an output file of optimal detention basin operating policy and sets of x,y coordinates for plotting the detention basin yield curve for each season.

```
> F77 DPRC1.FOR
> LINK DPRC1
> RUN DPRC1
> F77 GAUSS.FOR
```

> LINK GAUSS

> RUN GAUSS

> F77 SORT.FOR

> LINK SORT

> RUN SORT

Separate compilation and linking are required to keep the size of the executable code less than the available operational memory. Variations in the compilation, link, and run commands should be expected depending on the particular FORTRAN compiler in use.

This series of programs creates four additional files. The file OUTPUT.TBL contains the input value in tabular form and every combination of season, storage level, inflow level, release level, and deviation from target values. At the end of that file are the optimal release schedule and release schedule independent of current inflow. The file OPTPOL.DAT only includes the optimal policy for input to GAUSS. The file GAUSS.DAT is the output file from GAUSS and serves as the input file to SORT. It contains the joint probabilities for the optimal solution policy. The file SORT.DAT contains the cumulative distribution points for plotting release versus probability for each season based on the optimal operation policy given sets of target values.

Appendix B contains the FORTRAN source codes for DPRC1, GAUSS, and SORT.

7.3.5 Detention Basin Yield Curves

Detention basin yield curves for each season can be plotted directly from the file SORT.DAT. They are a plot of the cumulative distribution function of the detention basin release for a seasonal period. In other words, they show what fraction of years a release amount will be equaled or exceeded for a seasonal period. Water supply planners can use these probability curves to plan for a range of available water from each detention basin. Storage and release targets, especially for intermediate levels, can be adjusted and the programs rerun to find out if the yield curves can be improved. With a better estimate of water conservation benefits through the use of these curves, perhaps the cost of constructing the detention basin can be offset.

CHAPTER 8

CONCLUSION AND SUMMARY

A tractable methodology for the hydrologic modeling used in flood control detention basin design and operation has been developed and presented in the preceding chapters. It is my belief that this dissertation benefits the community of civil engineers, planners, and hydrologists, as well as the public seeking flood protection to prevent property damage. The methodology developed in this dissertation is tailored to utilize computer simulation and includes recent advances in hydrology, while recognizing data availability and political reality in the design process. Accounting for natural variability in the rainfall-runoff relationship through the use of stochastic modeling should provide a more realistic analysis compared to old deterministic methods, which may result in economic inefficiency caused by factors of safety. Although economic objectives are not explicitly treated, this methodology provides a way to determine the relation between detention basin size and inundation during extreme stormwater runoff events which would be used in an economically based decision on the viability of the design. Design variables including outlet capacity, storage volume, spillway capacity, and bypass channel capacity, which rely on hydrologic analysis, are determined using this methodology. Furthermore, conjunctive land and water uses are addressed which should be included in an economically based decision. New and original modeling techniques, in addition to documentation of personal experience, can be found throughout this treatise and summarized in the next paragraph. However, there is no intention that this methodology is the best approach to every flood design problem. In addition, some components of this methodology may be better suited than others for a particular situation. Experience, common sense, and engineering judgment have no substitutes and are required to use the components in this methodology. By its very nature, construction of a detention basin will affect the local environment. These effects may be major or minor, adverse or enhancing, but should always be considered.

To summarize the new and original work presented herein, the following statements about the overall topic should be made. The subject of hydrologic modeling for flood control detention basin design and operation has never been specifically addressed as a tractable and comprehensive methodology. Incorporation of personal experience with state-of-the-art model components runs throughout the document. In addition, each chapter contains new and original work. Chapter 2 (on precipitation data analysis) includes discussion on storm direction with respect to watershed orientation, which is potentially useful with adequate wind and rain data; discussion of climatology and operational storage and release targets for use in the optimal operation scheme in Chapter 7; an integral limit ($0.5\,\chi^2$, 0.05, ν) in the Chi-square histogram test for choosing the best distribution for frequency analysis; rainfall thresholds for extreme storm candidates in order to quickly and automatically select them out of copious data; autocorrelation of hourly data from extreme storms for construction of multi-day design storms; and application of multiple discriminant analysis for storm classification, especially useful when a single storm-generation mechanism cannot be determined by examination of the daily weather maps. For detention basin inflow (the subject of Chapter 3), new and original work includes discussion of experience with watershed mapping never documented before with suggestions to make it easier, including initial mapping, watershed delineation, anthropogenic changes, existing information, and field mapping techniques. That chapter also contained a discussion of land-use anticipation and discussion of soil sampling including in-the-field estimates of clay content, both of which are useful for predicting the rainfall-runoff relation; tips for modeling undersized culverts which may perform as a hydraulic control limiting peak discharge at their location in the watershed; use of the Beta distribution to stochastically model the ultimate discharge parameter in a unit hydrograph method; channel routing using as-built slopes excluding drops at culverts to more realistically model the velocity of flow; speculation on a causal link between El Nino climatic episodes and potential bimodal distributions within a transitional probability matrix of streamflow for use in the optimal operation scheme in Chapter 7; and a discussion of the need for critical storm area analysis. The chapter on detention basin sediment inflow (Chapter 4) includes a discussion on urban debris in detention basins which reduce the storage volume; combination of detention basin trap efficiency equation with sediment yield equation to obtain annual sediment trap estimates for detention basins; fire-contingency planning to anticipate increased sedimentation; side-channel sediment traps for ease of cleaning; right-of-way limits and the effect on a sediment trap design; and downstream sediment restoration to prevent downstream erosion and beach degradation; and conflicting regulatory objectives which complicate the design process. The loss of stored water to evaporation and infiltration is the topic of Chapter 5. New and original work in that chapter includes a modified version of the Penman equation in SI units for direct input of meteorological state variables, re-establishment of the psychrometric constant as 0.667 mb/K missing in the literature since 1960 (Tanner and Pelton, 1960), and extension of the Green-Ampt method for estimating infiltration to include a fluctuating static water depth in a detention basin. Chapter 6 (on detention basin design) begins with an original discussion on "What is design?"; other new and original work includes a discussion of right-of-way limits and utility locations as design constraints, conjunctive land uses for detention basins, a side-weir hydraulic design procedure incorporating various state-of-the-art methods taken from recent literature, incorporation of water surface elevations given by local regulatory agencies for hydraulic modeling and design of outlet works, and a discussion on flap gate design so that water does not back into the detention basin filling it unintentionally. Chapter 7, in which detention basin operation is discussed, also includes original discussion on conjunctive water uses for flood control detention basins, establishment of storage and release targets which are used in the optimal operation algorithm, and a new routine for developing a closed-form solution for detention basin yield curves by coupling a Gauss elimination algorithm with dynamic programming executable on MS-DOS personal computer systems. The yield curves reflect predicted performance of the detention basin with respect to water use and flood control policies.

