US Army Corps
of Engineers ®
Sacramento District

Summary Report:

Investigations to Determine Regional Flow-Frequency Relationships and Watershed Modeling Recommendations for Hydrologic Design Criteria for the Lake Tahoe Basin

Prepared for the Lake Tahoe Storm Water Quality Improvement Committee

October 2006

Lake Tahoe Investigations to Determine Regional Flow-Frequency Relationships and Watershed Modeling Recommendations Useful for Developing Hydrologic Criteria for Drainage Design and Best Management Practice

EXECUTIVE SUMMARY

Purpose

The purpose of this report is to provide recommendations for traditional drainage and best management practice (BMP) hydrologic design criteria to be applied in the Lake Tahoe Basin. These recommendations are based on detailed studies of the hydrologic characteristics for the study area (see SPK 2005a and 2005b) and were developed by the Sacramento District Corps of Engineers at the request of the Lake Tahoe Storm Water Quality Improvement Committee.

The criteria will be used to help develop a drainage design criteria manual that can be used by Lake Tahoe Basin counties and other federal, state and local interests responsible for developing designs to control storm water runoff.

Methodology

The recommendations focus on watershed modeling methods for estimating risk based design flows (e.g., the 100-year peak annual flow). Separate recommendations were made for watersheds located at elevations above and below 7000 feet. This division was made because useful stream gage data for calibrating water models was not available below 7000 feet. Reviews of current county and professional practice (SPK, 2005b) and watershed calibration modeling studies (Cold Regions Research and Engineering Laboratory, 2005) provided the information needed to develop the recommendations for basins lying below 7000 feet. In this case, balanced design storms, constant loss rates obtained from the model calibrations, and generally recognized flow routing procedures are recommended. In the case of watersheds lying at or above 7000 feet, the issue is runoff from natural (forest or pasture) areas. The key recommendation for these watersheds is to use the regional regression estimates (SPK, 2005a) of flow-frequency curves as base information to estimate watershed modeling parameters for drainage areas greater than 0.5 square miles. Comparison of regional regression estimates and watershed model simulation of design storms might be used to judge the value of model parameters for areas with drainage areas smaller than 0.5 square miles (see recommended future studies).

Runoff coefficient methods are recommended instead of watershed models for very small watersheds (< 200 acres) irrespective of the elevation. Gage information for these small basins does not exist. Consequently, ungaged analysis approaches accepted in professional practice were relied upon for the recommendations. Typically, the Rational Method is used in estimating design peak discharges for these small drainage areas. Unfortunately, published Rational Method coefficients are not particularly relevant to the snow-affected runoff in the Lake Tahoe Basin. In lieu of further studies, a conservative approach, with a runoff coefficient in the range 0.9 – 1.0, is suggested in applying the Rational Method. However, there is an issue that needs to be considered when modeling the effects of urban development (i.e., increasing the drainage area percent impervious). Under these circumstances, existing natural (forest and pasture) or previously landscaped drainage areas might be considered to have less runoff potential. Assuming less runoff potential would require a greater effort to mitigate the potentially increased runoff from the future development. Given the lack of data, this may require an operational decision by regulatory agencies. Further studies might use watershed models to estimate the runoff coefficients for the Rational Method. Here, the information gained from the large watershed model calibration studies could be used to simulate the precipitation – runoff estimates needed to calibrate the runoff coefficients.

The NOAA 14 (NWS) precipitation depth-duration frequency curves should be used in estimating design precipitation in application with either the Rational Method or in creating design storms for watershed modeling studies. These precipitation frequency curves were found to be consistent with local Lake Tahoe Basin gage data, although the user should be aware of the limitations of the results, given the lack of precipitation data for durations less than 60 minutes and elevations greater than 7000 feet (see SPK, 2006).

Future Studies

The following future studies would provide additional information and guidance for estimating discharges for drainage design:

- Published coefficients for application of the Rational Method to small drainage areas (< 200 acres) are probably not relevant to the snow-affected runoff problem important to the Lake Tahoe Basin. A watershed model simulation study, much as was done for the Placer County Manual (1990), using the results of the model calibration study (Cold Regions Research and Engineering Laboratory) could be performed to develop more appropriate coefficients.
- A national study (WRC, 1981) of flow-frequency curve estimation methods demonstrated that regional regressions were somewhat more accurate than simulation of design storms with watershed models in application to ungaged watersheds. Consequently, a future effort to develop guidelines for use of the Lake Tahoe Basin regional regression equations (SPD, 2005a) to aid in watershed model calibration would improve model prediction accuracy.

Recommendations for Best Management Practice

Recommendations for best management practice hydrologic design criteria focused on developing design water quality volumes (WQVs). Using WQVs for design is commensurate with standard practice in the profession and can be used easily with the design event concepts recommended for drainage design. The Tahoe Regional Planning Authority (see LRWQCB, 1994) and Caltrans (2003) currently employ this approach. It is also used in the well-known Denver Drainage Manual (see USDCM, 2003).

The current Tahoe Regional Planning Authority criteria are not well substantiated by studies that relate the WQV to water quality objectives for the Lake Tahoe Basin. Modeling studies are needed to derive WQV values for this purpose. Current modeling studies being performed by the Lahontan Regional Water Quality Control Board may serve this purpose. The major challenge to new modeling studies will be:

- The lack of precipitation-runoff data. In particular, very little short interval precipitation exists. Furthermore, data does not exist for the urban watersheds, which are the focus of the hydrologic design criteria.

- Modeling studies need to assess the margin of safety (MOS) required by EPA (1999) to assure that a particular design will meet total maximum daily load constraints. Currently, MOS is implemented without regard to the tradeoff between water quality benefits and design costs. New modeling studies need to examine the tradeoff between incremental benefits and costs as a function of incremental increases in MOS.

Regional Regression Equations

Regional regression equations were developed to provide predictive relationships for inclusion in the hydrologic design criteria (see SPK 2005a). These equations are not only useful for this purpose, but may also be valuable for water quality and stream restoration studies.

Regression equations and examples of their application are described in the Appendix, Section 5.0 of this report. Section 5.0 also gives a comparison of the peak flow regression equations developed in this report with other studies relevant to the Lake Tahoe Basin.

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2. SPK, 2004b. Comparison of Depth Duration Frequency Curves Obtained from NOAA 14 and Estimated from MM5 Simulated Precipitation, Sacramento District (SPK), U.S. Army Corps of Engineers
3. SPK, 2005a. Investigation of Regional Regression Equations for Flow Duration, Peak and Annual Maximum Flow, and Low-flow Frequency Curves for Lake Tahoe Basin, Sacramento District (SPK), U.S. Army Corps of Engineers.
4. SPK, 2005b. Recommended Watershed Modeling Techniques for Hydrologic Design and Best Management Practice, Sacramento District (SPK), U.S. Army Corps of Engineers
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1. INTRODUCTION

1.1 Report Topics

This report provides recommendations for traditional drainage and best management practice (BMP) hydrologic design criteria in the Lake Tahoe Basin. These criteria will be useful for the development of a drainage design manual that can be used by the Lake Tahoe Basin counties and other federal, state, and local interests responsible for designs intended to control storm water runoff. The studies performed by the Sacramento District, Corps of Engineers, to develop these recommendations were requested by the Lake Tahoe Storm Water Quality Improvement Committee (SWQIC).

