ATTACHMENT 4

Recommended Watershed Modeling Techniques for Hydrologic Design and Best Management Practice

LAKE TAHOE, CALIFORNIA AND NEVADA

Prepared for:

SWQIC
Lake Tahoe Storm Water
Quality Improvement Committee

Prepared by:

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of Engineers®
Sacramento District

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Executive Summary

- The primary purpose of this report is to provide recommendations for watershed modeling approaches useful for developing a hydrologic design criteria manual that addresses traditional drainage design and best management practice. Precipitation and stream flow gage observations, almost certainly, will not be available for watershed model applications to design problems. Consequently, the focus of the recommendations will be on approaches addressing the ungaged analysis problem.
- Traditional drainage design problems invariably require the estimate of a flow frequency curve. For example, a culvert in an urban area may need to convey the 10% chance exceedance (10 year return interval) peak flow, or a retention pond may need to control the 1% exceedance (100 year flow) inflow hydrograph. The hydrologic design criteria necessarily need to focus primarily on watershed modeling techniques that can be used to estimate these frequency curves for an ungaged watershed.
- As noted by Urbonas (personal communication, 2004), who has extensive experience in developing drainage criteria for the well known Denver Urban Storm Drainage Criteria Manual (see USDCM, 2003), the reliability of watershed model predictions depends largely on calibrating model parameter with observed precipitation-runoff data. His experience, which is a common one, is that there is significant uncertainty in un-calibrated model predictions .
- At this time, studies have not been performed in the study area to calibrate watershed models for drainage design. However, data does exist that could potentially be used for this purpose. Consequently, the report will both recommend methods based on accepted watershed modeling approaches and propose future studies using available study area data that would improve these recommendations.
- A general review of the state of the art was performed to serve as a basis for evaluating the current application of modeling techniques being applied by counties to obtain design flows.

    - Methods were examined that focus on both small (less than 200 acre) and large drainage area problems. The rational method is used in the vast majority of drainage design problems for small areas. A wide variety of methods are used for modeling runoff from large areas.
    - The rational method is generally applicable to most small area problems. The method predicts design peak flow for a given exceedance probability based on a runoff coefficient, time of concentration for the watershed, rainfall intensity and drainage area. Ideally, the runoff coefficient is determined as a function of rainfall intensity return interval; but this requires that the method be calibrated to observed data. Consequently, a value independent of return interval is usually employed.
    - The discussion of watershed modeling for large areas focused on the accepted methods for computing a annual maximum flow-frequency curve. The frequency curve is determined by simulating design storms of a particular exceedance frequency to obtain discharge assumed to have this same exceedance frequency.
    - Basically, the watershed model was envisioned as a set of components that simulate an input storm hyetograph to obtain an outflow. Each component employs computational methods representing various aspects of the runoff process. The following describes what was judged to be a reasonable set of component methods that could be used to obtain design discharges by simulating a design storm given the present state of the art:

- Create a balanced design storm using depth-duration-frequency information from NOAA14;
- Snowmelt volume is determined based on melt rates found from modeling studies (e.g., Jeton (1999)), total melt volume is constrained by available snow water equivalent shown in figure 4.8;
- Direct runoff volume is computed using the NRCS CN method, frozen soil conditions selected based on degree of conservatism and design problem of interest (see section 6 for further discussion);
- Direct runoff routing performed using the NRCS lag UH method for open areas, distributed modeling using kinematic wave/Muskingum-Cunge routing for urban areas;
- Channel routing performed with the Muskingum-Cunge method or Muskingum method if data available for computing K and X.

- The current design practice was considered by reviewing drainage design manuals of counties (Placer, El Dorado, Washoe and Douglas)in the Lake Tahoe Basin. The Placer county manual addressed the greatest range of conditions, including snowmelt and frozen ground. The other counties do not provide guidance on snow and related problems probably because the manual do not specifically address Lake Tahoe. The following compares the methods used by the counties and provides a review based on the previous discussion of state of the art modeling methods:

Rational method

- Placer County does not use the rational method for small areas, but rather estimates a unit area discharge that is a function of drainage area time of concentration and other drainage area characteristics. These unit area discharges were determined based on application of the HEC-1 model to a range of watershed characteristics. The other counties employ the more traditional rational method approach. The methods used to estimate either runoff coefficient or time of concentration differ substantially. El Dorado County uses the TR-55 methodology for computing time of concentration which results in substantially greater travel times than obtained by Placer and Washoe county, especially for forest or natural open areas. This may result because the other methods for estimating time of concentration were developed for an urban land use. The development of the Placer County equations for time of concentration are not well documented, nor is the development of the Washoe County method for estimating runoff coefficients.

Watershed modeling approaches

- **Design storm**: The counties employ various methods for developing design storms (balanced ,NRCS type Ia and II are employed) and means for estimating basin average design storm depth.
- **Loss rates:** Placer County distinguishes between snow covered and no snow drainage areas based on drainage area elevations. Snow covered areas are assumed to have frozen ground; whereas, snow-free ground assumes constant loss rates that are determined for a wet condition NRCS curve number (CN). El Dorado and Washoe County use the NRCS CN assuming average antecedent wetness conditions. Douglas County provides no recommendations.

- **Snowmelt:** Only Placer County provides estimates of snowmelt contribution to direct runoff. El Dorado county notes that it should be considered, but provides no specific recommendation except to contact the county for guidance.
- **Runoff Routing:** The counties do not distinguish between the runoff dynamics described by the hillslope mechanism important for forest/natural-open areas and the surface flow mechanism important to urban areas. Placer County recommends the use of a distributed modeling approach, kinematic wave/Muskingum-Cunge, which is best suited to simulating surface runoff. El Dorado and Washoe County recommend use of the NRCS lag unit hydrograph (UH). Although the UH method is the same, El Dorado and Washoe counties use different means for calculating the time of concentration to obtain the UH lag (as in the case of the application to the rational method). Douglas County accepts either the distributed or UH approaches.

  - In review: The county methods follow commonly accepted engineering practice. The chief difficulty is in parameter estimation. For example, the NRCS CN or lag UH were developed based on data for small agricultural watersheds in the Midwest. This does not recommend their use in Lake Tahoe. Also, the effects of snow on routing parameters is ignored; but this is typical of most guidance on parameter estimation. Assumptions regarding frozen ground will have a very large impact on the computed hydrograph. In terms of methodology, the application of the distributed (kinematic wave) versus lumped (UH) is notable, but much less significant than issues regarding selection of design storm, initial conditions and loss rates. The distributed approach should not be used in natural areas (e.g., forest areas) where subsurface flow is important to computing runoff. However, the distributed approach probably has some advantage in capturing the differing response from open and impervious areas in urban catchments.

- **Channel routing:** Placer, El Dorado and Washoe County provide various recommendations with regard to the channel routing methods to employ. Douglas County does not provide any recommendations.

  - In review: the Muskingum-Cunge method should always be used instead of kinematic wave, being more applicable to a wider range of hydraulic conditions and requiring exactly the same parameters. Although often used as a "hydraulic method", the modified-Puls method should not be used for channel routing. The method is effectively a poor solution to the kinematic wave equations, where the numerical computational error resulting in an apparent subsidence of the hydrograph is a function of the computation interval rather than any affect of storage.

- **Recommendations regarding hydrologic modeling criteria were made based on the review of the generally accepted modeling techniques and county practice.** These recommendations are likely to be acceptable in that the methods for computing runoff are widely applied. The conservative approach to parameter estimation certainly is worth debate by regulatory agencies in the study area. The most reasonable approach to obtaining better parameter estimates would be to calibrate model parameters to observed data as is discussed in section 7. The following criteria are recommended for estimating direct runoff for hydrologic design:

Rational method

- o The rational method will be applied using the recent NOAA14 depth-duration-frequency curves and the TR-55 (NRCS, 1986) methodology for computing time of concentration. The recommendation provided for maximum overland flow length should be replaced by the most recent recommendation to limit this length for sheet flow to 100 feet rather than 300 feet (NRCS, 2004a and 2004b, and, personal communication: Woodward, 2004). The runoff coefficient estimates can be obtained from EPA (1983).
- o The maximum basin size to use for application of this method depend largely on the variation in runoff properties and complexity of the drainage system in a drainage area being analyzed. Estimating a composite runoff coefficient and the appropriate time of concentration for a drainage area becomes increasingly difficult as the drainage area contributions to runoff become more distributed The typical rule of thumb is that drainage areas less than 200 acres have drainage patterns simple enough to be captured in a rational method analysis.

Watershed modeling approaches

Design storm

- o Use a balanced storm approach which captures the critical peak-intensity-duration characteristics defined by precipitation depth-duration-frequency curves. This is a conservative approach in that the analysis of available storm data for regions surrounding the study area (see section 4.4.2) found that actual storms for a significant portion of the period of record were not balanced.
- o The duration of the storm should consider both the time of concentration of the watershed, and, the design of a detention/retention storage (if relevant). At the very least, the storm duration should be great enough to where the whole basin will be contributing to the computed peak runoff needed for design. If a detention/retention structure is being designed, then the volume of runoff is important. The duration of the storm should be great enough to where increments in storm duration do not affect the design of the detention/retention structure to control storm water.
- o Depth area reduction correction to the point estimates of precipitation will not be used because: 1) of the increase in precipitation with elevation; and, 2) the lack of studies analyzing the change in average storm depth with drainage area for the study area.

Frozen soil

- o The decision to assume a frozen soil for a watershed depends to some extent on the design problem and the degree of conservatism of interest. For example, a frozen soil assumption is conservative when interest focuses solely on the maximum runoff for design of conveyance capacity or determining the regulatory flood plain. Alternatively, the assumption is not conservative when designing retention storage to reduce runoff from an urban development to a pre-development magnitude A frozen soil condition would produce the same volume of runoff as the impervious area created by the planned development. Under the assumed frozen soil condition there would be no need to provide detention/retention storage to reduce runoff to the pre-existing condition. The assumption of an unfrozen soil for design problems where runoff needs to be controlled to a pre-project level is more conservative than the frozen soil assumption. **Consequently, the recommendation is to take a conservative approach where the frozen soil (zero loss rate) assumption for**

sizing conveyance and determining the regulatory flood plain, and, an unfrozen ground assumption for sizing detention/retention storage.

Snowmelt

- Snowmelt will be included as part of the runoff excess volume in terms of an average rate. This rate should be based on previous modeling studies of snowmelt in the study area (e.g., Jeton, 1999). The melt rate will limited to the amount of available storm water equivalent provide in figure 4.8.

Loss Rates

- The runoff excess for unfrozen soils, and corresponding loss rates, will be calculated based on the NRC CN, assuming an average (AMC II) antecedent wetness condition (see NRCS, 1986).

Runoff routing

- Natural/open areas will use TR-55 (NRCS, 1986) for estimating the NRCS Llag Unit Hydrograph. Urban areas will employ a distributed approach using kinematic wave overland flow planes and Muskingum-Cunge channel routing (see HEC 1990, 2001). The unit hydrograph method is applied to natural/open areas because direct runoff can be due to both surface and subsurface flow. The distributed approach is recommended for urban areas because surface flows dominate the contribution to direct runoff, it is simpler to apply than the unit hydrograph method and can capture the separate responses from pervious and impervious areas. This application should use the most recent research which has found that limits the maximum overland flow length for sheet flow to 100 feet rather than 300 feet (NRCS, 2004a and 2004b, and, personal communication: Woodward, 2004)

Channel Routing

- Muskingum-Cunge method will be used to perform hydrologic channel routing (see HEC 1990, 2001). Standard published values of roughness coefficients will be employed (see TR-55, 1986) . The Muskingum method can be employed in circumstances where flood travel time can be estimated.

Applications with regional regression equations

- Regional regression equations relating annual peak and maximum daily volume duration frequency curves to watershed meteorologic and physical characteristics have been developed for the study area (see SPK, 2005). These regression equations are useful for relatively large drainage areas (greater than 0.5 square miles) that experience a significant proportion of storm runoff from snowmelt (watersheds with a significant proportion of drainage area above 7000 feet).
- A nation wide study (U.S. WRC, 1981) demonstrated that, for the most part, the USGS regression equations were more accurate than event oriented watershed models in predicting peak annual flow frequency curves. This study provides good reasons for using the regression equations to validate watershed model predictions in ungaged areas. Consequently, the regression equations can be used to calibrate/validate watershed model predictions by: 1) adjusting model loss rates so that the model predicted frequency curves agree with the regression prediction within some reasonable tolerance; 2) adjust the model loss rates if necessary to ensure that model predictions lie within predicted regression confidence limits on frequency curves of

interest; or 3) average model and regression predicted frequency curves. Results from (1) or (2) could be used to estimate loss rates for open areas in urban watersheds, even though regressions are not directly applicable to these watersheds. Of the three described, the method to use will depend on confidence placed in watershed model predictions.

- The hydrologic modeling criteria could, potentially, be greatly improved by performing calibration/validation studies using both observed precipitation-runoff events and stream gage/regression equation estimates of flow frequency curves.
- Probably the most important decision to be made in performing the model validation/calibration studies is in selecting criteria for evaluating the difference between watershed model predicted and gage/regression estimated frequency curves. As noted in section 6, a national test (U.S. WRC, 1981) demonstrated that regional regression equations were, generally, more accurate than uncalibrated watershed model predictions of flow-frequency curves. Consequently, there is some evidence for adjusting watershed model parameters to obtain some reasonable agreement between model and gage/regression equation estimates of flow frequency curves To what degree the parameters should be adjusted is an open question. Two principles should be considered in adjusting parameters in these comparative studies:

    1) The watershed model predictions should agree on the average with the stream gage/regression estimated flow frequency curves over a reasonable number of comparisons;
    2) The model parameters should be constrained to some physically reasonable values.

    These can, and most likely will be, competing requirements. Ideally, physically reasonable watershed model parameters will result in model predictions that agree on the average with the gage and regression flow frequency estimates. However, this will not necessarily be true in practice. Ultimately, guidelines would be developed that specified to what extent model parameters would be adjusted to bring into agreement model predicted and regression estimates of flow-frequency curve for ungaged watersheds.
- The comparative studies performed will depend on research done to improve parameter estimates for ungaged basins and develop alternative depth-duration-frequency curves for developing design storms.
- Future calibration studies would involve the following efforts (see section 7.2):

    - Evaluate precipitation/gage flow record useful for calibrating watershed models (see section 7.2.2). If this data is not valuable then the calibration should not be undertaken.
    - Study the relationship between frozen soil (zero loss rate) conditions and annual maximum runoff (see section 7.2.3) This is important for evaluating the necessity of conservative recommendations made in section 6 and providing information for setting loss rates in model calibration efforts to precipitation runoff data.
    - Determine degree of balance in gage precipitation events (see section 7.2.4). This study is important for improving estimates of temporal patterns in design storms.
    - Calibrate watershed models to observed precipitation/runoff events, assuming existing precipitation/flow data is useful for this purpose (see section 7.2.5).

- The design storms used in watershed model simulations could be improved by (see section 7.3):

    - Estimating rainfall depth-duration-frequency curves by adjusting NOAA14 precipitation DDF curves using available gage records. (see section 7.3.1). Rainfall causes major flooding perhaps making rainfall DDF curves more relevant to the flow-frequency problem.
    - Estimate seasonal precipitation/rainfall DDF curves from the NOAA14 annual DDF curves using available gage records (see section 7.3.2). Application of mixed distribution analysis using seasonal precipitation/rainfall depth-duration-frequency curves to obtain design storms may improve watershed and gage/regression flow-frequency curve estimates.

- The value of various parameter estimation schemes and the selection of design storms is best evaluated by comparing watershed model predictions and stream gage/regional regression estimates of flow-frequency curves (see section 7.4):

    - The goal of the comparative study would be to evaluate to what extent model parameter estimates, obtained based on either the recommendations in section 6 or from calibration studies, should be adjusted to agree with stream gage/regression equation estimates.

- Finally, the results of the calibration and comparative studies could be used to regionalize certain model parameters to ungaged watershed drainage areas which are not typical of the drainages used in developing the regional regressions (see section 7.5).

- Recommendations regarding hydrologic design criteria for best management practice (BMP)focused on the current state of the art and the potential for its application in the Lake Tahoe Basin. In particular the focus of the criteria is for BMP design for ungaged urban catchments.
- The objective of BMP is to meet receiving water quality objectives by controlling non-point source pollution. Water quality objectives in the study area are certainly focused on protecting Lake Tahoe; but also, have relevance to other receiving waters in the basin.
- Allowable total maximum daily loads, TMDLs, are used by EPA as a tool for constraining pollutant discharge. A TMDL quantifies the allowable pollutant loading that meets a receiving water quality objective. The TMDL is most likely to be determined by some type of modeling study.
- Reduction of loading may be achieve by controlling point or non-point sources. The allowable load from either of these sources is allocated as (see EPA, 1999, pg. 1-1):

$$\mathrm{TMDL} = \sum \mathrm{WLA} + \sum \mathrm{LA} + \mathrm{MOS}$$

  where WLA is the waste load allocation or the portion of the TMDL allocated to existing or future point sources, LA is the load allocation or portion of the TMDL allocated to existing or future non-point source and natural background, and MOS is a margin of safety.
- As EPA notes, the effectiveness of any reduction due to BMP is difficult to determine by a predictive model. Consequently, the MOS is included to provide some degree of assurance

that the TMDL constraint will be met. Still, EPA requires that monitoring be used to ensure that receiving water quality objective is attained.

- The hydrologic criteria use to meet an allowable LA typically provided in drainage design manuals is based on containing or treating a water quality volume (WQV).
  - The Tahoe Regional Planning authority currently specifies that the 20-year, 1-hour design storm be used to determine the WQV. The basis for establishing this WQV is not explained. Seemingly, the guideline was judged to be a reasonable criteria for reducing loading to meet receiving water objectives.
  - Caltrans (California Department of Transportation) determines a WQV based on precipitation frequency information and basin impervious fraction (see Caltrans, 2003). This procedure is largely based on studies done to develop the well known Denver Drainage Manual (see USDCM, 2003).
  - The method described in the Denver Drainage Manual to estimate WQVs is based on both field studies of removal rates from various BMP designs, such as retention structures, and modeling studies.

    - The studies demonstrated that a significant reduction in pollutant loading (80-90%) could be achieved by controlling up to the 80th -90th quantile runoff event (i.e., controlling all events not exceeded 80%-90% percent of the time).
    - Simplified models were developed that related WQV to drainage fraction impervious and mean annual precipitation. These models were calibrated to results obtained with more sophisticated continuous simulations models. Case studies were used to verify the results of the calibration.

- Recommendations for improving the current approach to specifying hydrologic criteria for BMPs in the Lake Tahoe Basin depend on implementing future modeling studies to determine WQVs.

  - Modeling studies relevant to the hydrology and water quality problems in the Lake Tahoe Basin are needed to calibrate simplified methods for estimating WQVs.
  - The California Lahontan Regional Water Quality Control Board (LRWQCB) is currently involved in large area continuous simulation watershed modeling studies that will determine allowable TMDLs for Lake Tahoe and the other receiving waters in the basin. Models developed in this study will provide the experience and methodologies valuable for studies of smaller urban areas that could be useful in developing simplified approached to estimating WQVs.
  - Application of continuous simulation watershed models will face the following challenges:

    - Gage precipitation records are limited for the basin. Daily precipitation is available, but shorter interval information is limited (see Table 7.1). The LRWQCB is using a 40-year period of record estimated from simulations of a physically-based atmospheric model (MM5). The simulated precipitation could prove useful for smaller scale urban studies, but needs to be validated to the extent possible in comparison to the available precipitation gage record.
    - Gage measures of runoff from urban basins is limited. Calibration of the simulations models to this data is likely to be useful.
    - The model precipitation-runoff algorithms will need to simulate snowmelt runoff. Very little information exists on how to model snowmelt in urban areas. Human

activity, (e.g., plowing) has a significant impact on the thermal properties of snow. Furthermore, urban impervious areas have impacts on the energy budget which are different than natural-open areas where most snowmelt studies have been performed. Consequently, parameter typically used in either energy-budget or degree-day snow-melt models are not likely to be appropriate for urban snowmelt runoff simulations. Field studies of urban-snowmelt would provide a basis for developing urban snowmelt runoff-models.

- The LRWQCB might also consider using the regional flow regression equations (see SPK, 2005) to calibrate/validate continuous simulation model applications in ungaged watersheds.

- Developing criteria for estimating the MOS applied together with a WQV needs to be an important part of any modeling study. Currently, MOS is estimated in an arbitrary fashion without regard to design cost. Modeling studies could provide information on the benefits versus costs of incremental reductions in pollutant loading. Metrics for the benefits of water quality are not easily identified. However, a simple approach would be to develop relationships between incremental reduction in pollutant load versus cost as a function of MOS. Judgments can then be made regarding the worth of increased MOS from these relationships. In the long run, only monitoring studies will determine if the MOS selected needs revision.

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1. Introduction

The purpose of this report is to provide recommendations for analysis methods useful for developing a hydrologic design criteria manual that addresses traditional drainage and best management practice within the Lake Tahoe Basin. Precipitation and stream flow gage observations, almost certainly, will not be available for watershed model applications to design problems. Consequently, the focus of the recommendations will be on approaches addressing the ungaged watershed analysis problem.

Traditional drainage design problems invariably require the estimate of a flow frequency curve. For example, a culvert in an urban area may need to convey the 10% chance exceedance (10 year return interval) peak flow, or a retention pond may need to control the 1% exceedance (100 year flow) inflow hydrograph. The hydrologic analysis method recommended necessarily will focus on watershed modeling techniques that can be used to estimate these frequency curves for an ungaged watershed. Of particular importance are methods useful for smaller (less than 0.1 square mile) ungaged urban basins.

In contrast to the traditional design problem, best management practice (BMP) design problems involve controlling non-point source pollution to meet a loading constraint that meets a receiving water quality objective. In this case, the receiving waters are Lake Tahoe and other water bodies within the Lake Tahoe Basin. Hydrologic design criteria need to be developed design flows to be used for the BMP design objective. In this report, recommendations will be provided for the analysis methods to use in estimating design flow for the BMP design.

In terms of the traditional design problem, watershed modeling methods could be recommended where:

- applications have validated model prediction of flow frequencies;
- model parameters can be reasonably estimated for ungaged areas;
- model predictions addresses the appropriate design requirements.

As noted by Urbonas (personal communication, 2004), who has extensive experience in developing drainage criteria for the well known Denver Urban Storm Drainage Criteria Manual (see USDCM, 2003), watershed model validation depends largely on calibrating the model observations. His experience, which is a common one, is that un-calibrated model predictions will have a high degree of uncertainty.

The development of the criteria for the Denver manual benefited from the availability of a significant amount of observed data. Furthermore, a great deal of effort was made to calibrate watershed model parameters to this data.

Potentially, this could be done for the Lake Tahoe study area given the observations available. However, this would take a significant effort. In lieu of this effort, the methods recommended for use in hydrologic design criteria will focus on modeling techniques where parameters can be estimated based on information available for ungaged areas; and, that make predictions which address the drainage design problem.

The kinds of design problems that need to be addressed by watershed model applications are described in section 2. The range in problems presented will require a suite of different modeling approaches. Section 3 and 4 discuss and evaluate modeling approaches for respectively the small and large drainage areas that have been applied to the ungaged watershed analysis problem important to drainage design problems in the study area. This discussion provides the basis for reviewing the various hydrologic modeling approaches and criteria proposed by the study area counties. Section 6 provide recommendation regarding the hydrologic criteria for the study area based on the previous discussion of modeling approaches in sections 3-5.. Section 7 describes future studies and research which would be very useful in improving recommendation regarding hydrologic design criteria.

Recommendations for hydrologic analysis methods applicable to BMP design focus on methods that address a range off hydrologic flow magnitudes needed to control non-point source pollution rather than the annual maximum flows important to traditional drainage design. However, like the traditional design problem, the methods need to focus on ungaged and mostly smaller urban drainage basins. In section 8, a discussion is provided of the current state of the art used in modeling flows for BMP design and how these methods might need to be modified for application to the Lake Tahoe Basin.

2. Traditional Drainage Design Problems

2.1. Introduction

Traditional hydrologic designs require an estimated peak flow, or a combination of flow peak and volume represented by a hydrograph. Most typically the flow magnitude is related to a risk related design capacity. For example, a retention basin design may need to reduce the 1% exceedance frequency peak flow (e.g., 100 year flow) to pre-project levels or a culvert design may need to have capacity to convey the 2% chance exceedance flow.

The methods that would be used to address either of these drainage design problems are a function of scale, i.e., the drainage area of the watershed. Modeling methods that only consider peak flow cannot adequately account for variation of watershed properties within a watershed or complex drainage design problems. These methods are relegated to small drainage areas. More complicated problems need to be addressed by models that can simulate runoff hydrographs. Sections 2.2 and 2.3 describe respectively the small area drainage area problems that can be addressed simply by considering only peak flows or larger area problems that require a hydrograph for design.

2.2. Small drainage area problems

Small drainage area problems needing peak flow estimate are invariably related to road/highway drainage design. These problems involve spacing of catch basins/curb inlets; and, estimating culvert capacity . The magnitude of the peak flows usually correspond to a 10% (10 year return interval) or more frequent design event.