This dissertation presents a methodology for performing the hydrologic analysis to be used in the design and operation of flood control detention basins. Up until now, the subject has not been addressed separately, comprehensively, or tractably. There is no single standard procedure to perform the hydrologic modeling necessary for detention basin design and operation. Techniques in current use do not take advantage of computer technology or recent advances in the science of hydrology. Perhaps this work will serve as a technology transfer between academia and engineering practice.

It is my hope that this work will be used for stormwater management. Although regulatory agencies cautiously adopt new methods, the advent of computer technology and the complexity of hydrologic modeling will replace old rules-of-thumb, nomographs, and factors of safety with sophisticated statistical analyses and assessments of natural variability. The old techniques may have worked and should not be forgotten, but we have better tools and better methods.

APPENDIX A

TABLES

Table 1. Example of Multi-Day Design Storm (ref. Section 2.2.7).

Time (hour)	Rainfall (in/hr)	Time (hour)	Rainfall (in/hr)
1		41	0.46
2		42	2.25
3		43	0.31
4		44	
5		45	
6		46	
7		47	
8		48	
9		49	
10		50	
11	0.27	51	
12	2.25	52	
13		53	
14		54	
15		55	
16		56	
17		57	
18		58	
19		59	
20		60	0.10
21		61	0.10
22		62	0.10
23		63	0.10
24		64	0.10
25		65	0.15
26		66	0.15

27		67	0.15
28		68	0.16
29		69	0.19
30		70	0.20
31		71	0.46
32		72	2.25
33		73	0.31
34		74	0.20
35		75	0.15
36		76	0.15
37		77	0.15
38		78	0.15
39		79	0.10
40	0.09	80	0.05

Table 2. Curve Numbers (AMC II) (ref. Section 3.2.4).

	Hydrologic Soil Group			
Land Use	A	B	C	D
Residential				
Single-Family Lots with Lawn/Shrub Landscaping (in acres)				
0.100	85	90	92	93
0.125	77	85	90	92
0.167	65	77	84	88
0.250	61	75	83	87
0.333	57	72	81	86
0.500	54	70	80	85
1.000	51	68	79	84
2.000	46	65	77	82
2.500	39	60	72	77
Single-Family Lots with Natural Desert Landscaping (in acres)				
0.100	91	94	95	96
0.125	86	91	93	95
0.167	81	88	92	93
0.250	76	85	90	92
0.333	74	83	89	91
0.500	72	82	88	90
1.000	70	81	87	89
2.000	67	80	86	88
2.500	66	79	85	88

Single-Family Lots with Artificial Desert Landscaping				
0.100	98	98	98	98
0.125	97	97	97	97
0.167	97	97	97	97
0.250	97	97	97	97
0.333	97	97	97	97
0.500	97	97	97	97
1.000	96	96	96	96
2.000	96	96	96	96
2.500	96	96	96	96
Mobile Home Park				
Lawn/Shrub Landscaping	82	88	91	92
Natural Desert Landscaping	89	93	95	96
Artificial Desert Landscaping	97	97	97	97
Condominiums				
Lawn/Shrub Landscaping	77	85	90	92
Natural Desert Landscaping	86	91	93	95
Artificial Desert Landscaping	97	97	97	97
Apartments				
Lawn/Shrub Landscaping	85	90	92	93
Natural Desert Landscaping	91	94	95	96
Artificial Desert Landscaping	98	98	98	98