The recommendations are based on investigations of Lake Tahoe Basin hydrologic characteristics and a review of current study area design practice. The focus of the hydrologic investigations was on the common drainage design problem: flow prediction for ungaged watersheds. The ungaged watershed problem was investigated by: (1) estimating regional flow-frequency relationships relevant to watersheds draining relatively high elevation areas (at or above 7000 feet), (2) obtaining regional loss rate information for lower elevation watersheds (less than 7000 feet), and (3) reviewing the most recent approaches described in the engineering literature used for watershed model prediction of design flows. Current drainage design manuals for Lake Tahoe counties were reviewed to assess differences in design criteria. Recommendations were based on this review information, knowledge of the current state of the art, and the hydrologic investigations.

Section 2 describes the development of the recommendations for traditional drainage design problems, specific design problems considered, and the methods that should be used to develop design flows. Section 3 describes the approach to BMP design outside of the Lake Tahoe Basin and the studies that need to be performed to modify these approaches for application to the basin. Section 4 describes other applications of the regional regression equations that could prove useful for water quality and restoration studies. The Appendix, Section 5, describes the regional regression equations developed for the study area and presents example applications.

1.2 Tasks Completed

Table 1.1 describes the tasks completed in the joint investigations with the SWQIC. The requirements for the study were developed based on extensive discussion with the SWQIC (see SPK, 2004a, Task 1.0).

The main study effort resulted in reports (1) reviewing study area precipitation depth-duration frequency curves (see, SPK 2004); (2) describing the development of regional flow-frequency regression relationships (SPK, 2005a); (3) providing detailed criteria for performing watershed modeling in the Lake Tahoe Basin (SPK, 2005b); and (4) calibrating watershed models that resulted in regional loss rates useful for drainage design (Cold Regions Research and Engineering Laboratory, 2005). The review of depth-duration-frequency curves (see tasks 3.1

and 3.2) involved comparing estimates from NOAA 14 (National Weather Service, 2004) with simulated precipitation derived from applications of a meteorologic model MM5 (the MM5 unpublished synthetic precipitation estimates are currently being used by the California State Lahontan Regional Water Quality Board for hydrologic investigations within the study area). This comparison was made at the request of the SWQIC to quantify the difference between precipitation depth-duration-frequency characteristics from NOAA 14 and those derivable from the MM5 simulations.

The studies to develop the regional frequency relationships, snow-water equivalent maps, and studies of precipitation and antecedent conditions involved an extensive data collection effort (see Task 2). Precipitation gage data and snow-water equivalent data were used to develop maps of snow-water equivalent and design storm recommendations useful for performing watershed modeling (see SPK, 2005b, Tasks 3.3 and 3.4).

The database effort also produced GIS information on study area topographic and meteorologic characteristics necessary to develop regional regression flow-frequency relationships (see SPK, 2005a). The topographic database was used to estimate watershed drainage areas and mean elevations for the gaged basins used in the regression study. Results from applications of the PRISM model (see Taylor, et al., 1993) were used to estimate watershed mean annual precipitation, mean annual temperature, and mean total annual snowfall for these areas.

The regional regression effort (see Task 4.0) resulted in regression equations which use watershed topographic and meteorologic characteristics to predict (1) annual peak flow-frequency curves, (2) annual maximum volume-duration-frequency curves, (3) 7-day low-flow-frequency curves, and (4) annual flow-duration curves (see SPK, 2005a).

Initially, recommendations for precipitation-runoff modeling approaches for study area hydrologic design and best management practice were developed based on analysis of available hydro-meteorologic data, review of the current county practice, and the state of the art (see, SPK 2005b, Task 5.0). These recommendations were improved when the resources became available to estimate regional loss rate parameters from water model calibration studies (Cold Regions Research and Engineering Laboratory, Task 6.0)

The evaluation of NOAA 14 applicability to the Lake Tahoe Basin in Task 7.0 was added after the completion of the other tasks. This evaluation is important because the precipitation depth-duration frequency curves are a key input to the drainage design analysis.

Finally, this summary report was developed to provide a concise explanation of the results and recommendations resulting from tasks 1.0 – 7.0 (see Task 8).

Table 1.1. Study Tasks

Task	Description	Deliverable
1.0	INITIAL ASSESSMENT	Initial report describing state of the practice, ongoing studies and consensus on needed criteria
1.1.	Assess existing practice and ongoing studies	
1.2.	Define Criteria/Methods Selection Process, develop stakeholder consensus	
2.0	DATABASE	Hydro-meteorological time series and GIS data base
3.0	PRECIPITATION/METEOROLOGIC EVALUATION AND ANALYSIS	Report providing precipitation depth-duration frequency curves, snow-water equivalent mapping, and frequency based design storms with corresponding initial conditions
3.1.	Review current NWS depth-duration frequency study (NOAA 14)	
3.2.	Compare NOAA 14 vs. MM5 precip.	
3.3.	Snow-water equivalent mapping/frequency analysis	
3.4.	Develop design storms/antecedent conditions	
4.0	FLOW-FREQUENCY ANALYSIS	Report describing frequency curves for low and high flows, flow-duration curves, and regional regression equations for each flow type
4.1.	At-site Flood (high flow) frequency analysis	
4.2.	At-site low-flow-frequency analysis	
4.3.	At-site flow-duration analysis	
4.4.	Regional regressions for high flow-frequency curves, low-flow-frequency curves and flow-duration curves useful for ungaged watershed analysis	
5.0	RECOMMENDED PRECIPITATION-RUNOFF MODELING APPROACHES	Report describing recommended modeling approaches
6.0	PERFORM WATERSHED MODEL CALIBRATION STUDIES TO IMPROVE MODELING APPROACH RECOMMENDATIONS	Report describing water modeling effort to obtain regional loss rate values for drainage design applications.
7.0	Evaluate NOAA 14	Report evaluating applicability of NOAA 14 precipitation-depth-duration frequency curves to Lake Tahoe Basin
8.0	SUMMARY REPORT	Summary report

2. RECOMMENDED HYDROLOGIC CRITERIA FOR TRADITIONAL DRAINAGE DESIGN PROBLEMS

2.1 Introduction

The purpose of this section is to recommend hydrologic design criteria for the traditional drainage design problem (i.e., drainage problems caused by floods or high storm runoff) where risk-based estimates of flow are required (e.g., the 100-peak annual flow). Section 2.2 describes the drainage design problems of interest. The regional regressions developed to estimate design flows are described in Section 2.3. An alternative approach to estimate design flow is to simulate design storms with a hydrologic model. Section 2.4 reviews the criteria for employing these models described in Lake Tahoe counties' drainage design manuals. Section 2.5 provides recommended design criteria for estimating design flow rates based on regional regression estimates, results of the watershed model calibration studies, and review of the current county modeling practice. These recommendations are based on the review of the county practice, the state of the art in applying hydrologic modeling methods, the investigation of precipitation runoff characteristics, and the availability of the regional regressions for estimating design flows.