The rational method is used by county and state engineers in the study area to compute the peak discharge for these small drainage area design problems. In applying this approach, the most important issues are determining the maximum area where the method is applicable and determining the parameters of the method as is discussed in section 3.

2.3. Large drainage area problems

Large drainage area problems typically require a design hydrograph that represents the flow peak and volume-duration-frequency relationship for a drainage area. The design hydrograph is computed by watershed model simulation of a design storm (see figure 2.1). The design storm is constructed from precipitation depth-duration-frequency (DDF) curves for the drainage area. For example a 1% chance exceedance design storm (i.e., the 100 year storm) is simulated to estimate the 1% chance design hydrograph.

As is discussed in section 4, a watershed model can also be used to simulate a continuous period of stream flow that can be analyzed for design purposes. However, the design hydrograph approach is used much more often than the continuous approach, being more commensurate with the information available for ungaged analysis.

The large area hydrologic design problems are as follows:

- major highway culverts;
- channel construction;
- regulatory flood plain definition;
- retention basins;
- spillway design for dam safety;

Culvert and channel design, as well as regulatory flood plain definition only require a peak flow for design. The design conveyance needs to be sufficient convey a peak flow for given exceedance probability. Flood profile analysis for the regulatory flood plain is most typically determined by a steady-state non-uniform channel flow hydraulic model which only requires peak flows as input. However, hydrograph simulation is needed to estimate the peak flows accurately for large drainage areas. Design hydrographs are required for both retention basin and spillway design.

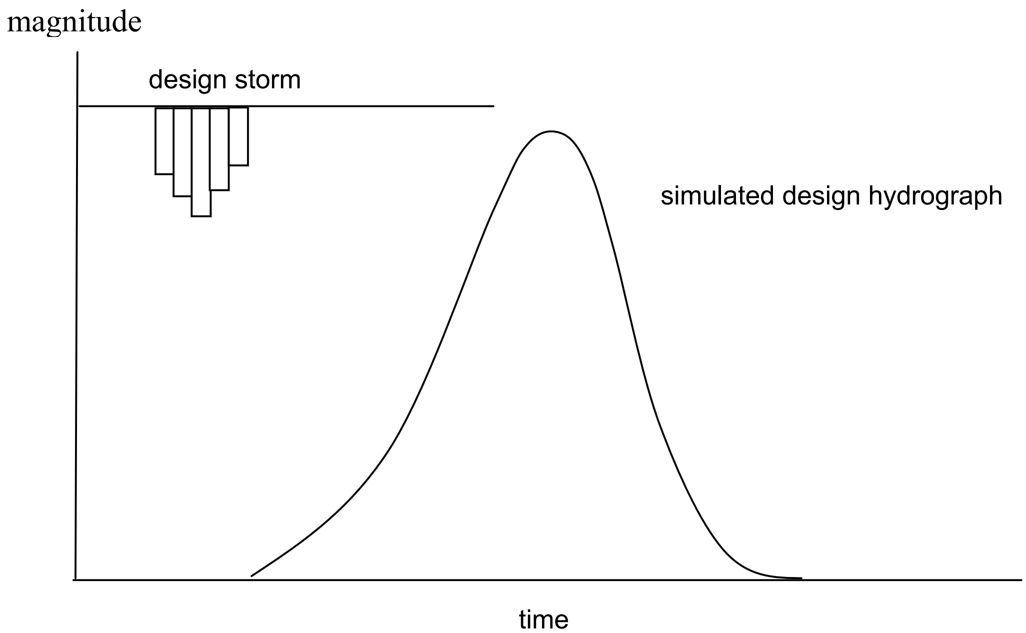

Figure 2.1: Simulated design runoff

3. Peak discharge estimation for small drainage areas

The rational method is the most popular approach to estimating peak flows for drainage design in the U.S. (see Pilgrim and Cordery, 1992, and Urbonas and Roesner, 1992). The method estimates peak discharge for a particular exceedance probability as:

$$Q_p = FCi_pA \tag{0.1}$$

where i_p is the rainfall intensity for exceedance probability p and duration equal to the time of concentration to the basin outlet, A is the drainage area, C is a runoff coefficient and F is conversion factor. If Q_p is in cfs (cubic feet per second), i_p in inches/hour and A in acres, then F is 1.00.

The method assumes that the precipitation is uniformly distributed over the basin. Furthermore, the runoff coefficient does not distinguish between the different process in urban, agricultural and natural (forest and rangeland) watersheds which contribute to peak flow. Surface flow processes are most important in urban and agricultural watersheds; whereas, subsurface and surface runoff contribute to peak flow in natural areas (see section 4 for a further discussion of the different factors contributing to runoff).

The method application requires determination of the time of concentration and estimating C. The time of concentration can be defined, assuming no runoff storage within the basin, as either: 1) the surface runoff travel time from the point on the basin boundary to the basin outlet; or, 2) the time at which the outflow reaches a steady state given a uniformly distributed precipitation rate over the basin.

C is equal to the ratio of runoff to rainfall intensity. Research has found that this coefficient is a function of the rainfall intensity (see Schaake, J.C., et al., 1982).

Ideally, the time of concentration and runoff coefficient are estimated from gage data or regional relationships appropriate for the study area of interest. However, if this information is not available, then general information is available for estimating the time of concentration (see, NRCS, 1986) and runoff coefficients ((see Pilgrim and Cordery, 1992, and Urbonas and Roesner, 1992).

In an urban or agricultural setting where surface flow predominates, the time of concentration can be computed as the sum of the travel time of overland or sheet flow and channel flow along the longest flow path found for the watershed. Overland flow lengths should not exceed more than a few hundred feet. This is reasonable considering that in a an urban setting, flow length to a street gutter probably will not exceed this value. In an agricultural setting, this distance represents the maximum distance to where the flow depth becomes great enough to be more characteristic of the flow in a rivulet or drainage ditch.

The method seems rather simplified in comparison to other watershed modeling techniques, but as Urbonas and Roesner point out (1992, pg. 28.15).

Despite many critics the rational method continues to dominate hydrologic practice for the design of storm sewers, especially in the field of land development. Other, more complicated design tools, may, in fact, not offer more accurate results unless rainfall and runoff data are available for the design subbasin. At the same time, many of the more complicated procedures and models incorporate some aspects of the rational method. As an example, another procedure that is ingrained in hydrologic practice, the SCS method, utilizes the time of concentration and an empirically redistributed intensity-duration-frequency curve as input hyetograph. The U.S. Army Corps of Engineer's STORM model uses runoff coefficients to estimate runoff rates and volumes. Thus, what may appear to be a more sophisticated model is often built by using some rational formula components to estimate runoff rates.

In deciding whether not to apply the method, the following factors should be considered:

- The complexity of the watershed and design problem. If multiple land uses and topographic features (e.g., change in slope) need to be considered then an application of a distributed watershed model might be considered.
- Averaging of runoff coefficients even for a fairly simple problem may not provide a good estimate of the peak flow required for the design capacity. Here, again a more distributed approach to estimating peak discharge might be considered.

Finally, application of a watershed model instead of the rational may be beneficial because hydrographs would be produced rather than peak discharges alone. Although not immediately needed, future studies may require this information.

In conclusion, the value of the rational method depends on the available information for estimating the time of concentration and runoff coefficient. The information available from studies of the method are probably reasonable given the limited drainage areas involved in application of the method. The greatest drawback, is that there does not seem to be much information available for application of the method in the case that peak discharge is due to a combination of rainfall and snowmelt.

4. Watershed modeling approaches for large drainage areas

4.1. Introduction

The purpose of this section is to: 1) describe ungaged watershed modeling approaches useful for estimating design flows for hydrologic design; and, 2) propose from among these the most reasonable approaches for modeling study area watersheds. This discussion will be used as a basis for reviewing the current county practices described in section 5.

The basic assumption made in describing modeling approaches is that direct runoff is the aspect of the precipitation-runoff process of most interest. Base flow, the long term contribution of groundwater to stream flow, can be estimated empirically if it is considered to be important to the design problem. However, it is usually a small fraction of the direct runoff hydrograph estimated for the drainage design problem.

Direct runoff is that portion of the precipitation together with snowmelt that concentrates to a design location in a relatively short period of time. Surface flow resulting from precipitation and snowmelt in excess of surface storage and soil infiltration capacity together with rapidly responding sub-surface flow combine to produce this runoff (see figure 4.1). It is easily the most significant contributor to large floods, and will determine design capacity for drainage systems.

Almost exclusively, the focus of the watershed modeling approach needs to be on design problems for ungaged watersheds. Section 4.2 discusses the issues important to model formulation to obtain estimates needed for design under these circumstances.

Section 4.3 describes model formulation alternatives that can be used to describe the precipitation-runoff process important for design. The model formulation envisions the runoff process as being represented by the following components described in sections 4.4-4.6:

- precipitation-runoff
- channel routing
- storage routing

This is a minimal list of components that reflect the analysis needed for the direct runoff nature of the design problem of interest. Other aspects of the hydrologic cycle such as evaporation or groundwater, and, or complex water resource system modeling involving diversions, or reservoir operations are not considered, not being important to the drainage design problem.

Section 4.7 identifies the best modeling and parameter estimation approaches that should be considered from among various methods described for each of the runoff components. The methods identified will be compared with those currently being recommended by Lake Tahoe Counties in their drainage design manuals in Section 5. Section 6 provides final recommendations regarding approaches as a compromise between the counties experiences and the value of each approach identified in this section.

4.2. Issues for ungaged watershed modeling

4.2.1. Selection of modeling approaches

A vast number of watershed models have been applied in practice to estimate storm runoff for drainage design purposes (e.g., see Urbonas, B. R. and Roesner, L. A., 1992, and Pilgrim, D. H., and Cordery, I. 1992 for a summary of the state of the art in engineering applications, see Singh and Woolhiser, 2002, for a comprehensive description of models used in research and practice). The general categories of watershed models that could be used to estimate runoff for drainage design purposes are as follows:

- Event oriented

Individual precipitation events are simulated to obtain runoff hydrographs at design locations within the watersheds. The methods used to estimate the runoff can be based on computation schemes derived from either empirical or physically based relationships describing watershed precipitation-runoff processes. The models have the capability to compute runoff using these methods for multiple sub-areas which have reasonably uniform runoff characteristics. Runoff from these sub-areas can be aggregated by stream channel routing procedures to obtain total outflow from a watershed. The models do not account for inter-storm watershed processes such as evapotranspiration which affect the water balance within the watershed. In design applications, a design storm is simulated to obtain design runoff associated with a specific risk (e.g., for example the 1% chance exceedance probability flow). HEC-1 (HEC, 1990), HMS (HEC, 2001) and TR-55 (NRCS, 2004b) are popular event oriented models used for design purposes.

- Continuous simulation models

Event and continuous simulation models differ in that the annual water balance is captured as part of the continuous simulation. To do this, such process as evapotranspiration are considered to calculate annual and seasonal water balances, as well as simulating storm runoff. Application to drainage design can involve estimating the design risk due to the continuous period of precipitation simulated or a frequency analysis of the continuous period can be performed to develop design hydrographs. An example of this approach is the use of PRMS by Jeton (1999) to simulate runoff in the Lake Tahoe Basin.

- Physically based models

Both event oriented and continuous simulation models can use physically based methods for simulating watershed runoff. For example, conveyance of flow through stream channels might be computed using a diffusion routing technique such as Muskingum-Cunge. However, the scale at which the sub-areas representing the watershed are formulated usually are considered to lump or average watershed runoff properties, at least to a greater extent than physically based models. Physically based models generally are considered to employ physically based approaches to simulating runoff at a finer scale than the event/continuous models; and, perhaps be more faithful to the fundamental equation for water movement throughout a watershed. Generally speaking these physically based models are applied in the same manner as continuous simulation models. Examples of these models are the SHE model (see Abbot et al., 1986) and the WEHY model (see, Kavvas, et al., 2004)

Precipitation-runoff data availability for the Lake Tahoe basin and potential model prediction accuracy needs to be considered when selecting among these methods in selecting from among these different approaches. Design problems for the Lake Tahoe Basin will be mostly for ungaged watersheds. However, precipitation-runoff data for gage basins is important for both verifying model prediction accuracy, and developing model parameters that can be used for simulating runoff in ungaged watersheds. Unfortunately, very little short interval (hour or less) precipitation data exists for the Lake Tahoe Basin (see section 7.2.2). This makes the application of continuous simulation models particularly difficult for the Lake Tahoe Basin.

A certain amount of controversy exists when it comes to comparing the prediction accuracy of physically based and simpler lumped approaches to watershed modeling. In reviewing the various claims regarding the accuracy of either of these approaches, Woolhiser, 1996, quotes various opinions on the problem:

> *There have been several references in the literature to the problems of overselling of physically based models and the dangers involved in using models without adequate understanding. For example. "I have considerable concern about the practical application of the current generation of physically-based models. Software packages will soon be available to consulting engineers to allow such models to be used in a wide range of applications. There is a great danger that the theoretical rigor that underlies these models will engender uncritical belief in their predictions" (Beven 1989). ……*
>
> *… I cannot disagree with any of these comments. Certainly, at each step in any analysis with any model, the user should ask these questions "Does this make sense?"; "What is the level of uncertainty of my prediction?"; "Does this level of uncertainty render the analysis meaningless?", etc. However, these concerns apply to simpler models as well as the more complicated physically based models.*

Part of the difficulty here is that there has been no comprehensive comparison of modeling approaches. Singh and Woolhiser, 2002, provide a comprehensive review of the state of the art of watershed modeling. This review discusses the various capabilities of about 70 watershed models (see Table 1 of their paper). As they note, the World Meteorological Organization in 1986 performed a comparative study of watershed modeling techniques. Since that time, Singh and Woolhiser note:

> *Except for the WMO reports, no comprehensive effort has been made to compare most major watershed hydrology models. However, efforts have been made to compare models of some component processes. Also, developers of some models have compared their models with one or a few other models.*

They finally conclude at their end of reviewing the current state-of-the art:

> *…A basic questions is: What modeling technology is better? Because of the confusion, the technology developed decades ago is still in use in many parts of the world. This state of affairs is partly due to the lack of consensus as to the superiority of one type of technology to the other. Also, we have not been able to develop physically based models in a true sense and define their limitations. Thus it is not always clear when and where to use which type of a model.*

Consequently, given the lack of precipitation data available for the study area and the lack of evidence that more sophisticated models will out perform simpler techniques, the focus of the modeling recommendations will be on event-oriented models. This is done with the caveat that the work being performed by the Lahontan Regional Water Quality Control Board (LRWQCB) and Nevada Department of Environmental Protection (NDEP) to set TMDL standards might be also considered. In this work, a continuous simulation model is being used to estimate runoff for a 40-year period of runoff for the lake. Synthetic precipitation is being used as part of this study. As an alternative to the recommendations made herein regarding the use of event-oriented modeling, the Lake Tahoe Counties may wish to consider applications with the models being developed by LRWQCB and NDEP depending on the results of their modeling applications.

4.2.2. Event orient approach to modeling direct runoff

The approach to modeling direct runoff needs to be commensurate both with the design problem of interest and the limitations imposed by the information available for ungaged analysis. The design problem requires estimates of a discharge-frequency relationship. The assumption is that this discharge frequency relationship will be obtained by simulating design storms associated with specific precipitation frequency to obtain discharge of the same frequency.

This presents the following two competing parameter estimation issues for a modeling approach:

(1) the parameters reflect some physically identifiable characteristics of the watershed;
(2) the parameter estimates must reflect the coincidence of watershed conditions that results in a particular exceedance frequency discharge.

Ideally, model parameters would be identified from calibration to gage precipitation and runoff data. In model calibration, parameters are determined from as large a number of observed precipitation-runoff events as possible. A range in parameter values will be obtained because of both errors in the estimated watershed average precipitation and approximations made in model formulation. A best estimate of watershed parameters is selected from this range of results. The expectation is that the best estimate of parameters will reflect watershed precipitation-runoff characteristics. For example, estimated NRCS curve numbers will reflect the hydrologic classification of watershed soils; or, hydraulic conductivity will be consistent with given watershed texture classes. In reality, this may not occur not only because of precipitation estimation errors and approximations made in model formulation errors; but also, because watershed physical characteristics are not easily relatable to precipitation-runoff properties. For example, soil texture class is a poor indicator of hydraulic conductivity.

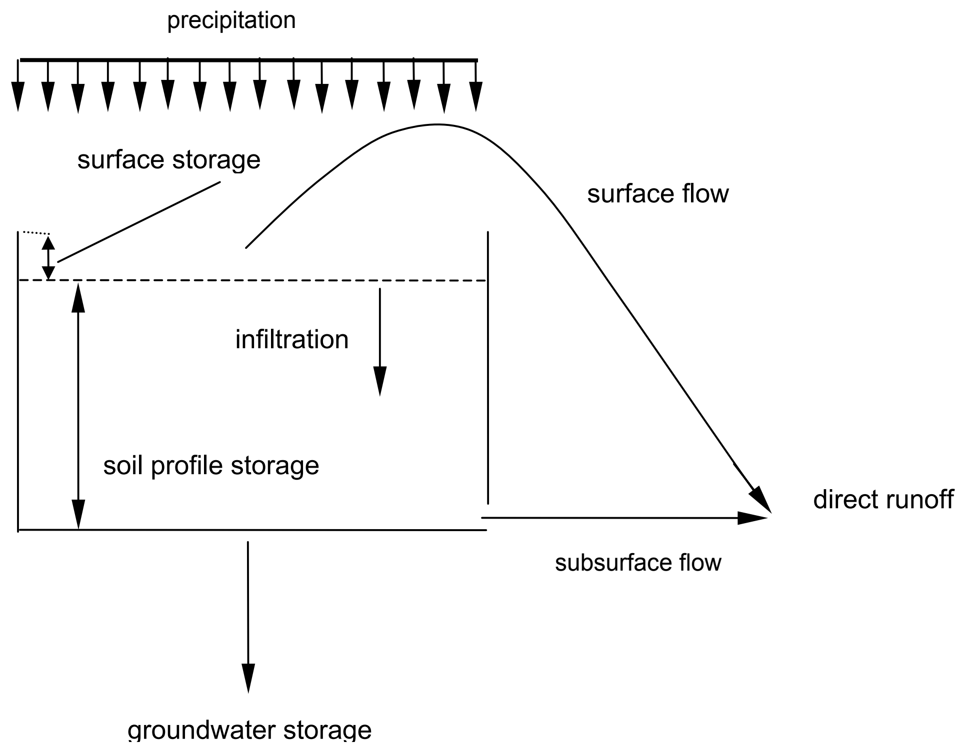

Figure 4.1: Computation of direct runoff volume

Watershed model simulations of design storms to estimate flow-frequency curves for design purposes present a different problem then in calibration or prediction of observed events. The assumption in this simulation is that a single design storm representing a precipitation frequency can be used to obtain runoff for the same frequency (e.g., the 1% chance design storm causes the 1% outflow). This estimated flow-frequency is of course an approximation; because, in reality many possible combinations of storm size, location, and watershed conditions could cause a flow to exceed the 1% flow level. **Basically, the key assumption in developing a design storms is that a combination of storm characteristics and watershed model parameters can be found to estimate a flow-frequency curve which ideally should represent the likely coincidence of many random storms and watershed conditions that cause maximum annual runoff.**

Consequently, a combination of model states (such as initial wetness conditions or snow water equivalent) and parameters need to be selected to best estimate a flow hydrograph for a particular frequency. In gaged analysis, an additional calibration is performed; where typically, initial conditions and loss rate parameters are adjusted to agree with flow-frequency curves estimated from the period of record (see Bulletin 17B, IACWD 1982). The range in loss rates used to obtain agreement, it is hoped, will be commensurate with those found in calibrating the watershed model to observed events.

The problem, of course, is much more difficult, for ungaged analysis because observed data does not exist for estimating model parameters or inferring flow-frequency curves from a period of record. Instead, either: 1) regional information for parameters or frequency curves needs to be available; or, 2) a decision needs to be made with regard to the degree of conservatism to use in selecting parameters and model initial conditions for simulating design hydrographs. In regard to the degree of conservatism required, Pilgrim and Cordery (1992, pg. 9.13) note:

> ***....Use of median values of losses , baseflow, temporal pattern of rainfall, and hydrograph model parameters.*** *Extreme values would convert a design rainfall of selected exceedance probability to a flood with a much smaller probability. If values of these variables are derived from several observed events, the probabilities of the occurrence of values higher and lower than the medians should be equal. Use of these median values in design should minimize the problem of joint probabilities and produce a flood estimate of similar probability to that of the design rainfall.*

In other words, combining conservative estimates of model parameters and inputs will result in drainage designs that do not correspond to the desired level of failure risk.

In the study area, parameter estimates could be obtained by calibration to regional frequency curves estimates available for undeveloped/natural drainage areas greater than 0.1 square miles (see section 7). The parameter estimates obtained from these studies could aid in estimating parameters for smaller areas. For example, estimates of NRCS curve numbers obtained for larger natural/open areas could be used for open areas in small urban basins.

Regional studies do not exist that would relate model parameters to watershed physical characteristics. Consequently, if further research is not performed as suggested in section 7, then the application of watershed models will depend on estimating model parameters from watershed physical characteristics.

In summary, estimating a flow-frequency relationship for design purposes by watershed simulation of design storms presents some very difficult parameter estimation issues. In application to gaged watersheds, a best parameter estimate can be obtained from a range of values realized from calibration to a group of precipitation runoff events. However, these best estimates are likely to need some adjustment, particularly those relating to loss rates, to obtain agreement between frequency curves estimated from the model simulation of design storms and that obtained from a statistical analysis of gage information. The adjustments are needed both because of approximations made in both calibrating the model to observed events and the approximation made in equating the exceedance probabilities of a computed flow and the design storm.

The problem becomes even more difficult for the ungaged problem faced in the study area. The alternative is to use regional information or watershed physical characteristics to estimate model parameters. Regional frequency curves do exist for the undeveloped/natural areas in the study area but not for the urban areas. Unless more research is done to develop regional relationships, as described in section 7, model parameter estimates for urban areas will need to be estimated from watershed physical characteristics.

4.3. Model Components

A standard modeling approach is to represent a watershed as an integrated collection of components (e.g., see HEC, 1990 , model HEC-1) each representing some aspect of the precipitation-runoff process. Figure 4.2 shows schematics of lumped versus distributed approaches to representing the watershed with these components. The lumped approach presume a "black-box" representation for each element. In this representation, the component is defined by average parameters that are used to simulate runoff process given an input. For example, the input to a precipitation-runoff component is a sub-area average precipitation and the output is a direct runoff hydrograph. The hydrograph is computed using "lumped" parameters. These parameters are "lumped" in the sense that the spatial variation of watershed properties are represented in a single average value. For example, the loss rate within a component will be given by a single set of parameters which are average values attempting to capture the spatially varying nature of surface cover and soil hydraulic properties within in sub-area of the watershed. The approximation made using lumped parameters becomes more accurate as the area size becomes smaller and the watershed properties more uniform for the sub-area represented by the component.

This "black-box" representation is relaxed somewhat in a distributed approach to modeling. In the distributed approach, the precipitation-runoff process can be represented by different elements within the component; and, inputs can be combined with the runoff from the component elements. For example, figure 4.3 shows lateral runoff from overland flow planes being combined with an upstream inflow uniformly along the length of a receiving channel. Using lateral runoff in this manner is a distributed approach to modeling the precipitation-runoff process.

The difference in the application of lumped versus distributed approaches can be seen in the manner which components are interconnected in the schematics of figure 4.2. Notice that the output runoff components in the lumped approach are always combined at a control point; whereas; the outputs from distributed type components can be directly connected.

The difference between the components is perhaps not significant given that at some point an average or "lumping" of watershed properties is performed for either kind of component. For example, the properties of an overland flow plane (e.g., width, length, loss rates) are considered to be an approximation to some average characteristics of a sub-area. The more important difference between these two types of components is the difference in computation techniques possible as will be discussed in the subsequent sections describing each modeling component.

4.4. Precipitation-runoff component

4.4.1. Introduction

The precipitation runoff component is used to simulate the direct runoff due to precipitation based on watershed physical characteristics. The elements of this calculation are:

- design storm rainfall;
- snow pack melt;
- loss rates;
- channel routing

The following sections describe the various methods used to represent these elements in the computation of direct runoff.

4.4.2. Design storm rainfall

Design storms are idealized rainfall temporal and spatial patterns of a specified exceedance frequency created from precipitation depth duration frequency curves. The assumption is that the precipitation phase of interest is rainfall given the nature of the design problem.

The design storms are simulated with a watershed model to obtain estimates of flow frequency curves at design locations. This involves a significant approximation in that a set of design storms is used to obtain a flow-frequency which is the realization of the random combination of pattern of storm patterns and watershed conditions. Consequently, a design storm at best represents some average condition which taken together with estimated watershed model parameters is used to simulate a hydrograph.

The various characteristics of the design storm that affect the estimated flow values are:

- estimated depth-duration frequency curves;
- depth-area-reduction factor;
- temporal pattern of rainfall;
- duration of rainfall;
- spatial distribution of rainfall.