Commercial and Business Districts				
Lawn/Shrub Landscaping	89	92	94	95
Natural Desert Landscaping	93	95	96	97
Artificial Desert Landscaping	98	98	98	98
Industrial Districts				
Lawn/Shrub Landscaping	81	88	91	93
Natural Desert Landscaping	88	92	94	95
Artificial Desert Landscaping	97	97	97	97
Schools	58	73	81	84
Urban Open Space (lawns), Parks, Golf Courses, Cemeteries				
Poor Conditions (Grass <50%)	68	79	86	89
Fair Conditions (50% < Grass < 75%)	49	69	79	84
Good Conditions (Grass >75%)	39	61	74	80
Other Landscaped Areas				
Lawn/Shrubs	32	56	69	75
Turf (Poor)	58	74	83	87
Turf (Fair)	44	65	77	82
Turf (Good)	33	58	72	79
Natural Desert	63	77	85	88
Artificial Desert	96	96	96	96

Developing Urban Areas				
Newly Graded Areas	77	86	91	94
Paved Areas				
Parking Lots, Driveways	98	98	98	98
Streets and Roads				
Paved, Curbs, and Storm Sewers (excluding right-of-way)	98	98	98	98
Paved with Open Ditches (including right-of-way)	83	89	92	93
Gravel	76	85	89	91
Dirt	72	82	87	89
Agricultural Areas				
Fallow				
Bare Soil	77	86	91	94
Crop Residue Cover (Poor)	76	85	90	93
Crop Residue Cover (Good)	74	83	88	90
Row Crops				
Straight Row (Poor)	72	81	88	91
Straight Row (Good)	67	78	85	89
Contoured (Poor)	70	79	84	88
Contoured (Good)	65	75	82	86
Contoured and Terraced (Poor)	66	74	80	82
Contoured and Terraced (Good)	62	71	78	81

Small Grain				
Straight Row (Poor)	65	76	84	88
Straight Row (Good)	63	75	83	87
Contoured (Poor)	63	74	82	85
Contoured (Good)	61	73	81	84
Contoured and Terraced (Poor)	61	72	79	82
Contoured and Terraced (Good)	59	70	78	81
Close-Seeded Legumes (Alfalfa)				
Straight Row (Poor)	66	77	85	89
Straight Row (Good)	58	72	81	85
Contoured (Poor)	64	75	83	85
Contoured (Good)	55	69	78	83
Contoured and Terraced (Poor)	63	73	80	83
Contoured and Terraced (Good)	51	67	76	80
Irrigated Pasture				
Poor	58	74	83	87
Fair	44	65	77	82
Good	33	58	72	79
Meadow (Hay)	30	58	71	78
Pasture or Range for Grazing				
Not Contoured (Poor)	68	79	86	89
Not Contoured (Fair)	49	69	79	84
Not Contoured (Good)	39	61	74	80

Contoured (Poor)	47	67	81	88
Contoured (Fair)	25	59	75	83
Contoured (Good)	6	35	70	79
Evergreen Orchards (Citrus, Avocado)				
Poor	57	73	82	86
Fair	44	65	77	82
Good	33	58	72	79
Deciduous Orchards (Apples, Walnuts)				
Poor	57	73	82	86
Fair	43	65	76	82
Good	32	58	72	79
Farmsteads	59	74	82	86
Natural Areas				
Barren (Rockland, eroded land)	78	86	91	93
Grass				
Poor	67	78	86	89
Fair	50	69	79	84
Good	38	61	74	80
Meadows and Cienegas				
Poor	63	77	85	88
Fair	51	70	80	84

Good	30	58	71	78
Open Brush (buckwheat)				
Poor	62	76	84	88
Fair	46	66	77	83
Good	41	63	75	81
Sagebrush				
Poor		67	80	85
Fair		51	63	70
Good		35	47	55
Desert Brush (creosote, palo verde, mesquite, cactus)				
Poor	63	77	85	88
Fair	55	72	81	86
Good	49	68	79	84
Herbaceous (short desert grasses with some brush)				
Poor		80	87	93
Fair		71	81	89
Good		62	74	85
Brush (brush-weed-grass mixture, mostly brush)				
Poor	48	67	77	83
Fair	35	56	70	77

Good	30	48	65	73
Broadleaf Chaparral (Manzanita, scrub oak)				
Poor	53	70	80	85
Fair	40	63	75	81
Good	31	57	71	78
Narrowleaf Chaparral (Chamise)				
Poor	71	82	88	91
Fair	55	72	81	86
Oak-Aspen-Mountain Brush				
Poor		66	74	79
Fair		48	57	63
Good		30	41	48
Pinyon-Juniper Grassland				
Poor		75	85	89
Fair		58	73	80
Good		41	61	71
Ponderosa Pine				
Poor		70	80	
Fair		64	75	
Good		58	70	

Grassy Woodland (canopy density < 50%)				
Poor	57	73	82	86
Fair	44	65	77	82
Good	33	58	72	79
Woodland (canopy density > 50%)				
Poor	45	66	77	83
Fair	36	60	73	79
Good	25	55	70	77

Table 3. Adjustment of Curve Numbers for Non-Moderate Antecedent Moisture Conditions (ref. Section 3.2.4).