2.2 Traditional Drainage Design Problems

Hydrologic design usually focuses on the magnitude of peak flow, or a combination of flow peak and volume represented by a hydrograph. The peak flow or hydrographic characteristics are used to provide information for regulatory flood plain definition, or to size:

- Small drainage features, such as catch basin spacing and corresponding drainage pipes;
- Major highway culverts;
- Channel conveyance;
- Detention/retention basin storage and outlets;
- Spillways for dam safety.

Culverts and channel conveyances need to be sufficient to convey a peak flow for a 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 detention/retention basins and spillway design.

Most typically, design flow magnitude is related to risk-related design capacity. For example, a detention/retention basin's 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.

2.3 Estimating Design Flows Using Regional Regressions

Regional regressions relate a flow quantile (e.g., the 1% chance exceedance peak annual flow) to watershed meteorologic and physical characteristics. A typical relationship has the form:

$$\log_{10}(Q_p) = b_0 + b_1 \log_{10}(A) + b_2 \log_{10}(MAP)$$

where Q_p is the design flow quantile, A is the drainage area, MAP is the mean annual precipitation for the watershed, and the regression coefficients, b_i, i=0,1,2, are determined from a regression analysis of gaged data.

A detailed study was performed (SPK, 2005a) to develop regional regression equations for:

- Peak annual stream flow
- 1-, 3-, 7-, 10-, 15- and 30-day maximum annual stream flow volumes.

A description of the regression equations and example applications are provided in the Appendix, Section 5.

2.4 Watershed Model Methods: Current Practice

The purpose of this section is to provide a critical review of the watershed modeling methods currently used by the Lake Tahoe counties to estimate design flow values. This review is intended to provide a perspective on the information available throughout the study area for performing ungaged watershed analysis. Recommendations for methods to be used in a drainage design manual will be based on both the county perspective and current state of the art.

The Lake Tahoe counties (Placer, El Dorado, Douglas and Washoe) provide different hydrologic design criteria for estimating design flows, and in varying levels of detail. (Alpine County, which also includes a portion of the lake, does not have a drainage design manual.) The counties do identify alternative approaches to estimating the design discharge, as is typically done, depending on the drainage area size and complexity of the drainage system. Runoff coefficient methods are used for smaller drainage areas (areas on the order of 100 acres or less) that lack complex drainage systems (e.g., no detention/retention structures). Watershed models are used for larger areas with potentially more complex drainage systems.

2.4.1 Runoff Coefficient Methods

Typically, the well-known “Rational Method” is used as the runoff coefficient approach. This method computes the design peak discharge as the product of (1) drainage area, (2) a runoff coefficient, and (3) a peak rainfall intensity for a particular exceedance probability. The peak rainfall intensity is determined from a precipitation depth-duration frequency relationship by using a duration equal to the drainage area time of concentration. The runoff coefficient is a function of the watershed land surface characteristics. Studies have been done relating the runoff coefficient to the exceedance probability of the precipitation.

Placer, El Dorado, and Washoe counties provide guidelines for application of runoff coefficient methods (see Table 2.1). Placer County does not use the rational method for small areas, but instead 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 watershed model (HEC, 1990) 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 values than those obtained by Placer and Washoe counties, especially for forest or natural areas. This may result because the Placer and Washoe county methods to estimate time of concentration were developed for urban land use. Development of the Placer County equations for time of concentration is not well documented, nor is the development of the Washoe County method for estimating runoff coefficients (see SPK, 2005b, Section 3).

Table 2.1. County Methods for Application of 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) $V_{open} = 16.1435\sqrt{S}$ $V_{paved} = 20.3283\sqrt{S}$ $t = L/3600V$	
Channel Flow	[2]travel time minutes $\frac{.00735 L n_c^{0.75}(1+Z^2)^{0.25}}{S^{0.375}(A_c Z)^{0.25}}$	velocity Manning equation 2-year 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.

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

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

El Dorado County Parameters:

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

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)

2.4.2 Watershed Modeling Methods

The purpose of this section is to critically review the watershed modeling techniques recommended in county drainage manuals in the context of the current state of the art. This review will provide a good basis for recommending criteria for the application of watershed models.

A design storm is simulated by a watershed model to obtain a specific exceedance probability design flow or hydrograph. To do this, the watershed is represented as an interconnected set of modeling components (see Figure 2.1).

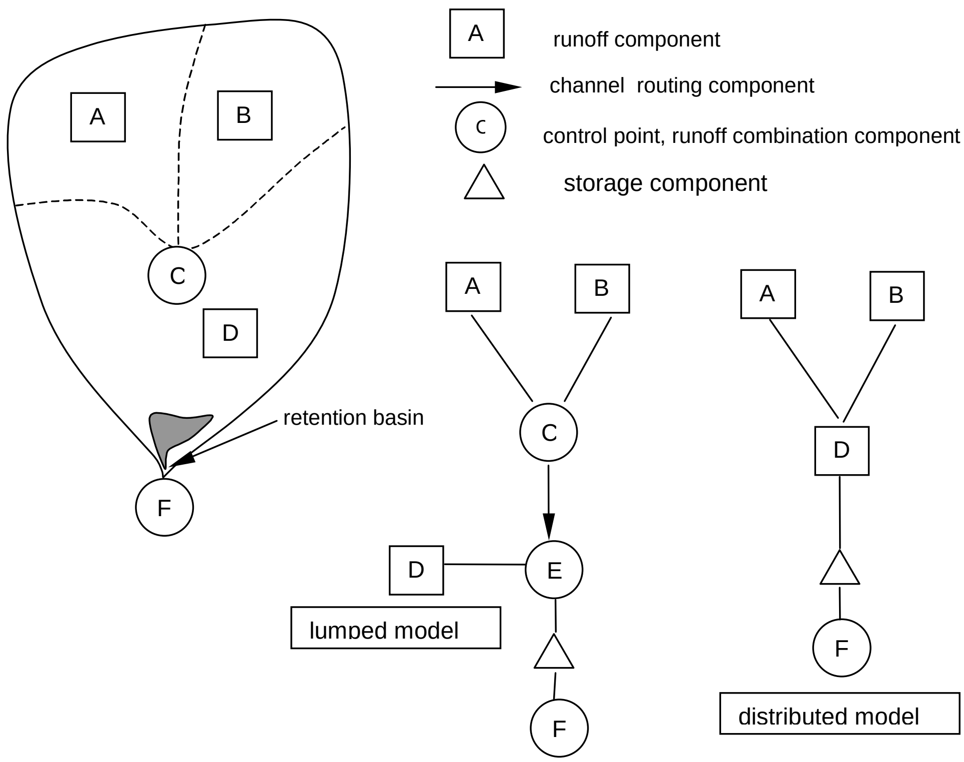

Figure 2.1. Watershed Model Components

The methods used by each county were compared based on how each county represents model components (see SPK 2005b, Section 4, for a more detailed discussion). The following component elements were considered when comparing county criteria:

Rainfall Runoff

- **Input:** design storm
- **Initial Conditions:** antecedent wetness (snow-water equivalent, soil moisture, frozen ground)

- **Computation Methods:** rainfall excess volume (a function of surface and soil loss rates), snowmelt volume, volume transform (e.g., unit hydrograph or kinematic wave)
- **Output:** runoff hydrograph

Channel Routing

- **Input:** upstream inflow hydrograph
- **Initial Conditions:** channel outflow or depth
- **Computation Methods:** hydrologic routing (e.g., Muskingum), hydraulic routing (Muskingum-Cunge)
- **Output:** routed channel hydrograph

Storage Routing

- **Input:** upstream inflow hydrograph
- **Initial Conditions:** initial storage volume
- **Computation Method:** level-pool reservoir routing
- **Output:** routed hydrograph

The counties' criteria differed with regard to initial conditions and computational methods, except for the storage routing component. All the counties recommended using standard level-pool reservoir routing. Otherwise, the counties differed in their criteria as summarized below:

Rainfall Runoff

- **Design Storm**. The counties employ various methods for developing design storms (balanced, NRCS type Ia and II are employed) from depth-duration-frequency curves and for estimating basin average design storm depth.