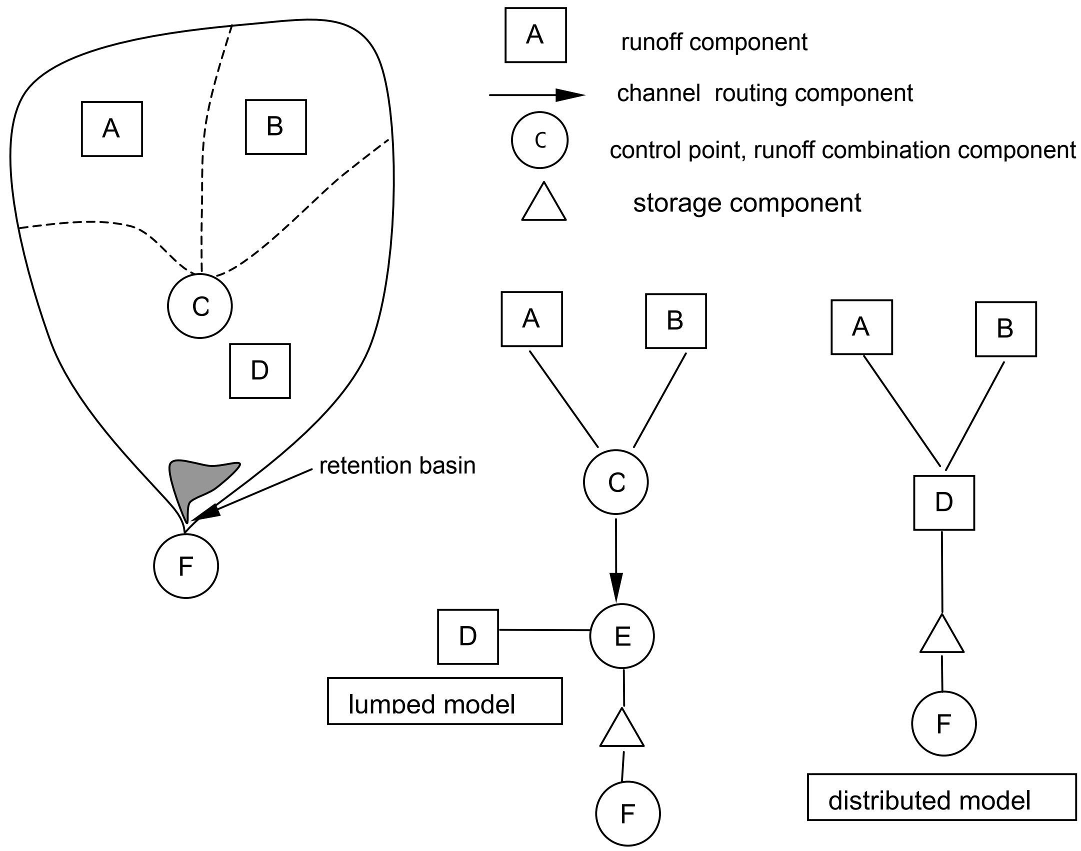

Figure 4.2: Watershed modeling approaches

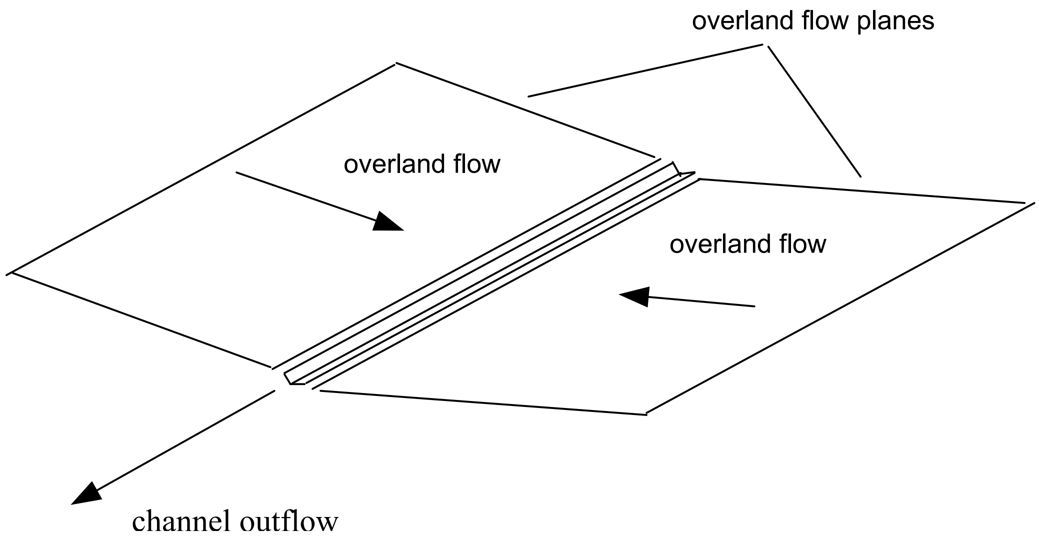

Figure 4.3: Distributed model of surface runoff

The following discusses the issues involved in estimating these characteristics for the study area.

depth-duration-frequency curve
A depth-duration-frequency (DDF)curve provides an estimate of precipitation exceedance frequency at a point for a given duration. NOAA14 (NOAA, 2004) provides the most current DDF curve estimates for the study area.

depth-area reduction factor

A depth-area-reduction factor is typically used to reduce the point estimate from the DDF curves to reflect the reduction in basin average precipitation from the measure gage maximum intensity observed for actual storms. However, the reduction provided by NOAA (National Oceanic and Atmospheric Administration) for the mountainous western U.S. was established for observed storms east of the Mississippi River (see Hershfield, 1961); and, is not relevant for regions where orographic features affect precipitation as in the study area. This factor probably is not important for the small drainage areas of interest in drainage design (less than a square mile). Further research would be needed if this factor becomes an issue for larger drainage area because it is not provide by NOAA14.

temporal pattern of rainfall

Figures 4.4a and 4.4b provide an example of the creation of a temporal pattern of a design storm from a DDF curve. Notice that the peak intensity (assuming a 1hour minimum computational interval) is captured within the storm. Preservation of incremental maximum intensity for the selected duration of the DDF curve results in a balanced storm.

Decisions regarding the balance of the storm should be made based on the characteristics of gage information. Unfortunately, extended short interval (hour or less) gage information is not readily available for the study area. A study of gages in the areas surrounding the study area investigated the degree of storm balance in actual precipitation (see Sacramento, District 2004). The annual maximum 24-hour storms were determined from hourly data (see Table 4.1). The storms were separated annually to determine the effects of season on the degree of storm balance. Table 4.2 shows that storm balance is, to some extent a function of season, the degree of balance varies with duration (1, 6 and 12hours), and that actual storms are balanced between 25-100% of the cases depending on duration and location. **Consequently, assuming a balanced storm for design is likely to be conservative.**

Irrespective of the balance of a storm, the overall shape, including the location of the peak intensity needs to be determined in the context of the loss rate method being used in the watershed model simulation. Figure 4.4, shows how the interaction between storm shape and loss rates will determine the volume of the rainfall that will be available for surface runoff.

The time interval used for specifying the design hyetograph should be equal to the minimum interval available from the DDF curves. The volume of runoff computed will be preserved in simulation irrespective of the segmentation of the watershed and resulting computation intervals used in the simulation. For example, watershed response time may only require a 1 hour computation interval, but the design hyetograph time interval and corresponding loss rate calculation should still be carried out at the smallest increment possible. The resulting time

Table 4.1: Hourly rain gages

Gage	[1]Source	Record Period		Latitude	Longitude	Elevation (ft)
Blue Canyon	NCDC	1-Jul-48	31-Dec-99	39.30000	-120.71700	5280
Hell Hole Reservoir	NCDC	1-Jan-54	31-Jan-99	39.06670	-120.41700	4850
Robbs Power House	NCDC	1-Jan-67	31-Jan-98	38.90000	-120.38300	5120
Woodfords RS	NCDC	1-Jan-79	31-Aug-90	38.78300	-119.81700	5670
[2]Stampede reservoir	Corps	1-Jan-95	30-Sep-94	39.4710	-120.1030	5956
Martis Lake	Corps	1-Jan-95	30-Sep-94	39.3270	-120.1130	5745
Prosser reservoir	Corps	1-Jan-95	30-Sep-94	39.3794	-120.1367	5622

[1]NCDC, the National Climatic Data Center, Asheville; Corps, Sacramento District (2004)

Table 4.2: Fraction of storms where maximum annual 1, 6 and 12 hour depths are contained within the 24 hour annual maximum depth (see Sacramento District, 2004)

Season		[1]Winter			Summer		
Gage	events	1hr	6hr	12hr	1hr	6hr	12hr
Blue Canyon	51	0.39	0.61	0.76	0.75	0.88	0.94
Hell Hole Reservoir	32	0.41	0.63	0.72	0.59	0.81	0.88
Robbs Power House	31	0.26	0.61	0.61	0.45	0.71	0.84
Woodfords RS	12	0.50	0.83	0.83	0.50	0.83	0.92
Stampede reservoir	9	0.67	0.78	0.89	0.67	1.00	1.00
Martis Lake	9	0.56	0.78	0.78	0.67	0.67	0.78
Prosser reservoir	9	0.56	0.78	0.78	0.67	0.67	0.78

[1]Winter period 01 October to April 14, Summer period 15 April to 30 September

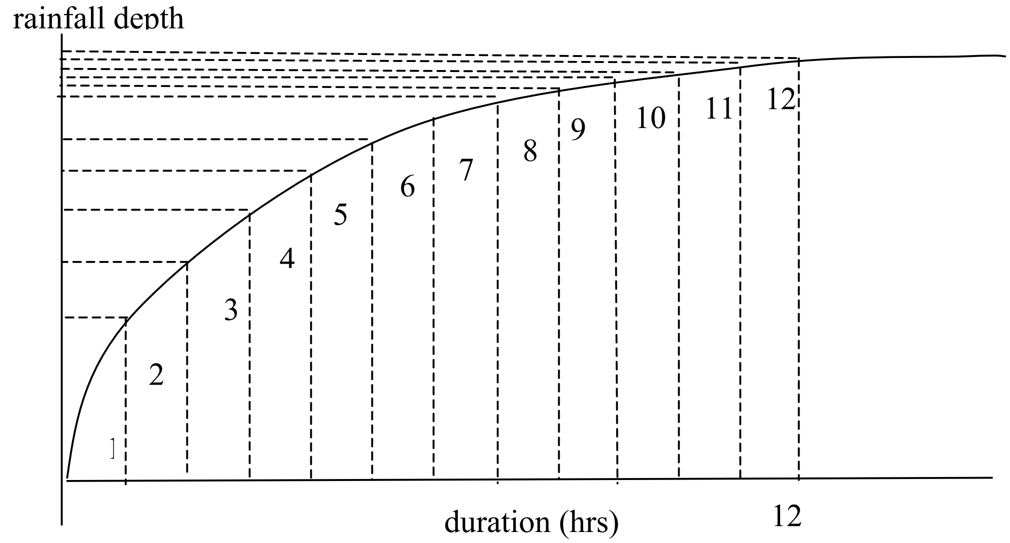

Figure 4.4a: Hourly increments obtained from a depth-duration curve for a particular frequency used to create a design storm

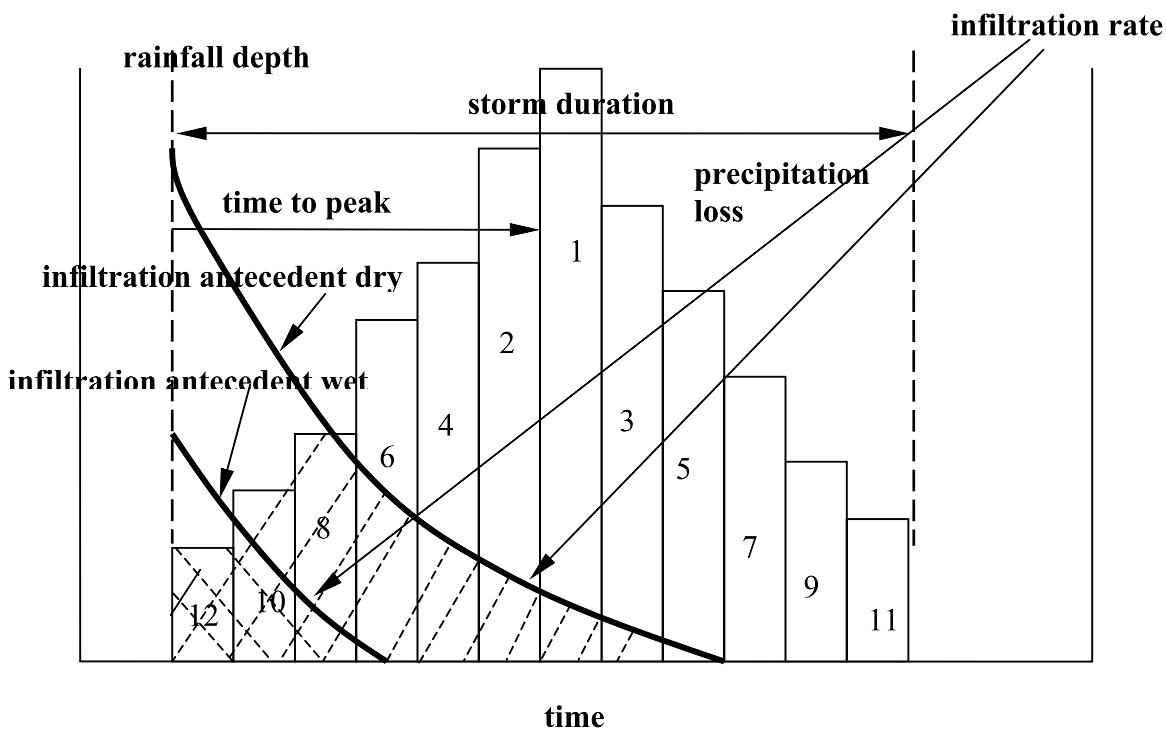

Figure 4.4b: Example design storm for a particular frequency, antecedent conditions impact on storm loss

history of runoff excess can always be aggregated to the hourly interval, while still preserving the runoff volume.

An alternative to the balanced storm approach is the NCRS (1986) 24 hour design storms. These storms are not balanced but are presumably based on characteristics of storms in a particular region. Figure 4.5 compares the 100 year balanced storm with the NRCS Type 1a and II storms for a location near the eastern edge of the lake. The Type Ia storm is be appropriate for the western portion of the study area and Type II storm for the eastern (see figure B-2, NRCS, 1986). As can be seen from the comparison. The type II storm is the most conservative in that the peak intensity is even greater than that of the balanced storm. The high intensity of the type II is probably partly due to the example location selected, and the older information used to establish the NRCS storm patterns. Of course, the overall difference in computed discharge using these various storms depends largely on the loss rate method used.

Storm duration

Storm duration depends on the response time of the drainage area and the design problem of interest. The duration needs to be great enough to determine the peak outflow at the design location. The peak of the outflow will not be realized if the storm duration is too short.

Spatial distribution

The spatial distribution of rainfall is difficult to choose because, as in the case of the temporal pattern, a single configuration needs to approximate the random centering and shape of actual storms that affect flow frequencies. Practically speaking, the design storm can be assumed to be uniformly distributed over a drainage area given the small watershed areas of interest in drainage design. However, the design storm's spatial distribution would need to be discussed if relatively large drainage areas are of interest.

The value of a particular design storm construction is best judged in comparison with estimates of flow-frequency curves obtained from gage data. If this type of comparison cannot been made, then comparisons with regional frequency curves would be desirable. Research is proposed in section 7 that would make these comparisons for the study area. If additional research is not performed, then the design storms would have to be created depending on the desired degree of conservatism.

In conclusion, , the design storms need to be created with a desired degree of conservatism in the context of the watershed model being used to compute design flows if no future modeling studies are performed. Ideally, modeling studies will be performed to see if flow frequency curves computed with the design storm correspond well with frequency curves estimated from gage records and regional regressions. In lieu of future research, a reasonable recommendation is to create balanced storms which are assumed to be uniformly distributed over the drainage area. This recommendation presumes that the drainage areas of interest are reasonably small (less than 10 square miles).

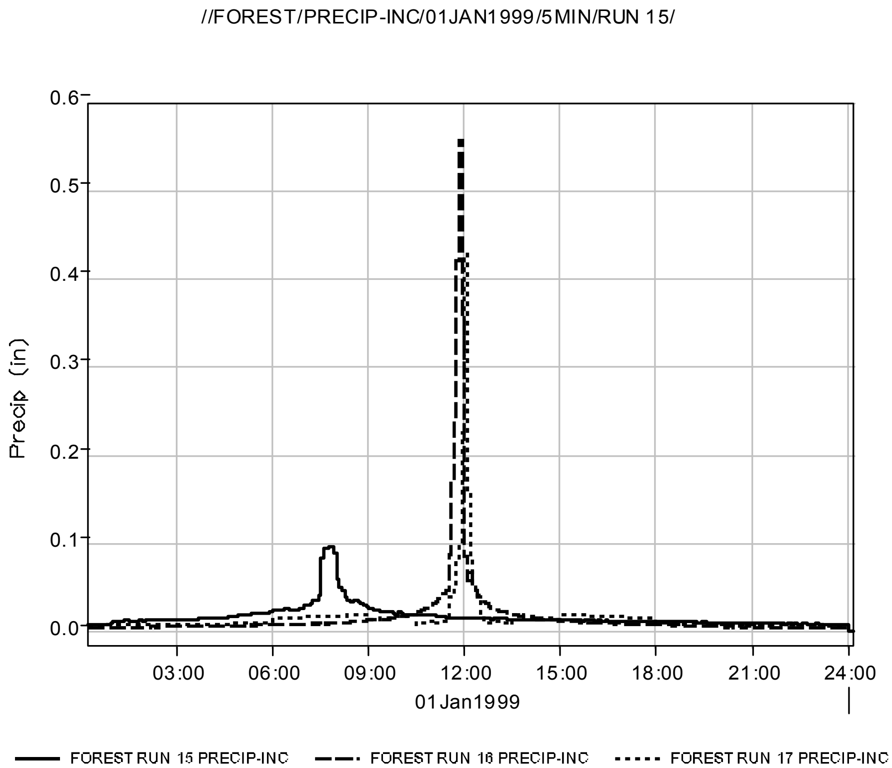

Figure: 4.5: 100-year, 24hour design storms, NOAA13 depth-duration-frequency curves for east side lake level watershed, 4.8 inch depth, NRSC type Ia (run 15), NRCS type II (run 16), balanced storm (run 17)

4.4.3. Snowmelt

Snowmelt is an important contributor to direct runoff in the study area as can be seen from figures 4.6 and 4.7. Snow cover, as measured by the snow water equivalent (SWE) , is available during all months where the maximum annual flow occurs, except at lower elevations for the spring months (e.g., Tahoe City) and for eastern portions of the study area (e.g., Mt. Rose). This means that snowmelt certainly should be a component of the direct runoff in the simulations design storms for higher elevation drainage areas. A more difficult problem will be in determining the relative snowmelt contribution to the computation of flow frequency for areas where snow cover does not exist for the spring months.

The assumed snow water equivalent together with the design storm rainfall determines the available volume for runoff (actual volume depends on the loss rates). Daly et al. (2004) found that the 1-day antecedent SWE is uncorrelated with the magnitude of the annual maximum flow. Although weighted towards the snowmelt floods in the spring months the average 1day antecedent SWE shown in figure 4.8 runoff can be used as an estimate of the upper limit of maximum snowmelt volume that is available for direct runoff.

The snowmelt rate can be estimated using either an energy budget or degree-day approaches. The method to use depends on the data available. The meteorology data required for the energy budget approach is usually not available, even in research catchments let alone gaged watersheds. Application to ungaged basins would depend on being able to regionalize results from gaged basins, or, use general climatologic relationships and watershed physical characteristics to determine energy budget hydro-meteorolgic inputs and parameters.

The following categories describe the kinds of data needed for the energy budget methodologies:

- meteorologic
    - solar insolation
    - cloud cover
- aerodynamic
    - wind profile
    - temperature profile
    - atmospheric pressure profile
    - relative humidity profile
- topographic
    - land surface elevation
    - land surface aspect
    - land surface cover
- snow pack
    - initial water equivalent
    - age of pack
    - heat flux from ground surface
    - area distribution of pack
    - initial water content
    - initial pack condition (temperature profile)

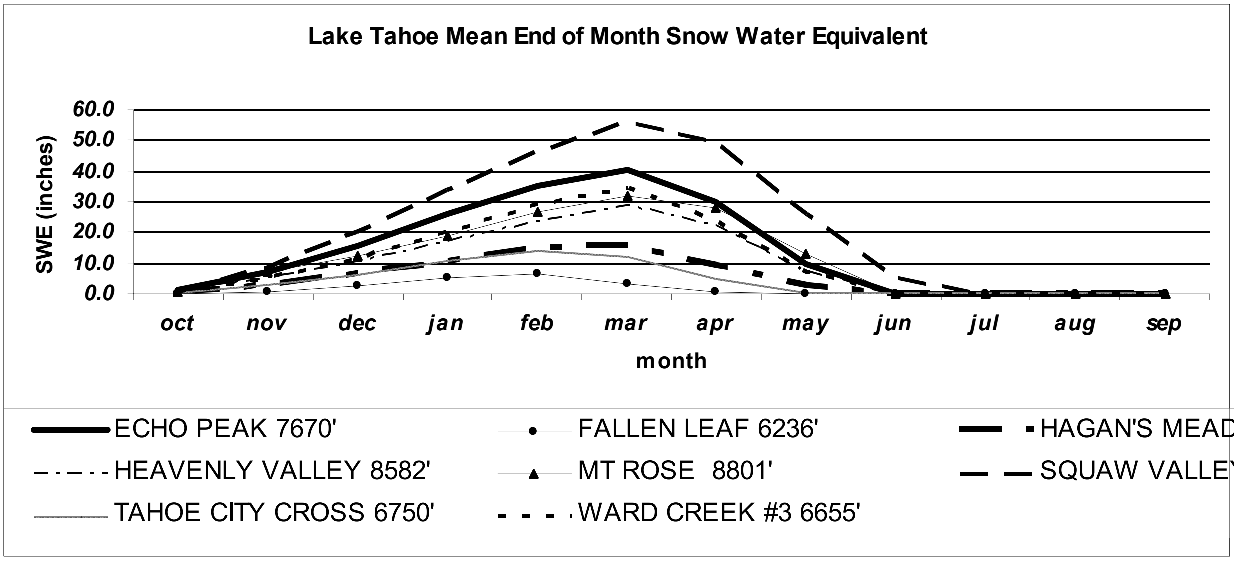

Figure: 4.6: End of month snow water equivalent

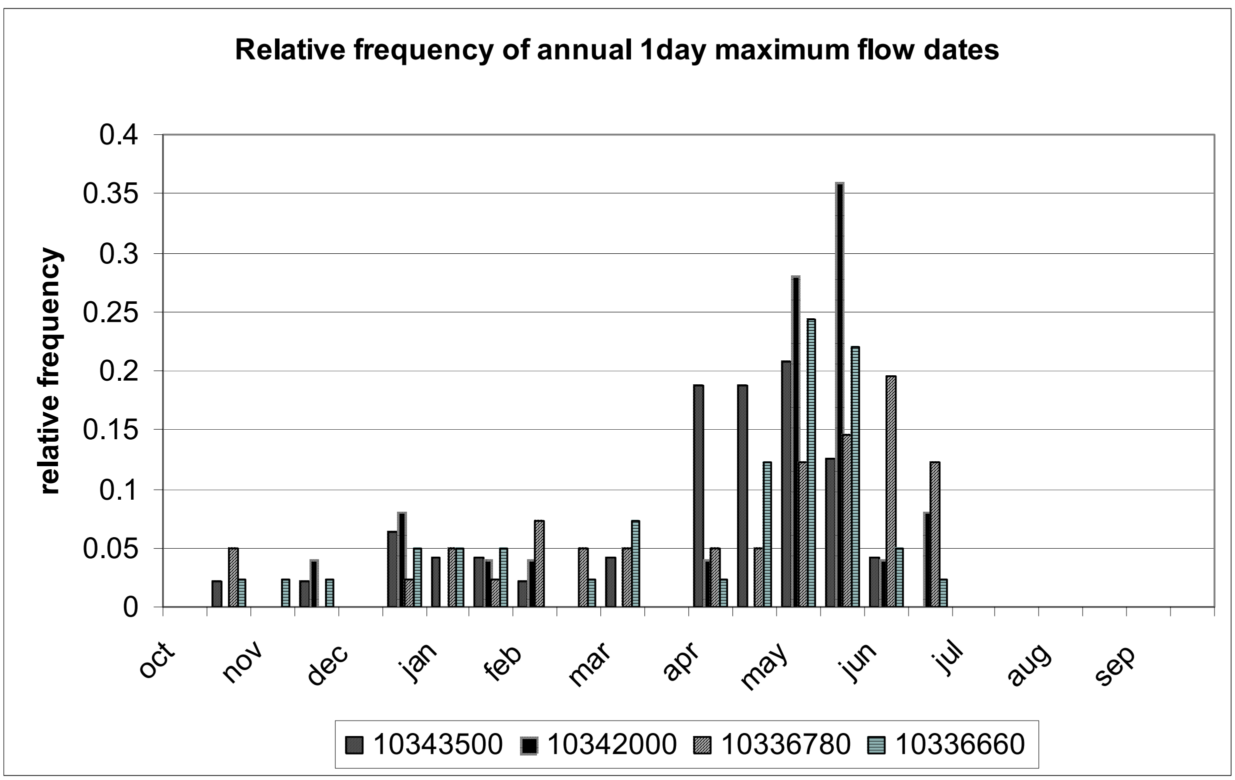

Figure 4.7: Seasonal distribution of annual maximum 1-day flow at Lake Tahoe Basin and near vicinity gages