Curve Number for AMC II	Curve Number for AMC I	Curve Number for AMC III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

Table 4. Bed Material Characteristics (ref. Section 3.3.2).

Bed Material Group	Bed Material Characteristics	Hydraulic Conductivity K (in/hr)
1 Very high loss rate	Very clear gravel and large sand	>5
2 High loss rate	Clean sand and gravel	2.0-5.0
3 Moderately high loss rate	Sand and gravel mixture with low silt-clay content	1.0-3.0
4 Moderate loss rate	Sand and gravel mixture with high silt-clay content	0.25-1.0
5 Insignificant to low loss rate	Consolidated bed material, high silt-clay content	0.001-0.10

Table 5. Decay Factor for Transmission Losses (ref. Section 3.3.2).

Bed Material Group	K Decay Factor $(ft \times mi)^{-1}$
1	> 0.030
2	0.0120-0.030
3	0.0060-0.018
4	0.0015-0.0060
5	$6 \times 10^{-6} \times 10^{-4}$

Table 6. Coefficient a_o for Transmission Loss Calculations (ref. Section 3.3.2).

Bed Material Group	a_o (acre-feet)
1	$<$-0.023
2	-0.0093 to -0.023
3	-0.0047 to -0.014
4	-0.0012 to -0.0047
5	-5×10^{-6} to -5×10^{-4}

Table 7. S-Graph Tabulations for the Unit Hydrograph Method (ref. Section 3.4.1).

Time Ratio (t/t_c)	SCS Dimensionless (Q/Q_{ult})	Valley Undeveloped (Q/Q_{ult})	Valley Developed (Q/Q_{ult})	Foothill (Q/Q_{ult})	Mountain (Q/Q_{ult})
0.0	0.000	0.000	0.000	0.000	0.000
0.1	0.003	0.020	0.015	0.014	0.026
0.2	0.012	0.050	0.035	0.037	0.055
0.3	0.045	0.092	0.068	0.065	0.119
0.4	0.107	0.152	0.129	0.102	0.208
0.5	0.193	0.230	0.207	0.150	0.313
0.6	0.300	0.320	0.292	0.215	0.395
0.7	0.412	0.415	0.395	0.315	0.458
0.8	0.522	0.500	0.500	0.500	0.500
0.9	0.625	0.570	0.600	0.583	0.539
1.0	0.700	0.622	0.694	0.644	0.569
1.1	0.775	0.660	0.770	0.685	0.595
1.2	0.822	0.693	0.827	0.720	0.618
1.3	0.862	0.721	0.870	0.750	0.640
1.4	0.890	0.745	0.903	0.777	0.658
1.5	0.917	0.765	0.930	0.802	0.675
1.6	0.934	0.782	0.947	0.823	0.691
1.7	0.950	0.799	0.963	0.843	0.707
1.8	0.961	0.812	0.973	0.861	0.721

1.9	0.970	0.825	0.980	0.876	0.734
2.0	0.977	0.838	0.985	0.890	0.747
2.1	0.983	0.849	0.988	0.902	0.760
2.2	0.987	0.859	0.990	0.913	0.772
2.3	0.990	0.867	0.992	0.924	0.782
2.4	0.993	0.875	0.994	0.933	0.791
2.5	0.994	0.882	0.996	0.942	0.800
2.6	0.995	0.889	0.997	0.951	0.809
2.7	0.996	0.897	0.998	0.957	0.816
2.8	0.997	0.904	0.999	0.963	0.822
2.9	0.998	0.910	1.000	0.969	0.828
3.0	0.999	0.916	1.000	0.973	0.834
3.2	1.000	0.928	1.000	0.979	0.846
3.4	1.000	0.937	1.000	0.983	0.858
3.6	1.000	0.944	1.000	0.987	0.870
3.8	1.000	0.951	1.000	0.990	0.880
4.0	1.000	0.958	1.000	0.991	0.890
4.2	1.000	0.962	1.000	0.992	0.899
4.4	1.000	0.968	1.000	0.993	0.907
4.6	1.000	0.973	1.000	0.994	0.914
4.8	1.000	0.978	1.000	0.995	0.920
5.0	1.000	0.982	1.000	0.996	0.926

6.0	1.000	0.997	1.000	1.000	0.955
7.0	1.000	1.000	1.000	1.000	0.970
8.0	1.000	1.000	1.000	1.000	0.980
10.0	1.000	1.000	1.000	1.000	0.998
12.0	1.000	1.000	1.000	1.000	1.000