- **Loss Rates.** Placer County selects loss rates based on design purpose. Frozen ground (zero loss rate) conditions are assumed when sizing conveyance structures or delineating flood plains, whereas, unfrozen conditions are considered when assessing the runoff increase due to development. Unfrozen ground constant loss rates are determined from a wet condition NRCS curve number (CN). El Dorado and Washoe counties 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 counties recommend use of the NRCS lag unit hydrograph (UH). Although the same UH method is used, El Dorado and Washoe counties use different means for calculating the time of concentration to obtain the UH lag. Douglas County accepts either the distributed or UH approach.

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 from data for small agricultural watersheds in the Midwestern U.S., which does not justify their use in Lake Tahoe. Also, the effect 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 (see discussion by Pilgrim and Cordery, section 9.6.3, 1992). However, the distributed approach probably has some advantage in capturing the differing response between open and impervious areas in urban catchments.

- **Channel Routing.** Placer, El Dorado and Washoe counties provide various recommendations on which hydrologic and hydraulic 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, since it is more applicable to a wider range of hydraulic conditions and requires exactly the same parameters (see Fread, pg. 10.4, 1992). Although often used as a "hydraulic method," the modified-Puls method should not be used for channel routing. The method is a poor solution to the kinematic wave equations, where the numerical error, resulting in apparent attenuation of the hydrograph, is a function of the computation interval and reach length, rather than any effect of storage. Generally, it gives poor results for channel routing (see Miller and Cunge, pg. 215, 1975, and also Fread, chapter 10, 1992 for discussion of level-pool reservoir routing).

To summarize, the counties generally approach the watershed modeling problem in the same manner. The greatest differences in computed runoff will come from the design storm chosen, assumed initial watershed antecedent conditions, and loss rate method and parameters.

2.5 Recommendations Regarding Hydrologic Modeling Criteria

Recommendations regarding hydrologic modeling criteria are made based on the hydrologic studies to develop regional regression estimates of flow-frequency curves, calibrated watershed model parameters, and review of the current county practice in applying watershed modeling techniques. The recommendations are somewhat subjective in that professional practice accepts various approaches as reasonable for estimating design discharges. For example, application of either the unit hydrograph or kinematic wave techniques are standard methods commonly applied to compute runoff. Either might be applied in an urban setting, resulting in somewhat different discharge estimates. The profession usually judges either of these methods to be acceptable techniques, as can be seen from reviewing hydrologic texts or drainage design manuals. Watershed modeling methods are approximate and different approaches provide useful design estimates.

The recommendations propose that watershed modeling methods derived from calibration to gaged precipitation and runoff data are superior to those based on an ungaged analysis (i.e., determined from the physical characteristics of the watershed). Consequently, the regional regression estimates and results of the watershed model studies provide model calibration information that adds value to the recommendations for estimating design discharges.

The calibration studies (CRREL, 2005) determined that snow-affected runoff is critical to determining design runoff within the Lake Tahoe Basin. As will be demonstrated in the following two sections, model parameters need to reflect this reality. Of concern is that most text estimates of runoff parameters (loss rates, runoff coefficients and routing parameters) have been developed for snow-free ground situations. Consequently, although smaller drainage areas lie outside the range of those used in the regression and calibration studies, the finding that snow-affected runoff is dominant within the basin needs to be considered.

2.5.1 Rational Method

Recommendations for application of the Rational Method have to be based on current practice. No gage information or modeling studies such as described in the Placer County Drainage Manual are available for other recommendations to be made.

The Rational Method probably should be applied using the recent NOAA 14 precipitation depth-duration-frequency curves and the TR-55 (NRCS, 1986) methodology for computing time of concentration. The NOAA 14 frequency curves were found to be consistent with precipitation in the local Lake Tahoe Basin, although the user should be aware of the limitations of the results, i.e., the lack of precipitation data for duration less than 60 minutes and for elevations greater than 7000 feet (see SPK, 2006). The computation of time of concentration critical to the application of the method depends partly on the estimated overland flow length. The TR-55 published recommendation for maximum overland flow length should be replaced by the most recent recommendation, which is to limit this length for sheet flow to 100 feet, rather than 300 feet (NRCS, 2004a and 2004b, and personal communication: Woodward, 2004).

Standard runoff coefficients (see EPA (1983)) are probably not relevant for the snow-affected design problem important to Lake Tahoe. A conservative approach would be to use a coefficient ranging from 0.9 – 1.0 for both open natural (forest or pasture) and urban (landscaped) areas. This would certainly produce maximum runoff. However, as in the Placer County Manual, the coefficient may be reduced for an existing open land use where development is planned. This would show some increase in runoff due to the loss of infiltration capacity from development. A tradeoff exists here, where the runoff from open areas may be underestimated by reducing the coefficient for regulatory purposes, where, in fact, snow-affected runoff may be higher. The regulatory agencies need to decide if the reduced time of concentration caused by urbanization captures enough of the development impact on runoff that needs to be mitigated in drainage design. If not, then perhaps reduction in the runoff coefficient is warranted. See section 2.6 for recommended future studies to improve applications of this methodology.

The maximum basin size to use for application of this method depends largely on the variation in runoff properties and complexity of the drainage system in the 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 varied or distributed. The typical rule of thumb is to limit application to drainage areas less than 200 acres with relatively simple drainage patterns (e.g., no detention/retention storage).

2.5.2 Watershed Modeling Approaches

Identifying a Modeling Approach (section 2.5.2.1)

The recommendations regarding hydrologic modeling criteria need to focus on a particular watershed modeling approach. A vast number of watershed modeling approaches 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).

The general categories of watershed models that could be used to estimate runoff for drainage design purposes are as follows:

- Statistical

Statistical approaches to directly explain the variation of stream flow and precipitation provide the basis for developing design flows (see Stedinger, et al. 1992). These techniques were used to develop the NOAA 14 precipitation depth-duration frequency curves and the gage flow-frequency curves used by the Corps to develop the regional regression relationships described in section 2.3. These statistical approaches are recommended for use in conjunction with the watershed models (mentioned subsequently) to develop design flows for ungaged basins.