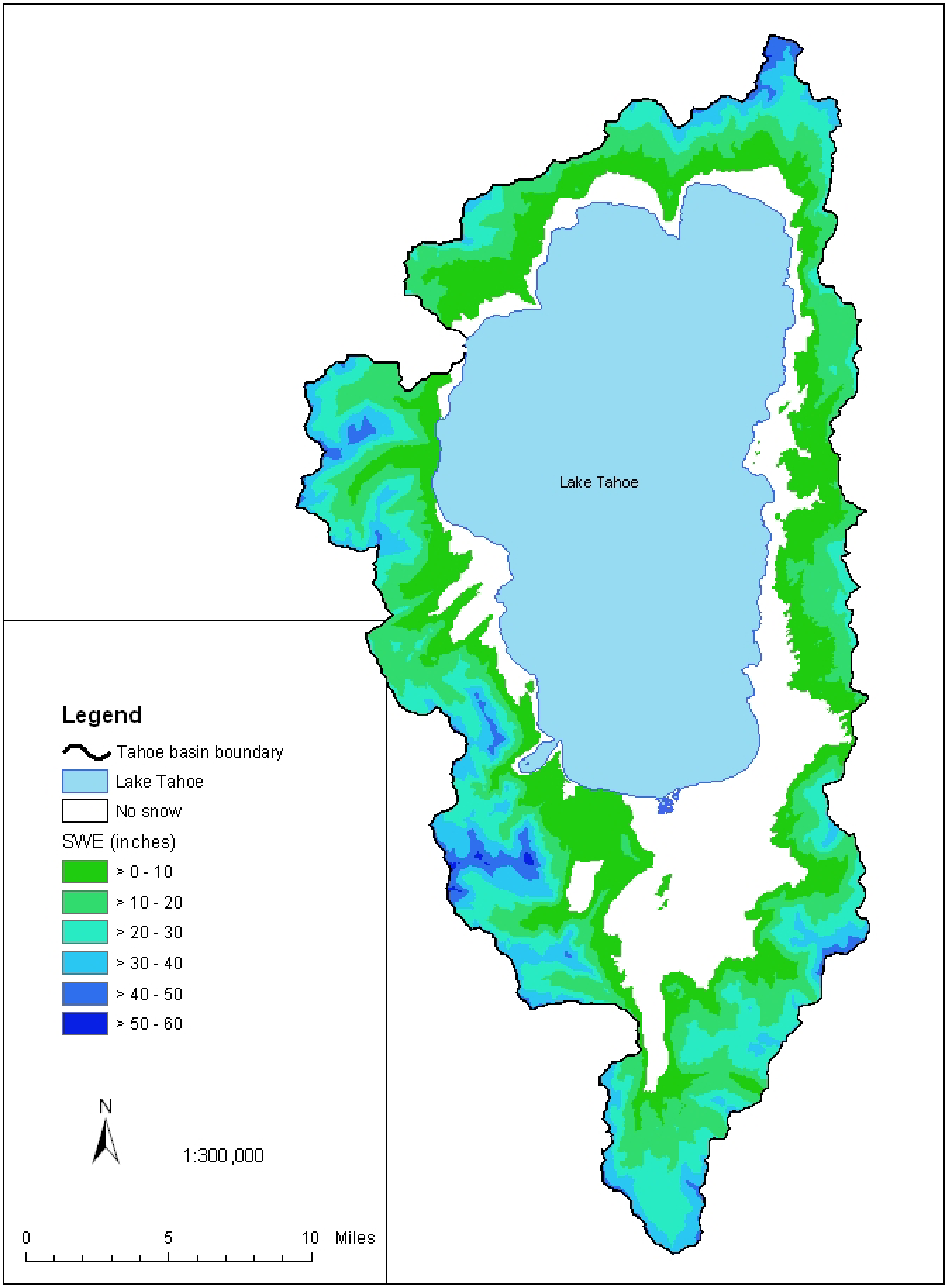

Figure 4.8: Average Snow Water Equivalent antecedent to annual maximum daily flow, (see Daly et al., 2004, reproduced in section 9, Appendix)

This data is needed to calculate the fluxes associated with radiated, convected, conducted, latent and sensible heat budget for the snow pack. For example, consider the data needed for estimating the radiation contribution to the energy budget. Relative humidity is important to estimating the emissivity of atmosphere which is used to determine the atmospheric black body contribution to the energy budget. The age of the pack is important in determining the albedo of the snow surface which measures how much of the incoming solar radiation is reflected at the snow surface. The level of data needed to estimating these kinds of energy fluxes when either calibrating energy budget model parameters or predicting observed runoff is almost never available.

In the case of simulating a design storms, simplifications to the energy budget calculation can be made. For example, the cloud cover can be assumed to be complete, and solar insolation negligible during the storm. Other parameters can be assumed to have average values given experience in application of the method. However, the goal of simulating a design storm to produce a particular exceedance frequency flow needs to be kept in mind in estimating these parameters. Consequently, the parameters need be chosen to best approximate the random coincidence of watershed conditions that result in flow frequencies. This is a very difficult problem when considering the numerous number of parameters involved in the energy budget approach. For example, the specification of the initial snow pack properties (e.g., the value of the initial water equivalent, temperature profile and age of the pack) is just one aspect of the energy budget approach that needs to be specified for each design storm simulates. Arriving at these estimates for a gaged basin would be very difficult; and obviously, would be even more difficult for an ungaged basin.

Jeton (1999) has used the energy budget approach in performing continuous simulation modeling to assess the water balance for the Lake Tahoe Basin. However, this was a very involved study of gaged natural watersheds. The energy budget approach probably provides no advantage to simpler approaches to estimating snowmelt given the difficulty in estimating the parameters for the drainage design problem.

An alternative approach is to use the simpler degree-day method. In this approach, the product of a single coefficient and the difference between the air and freezing temperature gives the melt rate. The difficulty with this approach is that the coefficient is empirical; so that, the value needs to be determined based on gage information. Applications to ungaged basins requires that the value be regionalized in some manner.

A great deal of experience exists in applying both methods to snowmelt problems in most natural/undeveloped watersheds. However, urban areas present a more difficult problem because of the impact of the landscape (buildings, roadways) and snow removal activities on the snow pack.

In conclusion, the application of the energy budget approach would not be commensurate with either the information available for or goals of design storm simulation. The degree-day method is probably a more reasonable approach, being parsimonious in parameters and simple to apply. However, this method depends on being able to obtain the degree-day melt coefficient by calibration.

Perhaps the best approach is to examine the melt rates resulting from Jeton's (1999) energy budget approach or other modeling studies to determine a melt rate for design. Placer County's hydrologic design criteria (see section 5) includes a constant melt rate as part of the computation of design runoff (the methodology used to estimate this melt rate is undocumented). A constant melt rate that can be estimated from previous modeling studies and/or an analysis of gage snow water equivalent is probably a useful practical approach. The maximum amount of melt should be constrained by the average snow water equivalent antecedent to annual maximum runoff shown in figure 4.8. The drawback to this approach is that very little information on melt rates for urban areas exists, and apparently none for the study area. Consequently, urban snowmelt rates would have to be approximated from the melt rates obtained from the sources mentioned.

4.4.4. Loss rates

The application of loss rates to the available runoff volume (the sum of snowmelt and rainfall) results in a direct runoff volume (see figure 4.1). The loss rate is equal to the available runoff volume that infiltrates to groundwater storage and does not contribute to the rapidly responding portion of a watershed outflow hydrograph.

Estimating loss rates for design needs to consider antecedent conditions (snow cover, frozen soil condition and antecedent precipitation), the available surface storage, soil profile infiltration capacity and the dynamics of subsurface flow. A frozen soil generally would be assumed to result in no loss rates. If frozen ground is not present, then the surface storage available for intercepting rainfall is replaced by whatever storage is available due to pack porosity.

The relative importance of surface infiltration and subsurface flow determines the dominant mechanism affecting direct runoff volume. The "hill slope" mechanism is considered most important when subsurface flow dominates, and, "Horton" surface runoff mechanism is dominant when infiltration capacity controls. The hill slope mechanism is important in natural watersheds (i.e., forest and pasture) because of the high infiltration capacity of the well developed soil profiles in these watersheds. A portion of the infiltrated volume contributes to direct runoff via subsurface flow outflow to surface source areas in proximity to rivulets and channels in the watershed.

"Horton" surface runoff is most dominant in watersheds with limited infiltration capacity such as in agricultural or possibly urban areas. In agricultural areas, the limited infiltration capacity occurs due to crusting of bare unprotected soils. In urban areas, human activities can reduce infiltration capacity by watering of lawns, compaction of upper soil zones during construction periods, and other activities that alter the natural soil profile.

The runoff mechanism determines what type of computational approach should be used to compute the loss rate and resulting direct runoff volume. Either physically based infiltration methods or empirical runoff coefficient methods could be used to estimate loss rates for computing Horton surface runoff. Empirical methods will need to be used to estimate direct runoff volume for hill slope runoff.

Physically based infiltration methods estimate loss rates as the sum of a surface abstraction and an infiltration rate. The surface abstraction depends on the surface cover (e.g., forest, lawn or pasture); and places the initial demand on the available runoff volume. In the case of snow cover, the abstraction either can either be assumed to be a function of the water holding capacity of the snow pack or is assumed to be negligible. The available direct runoff greater than the abstraction is infiltrated at a rate proportional to the cumulative infiltrated volume. The proportionality between infiltration rate and cumulative infiltration rate has been investigated thoroughly both via laboratory experiments and theoretical numerical models of the infiltration process. Methods such as the Green and Ampt method (see, Rawls et al., 1995, Rawls et al., 1982) use measures of soil properties to obtain the infiltration rate.

In gaged analysis, the parameters of these method could be obtained by calibration, with the hope that the resulting values would reflect the watershed soil profile characteristics. In ungaged analysis, if regional values are not available from gaged analyses, then a great deal of work has been done to relate model parameters, such as porosity, matric suction and hydraulic conductivity at natural saturation to soil texture class (texture class is the soil) percent sand, silt and clay). However, the relationship between texture class and soil infiltration properties is tenuous at best, particularly for hydraulic conductivity.

Empirical methods for computing loss rates are based on calibration to precipitation runoff data. The most widely used of these methods is the NRCS runoff curve number, CN, (see SCS, 1972). CN relates cumulative runoff (available direct runoff minus loss rate) to cumulative rainfall as a function of land use, antecedent precipitation, and soil hydrologic group. Soil surveys are available for the United States which provide the information needed to estimate the curve number. However, the curve numbers were developed based on rainfall-runoff data for small (on the order of a few acres) agricultural watersheds in the Midwestern U.S. The usefulness of these curve numbers for the study area is questionable. Consider, for example, comments regarding the general applicability of the method by former chief hydrologists of the NRCS, Rallison and Miller (1981, pg. 361):

> *..... There are other concerns regarding use of the procedure* ***[the runoff curve number method]****. Data for developing reliable curve numbers are not equally available throughout the United States. Information on rainfall, runoff, and soil is deficient as a consequence, there are many soil cover complexes that a are either unclassified or lack data for verification. The sparseness of rainfall-runoff data in urban or urbanizing areas has forced reliance on interpretive values with little "hard" data available for verification. ....*

note: information added within **[ ]** to explain quote.

The Holtan method is an alternative method to the curve number (see Rawls, et al., 1995). The method was developed primarily for agricultural areas and suffers from many of the same parameter estimation limitations of the curve number method.

The availability of parameter estimates is the important determinant in selecting between these loss rate methods. Although perhaps not as theoretically justifiable, the loss-rate functions of the empirical methods are likely to be as useful for explaining runoff volumes in actual watershed soils as a physically based approaches. For example, the work done to relate Green and Ampt parameters by Rawls, et al., 1982, relied on soil from the same limited set of watersheds used to originally develop the curve number method. Studies that related loss rate parameters to either urban soil characteristics or natural/open areas within the study area do not exist.

Little data exists which is valuable for estimating the loss-rate characteristics for study area watersheds. A parsimonious approach to this problem is to select the NRCS CN to determine the volume available for direct runoff in lieu of performing further modeling studies. The method is at least based on rainfall runoff data, is commonly accepted and understood in the profession.

Application of the CN method requires that one of three antecedent moisture conditions be selected (see, Rallison and Miller, 1981). Presumably, this antecedent condition is related to antecedent precipitation and meteorologic conditions. In lieu of this relationship, the NRCS recommendation is to use the average moisture condition CN.

Selecting the antecedent condition is very difficult because of the potential for snow cover and frozen ground conditions that obviously would have a great impact on loss rates. The relationship between major runoff event exceedance probability, precipitation and initial snow water equivalent is complicated as is shown in Table 4.3. For example, the 1997 event, the event of record at most gages is on the order of a 0.02 – 0.01 exceedance probability; and, occurred with a significant snow pack throughout the basin during the winter season. In contrast, the May 1996 event is on the order of a 0.1 to 0.025 exceedance probability event; and occurred with essentially no antecedent snow pack. Consequently, selecting a single representative antecedent condition for estimating design runoff in the basin will be extremely difficult.

4.4.5. Direct runoff routing

The direct runoff volume is routed to a sub-area outlet either using a unit hydrograph or a distribute model of runoff. comprised of overland flow planes and an intercepting channel as shown in figure 4.2. As in the case of loss rates described in the previous section, the selection of which method to use depends partly on whether direct runoff is due to hill slope or Horton mechanisms. The ungaged nature of design problem also is an important consideration in selecting a method.

The unit hydrograph (UH) approach is probably most appropriate for representing hill slope aspect of runoff conveyance to the watershed outlet. This is a lumped approach where no attempt is made to model the complexity of subsurface and surface flows that occurs within a watershed. Rather, the UH is used to compute the runoff hydrograph given the time history of the direct runoff volume. Details of the actual dynamics of the water movement is not known or represented.

The unit hydrograph parameters are best estimated from gage data. The calibrated parameters could be regionalized for application to ungaged analysis. However, if regional estimates are not available, then UH parameters must be estimated from watershed characteristics.

The problem involved in estimating these parameters can be appreciated by considering the Clark Unit Hydrograph model (see Pilgrim and Cordery, 1990, and HEC, 1990). In this model a time-area curve is routed through a linear reservoir to obtain the unit hydrograph. The time area curve describes the time history of flow from incremental areas of the watershed assuming that the volume of available runoff is uniformly distributed throughout the watershed. Computing the time history depends on estimating the travel time from each incremental area within the watershed. This computation can perhaps be performed if the contribution to outflow is only due to surface outflow. However, this is not a very easy problem if sub-surface response is involved.

The time area curve would represent the UH if not for watershed storage. The Clark linear storage is used to represent any surface storage (e.g., lakes, out of channel storage areas) that would attenuate the time-area curve representation of the unit hydrograph. . If important, estimating this storage presents a significant problem for estimating the UH for a natural watershed.

Table 4.3: Exceedance probability, precipitation and initial storm water equivalent for major runoff events

Date	[1]Flow	[2]prob	[3]precipitation	[4]SWE				
Blackwood	[6]10336660		[7]Ward Creek	Heavenly Valley	Marlette	Squaw Valley	Tahoe City	Ward Creek
1-Jan-97	2000	0.0189	9	21.5	17.3	58.3	9	21
20-Dec-81	1370	0.0364	6.2	11.4	8.2	20.6	1.1	5
8-Mar-86	920	0.0699	6.7	38.6	36.2	57.9	14.4	42.2
16-May-96	607	0.1407	4.4	0.1	0.1	0.6	0	0.2
1-Jun-75	336	0.3968	-1	-1	-1	-1	-1	-1
Incline	10336700		Heavenly Valley					
1-Jan-97	112	0.0123	6.6	21.5	17.3	58.3	9	21
4-Jun-95	57	0.103	0	35.8	24.1	83.8	0	31.9
16-May-96	57	0.103	0.9	0.1	0.1	0.6	0	0.2
17-Mar-93	32	0.349	0.2	35.7	31.5	91.2	24.5	55.3
Trout	10336780		Heavenly Valley					
1-Jan-97	501	0.0257	3.4	21.5	17.3	58.3	9	21
1-Feb-63	352	0.0747	-1	-1	-1	-1	-1	-1
24-Dec-64	347	0.0778	-1	-1	-1	-1	-1	-1
8-Mar-86	328	0.0906	1	38.6	36.2	57.9	14.4	42.2
18-Jun-83	327	0.0913	0	13.8	9.9	64.5	0	13.6
16-May-96	267	0.152	0.4	0.1	0.1	0.6	0	0.2
Upper Truckee	10336610		Heavenly Valley					
1-Jan-97	3150	0.015	3.4	21.5	17.3	58.3	9	21
16-Feb-82	2010	0.0427	2.4	38.6	36.2	57.9	14.4	42.2
8-Mar-86	1870	0.0509	1	38.6	36.2	57.9	14.4	42.2
16-May-96	1430	0.1001	0.4	0.1	0.1	0.6	0	0.2
Ward	10336676		Ward Creek					
1-Jan-97	1390	0.007	1.9	21.5	17.3	58.3	9	21
19-Dec-81	709	0.0391	2.7	11.4	8.2	20.6	1.1	5
16-May-96	672	0.0451	2.5	0.1	0.1	0.6	0	0.2
8-Mar-86	504	0.0961	6.7	38.6	36.2	57.9	14.4	42.2
11-Jun-83	434	0.1389	0.5	22.1	22.3	78.1	0	28.5

[1]Annual maximum 1day flow (inches)
[2]Exceedance probability (Log-Pearson III, a site statistics)
[3]SNOTEL 1 day precipitation (inches)
[4]SNOTEL snow water equivalent (inches

[6]USGS stream gage ID
[7]SNOTEL gage

If the direct runoff mechanism is due to a Horton runoff mechanism, the routing problem becomes simpler because the time-area curve is easier to estimate for surface runoff. Still, estimating the impact of watershed storage is as difficult as in the hill slope application. The impact of watershed storage is probably minimal for most drainage design problems, particularly in the urban case.

A more reasonable approach, to the typical urban drainage design problem is to use the distributed modeling approach (see Pilgrim and Cordery, 1992). In this approach the routing of direct runoff via overland flow planes and channels replaces the unit hydrograph time-area curve. The value of the approach is that the application has all the information needed to perform the routing once the characteristics of the watershed are conceptualized in the overland flow planes and channels. In urban hydrology, overland flow plane can be used to represent separately the runoff characteristics from open and impervious areas. The channel can then be used to represent culverts or streams that convey the runoff to the watershed outlet. The kinematic wave method is typically used to route the overland flow and either kinematic wave, or more preferably a diffusion method, such as Muskingum-Cunge, for channel routing.

The difference between these approaches is that the UH provides a linear response and the distributed a non-linear response to available direct runoff volume. The unit hydrograph linear-response will double the peak hydrograph outflow when the direct runoff volume is doubled; whereas, a somewhat greater increase will be realized in the overland flow runoff with the application of the kinematic wave to the distributed approach.

In conclusion, the UH approach is preferable in natural areas where the hill slope process is important. Its application can be problematic if regional information does not exist to estimate the method's parameters. If regional information does not exist, then the NRCS lag UH the simplest approach. The NRCS provides detailed criteria for computing the lag needed to estimate this UH (see section 6). Estimating runoff parameters for urban ungaged watersheds is easier than for natural areas because surface flow predominates. Furthermore, the distributed approach is more easily applied, using the same type of information as the UH approach; but, automatically providing the time-area distribution characteristics once the watershed is conceptualized using overland flow planes and channels.

4.5.Channel routing component

Runoff hydrograph passage through watershed water courses is simulated using a channel routing component This concept has already been introduced in describing the distributed approach to simulating precipitation-runoff processes in the previous section. This component is used to route hydrographs between control points (see schematic in figure 4.2).

Application of these methods needs to be considered in the context of the headwater nature of watershed model channel routing. In this context, backwater affects **cannot** be captured by the routing scheme. In other words, the dynamic wave effects that occur because of the interaction between downstream stage and upstream inflow cannot be accounted for by these channel routing methods. If these dynamic effects are important, then an unsteady flow hydraulic model should be used to simulate lateral and boundary condition hydrographs computed by a watershed model.

Despite the limitations imposed due to backwater, headwater channel routing methods are generally very effective in estimating runoff hydrograph attenuation occurring in a river system. The resulting maximum river stage due to the estimated hydrograph peak flows can be computed using a steady non-uniform flow model.

Headwater routing methods employ both hydrologic and hydraulic approaches to route hydrographs. The parameters of the hydrologic methods are obtained either by calibration to gage observations or based on some estimate of travel time and regional experience with the method. Hydrologic methods fall into the following categories:

- lag and route
- storage routing

The lag and route methods simulate the channel travel time for the hydrograph using a time lag; and, the hydrograph peak subsidence caused by channel flow dynamics by averaging successive ordinates. The lag is a simple translation of the channel inflow hydrograph to reflect the channel travel time. The number of consecutive ordinates to average is a function of the desired hydrograph attenuation, the more ordinates average, the greater the attenuation.

If applied in an ungaged analysis, the lag might be computed based on an estimate of channel velocity for some assumed flow depth using a steady flow relationship such as the Manning equation. The number of ordinates to average is much more difficult to estimate. This difficulty in estimating the number of lags makes this approach unusable for ungaged analysis.

The Muskingum method is a much more popular approach where the continuity equation is combined with a channel rating relationship to perform storage routing. The rating is established by relating the volume of the hydrograph in channel storage at any time to the channel inflow and outflow using two parameters, K and X. In an ungaged watershed, K can be related to channel travel time, and X is required to have values between 0.0 and 0.5. Regional experience may provide information on how to select X within this range.

Hydraulic based channel routing approaches are more preferable to the empirical approaches because: 1) the parameters of the methods can be derived from the physical characteristics of the channel; and, 2) the method is derived from the fundamental hydraulic principles for fluid motion. The hydraulic based methods are the kinematic and diffusion (Muskingum-Cunge) wave methods described in the previous section on distributed precipitation-runoff methods (see also, Fread, 1992). These methods are approximations to the full one-dimension equations describing channel flow, known as the St. Venant equations. The application of these methods in watershed models is to simplified models of channel geometry and to normal depth downstream boundary conditions. A single channel cross-section is assumed for a routing component The normal depth boundary condition is applied given the headwater nature of the watershed model application, i.e., backwater condition are not modeled in the simulation.

The kinematic wave does not apply to as wide a range of conditions as a diffusion wave method such as the Muskingum-Cunge. Furthermore, numerical solution of the kinematic wave equations is more difficult because of the potential for the formation of a kinematic shock. This shock wave will always form in a kinematic wave if the upstream inflow hydrograph dominates over the channel lateral inflow and the channel is of sufficient length. Approximations in most numerical schemes do not capture the shock and the limitations of the kinematic wave approach are not apparent. The Muskingum-Cunge method is preferable to the kinematic wave because: 1) the method is a reasonable approximation to flow dynamics for a wider range of channel characteristics and inflow hydrograph shapes; 2) both method will produce the same result (no attenuation) under the channel conditions where kinematic wave is appropriate, and 3) diffusion waves provide a better representation of actual routed hydrographs because the wave diffusion prevents the occurrence of the kinematic shock.

Hydraulic models can be applied in ungaged reaches by specifying the channel cross-section and physical properties. These properties are the channel length, slope, and roughness factors.

A popular pseudo-hydraulic approach to channel routing uses level-pool reservoir routing, often referred to as modified-Puls routing. The level pool routing uses the continuity equation together with a rating curve determined from a non-uniform-steady flow model to perform the routing. Although popular, the method is little more than a hydrologic routing technique wrapped in a hydraulic cover. The problem with the method is that the definition of the channel reach length used to develop the rating curve is arbitrarily based on a stability criterion that relates routing reach length to computation interval. The incremental reach length is required to decrease with this computation interval. As the reach length decreases, the method approaches a kinematic wave approximation where there is no attenuation of the routed hydrograph. Consequently, the effects of routing is strictly related to the computation interval and has nothing to do with the application of a hydraulic model to obtain the rating curve. Basically, a great deal of analysis is done to develop a hydraulic model to determine a rating curve for this approach; which, in reality is irrelevant to the actual diffusion of the hydrograph obtained by the method.

Level pool-reservoir routing can be effective in estimating hydrograph subsidence through a reach if in fact the flow velocity within the reach is negligible, as in a reservoir (see the next section on the storage component). This may occur due to backwater conditions from a culvert or bridge, or perhaps at the junction of major streams. The rating curve can be computed readily for the culvert or bridge; but is a daunting problem for the river junction problem.