APPENDIX B

DYNAMIC PROGRAMMING COMPUTER CODE

```
      PROGRAM DPRC1

      INTEGER*2 IT,IP(10,10,10,3,2)

      CHARACTER*1 TX(72)
      CHARACTER*4 TR(2)

      DIMENSION AP(10,10,3),S(10,10,10,3),F(10,3),AL(10,3),
     *FS(10,10,3),AR(10,3)

      REAL*4 ST(3),R(3),EV(3),SE(10),RE(10)

      COMMON FS,SE,K,ITRACE,I,NX,M,IT
      COMMON /PROB/ IP,F,AL,AR,EV,LEVEL,NFLOW,NSEASN

      OPEN (2,FILE='INPUT.DAT',STATUS='OLD')
      OPEN (3,FILE='OUTPUT.TBL',STATUS='NEW')
      OPEN (7,FILE='OPTPOL.DAT',STATUS='NEW')

      TR(1)=' OFF'
      TR(2)=' ON '

   11 FORMAT (20X,'DISCRETE SYSTEM PERFORMANCE VALUES'/7X,
     *'PERIOD=',I2,' STORAGE TARGET=',F9.2,' RELEASE TARGET=',F9.2/
     *' INDICES   INITIAL INFLOW   FINAL   EVAP RELEASE SQ DEVN',
     *X,'SQ DEFICIT SUM OF SQ'/'  K  I  L   STORAGE',9X,'STORAGE',
     *2X,'LOSS   VOLUME FM STOR FM RELEASE DEVIATION'/)
   12 FORMAT (X,3I3,6F8.2,2F9.2)
   13 FORMAT (72A1)
   14 FORMAT (4I3)
   15 FORMAT (10F8.2)
   16 FORMAT (72A1/' PROBLEM PARAMETERS'/' NUMBER OF
PERIODS=',
     *I3/' NUMBER OF DISCRETE STREAMFLOWS=',I3/
     *' NUMBER OF DISCRETE RESERVOIR LEVELS/PERIOD=',I3/
     *' TRACE SWITCH IS',A4/' PERIOD NUMBER:',8X,4I8)
   17 FORMAT (' STORAGE TARGET  (AF): ',4F8.2)
   18 FORMAT (' RELEASE TARGET  (AF): ',4F8.2)
   19 FORMAT (' EVAPORATION LOSS (FT):',4F8.2)
   20 FORMAT (/' STREAMFLOW FOR PERIOD:',4I8)
   21 FORMAT (19X,I5,4F8.2)
   22 FORMAT (/25X,' RESERVOIR DATA'/3X,3(11X,'T=',I2,4X))
   23 FORMAT (/' TRANSITION PROBABILITIES FOR PERIOD =',I3)
```

```
24   FORMAT (/'     FLOW',10F6.2)
25   FORMAT (X,F8.2,10F6.2)
26   FORMAT (X,' ERROR-NUMBER OF PERIODS EXCEEDS 4')
27   FORMAT (X,' ERROR-NUMBER OF FLOWS EXCEEDS 10')
28   FORMAT (X,' ERROR-NUMBER OF LEVELS EXCEEDS 10')
29   FORMAT (//,' SOLUTION COMPLETED IN ',I4,' ITERATIONS')
30   FORMAT (//,25X,'SOLUTION POLICY'//5X,3('INITIAL',
    *4X),' RELEASE   FINAL'/6X,'LEVEL      FLOW      PERIOD',17X,
    *'LEVEL'/)
31   FORMAT (2(I4,F8.2),I7,7X,F8.2,I4,F8.2)
32   FORMAT (2X,I2,3X,3(F9.2,F10.6))
33   FORMAT (/' STAGE=',I3,' PERIOD=',I3/'  K  I  L=',I2,9I6)
34   FORMAT (38X,F8.2,I4,F8.2)
35   FORMAT (//5X,'RESERVOIR OPERATING POLICY STATED',
    *X,'INDEPENDENTLY OF CURRENT INFLOW',//' PERIOD INITIAL',4X,
    *'FINAL    RELEASE LIMITS   RELEASE    FINAL VOLUME'/10X,
    *'VOLUME   VOLUME    LOWER    UPPER    TARGET',7X,'LIMITS'/11X,
    *'S(KT)   TARGET',31X,'LOWER     UPPER'/)
36   FORMAT (2X,I2,3X,7F9.2)
37   FORMAT (6X,3('     ACTIVE     AREA ')/6X,3(' LEVEL (AF)  (ACRE)'
    *)/)
39   FORMAT (200I2,100I2)
40   FORMAT (10F8.4)

     READ (2,13) TX
     READ (2,14) NSEASN,NFLOW,LEVEL,ITRACE

     IF (NSEASN.GT.3) WRITE (3,26)
     IF (NFLOW.GT.10) WRITE (3,27)
     IF (LEVEL.GT.10) WRITE (3,28)
     IF ((NSEASN.GT.3).OR.(LEVEL.GT.10).OR.(NFLOW.GT.10)) STOP

     READ (2,15) (ST(M),M=1,NSEASN)
     READ (2,15) (R(M),M=1,NSEASN)
     READ (2,15) (EV(M),M=1,NSEASN)

     WRITE(3,16)TX,NSEASN,NFLOW,LEVEL,TR(ITRACE+1),(M,M=1,NS-
EASN)
     WRITE (3,17) (ST(M),M=1,NSEASN)