- Event-oriented

Individual precipitation events are simulated to compute 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 with reasonably uniform runoff characteristics. Runoff from these sub-areas can be aggregated/combined 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 software programs implementing 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 computer software 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. Event-oriented and continuous simulation models often lump or average subbasin runoff properties, at least to a greater extent than physically based models. Physically based models are generally considered to employ the physics of fluid motion to simulate runoff at a finer scale than the event/continuous models, and are perhaps 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).

Lake Tahoe basin precipitation-runoff data availability and potential model prediction accuracy need to be considered when selecting from among these different approaches. Design problems will be encountered mostly on the ungaged watersheds in the Lake Tahoe Basin. However, precipitation-runoff data for gaged 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 exist for the Lake Tahoe Basin, which makes the application of continuous simulation models particularly difficult.

In terms of prediction accuracy, most of the comparative studies done show that the simpler event-oriented/continuous simulation models are at least as accurate if not more accurate than the more sophisticated physically based models (see SPK, 2005b, section 4.2.1).

Consequently, given both the lack of precipitation data available for the study area, and the lack of evidence that more sophisticated models will outperform 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 should also be considered. In the LRWQCB and NDEP work, a continuous simulation model with synthetic precipitation is being used to estimate runoff for a 40-year period for the lake. 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 the modeling applications.

Recommendations for Event-oriented Watershed Modeling (2.5.2.2)

Design Storm. 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 an analysis of available storm data for regions surrounding the (see section 4.4.2, SPK, 2005b) found that actual storms for a significant portion of occurrences were not balanced. The NOAA 14 precipitation depth-duration frequency curves should be used for developing these balanced storms. The NOAA 14 frequency curves were found to be consistent with data from local gages in the Lake Tahoe Basin; although the user should be aware of the limitations of NOAA 14, given the lack of precipitation data for duration less than 60 minutes and elevations greater than 7000 feet (see SPK, 2006).

The duration of the design 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 great enough so that the whole basin will be contributing to the computed peak runoff needed for design. If a detention/retention structure is being designed, the volume of runoff is important. The duration of the storm should be great enough so that increments in storm duration do not affect the design of the detention/retention structure to control stormwater.

No depth area reduction adjustment will be made to the point precipitation estimates, due to the increase in precipitation with elevation and the lack of studies analyzing change in average storm depth with drainage area for the study area.

Loss Rates (application to drainage area elevations less than 7000 feet)

Developing recommendations for loss rates was complicated by both the importance of snow-affected runoff in Lake Tahoe Basin and the varying recommendations regarding loss rates provided in the County Guidance. Placer County provides very detailed recommendation regarding the effect of snow, while the other counties rely on the CN method. In fairness to the other counties, the guidance provided does not directly address snow-related runoff issues. For example, El Dorado County notes that specialized studies need to be undertaken to account for snow-affected runoff.

The Placer County recommendations were found to be potentially very conservative in that snow-affected loss rates were assumed to be effectively zero (a frozen ground assumption) when computing runoff for sizing conveyance or delineating flood plains. For this reason, a watershed model calibration study (see Cold Regions Research and Engineering Laboratory (CRREL), 2005) was undertaken to estimate loss rates relevant to the Lake Tahoe Basin that could be used together with design storms to obtain realistic design discharges.

The focus of the CRREL study was to estimate loss rates for watersheds located below the 7000 foot elevation. Loss rates, as well as other watershed modeling parameters, for watersheds with drainage areas at or above 7000 feet that exceed 0.5 square miles can be obtained in model calibration studies with the regional regression equations (see section below on regional regression equation applications). Future modeling studies are recommended for smaller drainage areas located in this elevation range.

In this calibration study, a simple approach was taken, with the loss rates being estimated to account for the snow-affected runoff, which is important to drainage design in the Lake Tahoe Basin. These loss rates were determined as described below (see Cold Region Research and Engineering Laboratory, 2005, for a detailed description):

- The HEC-1 watershed model routing parameters were developed by calibration to gaged data for observed major events;
- The 100-year and 2-year discharges for watersheds below 7000 foot elevation were computed by calibrating loss rates in HEC-1 to reproduce 100-year and 2-year frequency curve estimates in simulations using templates based on historic storms of a similar frequency. That is, these two design events were created by proportioning the storm event templates to have the 100- and 2-year 24-hour storm depths from NOAA 14.
- The loss rates were calibrated in the HEC-1 model simulation with the balanced design storms to reproduce the 100-year and 2-year discharges obtained from simulating the design storm template events.

The simulation of storm templates had the advantage of capturing the coincidence of antecedent loss rate conditions, snow pack condition, snow pack melt rates, temperature lapse rates, and precipitation rates that occur in typical storms in the Lake Tahoe Basin. Calibrating loss rates to the 100-year and 2-year discharges determined from the templates resulted in one single, simple runoff factor that could be easily applied with a balanced storm to obtain design discharges. This avoids the messy problem of having to estimate all the coincident conditions that typically affect runoff. The resulting recommended loss rates are provided in Table 2.2 as a function of return interval and watershed location within the Lake Tahoe Basin. Simple interpolation and judgment can be used to determine loss rates for other Lake Tahoe watersheds and return intervals (see section 2.6).

Table 2.2. Recommended Constant Loss Rates (in/hr) for Open Areas Between Elevations (6200-7000 feet)

Watershed	100-year	2-year
Upper Truckee	0.2	0.1
General	0.2	0.1
Ward	0.05	0.1
Incline	0.3	0.1
Third	0.3	0.1
Glenbrook	0.3	0.1
Trout	0.3	0.1

Runoff Routing. TR-55 (NRCS, 1986) methods will be used to estimate the NRCS lag unit hydrographs for natural/open areas. 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 (e.g., forest and pasture) areas because direct runoff can be due to both surface and subsurface flow. The distributed approach is recommended for urban areas (e.g., paved and landscape surfaces) because (1) surface flows dominate the direct runoff, (2) it is simpler to apply than the unit hydrograph method, and (3) it can easily capture the separate responses from pervious and impervious areas. This application should use the most recent research, 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. The 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, then the Muskingum method can be employed.

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 Section 5 and also SPK, 2005a). These regression equations are useful for relatively large open-surface 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 7,000 feet). Additional modeling studies are recommended for estimating runoff for smaller watersheds (see section 2.6).

A nationwide 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. These regression equations can be used to calibrate/validate watershed model predictions by using any of the following approaches: (1) adjusting model loss rates so that the model-predicted frequency curves agree with the regression prediction within some reasonable tolerance; (2) adjusting the model loss rates, if necessary, to ensure that model predictions lie within predicted regression confidence limits on frequency curves of interest; or (3) averaging 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. The method to use will depend on the confidence placed in watershed model predictions.

Neither the regional regression equations nor the watershed model calibration studies address the design problem for watersheds at around 7000 feet with drainage areas less than 0.5 square miles. Without additional study (see the next section) the regional regression equations could be used to estimate loss rates in comparative studies with watershed model application with design storms. Comparison of design flows obtained by either method (regional regression equations or watershed model calibration) could be used to evaluate water model parameters. These parameters could then be applied to smaller drainage areas.