In terms of ungaged analysis, the Muskingum-Cunge is clearly the best approach to channel routing. The parameters can be reasonably identified from channel physical characteristics, and the results will be consistent irrespective of the segmentation of the basins into a representative group of model components. If regional information exists, or perhaps if travel time within channel reaches are reasonably short and hydrograph attenuation unimportant, than application of a hydrologic approach, such as a lag or the Muskingum method is reasonable.

4.6. Storage component

The storage component is used to model the attenuation of hydrographs due to retention basins, reservoirs, and highway culverts. The storage routing is accomplished in the same manner as in the case of channel routing discussed in the previous section where the continuity equation together with a storage-outflow relationship is used to route the hydrograph.

The difference between channel and storage routing is how the storage-outflow relationship is calculated. This relationship is developed by combining storage-elevation and elevation-outflow functions for the retention structure being modeled. The storage-elevation relationship can be obtained from topographic maps or survey information. The elevation-outflow function is obtained from the hydraulic capacity of the storage outlet structure. In the case of a retention basin or reservoir this would include spillways and outlets culverts. The hydraulic capacity of top of the embankment and culvert would be included in the elevation-outflow function for a roadway which acts as an impoundment of upstream flow.

4.7. Method selection summary

The following provides a summary of proposed methods described in the previous section for application in watershed modeling of direct runoff for drainage design. These proposed methods are made in lieu of further studies discussed in section 7 which would lead to better approaches to watershed modeling. The methods that should be considered are as follows:

1) Balanced design storm created using depth-duration-frequency information from NOAA14;
2) Snowmelt volume determined based on melt rates found from modeling studies (e.g., Jeton (1999)), total melt volume constrained by available snow water equivalent shown in figure 4.8;
3) Direct runoff volume computed using the NRCS CN method, frozen soil conditions selected based on degree of conservatism and design problem of interest (see section 6 for further discussion);
4) Direct runoff routing performed using the NRCS lag UH method for open areas, distributed modeling using kinematic wave/Muskingum-Cunge routing for urban areas;
5) Channel routing performed with Muskingum-Cunge or Muskingum if data available for computing K and X.

Other approaches to modeling runoff could be adopted. Section 5 provides a review and evaluation of the different methods adopted in drainage models of counties located in the study area. Final recommendations on modeling approaches will be made in section 6 based on both the method proposed in this section and the counties' perspective. .

5. Review county watershed modeling approach

5.1.Introduction

Lake Tahoe Basin runoff modeling approaches need to consider both rainfall and rainfall/snowmelt induced runoff. The purpose of this section is to compare the various modeling methods recommended in county drainage manuals (Placer, El Dorado, Washoe and Douglas) for estimating this runoff for drainage design purposes. These recommendations will be reviewed in the context of the methods recommended in sections 3 (see section 5.2) for small drainage areas (application of the rational method < 200 acres) and section 4 for watershed models of large areas (see sections 5.3-5.6).

Only Placer County directly addresses both the rainfall and rainfall/snowmelt runoff problem. El Dorado and Washoe County provide very detailed criteria for estimating design flows; but do not directly address the snowmelt issue important to the Lake Tahoe Basin. Douglas county provides minimal information. Although only one county considers snowmelt, common issues with regard to estimating precipitation, loss rates and routing flows important to the modeling problem can be compared.

5.2. Rational and Coefficient methods

Table 5.1 summarizes the county "coefficient" methods for estimating peak runoff. The methods are referred to as "coefficient" because these methods differ from the traditional application of the rational method employed. Placer County's approach represents perhaps the greatest deviation by developing a "unit area" discharge by application of HEC-1 to a full range of watershed characteristics. The unit discharge is provided as a function of return interval, flow travel time, elevation and east-west location with respect to the Sierra Nevada Crest.

This application of HEC-1 reflects the assumptions/approximations made in the Placer County recommended watershed modeling methods described in the section 5.3. With respect to the Lake Tahoe Basin, the most significant of these assumptions is that a watershed is snow covered (except for 10% of the design drainage area for any watershed located between 6000-7000 feet) and that zero infiltration occurs due to frozen ground conditions in snow covered areas. The runoff from a drainage areas is then computed as a simple product of unit discharge/area and drainage area. The amount of runoff is corrected for snow free areas based on an empirical relationship involving infiltration rate and elevation (see Table 5-3, Placer County 1990).

El Dorado and Washoe Counties employ the rational method using standard methods for determining the time of concentration needed to estimate the rainfall intensity; but, approach the problem of estimating the runoff coefficient, C, differently. El Dorado County considers two alternative means of estimating C. One method relies on published guidelines and a second is based on results from watershed model simulations. The simulations were used to relate C to the NRCS CN and the drainage area time of concentration. Washoe differs by using previous research that estimates C as a function of return interval (see discussion in section 3).

The Placer County unit discharge is related to the time of concentration, t_c, of the drainage area. The Placer County t_c is based on undocumented equations for overland and channel flow travel times (personal communication, Placer County 2004). The equations must have been specifically developed for the county since the equations do not contain a variable related to precipitation depth or channel flow rate.

El Dorado County uses methods described in TR-55 (NRCS, 1986) to estimate the t_c needed to determine the rational method rainfall intensity from precipitation depth-duration-frequency curves. Washoe County takes a different approach where t_c is estimated from an older study by the FAA (1970) and constrained for urban areas by a relationship obtained from the well known Denver Drainage Manual (USCDM, 1989).

Tables 5.2-5.4 compare the overland sheet flow times, one component of the t_c calculation (see Table 5.1) for various surface cover and catchment slopes, and for an overland flow length of 100 feet. As can be seen, the TR-55 approach recommended by El Dorado County results in the greatest overland sheet flow travel time, and this difference is very significant for forest or open surface cover types.

Each county prescribes limitations on the application of runoff coefficient approaches. All the counties limit the application to simple runoff problems where, for example, where surface storage does not affect runoff. Placer County further limits the application to drainage areas less than 200 acres. El Dorado and Washoe Counties limit application based on recommendations from the respective sources of the estimation equations used to obtain runoff coefficients and time of concentration. The drainage area limitation on drainage area for runoff coefficient methods seems to be mostly based on judgment and not based on any evaluation of method applications.

The Washoe County reference to TR-55 is out of date in that the maximum overland flow length recommended by NRCS is limited to 100 feet (NRCS, 2004a, personal communication: Woodward, 2004). The current limitation is now included in the most current release of the TR-55 watershed model (NRCS, 2004b).

In review:

- The relevance of the methods for estimating overland flow travel time for snow covered areas is not apparent. Placer County uses an equation which probably was obtained assuming snow free ground to estimate unit hydrograph parameters. Presumably the resulting unit hydrograph was used in simulations to derive their peak flow runoff equation. El Dorado and Washoe County have not as yet considered snowmelt in developing criteria. In fairness to the criteria developed by Placer County, very little work has been done to account for the impacts of snow on routing parameters.
- Irrespective of snow cover influence, estimating travel times for natural areas, where subsurface flow is important to the hill slope aspects of direct runoff, is very difficult. The methods described for computing travel time are relevant to surface runoff, and not applicable to hill slope direct runoff. These methods are most appropriate for urban areas where surface flow dominates.
- Placer County's assumption of frozen ground and the corresponding zero loss rate is not substantiated by any analysis, and its degree of conservatism will depend on the design application (see section 6).
- Placer County's equations are undocumented, probably depend on some specific depth-duration-frequency curve estimates. The relevance of these equations to applications where new depth-duration frequency curves are available is not apparent.
- Limitations on runoff coefficient method applications are probably arbitrary, not being based on any modeling studies; but rather, being based on judgment. Still, the recommendation to limit the application of this method to simple drainage problems is reasonable.
- Overland travel times computed using the TR-55 methodology are significantly longer than those obtained than other methods discussed.

Table 5.1: County methods for application to coefficient methods for estimating peak runoff

County	Placer	El Dorado	Washoe
method	[1]HEC-1	rational method	rational method
travel time			total travel time urbanized basins $\leq L_u/180 + 10$ (see USDCM, 2003)
overland sheet flow	travel time (minutes) $\frac{0.355(n_o L)^{0.6}}{S^{0.3}}$ (NRCS, 1986 and Overton and Meadows, 1976)	travel time (hrs) $\frac{0.007(n_o L)^{0.8}}{(P_2)^{0.5} S^{0.4}}$ (NRCS,1986)	travel time minutes $\frac{1.8(1.1-C_5)L_o^{1/2}}{S_o^{1/3}}$ (see FAA, 1970)
overland concentrated sheet flow		travel time (hrs) $V_{open}=16.1435\sqrt{S}$ $V_{paved}=20.3283\sqrt{S}$ $t = L/3600V$ (NRCS,1986)	
channel flow	[2]travel time minutes $\frac{.00375Ln_c^{0.75}(1+Z^2)^{0.25}}{S^{0.375}(A_c Z)^{0.25}}$	velocity Manning equation 2year flow used to compute travel time (El Dorado County, pg. 2-18)	channel travel time (see Washoe County, pg 703, 1996)
runoff factor	$Q = qA - A_p F_i$ $F_i = I(1+\frac{1}{1.3+0.0005E})$	rational C; WEF/ASCE (1992); as function of CN, time of concentration (see Figure 2.5.1, El Dorado County, 1995)	rational C; USDCM, 2003 (see Table 701, Washoe County, 1996) note coefficient function of return interval
application	areas < 200 acres	(see USDCM, 2003); WEF/ASCE (1992)	simple drainage problems, small areas

[1]Unit area discharges are generated by model application to a wide range of conditions and are a function of return interval, flow travel time, elevation and east-west location with respect to the Sierra Range Crest.

Placer County parameters:

L = flow length (ft), S = slope along flow length (ft/ft), $\mathbf{n_o}$ = overland flow roughness, $\mathbf{n_c}$ = Manning's n open channel flow, $\mathbf{A_c}$ = contributing area (acres), **Z** = triangular cross section side slope horizontal/vertical (ft/ft), $\mathbf{F_i}$ = infiltration factor, **A**=drainage area (acres), $\mathbf{A_p}$ = pervious area (acres), **I** = infiltration rate (in/hr), **E** = elevation (ft), **Q** peak discharge (cfs), **q** = unit area discharge (cfs/acre) based on HEC-1 applications.

[2]Assume lateral inflow to triangular shaped channel, if not appropriate, application to other cross-sections using Manning's equation acceptable.

[3]El Dorado County Parameters

$\mathbf{n_o}$ = overland flow roughness coefficient, **L** = overland sheet flow length (ft) < 300 ft, $\mathbf{P_2}$ = 2 year-24hour rainfall depth (in), **S** = slope in ft/ft along flow length

[4]Washoe County Parameters

$\mathbf{C_5}$ = 5 year rational method runoff coefficient, $\mathbf{L_o}$ = overland flow length (ft), $\mathbf{S_o}$ = average overland basin slope (percent), $\mathbf{L_u}$ = watershed length (feet),

Table 5.2: Overland flow roughness coefficients

flow	forest	open	lawn	impervious
[1]overland	0.6	0.4	0.15	[4]0.011
[2]C_5	0.05	0.05	0.05	0.82

[1]NRCS, 1986 (Placer and El Dorado counties)
[2]Washoe county 5 year return interval runoff coefficient used to compute overland flow travel time
[3]Placer county values
[4]Placer county (1990, Table5-5) gives value of 0.11 which is not in agreement with TR55 (NRCS, 1986, Table 3-1)

Table 5.3: Overland flow travel time (minutes) example (length = 100 ft, Slope=0.15 ft/ft)

cover	Placer	[1]El Dorado	Washoe
forest	7	17	8
open	6	15	8
lawn	3	7	8
impervious	3	1	2

[1]2-year 24hour precipitation 1.98 (in)

Table 5.4: Overland flow travel time (minutes) example (length = 100 ft, Slope=0.02 ft/ft)

cover	Placer	[1]El Dorado	Washoe
forest	13	57	15
open	10	41	15
lawn	6	19	15
impervious	5	15	4

[1]2-year 24hour precipitation 1.98 (in)

5.3. Design Storms

The counties use different sources for depth-duration-frequency relationships in their current drainage manuals. The presumption here is that the counties will adopt the most recent estimates of these relationships provided in NOAA14 (NOAA, 2004).

Criteria for developing design storms differ both in the detail provided; and with regard to requirements for: 1) temporal pattern, 2) depth-area-reduction factors, 3) spatial patterns; and, 4) storm duration (see Table 5.5). Both SCS type and symmetric balanced storms describing temporal patterns are employed by the counties (SCS storms do not necessarily center the peak intensity of the storm or capture the full precipitation depth-duration relationship). Depth-area-reduction factors, although referenced from different sources, actually come from Hershfield (1961). As discussed in section 4, these factors were developed for the eastern-half of the U.S. which is not typical of the study area. Washoe county does limit the application of this relationship to drainage areas less than 200 square miles, although no specific reason is given for this criteria.

Placer and Washoe Counties recommend using a uniform spatial distribution for a storm depth determined from the DDF curves centered at the centroid of the watershed. El Dorado County also recommends a uniform spatial distribution; however, the average depth is computed by obtaining a basin average maximum depth (El Dorado County, 1995, pg. 2-7) if the basin is sufficiently large or oriented in such a way that there is a significant variation of mean annual precipitation across the drainage area. The average is obtained from the iso-pluvials for a particular duration and frequency published for the watershed. The depth-area-reduction factor is then applied to the area weighted average precipitation depth. Douglas county does not make any specific recommendation. [Note: Placer county does require an elliptical storm shape for elevations less than 4000 ft msl, but this criteria is not relevant to the study area.]

Table 5.5: Summary Design storm requirements specified by Lake Tahoe Counties

County	temporal pattern	depth area reduction	spatial pattern	duration
Placer	[1]balanced	none	[4]uniform	[5]4*(response time)
El Dorado	[2,7]SCS type I, Ia	[3]Weather Bureau, 1958	uniform	[5](response time)
Washoe	[1]balanced	[3,6]NOAA, 1973	uniform	-------
Douglas	[7]SCS type II	"NOAA methods"	--------	[8]6,24 hour

[1]Peak intensity located midway through storm
[2] see SCS California Bulletin no. CA210-4-6
[3]depth area reduction factor applied to storm depth from depth-duration-frequency curves, no reduction factor for drainage areas < 10 square miles
[4]uniform for drainage area < 1.0 square miles or drainage areas > 4000 feet msl [otherwise elliptical pattern (see Table 5-1 Washoe County) but this is not relevant for Lake Tahoe)
[5]if reservoir/detention structure design, duration depends on storage/outflow characteristics
[6]limited in application to drainage areas < 200 square miles, greater drainages areas consult with local authorities
[7] see NRCS, 1972
[8] 6 hour storm for culverts, 24hour storm for retention structures

The storm duration is specified differently for Placer, El Dorado Counties, and Douglas counties, Washoe county making no specific recommendations. Placer county and El Dorado counties do specify that storm durations be selected to provide sufficient volume to examine retention structure performance.

The duration of the Placer County and El Dorado storm is intended to provide sufficient duration for the outflow hydrograph to reach its peak value. Douglas counties focus on 6hour and 24 hour durations depending on design purpose. Washoe County provides duration-frequency relationships for 1, 6 and 24 hour durations, but does not provide criteria for selection between these durations.

In review:

The county's different approach to storm design will produce different runoff volumes because of the relationship between storm shape and loss rates, and, the difference in estimating storm basin average depth. El Dorado County relies on SCS design storms which are not based on precipitation data from the study area. Three of the counties depend on the NOAA (1973) depth-area-reduction factor for adjusting point precipitation estimates to basin average depth. Unfortunately, this relationship is not relevant to the western U.S, even though it is used in many

NOAA publications covering this area. The importance of determining the appropriate adjustment to point estimates of design precipitation is difficult to evaluate because orographic effects on precipitation cause significant change in precipitation with elevation. The orographic nature of the precipitation probably means that some simple depth-area-reduction relationship does not exist for the study area. Still, a point estimate of precipitation obtained for the centroid may be a reasonable approximation given the limited size drainage areas of interest in drainage design. This reduction factor may not be important because most drainage design problems are usually for small drainage areas (less than a square mile).

5.4. Runoff volume

Runoff volume is equated to the direct runoff volume by Washoe and Douglas Counties. Placer and El Dorado Counties add an empirical base flow component to obtain this volume.

The counties do not provide a discussion of the difference between hill slope and surface runoff mechanism that contribute to direct runoff (see section 4.4.4). Computation of loss rates rely exclusively on the NRCS CN approach which is understandable given that hydrologic group is the only soil infiltration property information generally available. However as discussed previously, the method has not been calibrated for forested mountain regions characterizing the study area. Furthermore; the method has never been adequately applied to areas where hill slope runoff is important to determining direct runoff (see section 4.4.4).

The criteria specified for estimating runoff volume differ greatly between the counties (see Table 5.6). Placer county provides the most detailed criteria covering:

- snow cover;
- frozen ground;
- snowmelt;
- loss rates;
- base flow

The other counties provide varying levels of guidance for estimating the effect of these factors on direct runoff computation. The following sub-sections compare the guidance provided by each county.

Table 5.6: County criteria for estimating initial conditions, loss rates, snowmelt rate and base flow

County	snow cover	frozen ground	[1]ARC	loss rate	melt rate	base flow
Placer	[2]yes	[3]zero loss rate	saturated soil	[4]constant	[5]constant	[6]1.0 cfs/sq-mi
El Dorado	[7]yes	no	ARCII	[9]CN	[7]yes	[8]constant/HEC-1
Washoe	no	no	ARCII	CN	no	no
Douglas	no	no	no	no	no	no

[1]Antecedent runoff condition, for NRCS CN, ARCI, ARCII, ARCIII relate to dry, average and wet conditions
[2]Percent snow cover, 90% elevation 6000-7000 ft, 1000% elevation > 7000 ft (see Table 5-4, Placer County, 1990)
[3]Frozen ground assumed for snow covered area
[4]Computed from NRCS CN values for saturated soil conditions, (see Table 5-3, Placer County, 1990)
[5]Melt rates a function of elevation (see Table 5-2, Placer county 1990)
[6]Major streams
[7]Snowmelt should be included, no direct guidance given, (see El Dorado County, 1995, pg. 2-9)
[8]Constant value for ephemeral streams, empirical method such as in HEC-1 (HEC,1990) (see El Dorado County, 1995, Table 2.6.1)
[9]Composite curve numbers assume pervious areas are pasture in good hydrologic conditions, impervious area directly connected

5.4.1. Snow cover

As discussed in section 5.2, Placer County assumes that the snow covered area is assumed to be associated with frozen ground. In turn, frozen ground is assumed to have an associated zero loss rate. This assumption has a large impact because the criteria result in practically the whole Lake Tahoe Basin being assumed to be snow-covered for runoff calculations. The exception is that 90% snow-area coverage is assumed for watershed elevations between 6,000-7,000 feet.

The other counties do not provide any guidance on estimating the impact of snow cover on loss rates, or direct runoff.

5.4.2. Snowmelt

Placer county provides the only quantitative criteria for including snowmelt in a direct runoff computation. The approach is simple, where a constant flow is added to any direct runoff computed from design precipitation. The derivation of the snowmelt rate is undocumented.

El Dorado county does not directly specify snow cover or any contributions to runoff from snowmelt. However, the criteria recommend that in areas where snow is possible that snowmelt should be considered. No specific criteria is provided for estimating snowmelt; rather, the estimation method should be left for discussion with the county. Washoe and Douglas county provide no criteria for modeling a snowmelt contribution.

5.4.3. Loss rates

As was discussed in section 5.4.1, Placer County criteria would result in frozen ground, and a corresponding zero loss rate for the study area. The loss rates for the drainage area that would not be snow covered are minimal being based on “wet” initial soil conditions. A constant loss rate is provided as a function of the NRCS CN for a particular land cover and soil hydrologic group for this wet condition **The way in which the county derives the constant loss rates from the CN is not explained nor is it clear from the criteria description.**

El Dorado and Washoe Counties use average wetness condition CN values which result in greater loss rates than Placer County. Presumably, El Dorado county would recommend adjusting the loss rates , if snowmelt is important to computing runoff (see previous discussion on snowmelt).

5.4.4. Base flow

Placer County provides a constant inflow rate estimate for base flow contribution to runoff volume. El Dorado county provides criteria for adding base flow to the direct runoff volume depending on whether or not the stream is ephemeral or major. The derivation of the estimates from Placer or El Dorado county is undocumented. Washoe county does not address base flow and Douglas County provide no criteria regarding the runoff volume computation.

5.4.5. Runoff volume method review comments

Placer County's criteria result in significantly greater runoff volumes than would be obtained using the other counties criteria. However, this is probably partly because the other counties have not explicitly considered the impact of snow cover on the runoff process. For example, El Dorado County recommends that snowmelt be considered in estimating runoff; but provides no detailed guidance.

Irrespective of the snow modeling issue, the Placer County choice of frozen ground conditions and constant loss rates results in smaller loss rates for computing runoff volume. The magnitude of constant loss rates selected by Placer County also result in smaller losses than the CN values chosen by the other counties for equivalent land uses when applied to a typical design storm.

A common difficulty with all the counties' guidelines is the reliance on the NRCS CN and in the lack of an approach to modeling snowmelt. As discussed in section 4.4.4, published CN values are not likely to be relevant for the study area. Still this may be the best method available unless further studies are done to calibrate model parameters.

Annual maximum runoff from the study area is highly affected by snowmelt. Placer County, which is the only county that explicitly considers snowmelt, provides only a constant melt rate, without any documentation on the derivation of the rate. Alternatives to this approach need to be considered.

5.5. Runoff Hydrograph

The criteria for computing design runoff hydrographs differ among the counties (see Table 5.7). Placer county employs a distributed approach using kinematic wave; whereas, El Dorado and Washoe counties employ a lumped approach using the NRCS unit hydrograph (see section 4.4.5 for a discussion of lumped versus distributed runoff computation). Placer county will accept a unit hydrograph approach that produces equivalent results to that obtained using kinematic wave routing. Douglas county will accept either approach.

Kinematic wave will produce larger peak runoff than the unit hydrograph approach assuming that the watershed physical characteristics (overland and channel lengths, slopes and roughness characteristics) are represented equally in each method. This results because of the kinematic waves non-linear response to direct runoff volume.

Washoe and El Dorado counties (see Table 5.8) employ the NRCS lag unit hydrograph but employ different estimate for the lag given different criteria for computing the time of concentration, t_c. The Washoe county estimate of time concentration depends on area size and slope of the basin (see Table 5.8). For smaller (less than 1.0 square miles) moderately sloping basins, the time of concentration is the sum of an overland flow and channel travel time. The computation of the channel travel time is not detailed in the criteria. Also, the computation of the runoff coefficient, R, is provided with no reference. **The relationship is curious in that the NRCS CN is related to the rational method 5 year return interval runoff coefficient to obtain R. Documentation on how this relationship was developed would be very useful.** An upper limit on the time of concentration for urban watersheds is adopted from the Denver Drainage Manual (USCDM, 1989). For large areas or steep basins a single relationship using channel length, channel length to the drainage area centroid and channel slope is used to compute t_c.

As discussed in section 5.2, El Dorado county criteria computes t_c as the sum of overland flow travel time (due to both sheet flow and concentrated sheet flow) and channel flow travel time. Concentrated sheet flow is differentiated from sheet flow by the greater depth flow that is typical of rills of flow. The channel flow travel time is determined by application of Manning's equation using the 2-year return interval flow.

In review:

- The relationship $t_{lag}=0.6t_c$ was developed by the NRCS based on unpublished data (see NRCS, 1993 , pg. 15-6). This relationship was most likely developed for the data available from NRCS test watersheds, which were predominately small (on the order of acres), agricultural and located in the Midwest. There is not much evidence that this relationship is applicable to the study area. Consequently, a great deal of effort should not spent in selecting a procedure to compute t_c given the very rough approximation to estimate t_{lag}. Any reasonable method is probably acceptable.

- The roughness coefficient for overland flow referenced by El Dorado County, N=0.11, is in conflict with TR-55, and other sources of basic information. The value should be N=0.011 as reported by most basic research. The value reported by El Dorado has been incorrectly reported in other references as well.

- As discussed in section 4.4.4, the unit hydrograph approach is probably the most useful for simulating the watershed runoff response for natural areas where direct runoff is due to both subsurface and surface flow. A distributed approach using kinematic wave seems much more reasonable for simulating the surface flow dominated direct runoff in urban areas. The distributed approach is simpler to use and relies on the same watershed physical characteristics as the unit hydrograph approach to compute the runoff hydrograph.