     WRITE (3,18) (R(M),M=1,NSEASN)
     WRITE (3,19) (EV(M),M=1,NSEASN)

     DO IT=1,NSEASN
```

```
READ (2,15) (F(I,IT),I=1,NFLOW)
READ (2,15) (AL(L,IT),L=1,LEVEL)
READ (2,40) (AR(L,IT),L=1,LEVEL)
END DO

WRITE (3,20) (IT,IT=1,NSEASN)

DO I=1,NFLOW
WRITE (3,21) I,(F(I,IT),IT=1,NSEASN)
END DO

WRITE (3,22) (IT,IT=1,NSEASN)
WRITE (3,37)

DO L=1,LEVEL
WRITE (3,32) L,(AL(L,IT),AR(L,IT),IT=1,NSEASN)
END DO

DO IT=1,NSEASN

DO I=1,NFLOW
READ (2,15) (AP(I,J,IT),J=1,NFLOW)
END DO

WRITE (3,23) IT
M=IT+1
IF (M.GT.NSEASN) M=1
WRITE (3,24) (F(I,M),I=1,NFLOW)

DO I=1,NFLOW
WRITE (3,25) F(I,IT),(AP(I,J,IT),J=1,NFLOW)
END DO

END DO

DO IT=1,NSEASN
IIT=IT+1
IF (IIT.GT.NSEASN) IIT=1
LINE=55

DO K=1,LEVEL

DO I=1,NFLOW
```

```
      DO L=1,LEVEL
      EVAPOR=EV(IT)*((AR(K,IT)+AR(L,IIT))/2.0)
      SSQRD=(ST(IT)-AL(K,IT))**2.0
      REL=AL(K,IT)+F(I,IT)-AL(L,IIT)-EVAPOR
      RELTST=R(IT)-REL
      IF (RELTST.LT.0.0) RELTST=0.0
      RSQRD=RELTST**2.0
      S(K,I,L,IT)=SSQRD+RSQRD
      IF (REL.LT.0.0) S(K,I,L,IT)=-1.0*S(K,I,L,IT)
      IF (ITRACE.EQ.0) GO TO 100
      IF (LINE.NE.55) GO TO 95
      LINE=1
      WRITE (3,11) IT,ST(IT),R(IT)
 95   WRITE (3,12) K,I,L,AL(K,IT),F(I,IT),AL(L,IIT),EVAPOR,REL,
     *SSQRD,RSQRD,S(K,I,L,IT)
100   END DO

      END DO

      END DO

      END DO

      IT=NSEASN
      LAST=NSEASN+1
      N=1
      NX=2
      IF (ITRACE.EQ.0) GO TO 105
      WRITE (3,33) N,IT,(L,L=1,LEVEL)

105   DO K=1,LEVEL

      DO I=1,NFLOW

      DO L=1,LEVEL
      SE(L)=S(K,I,L,IT)
      END DO

      CALL OPTIMA
      END DO
      END DO

130   N=N+1
```

```
      IT=IT-1
      IF (IT.LE.0) IT=NSEASN
      LAST=LAST-1
      IF (LAST.LE.0) LAST=NSEASN
      M=(N-1)/NSEASN+1
      NX=M-2*(M/2)+1
      IF (ITRACE.EQ.0) GO TO 135
      WRITE (3,33) N,IT,(L,L=1,LEVEL)

135   DO K=1,LEVEL
      DO I=1,NFLOW

      DO L=1,LEVEL
      SE(L)=S(K,I,L,IT)
      IF (SE(L).LT.0.0) GO TO 138

      DO J=1,NFLOW
      SE(L)=SE(L)+AP(I,J,IT)*FS(L,J,LAST)
      END DO

138   END DO

      CALL OPTIMA
      END DO
      END DO

145   IF (N.LE.NSEASN*2) GO TO 130
      IF (N.NE.(N/NSEASN)*NSEASN) GO TO 130

      DO K=1,LEVEL
      DO I=1,NFLOW
      DO L=1,LEVEL
      DO IIT=1,NSEASN
      IF (IP(K,I,L,IIT,1).NE.IP(K,I,L,IIT,2)) GO TO 130
      END DO
      END DO
      END DO
      END DO

      WRITE (3,29) N

      WRITE (3,30)
      DO IT=1,NSEASN
```

```
      DO K=1,LEVEL
      DO I=1,NFLOW
      M=0

      DO L=1,LEVEL
      J=IP(K,I,L,IT,1)
     RE(M+1)=AL(K,IT)+F(I,IT)-EV(IT)*(AR(K,IT)+AR(J,IT))/2.0-AL(J,IT)
      IF (IP(K,I,L,IT,1).EQ.0) GO TO 195
      M=M+1
      END DO

195    J=IP(K,I,1,IT,1)
     WRITE (3,31) K,AL(K,IT),I,F(I,IT),IT,RE(1),IP(K,I,1,IT,1),AL(J,IT)
     WRITE (7,31) K,AL(K,IT),I,F(I,IT),IT,RE(1),IP(K,I,1,IT,1),AL(J,IT)

      IF (M.LE.1) GO TO 210

      DO N=2,M,1
      J=IP(K,I,N,IT,1)
      WRITE (3,34) RE(N),IP(K,I,N,IT,1),AL(J,IT)
     WRITE (7,31) K,AL(K,IT),I,F(I,IT),IT,RE(N),IP(K,I,N,IT,1),AL(J,IT)
      END DO

210    END DO
      END DO

      END DO

      WRITE (3,35)