2.6 <u>Future Studies</u>

2.6.1 Introduction

The goal of future studies will be to improve watershed model predictions of annual maximum flow-frequency curves for ungaged areas. These studies need to focus on Rational Method runoff coefficients and developing guidelines for using the regional regression estimates of maximum annual flows to calibrate watershed models in ungaged applications.

Published information for estimating runoff coefficients for application of the Rational Method to small drainage areas is probably not relevant to the snow-affected runoff for the Lake Tahoe Basin. As described in section 2.6.2, watershed model simulations may be performed that use model calibration study results to obtain effective runoff coefficients. Finally, a study described in section 2.6.3 may provide guidance for using the regional regression equations to help calibrate watershed models in applications for open (natural or urban landscaped) areas .

2.6.2 Runoff Coefficients from Watershed Model Simulations

Watershed model simulations could be performed to derive runoff coefficients for use with the Rational Method in Lake Tahoe Basin urban areas. Placer County (1990) used this approach to develop runoff relationships for a runoff-coefficient type method. The watershed model calibration studies (Cold Regions Research and Engineering Laboratory, 2005) provided the loss rate information that could be used in these model studies.

2.6.3 Application of Flow-Frequency Curve Regression Equations to Watershed Modeling

A national test (U.S. WRC, 1981) demonstrated that regional regression equations were, in general, more accurate than uncalibrated watershed models for estimating flow-frequency curves. This study provides a rationale for adjusting watershed model parameters to obtain some reasonable agreement between model and stream 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 for 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 specify to what extent model parameters should be adjusted to bring into agreement model-predicted and regression estimates of flow-frequency curves for ungaged watersheds.

3. RECOMMENDATIONS FOR DEVELOPING HYDROLOGIC DESIGN CRITERIA FOR BEST MANAGEMENT PRACTICE

3.1 Introduction

This section provides a recommendation for an approach for developing best management practice hydrologic design criteria. This recommendation differs from the specific recommendations provided in the previous section for traditional drainage design criteria. In this instance, both modeling studies and water quality monitoring studies will be necessary to ascertain the adequacy of the hydrologic design criteria.

Design criteria have been developed for other areas of the country where the hydrology, pollutant loading, and water quality objectives are different. Consequently, the recommendations in this report regarding hydrologic design criteria for best management practice (BMP) focus on the current state of the art and the modeling studies that are needed to modify these approaches for the Lake Tahoe Basin.

Section 3.2 discusses the role BMP design plays in reducing non-point source pollution to meet receiving water quality objectives. The current Lake Tahoe Basin hydrologic criteria together with other examples of different criteria are discussed in Section 3.3. These current approaches are the basis for the recommended approach to developing the hydrologic design criteria described in Section 3.4.

3.2 Best Management Practice Design Objectives

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. To meet these water quality objectives, EPA uses allowable total maximum daily loads (TMDLs) 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 achieved by controlling point or non-point sources. The allowable load from either of these sources is allocated as (see EPA, 1999, pg. 1-1):

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

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.

3.3 Current Approaches

The hydrologic criteria used to meet an allowable load allocation typically provided in drainage design manuals are based on containing or treating a water quality volume (WQV). The WQV approach is commensurate with the simple approach needed for typical drainage design problems.

Examples where the WQV approach is specified in design criteria are:

- 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 quality objectives.

- Caltrans (California Department of Transportation) determines a WQV based on rainfall 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 detention/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 80^{th} – 90^{th} 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.

The WQV approach used by Caltrans and described in the Denver Drainage Manual could be used in the Lake Tahoe Basin. However, future modeling studies are needed to develop simplified WQV predictive models relevant to the Lake Tahoe Basin.

3.4 Recommended Approach for Developing Design Criteria

Recommendations for improving the current approach to specifying hydrologic criteria for BMPs in the Lake Tahoe Basin depend on developing modeling studies relevant to the basin hydrology and water quality problems. These modeling studies are needed to help calibrate simplified methods to estimate WQVs. At this time, the key contributors to Lake Tahoe Basin non-point source pollution are fine sediment, nitrogen and phosphorous (LRWQCB, 2006).

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 useful in creating simplified approaches to estimating WQVs in smaller urban areas. 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 2.1). 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 are limited. Calibration of the simulation models to this data would 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, parameters typically used in either energy-budget or degree-day snowmelt 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, 2005b) to calibrate/validate continuous simulation model applications in ungaged watersheds. The regressions can be used to estimate the peak, 1-, 3-, 7-, 15-, and 30-day annual maximum flows for specific frequencies.

Developing criteria to estimate the margin of safety (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 an increased MOS from these relationships. In the long run, only monitoring studies will determine if the MOS selected needs revision.

4. OTHER APPLICATIONS OF REGIONAL REGRESSION EQUATIONS

Both 7-day low-flow and daily annual flow-duration frequency curves were estimated for the study area (see Section 5, and SPK, 2005a). These curves may help validate models used to develop hydrologic design criteria for best management practice, as discussed in Section 3.

These regressions may also be useful for water quality and restoration studies. The 7-day/10-year low flow is often used as a regulatory constraint for water quality. Restoration studies often use flow-duration curves to determine stream flow inundation important for design of riparian habitat boundaries.

5. APPENDIX: REGIONAL REGRESSION EQUATION DESCRIPTION AND APPLICATION

5.1 Introduction

This section provides example applications of the regional regression equations developed for the Lake Tahoe Basin. Sections 5.2, 5.3 and 5.4 provide, respectively, example applications for peak and annual maximum volume duration, 7-day low-flow, and flow-duration frequency curves. Refer to SPK (2005a) for further discussion on development of these frequency curves. Section 5.5 provides a comparison of the peak annual stream flow regression equations produced for this study and those developed in previous studies, to show the potential differences in estimates depending on approach and data used.

The application of these regression relationships should be restricted to the basin and for meteorologic characteristics used in the development of the regressions. Consequently, the regressions should be restricted to drainages areas:

- Greater than 0.5 square miles
- With significant contributions from areas above 7000 feet, where snowmelt contributes significantly to annual flow volumes
- Excluding the local drainage areas within the Upper Truckee watershed that are downstream of Meyers at Highway 50 (meaning a concentration point that receives zero runoff from upstream of Highway 50).
- With natural/open surface cover (e.g., forest, pasture, non-urban)

The peak flow-frequency curves were estimated using generalized least squares to account for the varying sampling error associated with quantiles (e.g., the 1% chance exceedance flow) due to varying gage record lengths and the inter-station correlation between gage peaks. The relationship between peak and daily volume quantile regressions was developed using ordinary least squares to obtain regularly varying frequency curve estimates as a function of duration. Ordinary least squares was also used to develop the regression relationship for 7-day low-flow and flow-duration frequency curves. Generalized least squares was not applied due to limitation of the study scope.

In the example applications, different measures of prediction error are produced depending on the least squares methodology used. An average prediction error is developed when using generalized least squares, whereas the standard error is used for ordinary least squares. See the technical appendix in SPK (2005a) for a complete discussion of the difference between these two measures of prediction error.