Table 5.7: Runoff Hydrograph computation methods

County	model	method
Placer	distributed preferred, lumped possible	kinematic wave, NRCS UH
El Dorado	lumped	NRCS UH
Washoe	lumped	NRCS UH
Douglas	distributed or lumped	kinematic wave or UH

Table 5.8: Computation method comparison for NRCS Unit Hydrograph Lag

[1]Washoe County			[2]El Dorado County	
lag components	estimate	constraint	lag components	estimate
		basins # 1.0 sq mi, and/or slope # 10%	t_s (sheet flow) (hrs)	$\frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{0.4}}$ (see NRCS, 1986)
t_o (overland) (minutes)	$\frac{1.8(1.1\text{-}R)L_o^{1/2}}{S_o^{1/3}}$ $R=0.132(CN) - 0.39$ (see FAA, 1970)	$L_o < 500$ feet	V_{sc} (concentrated sheet flow velocity unpaved)(ft/s)	$16.1435\sqrt{S}$ (see NRCS, 1986)
t_t (channel)	channel travel time (see Washoe County, pg 703, 1996)		V_{sc} (concentrated sheet flow velocity paved)(ft/s)	$20.3283\sqrt{S}$ (see NRCS, 1986)
			t_{sc} (concentrated sheet flow travel time) (hrs)	$L_{sc}/(3600V_{sc})$
			t_t (hrs)	Manning equation 2year flow (El Dorado County, pg. 2-18)
t_c time of concentration (minutes)	$t_o + t_t$	Urbanized basins $\leq L_u/180 + 10$ (see USDCM, 2003)	t_c time of concentration (hrs)	$t_s+t_{sc}+t_t$
t_{lag} (minutes)	$0.6t_c$		t_{lag} (minutes)	$0.6t_c$
t_{lag} (hours)	$(22.1)K_n(LL_c/S^{0.5})^{0.33}$ (see USBR, 1989)	basins > 1.0 sq mi, and/or slope > 10%		

[1]Washoe County Parameters

R = runoff coefficient, $\mathbf{L_o}$ = overland flow length (ft), $\mathbf{S_o}$ = average overland basin slope (percent), **CN** = NRCS curve number, $\mathbf{L_u}$ = watershed length (feet), $\mathbf{K_n}$ = roughness factor for channel **L** = length of longest water course (miles), $\mathbf{L_c}$ = length of longest water course measured upstream to a point opposite the centroid (miles), **S** = Representative (average)slope of the longest watercourse (feet per mile)

[2]El Dorado County Parameters

n = overland flow roughness coefficient, **L** = overland sheet flow length (ft) < 300 ft, $\mathbf{P_2}$ = 2 year-24hour rainfall depth (in), **S** = slope in ft/ft along flow length

5.6. Channel Routing

The counties recommended routing methods and application criteria are summarized in Table 5.9. El Dorado County provides the most detailed criteria, clearly specifying the conditions where the simplest to most sophisticated techniques should be applied.

Placer County recommends the use of Muskingum-Cunge method when "backwater" conditions do not exist. Curiously, the county recommendations restrict the modified Puls approach to situations where detailed cross-section information is available. Perhaps the feeling is that the storage-outflow relationship will not be sufficiently accurate with less detailed information. If detailed cross-section information is not available, the Muskingum method is recommended

Washoe County recommends application of the Muskingum-Cunge and kinematic wave approaches. Douglas County does not provide criteria.

In review:

The counties' recommendation to use kinematic wave and modified Puls for channel routing is not supportable. As discussed in section 4.5, diffusion wave routing, e.g., using Muskingum Cunge, is applicable to a wider range of conditions than kinematic wave routing and does not suffer from the occurrence of the kinematic shock. Basically, it is a better method for channel routing. As also discussed in section 4.5, modified Puls is applied in an incorrect attempt to try and account for storage and backwater effects in hydrograph routing. Irrespective of the storage outflow relationship used, the subsidence of the hydrograph will be a function of the reach length (the number of steps) used. For example, Placer County (1990, pg. V-20) recommends that the "..... travel time (based on celerity) through a reach is approximately equal to the simulation interval." Basically, this is a specification of the kinematic wave Courant Condition, that, as the reach length decreases, will result in no attenuation of the hydrograph, **independent of the storage outflow relationship used for method**.

Table 5.9: Comparison of channel routing method criteria

County	[2]Routing methods	Comments
Placer	MC, MP, MU	KW subbasin < 1.0 sq mi, MP requires detailed x-sections and backwater profiles, MU when detailed x-section not available
[1]El Dorado		
	MC, KW	ungaged impacts
	MP	backwater impacts
	MC, MP	overbank flow
	MC,KW,MK,MP	S > (0.002) & (T)(S)(V)/(Y)>117
	MC,MK,MP	0.0004 < S < (0.002) & (T)(S)(V)/(Y)>117
	MC	S < (0.0004) & $(T)(S)(gY)^{0.5}$ > 15
	Dynamic Wave	S < (0.0004) & $(T)(S)(gY)^{0.5}$ < 15
Washoe	MC,KW	
Douglas	-----------	no recommendations

[1](see El Dorado county, 1995, Table 2.7.1), T = duration of hydrograph, S = friction slope, Y= flow depth for hydrograph average discharge, V=cross section average velocity, g=acceleration due to gravity, all variables in consistent units
[2]MC = Muskingum-Cunge MU = Muskingum, KW = kinematic wave, MP = modified Puls (level-pool routing)

6. Recommendations for hydrologic design criteria

The purpose of this section is to provide recommendations for the hydrologic modeling approaches that could be used in a drainage design criteria manual. These recommendations are based on currently accepted practice for estimating runoff for drainage design. However, the recommendations could be significantly improved by performing the additional studies described in section 7.

The recommendations presented given no further research are admittedly based on some judgment, and certainly could be change based on the degree of conservatism desired by agreement among counties and other interested parties. As discussed in section 1, the reliability of a particular hydrologic method depends on some measure of calibration to gage observations. Without this information, the recommendations provided reflect a certain judgment, which could be modified based on the degree of conservatism desired by a regulatory authority.

These recommendations are largely based on NRCS method for computing runoff from ungaged areas. The limitations of these methods in applications to the Lake Tahoe Basin have been described in detail. However, at least the methods are based on some data and have gained acceptance in the profession. Still, calibration studies would almost certainly produce more reliable watershed model predictions.

The recommended methods have been adopted by one or more of the study area counties, and are also described in detail in many references (e.g., NRCS, 1986, and Maidment, 1992). The following recommendations are made for simulating each component of the precipitation-runoff process.

Rational method

The rational method will be applied using the recent NOAA14 depth-duration-frequency curves and the TR-55 (NRCS, 1986) methodology for computing time of concentration. The recommendation provided for maximum overland flow length should be replaced by the most recent recommendation to limit this length for sheet flow to 100 feet rather than 300 feet (NRCS, 2004a and 2004b, and, personal communication: Woodward, 2004). The runoff coefficient estimates can be obtained from EPA (1983).

The maximum basin size to use for application of this method should depend largely on the variation in runoff properties and complexity of the drainage system in a drainage area being analyzed. Estimating a composite runoff coefficient and the appropriate time of concentration for a drainage area becomes increasingly difficult as the drainage area contributions to runoff become more distributed. The typical rule of thumb is that drainage areas less than 200 acres runoff properties are simple enough to be captured in a rational method analysis.

Design storm

The HEC balanced storm approach captures the critical peak-intensity-duration characteristics defined by precipitation depth duration frequency curves. This is a conservative approach in that the analysis of available storm data for regions surrounding the (see section 4.4.2) found that actual storms for a significant portion of occurrences were not balanced

The duration of the storm should consider both the time of concentration of the watershed and the design of detention/retention storage. At the very least, the storm duration should be long enough to where the whole basin will be contributing to the computed peak runoff needed for design. If a retention structure is being designed, then the volume of runoff is important. The duration of the storm should be great enough to where increments in storm duration do not affect the design of the retention structure to control storm water.

Depth area reduction correction to the point estimates of precipitation will not be used because: 1) of the increase in precipitation with elevation; and, 2) the lack of studies analyzing the change in average storm depth with drainage area for the study area.

Frozen soil

The decision to assume a frozen soil for a watershed depends to some extent on the design problem of interest and the degree of conservatism of interest. For example, a frozen soil assumption is conservative when interest focuses solely on the maximum runoff for design of conveyance capacity or determining the regulatory flood plain. Alternatively, the assumption is not conservative when designing retention storage to reduce runoff from some urban development to a pre-project magnitude A frozen soil condition would produce the same volume of runoff as the impervious area created by the planned development. Under the assumed frozen soil condition there would be no need to provide retention storage to reduce runoff to the pre-existing condition. The assumption of an unfrozen soil for design problems where runoff needs to be controlled to a pre-project level is more conservative than the frozen soil assumption. Consequently, the recommendation is to take a conservative approach where frozen soil (zero loss rate)is assumed for sizing conveyance and determining the regulatory flood plain, and, an unfrozen ground assumption for sizing retention areas.

Snowmelt

Snowmelt will be included as part of the runoff excess volume in terms of an average rate in basin inches per hour. This rate should be based on previous modeling studies of snowmelt in the study area (e.g., Jeton, 1999). The melt rate will limited to the amount of available storm water equivalent provided in figure 4.8

Loss Rates

The runoff excess, and corresponding loss rates, will be calculated based on the NRCS CN, assuming an average antecedent wetness condition (see NRCS, 1986).

Runoff transform

Natural/open areas will refer to TR-55 (NRCS, 1986) for estimating the NRCS Unit Hydrograph. Urban areas will employ a distributed approach using kinematic wave overland flow planes and Muskingum-Cunge channel routing (see HEC 1990, 2001). The unit hydrograph method is applied to natural/open areas because direct runoff can be due to both surface and subsurface flow. The distributed approach is recommended for urban areas because surface flows dominate contributions to direct runoff, it is simpler to apply than the unit hydrograph method and can easily capture the separate responses from pervious and impervious areas. This application should use the most recent thinking which limits the maximum overland flow length for sheet flow to 100 feet rather than 300 feet (NRCS, 2004a and 2004b, and, personal communication: Woodward, 2004)

Channel Routing

Muskingum-Cunge method will be used to perform hydrologic channel routing (see HEC 1990, 2001). Standard published values of roughness coefficients will be employed (see TR-55, 1986). In circumstances where some estimates of travel time can be made, the Muskingum method can be employed.

Application with regression equations

Regional regression equations relating annual peak and maximum daily volume duration frequency curves to watershed meteorologic and physical characteristics have been developed for the study area (see SPK, 2005). These regression equations are useful for relatively large drainage areas (greater than 0.5 square miles) that experience a significant proportion of storm runoff from snowmelt (certainly watersheds with a significant proportion of drainage area above 7000 feet).

The regression equations have limited usefulness for application to small urban drainage systems, which will probably be the main concern of a drainage design manual. Future research discussed in section 7 performed with regression equations can be used to validate watershed modeling approaches to small ungaged urban basins.

A nation wide study (U.S. WRC, 1981) demonstrated, for the most part, that the USGS regression equations were more accurate than event oriented watershed models in predicting peak annual flow frequency curves. The results of the study were not universally accepted by those who favored the watershed modeling approach. However, the study did shows that the regression equations were: 1) unbiased; 2) the regression standard error gave an accurate picture of prediction accuracy; and, 3) more accurate than the watershed modeling approach, at least based on the criterion establishing the comparison. This study provides good reasons for using the regression equations to validate watershed model predictions in ungaged areas.

Consequently, where applicable (open areas, drainage areas greater than 0.5 square miles, and where snowmelt contributes to flood peaks), the regression equations can be used in any of the following ways to calibrate/validate runoff methods:

- Watershed model initial conditions and/or loss rates could be adjusted to have the model and regression predicted peak and annual volume-duration-frequency curves agree within some reasonable margin. The target duration frequency curve is a function of the design problem. For example, in the case of culverts, the peak flow frequency curves might be targeted; whereas for retention pond design, the 1day duration curve may be more important. In this approach, the calibration parameters are the initial conditions and loss rates.
- Watershed model parameters could be adjusted to require the model predicted frequency curve to be contained within a pre-selected regression predicted confidence interval (e.g., the 90% confidence interval). This is a variation of the previous approach, except that it is not nearly as restrictive on model predictions.
- Watershed model and regression estimates could be averaged. This average frequency curve could then be used to obtain both estimates of design peak flows and hydrograph volumes.

Applications where loss rates are determined in model prediction comparisons with regression equations could be used to estimate loss rates for open areas in urban watersheds, even though regressions are not directly applicable to these watersheds. Any of these options might be considered reasonable. Selection will depend on the level of accuracy of watershed model predictions. This tends to be a matter of judgment since estimating watershed model prediction error is note easily done.

7. Future studies to improve watershed model for application to traditional drainage design

7.1. Introduction

The goal of future studies will be to improve watershed model predictions of annual peak and flow frequency curves for ungaged areas. The studies performed will depend on the available resources, quality of observed precipitation-runoff data available, and the improvement in model prediction accuracy that can be obtained as a result of the studies.

Improvements to watershed model predictions could be obtained by performing calibration/validation studies using both observed precipitation-runoff events and stream gage/regression equation estimates of flow frequency curves. Flow frequency curves from stream gages and regression equations have been developed for the Lake Tahoe Basin (see SPK

2005). Precipitation-runoff data for the study area does exist, but is somewhat limited and the quality of the data needs to be evaluated.

The overall strategy for improving model predictions would be as follows:

1) Develop watershed models for gaged and ungaged watersheds;
2) Use the models to predict flow frequency curves at these locations;
3) Adjust the model parameters to obtain a reasonable agreement between model and stream gage/regression equation estimates of flow frequency curves;
4) Develop criteria for estimating watershed model parameters using regional frequency information in applications to ungaged watersheds;
5) Regionalize watershed model parameters for use for watershed with drainage areas outside the range of regional regression equation applicability.

In implementing this strategy, the watershed models could be formulated based on:

1) the recommendations described in section 6;
2) model calibration studies that estimate parameters from observed precipitation-runoff data;

An additional investigation that might prove to be worthwhile is to re-examine the procedure for developing design storms. Design storm characteristics together with parameter estimates play a significant role in determining model predictions. Research into improving design storm development would be very worthwhile for this reason.

The improvement in design storms would come from modifying the use of the NOAA14 precipitation depth-duration-frequency curve estimates to obtain: 1) rainfall depth-duration frequency (DDF) curves; and, 2) seasonal depth-duration frequency curves. The NOAA14 curves are based on precipitation; but rainfall causes flooding. Consequently, estimating rainfall DDF curves would perhaps be more useful in developing design storms.

Additionally, the difference in winter versus summer storm types in the study area might be a reason for developing seasonal design storms from seasonal precipitation or rainfall DDF curves. The seasonal design storm would be simulated by a watershed model to obtain season flow frequency curves. A mixed distribution analysis would then be used to combine the seasonal curves to obtain an annual frequency curve. Annual frequency curves simulated using design storms based on annual rainfall or seasonal DDF curves could be evaluated as part of the watershed model validation/calibration effort.

Probably the most important decision to be made in performing the model validation/calibration studies is in selecting criteria for evaluating the difference between watershed model predicted and gage/regression estimated frequency curves. As noted in section 6, a national test (U.S. WRC, 1981) demonstrated that regional regression equations were, generally, more accurate than uncalibrated watershed model predictions of flow-frequency curves. Consequently, there is some evidence for adjusting watershed model parameters to obtain some reasonable agreement between model and gage/regression equation estimates of flow frequency curves To what degree the parameters should be adjusted is an open question. Two principles should be considered in adjusting parameters in these comparative studies:

3) The watershed model predictions should agree on the average with the stream gage/regression estimated flow frequency curves over a reasonable number of comparisons;
4) The model parameters should be constrained to some physically reasonable values.

These can, and most likely will be, competing requirements. Ideally, physically reasonable watershed model parameters will result in model predictions that agree on the average with the gage and regression flow frequency estimates. However, this will not necessarily be true in practice. Ultimately, guidelines would be developed that specified to what extent model parameters would be adjusted to bring into agreement model predicted and regression estimates of flow-frequency curve for ungaged watersheds.

The following sections describe in more detail the future research that could be performed to improve watershed model predictions of annual flow frequency curves for ungaged watersheds. The discussion is separated into future studies that would parameter estimates based on calibration to precipitation runoff data, improvements in design storm development, and comparative studies needed to develop parameter estimates for ungaged watersheds. Future calibration studies would involve the following efforts (see section 7.2):

- Evaluate precipitation/gage flow record useful for calibrating watershed models (see section 7.2.2). If this data is not valuable then the calibration can not be performed.
- Study relationship between frozen soil (zero loss rate) conditions and annual maximum runoff (see section 7.2.3) This is important for evaluating the necessity of conservative recommendations made in section 6 and for providing information for setting loss rates in model calibration efforts to precipitation runoff data.
- Determine degree of balance in gage precipitation events (see section 7.2.4). This study is important for improving estimates of temporal patterns in design storms.
- Calibrate watershed models to observed precipitation/runoff events, assuming existing precipitation/flow data is useful for this purpose (see section 7.2.5).

The design storms used in watershed model simulations could be improved by modifying the DDF relationships obtained from NOAA14 by (see section 7.3):

- Estimating rainfall depth-duration-frequency curves by adjusting NOAA14 precipitation DDF curves using available gage records. (see section 7.3.1). Rainfall causes major flooding perhaps making rainfall DDF curves more relevant to the flow-frequency problem.
- Estimate seasonal precipitation/rainfall DDF curves from the NOAA14 annual DDF curves using available gage records (see section 7.3.2). Application of mixed distribution analysis using seasonal precipitation/rainfall depth-duration-frequency curves to obtain design storms may improve watershed and gage/regression flow-frequency curve estimates.

The value of various parameter estimation schemes and the selection of design storms is best evaluated by comparing watershed model predictions and stream gage/regional regression estimates of flow-frequency curves (see section 7.4):

- The goal of the comparative study would be to evaluate to what extent model parameter estimates obtained based on either the recommendations in section 6 or from calibration studies should be adjusted to agree with stream gage/regression equation estimates.

Finally, the results of the calibration and comparative studies could be used to regionalize certain model parameters to ungaged watershed drainage areas which are not typical of the drainages used in developing the regional regressions (see section 7.5).

7.2. Watershed model calibration studies

7.2.1. Introduction

The watershed model calibration effort will depend on finding valuable precipitation runoff gage data (see section 7.2.2). If the existing data is valuable, then the data can be used to improve estimates of the effects of frozen soil on loss rates, the appropriate temporal patterns for design storms, and to calibrate model parameters based on the observed runoff (see section 7.2.2 – 7.2.4).

7.2.2. Evaluate precipitation/gage data

Tables 7.1 and 7.2 summarize the available precipitation and stream gage information that will be useful for watershed model calibration. A significant effort would be needed to assure that the precipitation data would be useful for the calibration effort.

Table 7.1: Precipitation gage data recorded at break point intervals

[2]Gage	[1]Period of Record
Echo Peak	1979- 2004
Fallen Leaf	1979 - 2004
Hagan's Meadow	1979 - 2004
Heavenly Valley	1979 - 2004
Marlette Lake	1979 - 2004
Tahoe City Cross	1980 - 2004
Ward Creek	1979 - 2004

[1]Water year

[2]Break point interval data reports precipitation as it occurs at short intervals (as small as a minute increment)

Table 7.2: Lake Tahoe Basin U.S. Geological Survey Stream Gages, flow collected at short time intervals (1hour or less)

USGS ID	Description	[1]latitude	[1]longitude	[2]area
103366092	Upper Truckee River at Highway 50 above Meyers, CA	38.8485186	-120.0271275	34.28
10336610	Upper Truckee River at South Lake Tahoe, CA	38.92240778	-119.9915706	54.9
10336645	General Creek near Meeks Bay, CA	39.05185194	-120.1185208	7.44
10336674	Ward Creek below Confluence near Tahoe City, CA	39.14074	-120.2121378	4.96
10336676	Ward Creek at Highway 89 near Tahoe Pines, CA	39.13212917	-120.1576914	9.7
10336698	Third Creek near Crystal Bay, NV	39.2404633	-119.9465775	6.02
103366993	Incline Creek above Tyrol Village near Incline Village, NV	39.25879694	-119.9232439	2.85
10336700	Incline Creek near Crystal Bay, NV	39.24018556	-119.9449106	6.69
10336730	Glenbrook Creek at Glenbrook, NV	39.08740806	-119.9399056	4.11
10336760	Edgewood Creek at Stateline, NV	38.96601917	-119.937125	5.61
10336780	Trout Creek near Tahoe Valley, CA	38.91990778	-119.9724036	36.7

[1]Decimal degrees
[2]Drainage area in square miles, [3]Drainage area not reported by USGS estimated using GIS software

7.2.3. Frozen Soil Investigation

Frozen soil conditions are possible in the study area (NRCS, 2004c, Dan Greenlee, Snow Survey Manager, Nevada) depending largely on when major snowfalls occur. Early snowfall will insulate the ground, preventing freezing conditions. Snow free ground later in the winter is susceptible to freezing.

Frozen ground has the potential to limit loss rates and cause greater annual maximum floods. The relative frequency of this condition could be estimated by using a degree-day approach. Gage information is available for comparing degree-days (the total number of days where the mean daily temperature is below freezing) or some other temperature index to snowfall records. Previous research that has developed relationships between degree-days and frozen soil conditions could then be applied to the study area to estimate the relative annual frequency of frozen soil conditions.

This information could be used to either weighting flow-frequency curves based on the relative likelihood of soils being frozen (modifying the recommendations in section 6) or be used to aid in the estimation of loss rates in watershed model calibrations studies. The weighting approach would be applied by first obtaining watershed model predictions of flow frequency curves with and without the frozen soil assumption. Then, the frequency curves from the frozen and unfrozen soil scenarios would be weighted based on the relative likelihood of frozen ground conditions to obtain an annual frequency curve. This annual frequency curve would, obviously be less conservative than the frozen ground assumption recommended in section 6. The resulting annual flow frequency curve would be used in comparative studies such as described in section 7.2.

Also, the relationship between antecedent conditions and frozen ground could also be used to determine, as a first estimate, the loss rates for a particular event being used in watershed model calibration studies.

7.2.4. Balanced storm investigation

The degree of relative balance in observed storms for gages in proximity of the study area was described in section 4 (see Table 4.2). Information available from hourly gages (mostly data available from the National Climatic Data Center and the Corps of Engineers) was used to determine if annual maximum short duration and long duration depths were contained within a single storm. However, this investigation did not analyze the gage information shown in Table 7.1, partly because the data was not available at the time and partly because of the need to assure data quality. Investigation of this data would be worthwhile both because it involves gages within the watershed and provides depth data for intervals less than an hour. The results of the analysis would give a better indication of the degree of balance of study are storms.

7.2.5. Watershed model calibration

Model calibration is essential for building confidence in the methods for formulating watershed models and obtaining parameter estimates. The calibration effort will allow evaluation of techniques for estimating the contribution of snowmelt, the impact of frozen ground conditions and the value of the NRCS methods recommended in section 6 for computing runoff volumes and hydrographs. Although the NRCS Curve Number (CN) and Unit Hydrograph (UH) are very commonly applied across the country and used extensively by the study area counties, these approaches have not been calibrated for hydrologic conditions existing in the Lake Tahoe Basin.

Watershed model calibration could only be performed for the relatively large drainage areas where gage information exists. Still, the value of CN values determined from the soil hydrologic group of the watershed could be judged from the calibration. Furthermore, the calibration would be useful to see if standard relationship, $t_{lag}=0.6t_c$, where t_{lag} is the lag, and, t_c is the time of concentration, holds for the available data. There is also the possibility that other loss-rate and UH techniques could be evaluated. These results could then be extrapolated to smaller drainage areas.

7.3. Adjusting NOAA14 depth-duration-frequency curves for design storm development

7.3.1. Estimating rainfall depth-duration-frequency curves

The NOAA14 depth-duration-frequency (DDF) curves are for precipitation: both snowfall and rain. However, drainage design is intended to control rainfall induced storm runoff. Application of design storms developed from precipitation-depth-duration frequency curves might be overly conservative, particularly in the Lake Tahoe Basin, because of the effect of snow on the frequency curve estimates. An investigation of the effect of snowfall on these frequency curves could quantify the degree of conservatism and possible corrections that would provide rainfall rather than precipitation frequency curve estimates.

7.4.Estimating seasonal precipitation/rainfall frequency curves

Development of seasonal precipitation/rainfall depth-duration frequency (DDF) curves would be useful if mixed distribution flow frequency analysis is of interest. Precipitation or rainfall frequency curves would be developed depending on whether or not the study in section 7.3.1 is completed.

Runoff in the study area is due to a mixture of winter and spring events. The mixed nature poses a problem for setting watershed model parameters that are intended to capture some typical or average conditions for the simulation of design storms. The problem is that for different seasons and elevations the typical condition might involve, for example, snow cover-frozen ground versus no snow-cover and unfrozen ground. Setting an “average” loss rate in a watershed model simulation of a design storm does not capture very well the conditions that affect flow frequencies.

A potentially better approach would be to simulate seasonal design storms with a watershed model to obtain seasonal flow frequency curves. These curves are then combined a mixed population approach to obtain an annual flow frequency curve. Seasonal precipitation/rainfall

DDF would be needed to develop seasonal design storms. The existing gage precipitation record could be used to develop factors for computing seasonal curves from the NOAA14 annual frequency curves

The success of this effort would primarily depend on using the gage data period of record shown in Table 7.1 to develop the relationship between annual and seasonal depth-duration frequency curves. In turn this would depend on the quality of the period of record data, which needs to be investigated.

7.5. Comparative studies of watershed model predicted and stream gage/regional regression equation frequency curve estimates

A comparative study of watershed model predicted and stream gage/regression equation predicted flow frequency curves would be performed to either validate or improve the parameter estimates used for ungaged watersheds. As discussed earlier in section 7.1, two principles would be considered in adjusting parameters in these comparative studies:

1) The watershed model predictions should agree on the average with the stream gage/regression estimated flow frequency curves over a reasonable number of comparisons;
2) The model parameters should be constrained to some physically reasonable values.