      DO IT=1,NSEASN
      IIT=IT+1
      IF (IIT.GT.NSEASN) IIT=1

      DO K=1,LEVEL
      J=IP(K,1,1,IT,1)
      REL=AL(K,IT)+F(1,IT)-EV(IT)*(AR(K,IT)+AR(J,IT))/2.0-AL(J,IT)
      RELOW=REL
      REHIGH=REL
      FINLOW=AL(J,IT)
      FINHI=AL(J,IT)

      DO I=1,NFLOW
```

```
      DO L=1,LEVEL
      J=IP(K,I,L,IT,1)
      IF (J.LE.0) GO TO 220
      REL=AL(K,IT)+F(I,IT)-EV(IT)*(AR(K,IT)+AR(J,IT))/2.0-AL(J,IT)
      IF (REL.LT.RELOW) RELOW=REL
      IF (REL.GT.REHIGH) REHIGH=REL
      IF (AL(J,IT).LT.FINLOW) FINLOW=AL(J,IT)
      IF (AL(J,IT).GT.FINHI) FINHI=AL(J,IT)
220    END DO
      END DO

     WRITE (3,36) IT,AL(K,IT),ST(IIT),RELOW,REHIGH,R(IT),FINLOW,FINHI

      END DO

      END DO

      END

      SUBROUTINE OPTIMA

      INTEGER*2 IT,IP(10,10,10,3,2)

      DIMENSION FS(10,10,3),SE(10),F(10,3),AL(10,3),AR(10,3),EV(3)

      COMMON FS,SE,K,ITRACE,I,NX,M,IT
      COMMON /PROB/ IP,F,AL,AR,EV,LEVEL,NFLOW,NSEASN

      DO M=1,LEVEL
      BEST=SE(M)
      INDEX=M
      IF (BEST.GE.0.0) GO TO 110
      END DO

110    DO M=1,LEVEL
      IF (SE(M).LT.0.0) GO TO 125
      IF (SE(M).GE.BEST) GO TO 125
      BEST=SE(M)
      INDEX=M
125    END DO

      IP(K,I,1,IT,NX)=INDEX
      FS(K,I,IT)=BEST
```

```
      M=2

      DO N=1,LEVEL
      IF (N.EQ.INDEX) GO TO 130
      IF (ABS(SE(N)-BEST).GT.0.01) GO TO 130
      IP(K,I,M,IT,NX)=N
      M=M+1
130   END DO

      IF (M.GT.LEVEL) GO TO 150

      DO N=M,LEVEL,1
      IP(K,I,N,IT,NX)=0
      END DO

150   M=M-1
      IF (ITRACE.EQ.0) GO TO 160

160   RETURN

      END
```

```
      PROGRAM GAUSS

      INTEGER LROW(305)

      REAL X(305),Q,T

      DIMENSION P(10,10,3),B(305,305),RE(10,10,3),F(10,10,10,3)

      OPEN (2,FILE='INPUT.DAT',STATUS='OLD')
      OPEN (4,FILE='OPTPOL.DAT',STATUS='OLD')
      OPEN (5,FILE='GAUSS.DAT',STATUS='NEW')

      NPOL=302

      DO N=1,14
      READ (2,*)
      END DO
      DO N1=1,3
      DO L1=1,10
      READ (2,10) (P(L1,L2,N1),L2=1,10)
10     FORMAT (10F8.2)
      DO L3=1,10
      DO L4=1,10
      F(L4,L3,L1,N1)=0.0
      END DO
      END DO
      END DO
      END DO

      DO M=1,NPOL
      READ (4,12) K,I,IT,R,L
12     FORMAT (2(I4,8X),I7,7X,F8.2,I4)
      RE(K,I,IT)=R
      F(K,I,L,IT)=1.0
      END DO

      IROW=0

      DO N2=1,300
      LROW(N2)=N2
      DO N3=1,301
      B(N2,N3)=0.0
      END DO
```

```
      END DO

      DO IT=1,3

      IF (IT.NE.1) GO TO 100
      IIT=3
      GO TO 105
100   IIT=IT-1

105   DO L=1,10
      DO J=1,10
      IROW=IROW+1
      ICOL=0

      IF (IROW.EQ.1) GO TO 800
      IF (IROW.EQ.101) GO TO 810
      IF (IROW.EQ.201) GO TO 820
      GO TO 109

800   DO N11=1,100
      ICOL=ICOL+1
      B(1,ICOL)=1.0
      END DO
      B(1,301)=1.0
      GO TO 199

810   ICOL=100
      DO N12=1,100
      ICOL=ICOL+1
      B(101,ICOL)=1.0
      END DO
      B(101,301)=1.0
      GO TO 199

820   ICOL=200
      DO N13=1,100
      ICOL=ICOL+1
      B(201,ICOL)=1.0
      END DO
      B(201,301)=1.0
      GO TO 199

109   IF (IIT.NE.1) GO TO 120
```

```
      DO K=1,10