5.2 Peak and Annual Maximum Flow-Frequency Curves

5.2.1 Peak Flow Regression

Table 5.1 shows best and recommended regional regression equations. Regression equations that include mean annual precipitation (MAP) are recommended over the regressions with mean annual snowfall, because (1) the regressions using MAP are nearly as accurate as those using snowfall, (2) MAP is easier to estimate than snowfall, and (3) these regressions result in more consistent estimates at the extremes of the range of regression applicability.

Tables 5.2 to 5.4 provide an example application of the regression equations for the Upper Truckee River drainage area at USGS gage 10336580 near Meyers. As an example of applying the regressions, consider the computation of 0.01 exceedance probability discharge. The regression equation used is (see Table 5.1):

$$\log_{10}(Q_p)=b_0 + b_1\log_{10}(\text{area}) + b_2\log_{10}(\text{elevation}) + b_3\log_{10}(\text{MAP})$$

Apply this to the data in Table 5.2 with the regression coefficients shown in Table 5.1 and highlighted in bold in Table 5.2 to obtain:

$$\log_{10}(Q_p)= 23.3825 + 1.1254\log_{10}(14.09) -6.8861\log_{10}(8258.6) + 3.0215\log_{10}(51.9) = 2.886$$

Raising the log value to the base 10 of the logarithm gives the value shown in Table 5.3:

$$Q_p = 10^{2.8910} = 768 \text{ cfs}$$

An approximate 95% confidence interval bound on the regression curve can be computed using the average prediction error (avp) given in Table 5.1, as is shown in Table 5.4. Average prediction error is used because generalized least squares was employed to obtain the regression equations (note: avp is comparable to the standard error employed in estimating prediction error with ordinary least squares regression).

As an example, consider how the upper and lower confidence limits for the 0.01 probability quantile are computed. From the above computation, the $\log_{10}$ estimate is:

$$\log_{10}(Q_p) = 2.886$$

Table 5.1 gives the avp for each quantile, which , for this example, is shown in bold in Table 5.4. The approximate upper and lower confidence interval limits are given by either adding or subtracting two avp from the regression estimated flow:

$$\text{avp} = 0.18$$

$$\log_{10}(Qu) = 2.891 + 2(0.18) =3.246$$

$$\log_{10}(Ql) = 2.891 - 2(0.18) = 2.526$$

Raising the log value to the base 10 of the logarithm gives the value shown in Table 5.4:

$$Qu = 10^{3.246} = 1760 \text{ cfs}$$
$$Ql = 10^{2.526} = 335 \text{ cfs}$$

Figure 5.1 (page 27) provides a comparison of the peak flow-frequency curves estimated from the gage record and the regression equations tabulated in Table 5.4.

Table 5.1. Regional Regression for Peak Annual Quantiles (see Table 6.11, SPK 2005a)
(Regression equations should be limited to open land use drainage areas > 0.5 sq mi where a significant portion of drainage area exceeds 7000 ft msl. They should not be applied to local areas draining to the Upper Truckee River downstream of Meyers at Highway 50, nor to urban areas.)

[8]probability	constant (b_0)	[1]area (b_1)	[2]elevation (b_2)	[4]snow (b_3)	[5]se	[6]R^2	[7]avp
Best regression							
0.002	51.4905	1.0048	-14.1498	2.282	0.22	0.95	0.16
0.01	44.5481	0.9463	-12.4502	2.3831	0.19	0.96	0.15
0.02	41.0838	0.9222	-11.5941	2.4171	0.19	0.96	0.14
0.04	37.1691	0.9	-10.6206	2.4426	0.18	0.96	0.15
0.1	31.0127	0.874	-9.0837	2.4671	0.18	0.96	0.16
	constant (b_0)	[1]area (b_1)	[2]elevation (b_2)	[3]MAP (b_3)	[5]se	[6]R^2	[7]avp
Recommended regression							
[9]0.002	33.5078	1.1884	-9.3726	2.8118	0.29	0.91	0.20
0.01	**23.3825**	**1.1254**	**-6.8861**	**3.0215**	0.25	0.93	0.18
0.02	20.9166	1.0971	-6.3088	3.1346	0.22	0.94	0.17
0.04	16.8238	1.0678	-5.3176	3.2437	0.21	0.95	0.17
0.1	10.9192	1.0272	-3.8941	3.4092	0.19	0.96	0.16
0.2	5.7616	0.9957	-2.6617	3.5692	0.17	0.96	0.16
0.50	-5.4765	0.9553		3.9699	0.16	0.97	0.14
0.80	-6.2034	0.9493		4.2644	0.16	0.97	0.15
0.90	-6.5624	0.9454		4.4023	0.19	0.96	0.15
0.95	-6.8580	0.9428		4.5149	0.23	0.95	0.16
0.99	-7.4826	0.9402		4.7821	0.35	0.90	0.19

[1]drainage area (square miles) [2] mean basin elevation (feet msl) [3]mean annual precipitation (inches)
[4]mean annual snowfall (inches) [5]standard error ($\log_{10}$) [6]multiple coefficient of determination (adjusted) R^2 ($\log_{10}$) [7]average prediction error ($\log_{10}$)

[8]Best regression: application limited to drainage areas > 0.5 sq miles, basin average elevation > 7,000 ft msl (see discussion)
$\log_{10}(Q_p)=b_0 + b_1\log_{10}(\text{area}) + b_2\log_{10}(\text{elevation}) + b_3\log_{10}(\text{snow})$ p=0.1 to 0.002

[9]Recommended regression: application limited to drainage areas > 0.5 sq miles, basin average elevation > 7000 ft msl (see discussion)
$\log_{10}(Q_p)=b_0 + b_1\log_{10}(\text{area}) + b_2\log_{10}(\text{elevation}) + b_3\log_{10}(\text{MAP})$ p=0.2 to 0.002
$\log_{10}(Q_p)=b_0 + b_1\log_{10}(\text{area}) + b_2\log_{10}(\text{MAP})$ p=0.5 to 0.99
(recommended regressions result in predictions **10% less** than best regression predictions for all gages used in study)

Table 5.2. Example Regression Input Data for Peak Annual Stream Flow Regression

USGS ID	Name	[1]Area	[2]Elevation	[3]MAP
10336580	Upper Truckee River at S Upper Truckee Rd Near Meyers, CA	14.09	8258.6	51.9

[1]drainage area (square miles)
[2]elevation (feet msl)
[3]mean annual precipitation (inches)

Table 5.3. Regression-Computed Peak Annual Frequency Curve Example

[1]P	[2]0.002	0.01	0.02	0.04	0.1	0.2	[3]0.5	0.8	0.9	0.95	0.99
Q_p	964	**768**	695	610	494	399	269	159	119	93	63
b_0	33.5078	**23.3825**	20.9166	16.8238	10.9192	5.7616	-5.4765	-6.2034	-6.5624	-6.858	-7.4826
b_1	1.1884	**1.1254**	1.0971	1.0678	1.0272	0.9957	0.9553	0.9493	0.9454	0.9428	0.9402
b_2	-9.3726	**-6.8861**	-6.3088	-5.3176	-3.8941	-2.6617	0	0	0	0	0
b_3	2.8118	**3.0215**	3.1346	3.2437	3.4092	3.5692	3.9699	4.2644	4.4023	4.5149	4.7821