Ultimately, guidelines would be developed that specified to what extent model parameters would be adjusted to bring into agreement model predicted and regression estimates of flow-frequency curve for ungaged watersheds.

A number of different scenarios are possible depending on the information available for estimating watershed model parameters and developing design storms. Scenarios can be developed from the following options:

- parameter estimates and design storms from the recommendations in section 6;
- parameter estimates from calibrations studies using observed precipitation-runoff data
- design storms based on rainfall DDF curves
- design storms based on seasonal precipitation or rainfall DDF curves

Evaluating the model parameter estimation procedure for the recommendations given in section 6 certainly would be simplest. However, parameter estimates from the calibration schemes and alternative estimates of design storms may result in improved predictive capability and smaller differences between the alternative estimates of flow-frequency curves.

7.6. Regionalizing watershed model parameter estimates

Watershed model parameters such as loss rates and average snowmelt rates could be regionalized for use in ungaged drainage areas not covered by the frequency analysis studies. Possibly the most promising application would be to relate SCS CN values to soil hydrologic group or perhaps a surrogate measure of hydrologic group within a watershed (e.g., elevation and longitude). The CN values obtained from calibration would be for some composite soil group because of the relatively large drainage areas involved in the calibration. The variation in CN with the composite soil groups could still be used as an estimate for smaller areas. This will be approximate because the CN values are sure to be affected by scale, i.e., by the rainfall-runoff dynamics.

8. Developing design criteria for best management practice

8.1. Introduction

The purpose of this section is to propose approaches for developing hydrologic design criteria useful for the design of best management practices (BMPs) in the Lake Tahoe Basin. BMPs are implemented to meet receiving water quality objectives by controlling pollution from non-point sources of runoff. These practices generally fall into the category of non-structural or structural. Non-structural measures ("preventive maintenance" measures such as street sweeping measures) are the province of public outreach, administrative management, etc. efforts; but are not an issue for hydrologic design criteria and are not addressed here. **The focus here is on hydrologic criteria for structural measures typically implemented in urban areas, such as**:

- grass buffers and swales;
- sand filter strips;
- retention/detention basins

The important hydrologic design parameter for these measures is the volume of water that needs to be controlled over a specific time period to reduce pollutant loading. The non-point source load permissible would ideally be considered together with other point source loads to meet a total maximum daily load constraint (TMDL) for various pollutants.. The hope is that the water quality objectives will be attained in meeting the TMDL constraint.

The desired reduction in pollutant loading due to a BMP is related to the TMDL by the following relationship (see EPA, 1999, pg. 1-1):

$$\text{TMDL} = \sum \text{WLA} + \sum \text{LA} + \text{MOS} \tag{0.2}$$

where WLA is the waste load allocation or the portion of the TMDL allocated to existing or future point sources, LA is the load allocation or portion of the TMDL allocated to existing or future non-point source and natural background, and MOS is a margin of safety. Interestingly, EPA (1999) indicates that the MOS can be estimated as either a deterministic safety factor or a measure of risk computed based on the analytical approach used in estimating the loads.

The challenge in developing the hydrologic design criteria is to provide methods for estimating design water volumes which result in reasonable estimate of load allocation, LA. This is a formidable challenge as noted by EPA (pg. 7-3):

> Although LAs may be used to target BMP implementation within a watershed, translation of LAs into specific BMP implementation programs can be a problem. One reason for this difficulty is that often many agencies are involved in BMP implementation, rather than a single oversight agency, as for NPDES permits. In addition to numerous landowner-operators, BMP implementation can typically include federal, state and local involvement. Often, the objectives of the varying agencies are different, which make coordination difficult.

Moreover, it is not always easy to predict the effectiveness of BMPs. TMDL strategies heavily dependent on loading reductions through LAs should include long-term watershed quality monitoring programs to evaluate BMP effectiveness.

Establishing hydrologic design criteria will at least ensure a uniform practice for the Lake Tahoe Basin. The effectiveness of the criteria perhaps cannot ensure meeting the water quality objectives; at best, the criteria can implement the best approaches currently available. A significant monitoring effort is already under way for the lake; however, it is beyond the scope of this work to suggest improvements to the monitoring effort so that effectiveness of BMPs can be assessed.

Section 8.2 will describe the current hydrologic criteria used in the study area to establish BMPs. Suggested approaches to develop new criteria are described in section 8.3.

8.2. Current practice

8.2.1. Tahoe Regional Planning Authority

The Tahoe Regional Planning Authority's (TRPA's) Handbook of Best Management Practices (1988) is mostly concerned with the control of non-point source pollution from storm water runoff and associated erosion. In terms of hydrologic criteria, the water quality volume (WQV) specified for the practice is that the 20-year, 1-hour "design storm" be used for storm water control facilities (see LRWQCB, 1994, section 5-6). Pursuant to subsection 25.5.A of the TRPA Code of Ordinances, all property owners in the Tahoe Basin are required to install infiltration facilities designed to accommodate the volume of runoff from a six-hour storm with a two-year recurrence probability (or a twenty year/one hour storm, which is approximately one inch of precipitation in an hour). NDOT (Nevada Department of Transportation), Caltrans (the California Department of Transportation) and counties currently recognize this WQV as the requirement for design in the Lake Tahoe basin. The genesis of this design storm requirement is not described in TRPA documentation.

Apparently, the WQV and associated BMPs have been developed without any regard to the estimation of TMDLs needed to constrain point and non-point sources to meet water quality objectives (TRPA, 2001, pg. 3-19).

A major goal of the Lake Clarity Model (scheduled to be completed by February 2002) is to establish the nutrient and sediment-loading budget for the lake and predict the load reductions and major source allocations required to improve Lake Tahoe's clarity. Once the Lake Clarity Model is operational, and needed load reductions can be allocated, the management standards for individual water quality thresholds may need to be revised accordingly. Such load reductions also need to be applied to Environmental Improvement Program (EIP) projects in order to determine their efficiency in load reductions. A Total Maximum Daily Load (TMDL) or annual load approach to Lake Tahoe and the tributary watersheds may help to provide incentives to maximize treatment wherever possible based on load reduction potential of project treatments on a watershed contribution basis. However, it is likely such maximization of project load reductions would need to *occur Basin-wide regardless of particular watersheds' contributions to lake loading (as opportunities present themselves). ......*

Consequently, the recommended strategy, described in the previous section, of defining water quality objectives, setting TMDLs, and developing BMPs to address the TMDL requirements is not complete for Lake Tahoe or other water bodies with the Lake Tahoe Basin. The BMPs described in TRPA (1988) seem to have been developed without any target TMDLs in mind. Future modeling studies are being devised by the Lahontan Regional Water Quality Control Board (see section 8.3) to quantify these TMDLs.

8.2.2. Caltrans

The California Department of Transportation, Caltrans, has a significant responsibility for controlling non-point source pollution. Their methodology used throughout California provides a good basis for recommending new design criteria for BMPs in the Lake Tahoe Basin. The Caltrans method is based on controlling a design water quality volume (WQV) (see, Caltrans 2003, pg 2-15):

> Both storm volume and peak flow conditions must be considered in the evaluation of runoff conditions. The "Design Storm" is the particular event that generates runoff rates or volumes that the drainage facilities are designed to handle. Unlike flood control measures that are typically designed to store or convey the peak volumes or flows of infrequent storm events, treatment BMPs are designed to treat the lower volume or flow of more frequent storm events. The volume or flows associated with the frequent events are commonly referred to as the WQV for BMPs designed based on volume, and Water Quality Flow (WQF) for BMPs designed based on flow. Treatment BMPs are sized to accommodate the WQF or WQV from the contributing drainage area. Flows in excess of these values are diverted around or through the treatment BMP. Methods for determining the WQV are generally tied to an analysis of rainfall depths generated over 24-hour periods.
>
> The WQV of Treatment BMPs is based on using any one of the following methods:
>
> • Where they are established, sizing criteria from the RWQCB or local agency (which ever is more stringent) will be used; and• Where the RWQCB or local agency does not have an established sizing criterion,Caltrans will use one of the following methods:
>
> - Option 1: The maximized detention volume determined by the 85th percentile runoff capture ratio. This method is described in Chapter 5 of the *Urban Runoff Management WEF Manual of Practice No. 23,* 1998, published jointly by the Water Environment Federation (WEF) and the American Society of Civil Engineers (ASCE). Designers should note, however, that the information presented in the WEF manual cannot be directly applied to Caltrans facilities because it is based on large watersheds and oversimplified hydrologic data for California. This method requires the designer to assume a drawdown time. Any drawdown time between 24 and 72 hours can be used (the 24-hour limit provides adequate settling and the 72-hour maximum addresses vector

concerns). A design tool (Basin Sizer) that uses data from more than 700 California rainfall stations, has been created for Caltrans use. It is available at http://stormwater.water-programs.com. A detailed description of the method can also be found in: Guo, C.Y., and B.R. Urbonas (1996), "Maximized Detention Volume Determined by Runoff Capture Ratio," *Journal of Water Resources Planning and Management,* v. 122, n. 1, pp. 33-39.

- Option 2: The volume of annual runoff based on unit basin storage WQV to achieve 80 percent or more volume of treatment based on the sizing methods provided in the *California Storm Water Municipal Best Management Practice Handbooks,* published by the California Storm Water Quality Task Force, March 1993. This method requires the assumption of a 40-hour drawdown time. A design tool has been created for Caltrans use. It is available at http://stormwater.water-programs.com.

Alternatively, a WQV may be established by Caltrans subject to the review and approval of the RWQCB if one of the following situations applies:

• The site area is limited and cannot accommodate a Treatment BMP sized according to the methods described in Options 1 or 2; or
• Sizing a Treatment BMP using Options 1 or 2 in areas of the State with significant annual precipitation results in excessively large treatment units.

The estimation of the design WQV relies heavily on the work done to establish the BMP design requirements for the Denver Urban Storm Drainage Criteria Manual (see USDCM, 2003, and Guo and Urbonas, 1996). This methodology focus on the relationship between rainfall rates, percent impervious and pollutant loads. The BMP is designed to remove a reasonable percentage of the pollutant load. However, the design is not related to any TMDL constraint as given in equation (0.2).

8.2.3. Review of current practice

The current practice employed by TRPA and Caltrans reflects the same "event based" practical approach employed for drainage design. The application of a WQV to size BMP facilities is much the same as using a design storm to determine conveyance requirement to prevent flooding.

Although practical, the basis for the TRPA WQV is not supported by a published analysis, unlike the Caltrans WQV criteria. As discussed in the next section, current efforts by the Lahontan Regional Water Quality Control Board are aimed at improving the method for developing hydrologic criteria for BMP design.

8.3. Future research

8.3.1. Introduction

The possibility exists that a suite of hydrologic analysis methods will be used to design BMPs to meet the TMDL constraints described by equation (0.2). The variation of these methods will either be based on the current event approach to obtain a WQV or applications of continuous simulation modeling. Section 8.3.2 describes the methodology used in the Denver Urban Storm Drainage Criteria Manual (see USDCM, 2003) to establish basin and hydro-meteorologic parameters for estimating the WQV. This approach is considered both because Caltrans has based much of its guidelines on this approach; but also, because this manual has been developed by recognized experts in dealing with urban hydrology problems.

Establishing some simple relationships between WQV, hydro-meteorologic parameters and basin characteristics (such as percent impervious) by using the results of more detailed modeling studies with continuous simulation techniques is a practical approach to the design problem. Continuous simulation techniques **may** provide better estimates of both pollutant loading and the efficiency of various BMPs to reduce loading than other simpler techniques. However, application of this approach is impractical given the data availability and budget constraints typical of the standard design problem. Section 8.3.3. describes how studies with continuous simulation models have been applied in the past and can be used in the Lake Tahoe Basin to estimate WQVs useful for the BMP design problem.

An aspect of the TMDL constraint described in equation (0.2) is the MOS (margin of safety) that might be included in the design to allow for a margin of error in the analytic methods used to estimate pollutant loading. Section 8.3.4 describes the issues that would need to be addressed in establishing MOS for the Lake Tahoe Basin.

8.3.2. Relating hydrologic parameters to design WQV, the USDCM approach

The Denver Urban Drainage Manual (USDCM, 2003) relies on the study done by Guo and Urbonas (1996), among others, to select hydro-meteorologic parameters for estimating a WQV for designing a structural BMP. The focus here was on detention facilities, but the principles for estimating the WQV are the same as for other structural BMP designs.

The approach recognizes that there is a diminishing return in reducing pollutant loading when attempting to contain larger but more infrequent storms. Studies have been done showing that significant portions of the sediment load can be removed by designing “dry retention facilities” to capture the average annual runoff event. Even higher sediment removal rates (approximately 80%) can be obtained by capturing a WQV up to the 80-90 percentile event volume (i.e., capturing all magnitude events that are not exceeded in 90% of the observed events). Capturing larger, more infrequent events require longer facility emptying times, which would not be efficient in removing pollutants from the majority of events that contribute most significantly to annual pollutant loading.

These researchers proposed a simplified model that used runoff coefficients to convert period of rainfall records to a continuous set of runoff events. The value of the simplified model was established in comparison to more detailed modeling studies using continuous simulation watershed models. Based on these studies they found:

- the optimal precipitation event separation time is equal to half the detention facility brim full emptying time;
- that an optimal event or volume capture ratio for the facility could be defined based on the precipitation record; and finally,

- that the maximum detention volume could be defined as a function of the mean annual precipitation and the watershed runoff coefficient (here the runoff coefficient is the ratio of runoff to rainfall event volumes).

The general applicability of this approach depended on the following:

1) Calibrating the method based on more sophisticated modeling approaches;
2) Other studies relating pollutant removal to facility emptying time;
3) Rainfall induced runoff from frequent events provides the major contribution to the pollutant load.

The original modeling studies used to verify the simplified approach were performed for urban watersheds in Sacramento, California. Other investigations were later performed, including an actual case study application, to further validate this simplified modeling approach.

The hydro-meteorologic characteristics and pollutant removal characteristics assumed in the Guo and Urbonas study, for example, may not be relevant to addressing the water quality problems in the Lake Tahoe Basin. For example, fine material together with nutrient loading (nitrogen and phosphorous) are now thought to be critical to the Lake Tahoe clarity. The studies reported in Guo and Urbonas (1996) on observed sediment removal might not ensure that fine material is removed as well. Simulations studies probably need to be done that consider the removal characteristics of structural BMPs useful for controlling the pollutants important to Lake Tahoe..

In terms of hydro-meteorologic characteristics, the Guo and Urbonas study only focused on rainfall-runoff issues. However, snowmelt plays an important role in the annual runoff from urban areas in Lake Tahoe. The importance of potentially longer duration runoff due to snowmelt contribution could reduce the removal efficiency of a structural BMP in comparison to the removal efficiency found for an analysis that only considers runoff due to rainfall alone.

In conclusion, the USDCM methodology, based on research by Guo, Urbonas and others, provides a reasonable template for proposing simplified approaches to determining a WQV. As a recommendation, Caltrans has based its method for estimating a WQV on this approach. However, more detailed modeling studies such as those described in the next section, need to be performed to ascertain if this type of simplified approach is useful.

8.3.3. Future model studies

The Lahontan Regional Water Quality Control Board (LRWQCB) is currently involved in a significant modeling study to determine the TMDL constraints important for meeting Lake Tahoe water quality objectives. As part of this study, continuous simulation runoff and pollutant loading models are being developed for relatively large sub-basins. The experienced gained in developing these models will be important in applications to developing criteria for designing BMPs for controlling urban runoff. However, the following challenges need to be addressed in developing these models:

1) Gage precipitation records are limited for the basin. Daily precipitation is available, but shorter interval information is limited (see Table 7.1). The LRWQCB is using a 40-year

period of record estimated from simulations of a physically-based atmospheric model (MM5). The simulated precipitation could prove useful for smaller scale urban studies, but needs to be validated to the extent possible in comparison to the available gage record.

2) Gage measures of runoff from urban basins is limited. Calibration of the simulations models to this data is likely to be useful.
3) The model precipitation-runoff algorithms will need to simulate snowmelt runoff. Very little information exists on how to model snowmelt in urban areas. Human activity, (e.g., plowing) has a significant impact on the thermal properties of snow. Furthermore, the urban impervious areas have impacts on the energy budget which are different than natural-open areas where most snowmelt studies have been performed. Consequently, parameter typically used in either energy-budget or degree-day snow-melt models are not likely to be appropriate for urban snowmelt runoff simulations. Field studies of urban-snowmelt would provide a basis for developing urban snowmelt runoff-models.

The LRWQCB might also consider using the regional flow regression equations (see SPK, 2005) to calibrate/validate continuous simulation model applications in larger natural ungaged watersheds. The regression equations can be used to compute maximum and minimum daily volume duration an flow duration frequency curves. These same frequency curves can be estimated from the simulated daily record and compared to the regression estimates. Greater confidence can be placed in model applications to smaller areas given some reasonable correspondence between model and regression predicted curves in ungaged areas.

Irrespective of the challenges, the application of continuous simulation models to help "calibrate" a simplified WQV approach is a useful way to develop hydrologic criteria for designing structural BMPs. This approach has proven to be acceptable both in city of Denver (USDCM, 2003) and also by Caltrans (2003). However, as noted in EPA (1999), the success in controlling non-point source pollution needs to be verified by monitoring.

8.3.4. Implications of TMDL MOS on BMP design

The requirement for a margin of safety (MOS) in setting TMDL constraints (see equation (0.2)) has some significant implications for the capacity and corresponding cost of structural BMP designs. Obviously, as the MOS increases so does the cost of design.

EPA guidance on how to estimate MOS is not specific. Reckhow (2003, pg. 245) notes the following difficulty in the current practice for estimating the MOS:

> … Instead, TMDLs are typically proposed with either conservative modeling assumptions or an arbitrarily chosen MOS. Neither approach explicitly links the MOS to TMDL forecast uncertainty. However, by hedging the TMDL decision in the direction of environmental protection, the MOS effectively increases the assurance that water quality standards will be achieved. This may seem reasonable even desirable, but it is noted that this hedging comes at a cost, and the hedging cost is totally arbitrary in most cases.

Note in referring to TMDL forecast uncertainty the author is addressing the problem of model prediction uncertainty.

Some type of benefit-cost analysis needs to be performed in any modeling studies done to establish TMDL constraints important to developing BMP design criteria. Although the value of improving receiving water quality may not be easily estimated by a dollar metric, the cost of design certainly can be measured in this way. Consequently, any modeling study should include the incremental cost of design as a function of incremental increases in MOS. Simple analyses which show the incremental reduction in some parameter, such as the reduction in sediment contribution, versus increase cost as a function of increase in the MOS may help decisions regarding design criteria.

8.4. Summary

Establishing guidelines for estimating a WQV is a reasonable approach to providing hydrologic design criteria for BMP design. This approach is used both by Caltrans (2003) and in the Denver Drainage Manual (USDCM, 2003). However, a significant modeling effort will be required to develop the information needed to estimate the WQVs as a function of easily obtained parameters, such as mean annual precipitation and runoff coefficients for the Lake Tahoe Basin.

The LRWQCB is currently involved in a major modeling study to estimate TMDLs for Lake Tahoe Basin. The models developed could serve as a basis for developing simple method for estimating WQVs. However, a number of challenges exist in using these relatively large-natural area models for deriving WQVs for smaller urban areas that are focus of the BMP design.

9. Appendix: Cold Regions Research and Engineering Laboratory investigation of Snow Water Equivalent

Estimation of SWE for the Lake Tahoe Basin that Best Represents the Expected Snow Conditions when a One-day Annual Maximum Discharge Occurs

By Steven F. Daly, Tim Baldwin, Carrie Vuyovich
ERDC/CRREL, Hanover, NH 03755

Background

The distribution of Snow Water Equivalent (SWE) in the Lake Tahoe Basin that best represents the expected snow conditions when a one-day annual maximum discharge occurs is of interest to hydrologic modelers. This SWE distribution does not reflect the snow conditions on any specific date or time of year. Rather, it is a synthetic snow pack that best represents the snow conditions expected when a one-day annual maximum discharge occurs in a stream in the Lake Tahoe Basin. Conceptually, a one-day annual maximum discharge does not have a distinct time during the year when it can occur and in fact, one-day annual maximum discharges do occur throughout the year in the Lake Tahoe Basin. This lack of "seasonality" conflicts with our fundamental observations of mountainous snowpacks, which have definite accumulation and ablation periods that produce a SWE maximum in the late spring and a minimum in the late summer or early fall. In order to remove any seasonality from the representation of the SWE distribution, we estimated the mean or average snow conditions that existed at each of the snow gages in and around the Lake Tahoe Basin over all the dates that a one-day annual maximum discharge was recorded at a streamflow gage in the basin, regardless of the time of year. For each snow gage, the SWE found in this way is very nearly the same as the average SWE found on 15 May, the most likely day for a one-day annual maximum discharge to occur. The representative SWE values found by averaging over the one-day annual maximum discharge dates at each snow gage were then interpolated over the Lake Tahoe Basin. The interpolation relied on the observed relationship between the SWE values and the snow gage elevation. Separate relationships were used for the east and west side of the basin. The final result is a map displaying the distribution of SWE that best represents the expected snow conditions when a one-day annual maximum discharge occurs within the Lake Tahoe Basin.

Analysis

The 23 USGS streamflow gages that were used in this study are listed in Table 1 and their locations are shown in Figure 1. A series of one-day annual maximum discharges was available for each gage over its period of record. The period of record was different for each gage but in general, the periods were bounded by the years 1961 and 2003. There were a total of 481 one-day annual maximum discharge values for all the gages. The snow gages that were used in this study are listed in Table 2 and their locations are shown in Figure 2.
The first step in this analysis was to determine the value of SWE at each snow gage that best represents the average SWE on the dates that one-day annual maximum discharges were recorded at the USGS gages. This was done in two parts. First, the average SWE at a given snow

gage, i , based on the dates of the one-day annual maximum discharges that were recorded at a single USGS streamflow gage, j, $\overline{SWE^i_{USGS_j}}$, was found as

$$\overline{SWE^i_{USGS_j}} = \frac{1}{n_j}\sum_{k=1}^{n_j} SWE^i_{USGS_j Dates_k} \tag{3}$$

where $SWE^i_{USGS_j Dates_k}$ is the daily average SWE at snow gage i, recorded on the kth date of the series of one-day annual maximum discharges recorded at USGS gage j; and n_j = the number of years in the period of record of streamflow gage j. The average SWE at a given snow gage, i, $\overline{SWE^i}$, based on the dates of the annual maximum discharges recorded at all 23 of the stream flow gages listed in Table 1 was then found as

$$\overline{SWE^i} = \frac{\sum_{j=1}^{23} n_j \overline{SWE^i_{USGS_j}}}{\sum_{j=1}^{23} n_j} \tag{4}$$

The results for all the snow gages are provided in Table 3.
A histogram of the days during the year when the one-day annual maximum discharges occurred is shown in Figure 3. By examining Figure 3, it can be seen that one-day annual maximum events can occur any time of the year, although many occur in late spring. The most likely day of the year for a one-day annual maximum discharge to occur is 15 May, based on a simple averaging of the occurrence dates. An alternative approach to determining the representative SWE at each snow gage was to average the May 15 SWE over the period of record of each gage.

$$\overline{SWE^i_{15May}} = \frac{1}{n_i}\sum_{j=1}^{n_i} SWE^i_{15Mayj} \tag{5}$$

where n_i = the number of years in the period of record of snow gage i; and SWE^i_{15Mayj} = the SWE recorded at snow gage i on 15 May of the jth year of its period of record. The results are provided in Table 3, and a comparison of both methods of estimating the average SWE is shown in Figure 4. It can be seen that both approaches yield very similar results. As a result, it was decided to base the analysis on the representative SWE values found by averaging over the one-day annual maximum discharge dates.
Once the representative SWE was determined at each of the snow gages, the process of interpolating these values over the Lake Tahoe Basin could begin. This was done using GIS techniques based on a digital elevation model (DEM) of the Lake Tahoe Basin. The DEM was based on the USGS National Elevation Dataset (1 arc second). The basin was divided into grid cells 100 feet on a side.
A linear relationship between the SWE values found at each gage and the elevation of the gage was determined of the following form.

$$SWE_E = L_{apse}E + B \tag{6}$$

where SWE_E = the SWE at elevation E; L_{apse} = the estimated rate of SWE change with elevation; and B = a fitting constant. The gages fell into two distinct groups corresponding to the east and west sides of the Lake Tahoe Basin (See Figure 5), with the north-south dividing line passing through the approximate center of Lake Tahoe. The relationship for the east side was found to be 0.01437 inches of SWE per foot of elevation and 0.01724 inches of SWE per foot of elevation for the west side.