      DO I=1,10
      ICOL=ICOL+1
      B(IROW,ICOL)=P(I,J,IIT)*F(K,I,L,IIT)
      END DO
      END DO
      DO N4=1,100
      ICOL=ICOL+1
      IF (ICOL.NE.IROW) GO TO 110
      B(IROW,ICOL)=-1.0
      GO TO 199
110   END DO
      GO TO 199

120   IF (IIT.NE.2) GO TO 140
      ICOL=100
      DO K=1,10
      DO I=1,10
      ICOL=ICOL+1
      B(IROW,ICOL)=P(I,J,IIT)*F(K,I,L,IIT)
      END DO
      END DO
      DO N4=1,100
      ICOL=ICOL+1
      IF (ICOL.NE.IROW) GO TO 130
      B(IROW,ICOL)=-1.0
      GO TO 199
130   END DO
      GO TO 199

140   ICOL=0
      DO N4=1,100
      ICOL=ICOL+1
      IF (ICOL.NE.IROW) GO TO 150
      B(IROW,ICOL)=-1.0
      GO TO 151
150   END DO
151   ICOL=200
      DO K=1,10
      DO I=1,10
      ICOL=ICOL+1
      B(IROW,ICOL)=P(I,J,IIT)*F(K,I,L,IIT)
      END DO
```

```
      END DO

199    END DO
      END DO
      END DO

      DO K=1,299
      JS=0

      DO J=K,300,1
      IF (B(J,K).EQ.0.0) GO TO 300
      JS=J
      GO TO 301
300    END DO

301    IF (JS.NE.0) GO TO 310
      WRITE (*,9)
  9   FORMAT (X,'ERROR JS=0')
      STOP

310    IF (JS.EQ.K) GO TO 311
      DO ICOL=1,301
      T=B(K,ICOL)
      B(K,ICOL)=B(JS,ICOL)
      B(JS,ICOL)=T
      END DO

      MROW=LROW(K)
      LROW(K)=LROW(JS)
      LROW(JS)=MROW

311    DO J=(K+1),300,1
      IF (B(K,K).NE.0.0) GO TO 320
      WRITE (*,8) K
  8   FORMAT (X,'ERROR B(K,K)=0  AT K =',I4)
      STOP
320    IF (B(J,K).EQ.0.0) GO TO 399
      Q=B(J,K)/B(K,K)

      DO JP=(K+1),301,1
      T=B(J,JP)
      B(J,JP)=T-Q*B(K,JP)
      END DO
```

```
399   END DO
      END DO

      X(300)=B(300,301)/B(300,300)
      X(299)=(B(299,301)-X(300)*B(299,300))/B(299,299)

      DO I=3,300,1
      II=301-I
      DO JJ=(II+1),300,1
      B(II,301)=B(II,301)-X(JJ)*B(II,JJ)
      END DO
      X(II)=B(II,301)/B(II,II)
      END DO

      KROW=0
      DO IT=1,3
      DO L=1,10
      DO J=1,10
      KROW=KROW+1
      DO MM=1,300
      IF (LROW(MM).EQ.KROW) GO TO 400
      END DO
      WRITE (*,7)
  7   FORMAT (X,'ERROR - NO KROW VALUE')
      STOP
400   WRITE (5,20) L,J,IT,RE(L,J,IT),X(MM)
 20   FORMAT (3I8,F10.2,5X,F10.6)
      END DO
      END DO
      END DO

      END
```

```
      PROGRAM SORT

      REAL CDF(3)

      DIMENSION QI(3,300),QF(3,300),PDF(3,300),SPDF(3,300)

      OPEN (2,FILE='GAUSS.DAT',STATUS='OLD')
      OPEN (3,FILE='SORT.DAT',STATUS='NEW')

      N=100

      DO IT=1,3

      DO N1=1,N
      READ (2,10) QI(IT,N1),PDF(IT,N1)
 10    FORMAT (24X,F10.2,5X,F10.6)
      QF(IT,N1)=0.0
      SPDF(IT,N1)=0.0
      END DO

      DO N4=1,N
      DO N5=1,N
      IF (QI(IT,N5).LT.QF(IT,N4)) GO TO 100
      QF(IT,N4)=QI(IT,N5)
      SPDF(IT,N4)=PDF(IT,N5)
      J=N5
100    END DO
      QI(IT,J)=-1.0
      END DO

      END DO

      CDF(1)=0.0
      CDF(2)=0.0
      CDF(3)=0.0

      DO N6=1,N
      DO IT=1,3
      CDF(IT)=CDF(IT)+SPDF(IT,N6)
      END DO
      WRITE (3,11) (CDF(IT),QF(IT,N6),IT=1,3)
 11    FORMAT (3(F10.6,F10.2))
      END DO
```

END

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