Next, the SWE measured at the snow gages were adjusted to zero elevation as

$$SWE_{i\ elev=0} = B + \left(SWE_i - \left(L_{apse}E_i + B\right)\right) = SWE_i - L_{apse}E_i \tag{7}$$

where SWE_i = the measured SWE at the ith station at an elevation of E_i; and $SWE_{ielev=0}$ = the estimated SWE at an elevation of zero. Inverse Distance Weighting was then used to estimate the SWE in each grid cell:

$$\overline{SWE}_{m\ elev=0} = \frac{\sum_{i=1}^{n} \frac{1}{d_{mi}^2}\left(SWE_{i\ elev=0}\right)}{\sum_{i=1}^{n} \frac{1}{d_{mi}^2}} \tag{8}$$

where $\overline{SWE}_{m\ elev\ =\ 0}$ =the estimated SWE of the mth grid cell at elevation zero; d_{mi} = the distance between the mth grid cell and the ith measurement station; and n = the number of gages used.

Finally the estimated SWE of the mth grid cell at its elevation, E_m, was estimated as

$$\overline{SWE}_m = \overline{SWE}_{m\ elev=0} - B + \left(L_{apse}E_m + B\right) = \overline{SWE}_{m\ ele=0} + L_{apse}E_m \tag{9}$$

where $\overline{SWE}_m$ = the estimated SWE of the mth grid cell. Note that the fitting constant B is not required in the calculations. Grid cells with a final SWE estimate of less than zero were set to zero. This resulted in a clear demarcation of the snow-covered area (SCA).

The above process was done twice, once for the east side of Lake Tahoe and once for the west. To insure that there was a smooth transition between the west and east sides, a 10,000-foot wide transition zone was created. The SWE values inside this transition zone were based on taking a weighted average of the east side and west side SWE values; with the weighting values varying linearly between the two sides.

The final map of the expected SWE distribution corresponding to any one-day annual maximum discharge in the Lake Tahoe Basin is shown in Figure 6.

Gage	GAGE NAME	Latitude	Longitude	DA
10336660	BLACKWOOD C NR TAHOE CITY CA	39.1075	-120.1611	11.2
103367592	EAGLE ROCK CK NR STATELINE, NV	38.95657444	-119.9276806	0.63
103367585	EDGEWOOD CK AT PALISADE DRIVE NR KINGSBURY, NV	38.96657444	-119.9160136	3.13
10336645	GENERAL C NR MEEKS BAY CA	39.05185194	-120.1185208	7.44
10336730	GLENBROOK CK AT GLENBROOK, NV	39.08740806	-119.9399056	4.11
10336700	INCLINE CREEK NEAR CRYSTAL BAY, NEV.	39.24027778	-119.9438889	6.69
103366995	INCLINE CK AT HWY 28 AT INCLINE VILLAGE, NV	39.2454633	-119.9390772	4.54
103366993	INCLINE CK ABV TYROL VILLAGE NR INCLINE VILLAGE NV	39.25879694	-119.9232439	2.85
10336740	LOGAN HOUSE CK NR GLENBROOK, NV	39.0665747	-119.9354606	2.09
10336715	MARLETTE C NR CARSON CITY, NV	39.17213056	-119.907963	2.86
10336626	TAYLOR C NR CAMP RICHARDSON CA	38.92157444	-120.0612953	16.7
10336698	THIRD	39.24055556	-119.9455556	6.02
10336770	TROUT CK AT USFS RD 12N01 NR MEYERS CA	38.86324056	-119.9582367	7.4
10336775	TROUT CK AT PIONEER TRAIL NR SOUTH LAKE TAHOE CA	38.90339444	-119.9688917	23.7
10336780	TROUT C NR TAHOE VALLEY CA	38.9200	-119.9714	36.7
10337500	TRUCKEE R A TAHOE CITY CA	39.1662958	-120.1443586	507
103366092	UPPER TRUCKEE R AT HWY 50 ABOVE MEYERS CA	38.8485186	-120.0271275	34.28
10336600	UP TRUCKEE R NR MEYERS CA	38.84296306	-120.0246275	33.1
10336580	UPPER TRUCKEE R AT S UPPER TRUCKEE RD NR MEYERS CA	38.7962961	-120.0190719	14.09
10336610	UP TRUCKEE R A SOUTH LAKE TAHOE CA	38.92240778	-119.9915706	54.9
10336676	WARD C AT HWY 89 NR TAHOE PINES CA	39.13212917	-120.1576914	9.7
10336675	WARD C A STANFORD ROCK TRAIL XING NR TAHOE CITY CA	39.13685139	-120.1810256	8.97
10336674	WARD C BL CONFLUENCE NR TAHOE CITY CA	39.14074	-120.2121378	4.96

Table 1. USGS Stream gages used in the analysis

Gage	Gage Name	Elevation	Latitude	Longitude
20L06	ECHO PEAK	7800'	38º 51'	-120º 04'
20L10	FALLEN LEAF	6300'	38º 56'	-120º 03'
19L03	HAGANS MEADOW	8000'	38º 51'	-119º 56'
19L24	HEAVENLY VALLEY	8800'	38º 56'	-119º 54'
19K04S	MARLETTE LAKE	8000'	39º 09'	-119º 54'
19K02S	MT. ROSE	9000'	39º 21'	-119º 53'
19K07S	MT. ROSE SKI AREA	8850'	39º 19'	-119º 53'
20L02	RUBICON #2	7500'	39º 00'	-120º 08'
20K30	SQUAW VALLEY GOLD COAST	8200'	39º 11'	-120º 15'
20K27	TAHOE CITY CROSS	6750'	39º 10'	-120º 09'
20K13	TRUCKEE #2	6400'	39º 18'	-120º 12'
20K17	WARD CREEK #2	7000'	39º 08'	-120º 13'

Table 2. Snow Gages Used in the Analysis

Gage	Gage Name	SWE Peak Flow Dates		SWE 15 May	
		Average	Standard Deviation	Average	Standard Deviation
20L06S	ECHO PEAK	21.68	19.56	20.41	25.43
20L10S	FALLEN LEAF	1.03	2.21	0.00	0.00
19L03S	HAGAN'S MEADOW	5.61	8.91	4.85	10.33
19L24S	HEAVENLY VALLEY	14.32	12.63	15.30	17.57
19K04S	MARLETTE LAKE	11.60	11.77	11.42	14.80
19K02S	MT ROSE	19.13	14.27	22.59	20.33
19K07S	MT ROSE SKI AREA	33.77	20.20	33.38	28.97
20L02S	RUBICON #2	17.63	14.46	17.77	19.75
20K30S	SQUAW VALLEY	40.61	27.74	38.03	37.24
20K27S	TAHOE CITY CROSS	3.51	5.94	2.02	6.12
20K13S	TRUCKEE #2	4.67	7.41	3.13	7.32
20K25S	WARD CREEK #3	16.20	15.42	15.68	19.47

Table 3. Results of SWE analysis (in inches)

Figure 1. Stream gages

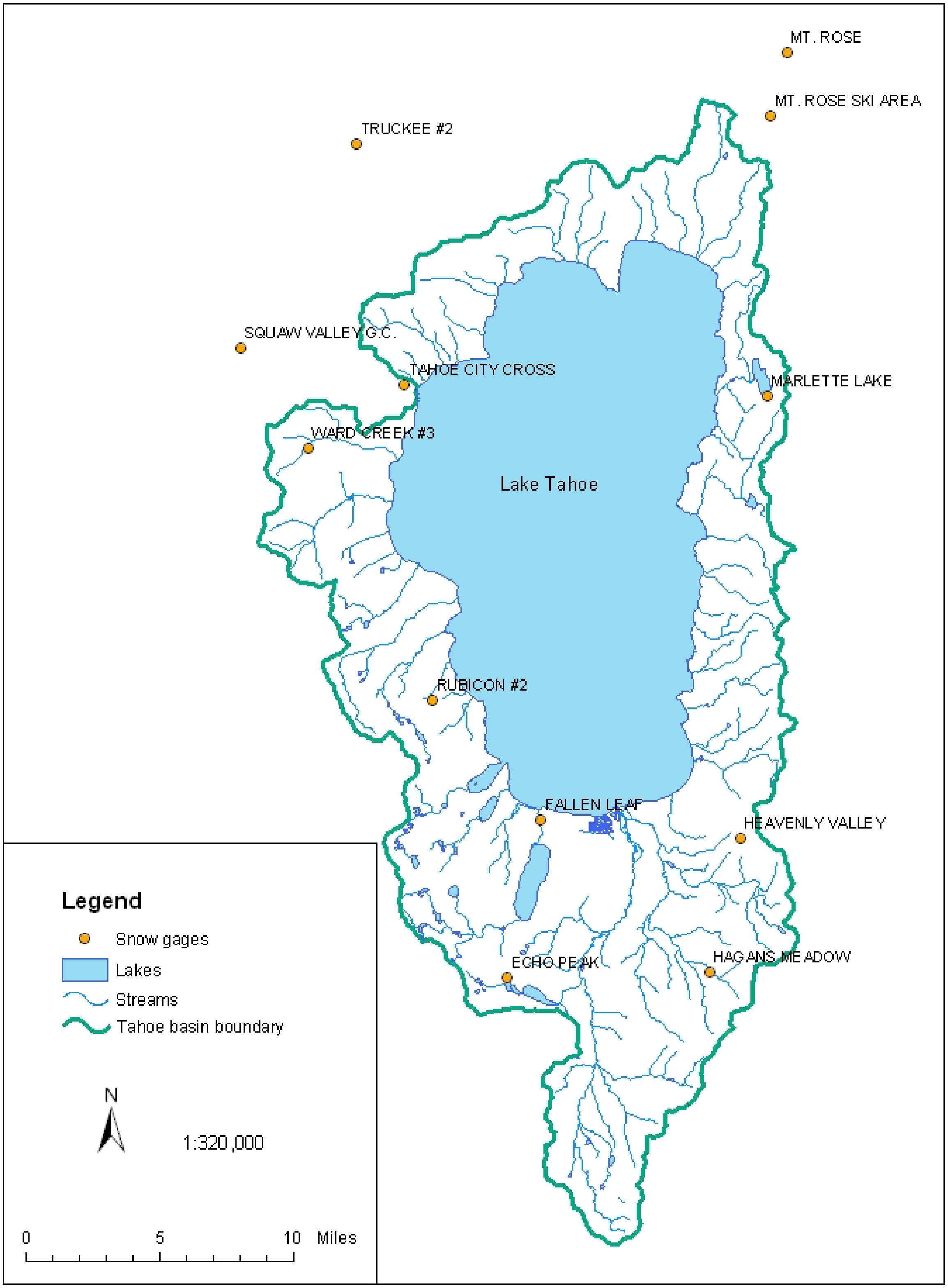

Figure 2. Snow gages

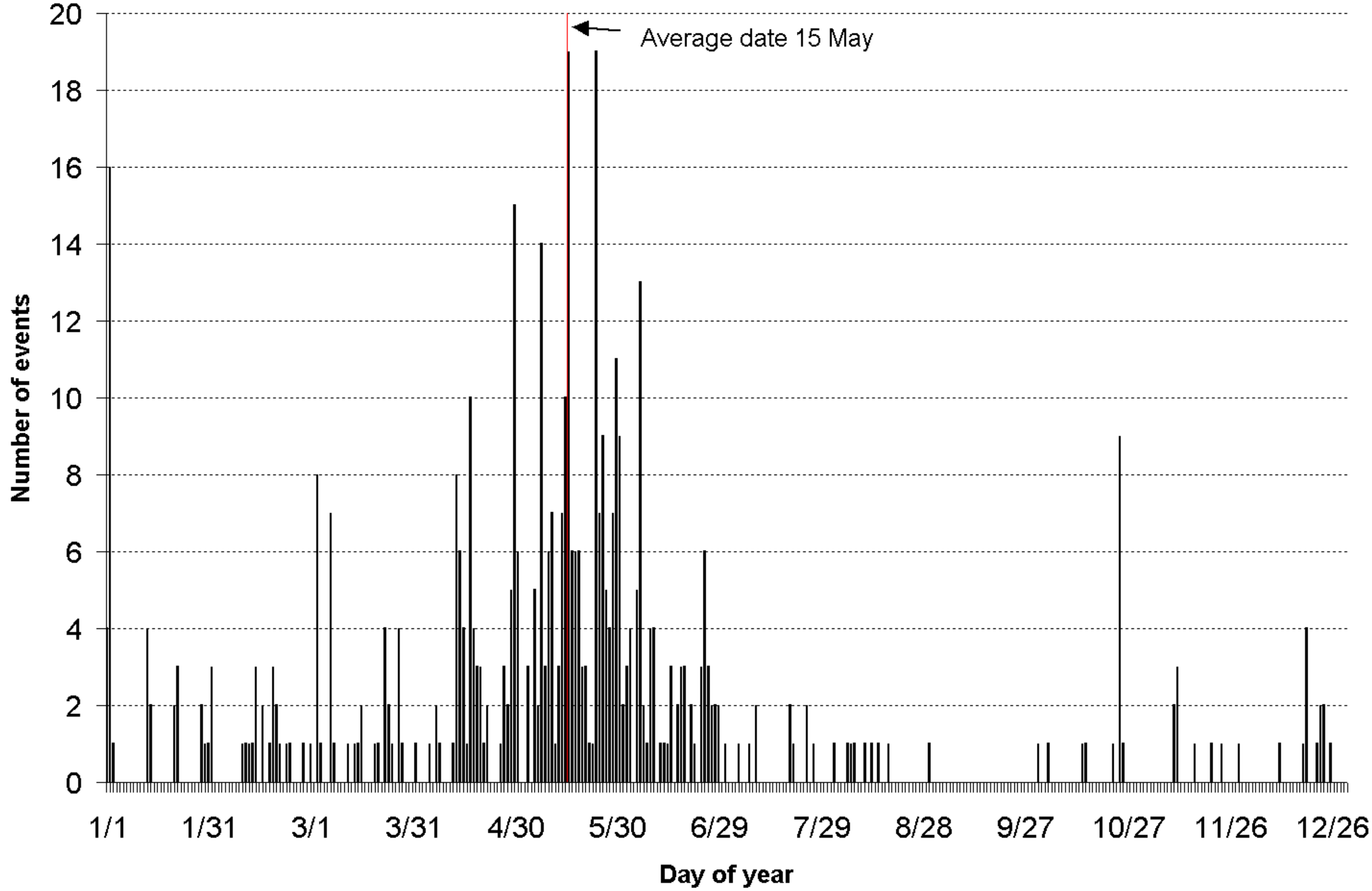

Figure 3. Dates of One-day annual maximum discharges for all gages in the Lake Tahoe Basin

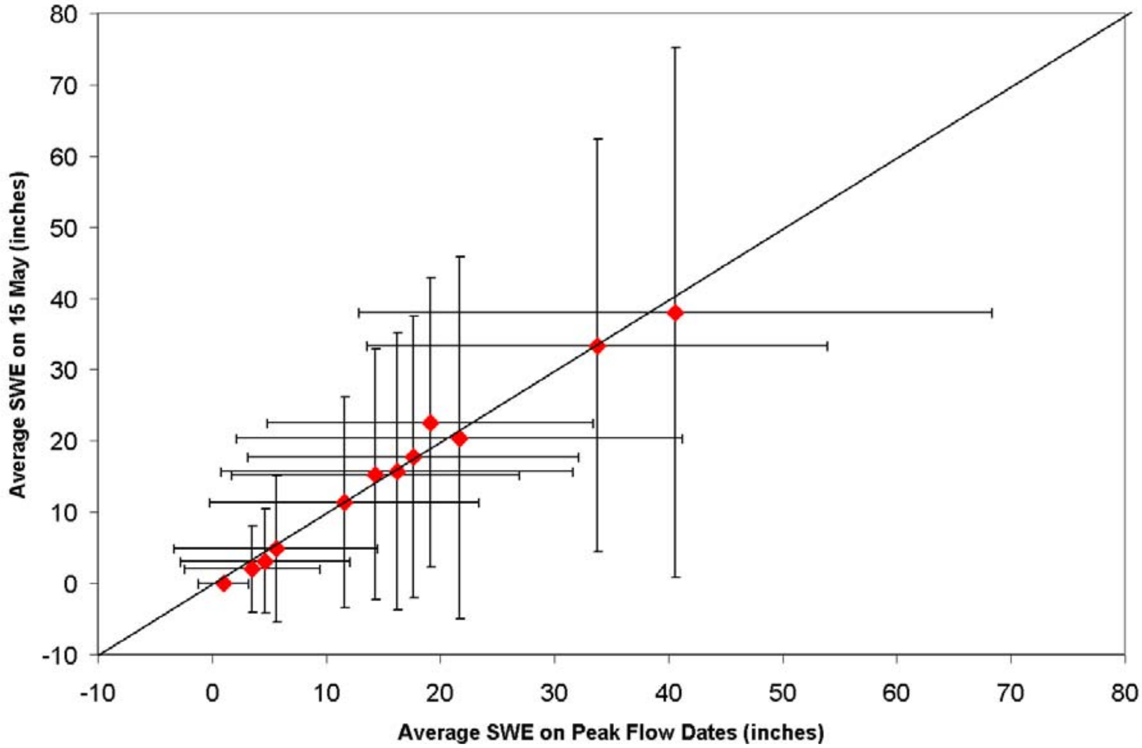

Figure 4. Comparison to average SWE on one-day annual discharge dates and average SWE on May 15.

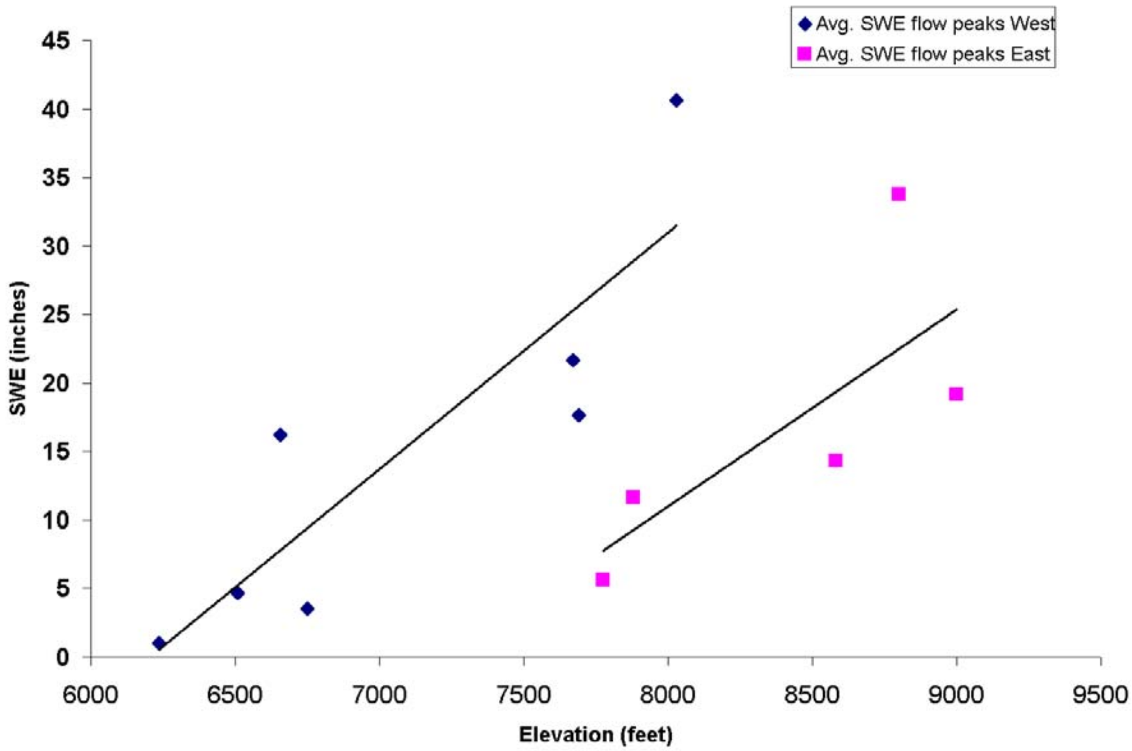

Figure 5. Average SWE as a function of elevation.

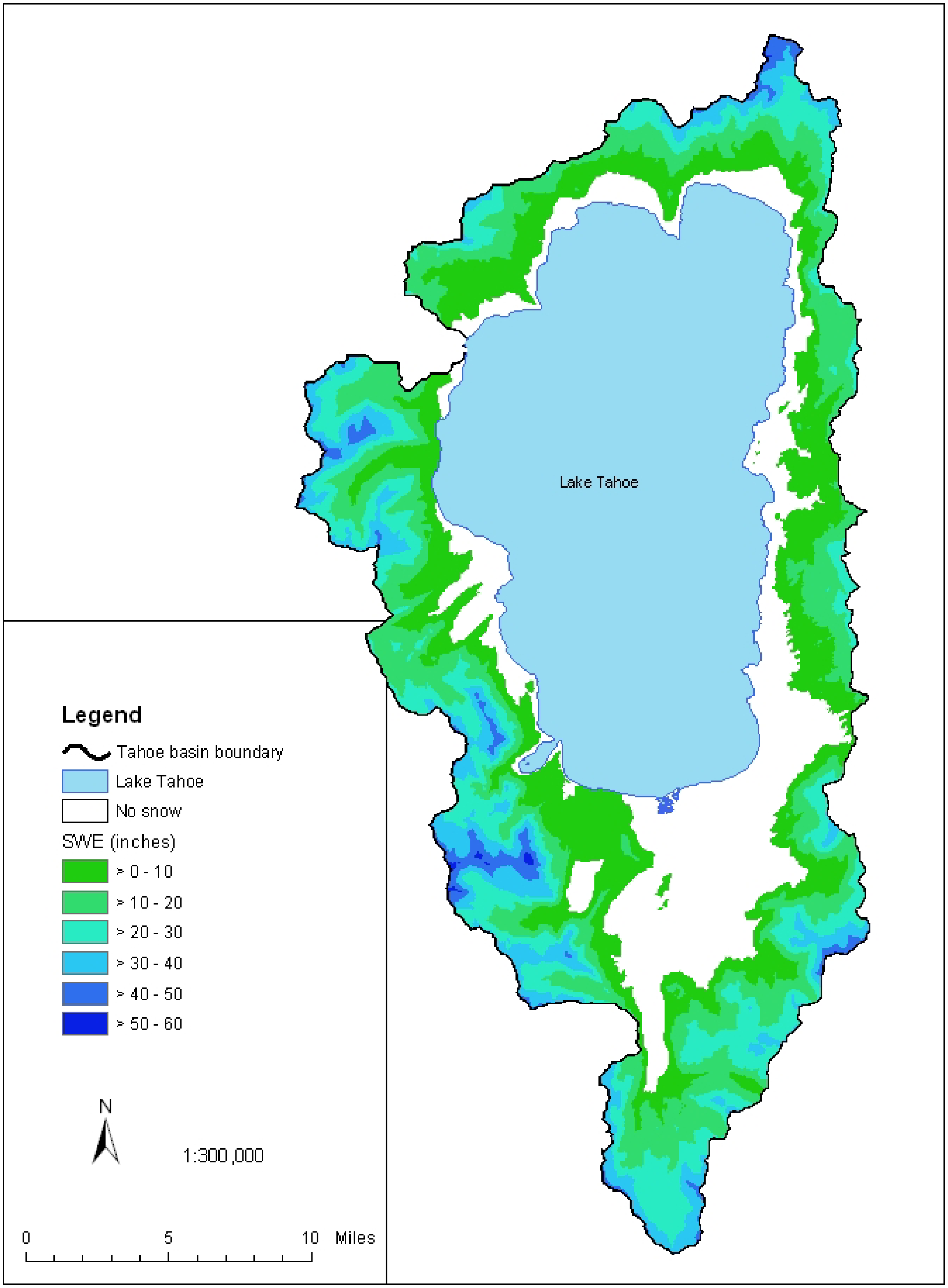

Figure 6. Expected SWE distribution corresponding to one-day annual maximum discharges in the Lake Tahoe Basin

The focus on developing hydrologic design criteria for best management practice(BMP) is limited to the design of retention facilities. The assumption is that the water quality volume that needs to be controlled by a best management practice to meet water quality objectives will be specified based on other studies. The problem is on designing a facility that addresses the competing requirements on a retention facility to control storm runoff and meet water quality objectives. This competition for retention storage occurs when area limitation prevent the construction of separate facilities to address each problem.

The regulatory requirements for storm water control focus on the need to control storm runoff to a pre-project level. For example, a retention basin needs to be designed to reduce the 1% chance exceedance (100 year return period) peak annual flow to a pre-project level. In contrast, regulatory requirements for BMP address the need to retain relatively low flows for a period needed to settle out sediment and related pollutants. The sediment and related pollutants retained is designed to meet receiving water total maximum daily load constraints. The design requirements conflict because the volume available for reducing storm runoff peaks is likely reduced by the encroachment of water volumes retained to settle out sediment and other pollutants. This likely encroachment is an additional factor in design which is not considered in the standard application of single event analysis for storm water control. Practically speaking, a simple adjustment factor accounting for the encroachment is needed to increase the storm water control design volume obtained from a single event approach.

The strategy to developing the adjustment factor for single event analysis will be to estimate inter-event arrival times to both consider the need to retain sediment/pollutants while estimating the risk of exceeding facility capacity (see P.S. Calabrò, "Design Storms and Water Quality Control," Journal of Hydrologic Engineering, ASCE, V9(1), January/February, 2004, p28-34). Future work could be performed to develop these factors for the hydro-meteorology of the Lake Tahoe Basin.