[1]exceedance probability, [2]flow quantile for exceedance probability
[2]$\log_{10}(Q_p)=b_0 + b_1\log_{10}(\text{area}) + b_2\log_{10}(\text{elevation}) + b_3\log_{10}(\text{MAP})$
[3]$\log_{10}(Q_p)=b_0 + b_1\log_{10}(\text{area}) + b_2\log_{10}(\text{MAP})$ p=0.5 to 0.99

Table 5.4. Regression-Computed Peak Annual Frequency Curve Confidence Limits Example

[1]Prob	Qg (1)	Q_p (2)	$\log_{10}Q_p$ (3)	avp (4)	+2avp (5)	-2avp (6)	Qu (7)	Ql (8)
0.002	2010	964	2.98404	0.20	3.389	2.578	2452	379
0.01	1420	**768**	**2.89077**	**0.18**	**3.251**	**2.530**	**1784**	**339**
0.02	1200	695	2.842189	0.17	3.187	2.496	1540	314
0.04	995	610	2.785279	0.17	3.118	2.451	1314	283
0.1	755	494	2.693595	0.16	3.015	2.371	1036	235
0.2	588	399	2.601412	0.16	2.917	2.285	826	193
0.5	373	269	2.429757	0.14	2.718	2.141	523	138
0.8	243	159	2.201056	0.15	2.494	1.907	312	81

[1]exceedance probability
(1) flow quantile for exceedance probability determined from gage record
(2) flow quantile for exceedance probability determined from regression
(3) $\log_{10}$ of regression quantile
(4) average prediction error (for $\log_{10}$ transformed flow)
(5) regression $\log_{10}$ flow quantile (column (2)) + twice the average prediction error
(6) regression $\log_{10}$ flow quantile (column (2)) - twice the average prediction error
(7) upper confidence limit on regression flow quantile prediction, $10^{\text{column(5)}}$
(8) lower confidence limit on regression flow quantile prediction, $10^{\text{column(6)}}$

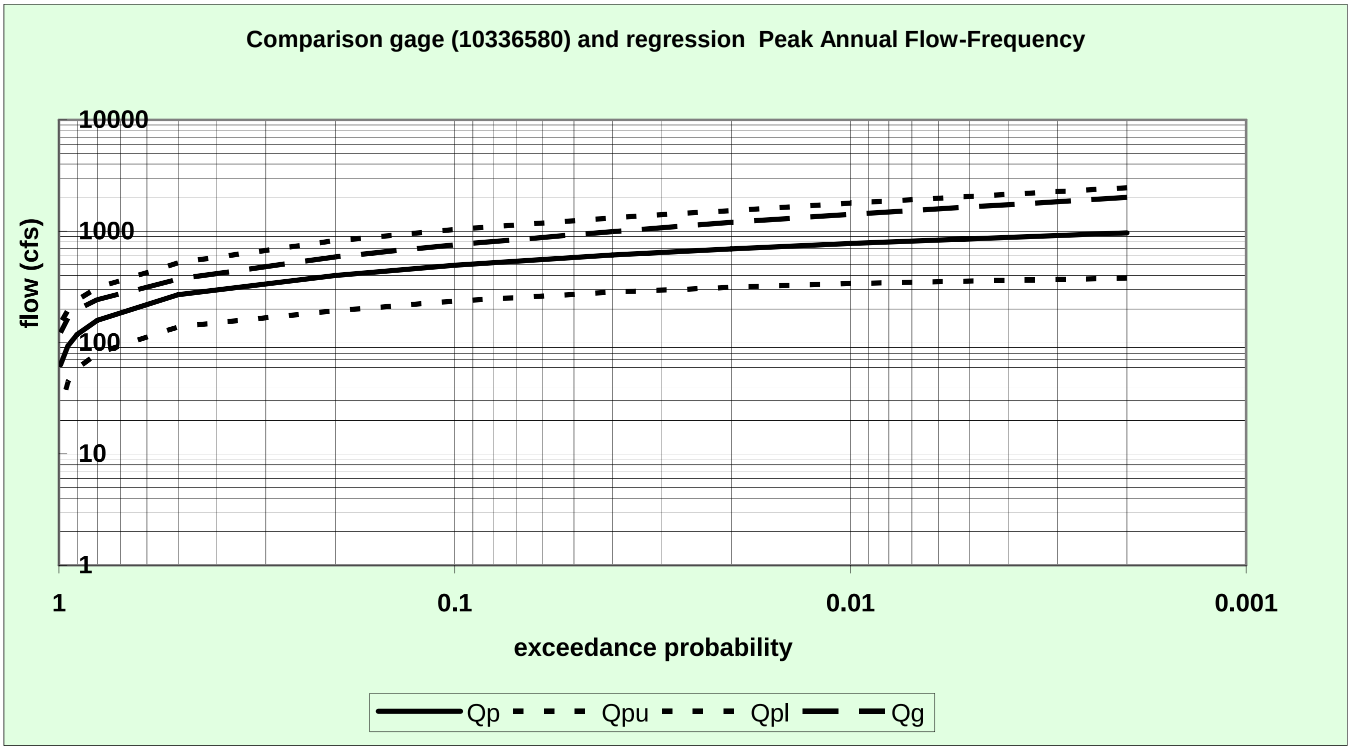

Figure 5.1. Comparison of Regression-Predicted (Qp) Flow-Frequency Curve and 2 Average Prediction Error Bounds (Qpu and Qpl) with Frequency Curve Estimated from Gage Period of Record (Qg)

5.2.2 Volume Duration Frequency Curve Regression

Ordinary least square regression relationships between peak and 1-day, and 1-day and other volume-duration frequency curves are shown in Tables 5.5 and 5.6. Consider, for example, the computation of the 1% exceedance probabilities 1-day and 3-day volumes, as shown in bold in Tables 5.7, using the example for the 1% peak discharge given in the previous example for the Upper Truckee. The 1% 1-day volume is computed as:

$$\log_{10}(Q_{1day}) = a + b[\log_{10}(Q_{peak})]$$

Substitute the regression computed peak and coefficients from Table 5.5 and as highlighted in bold in Table 5.7:

$$\log_{10}(Q_{1day}) = -0.09954 + 0.978921[\log_{10}(768)] = 2.725$$

The 1% 1-day flow is then obtained by raising the log value to base 10:

$$Q1day = 10^{2.725} = 531 \text{ cfs}$$

The 1% 3-day value is computed using the same regression equation form to obtain:

$$\log_{10}(Q_{3day}) = a + b[\log_{10}(Q_{1day})]$$

$$\log_{10}(Q_{3day}) = -0.00672 + 0.962746[\log_{10}(531)] = 2.6216$$

and then raising the log to base 10 to obtain the 1% 3-day exceedance value:

$$Q3day = 10^{2.6167} = 413 \text{ cfs}$$

The computation of the confidence limits on the volume-duration-frequency curve regression estimate is based on the peak discharge prediction error. Basically, the avp in e log-regression represents a fraction error, as can be seen from the basic regression equation:

$$\log_{10}(Q) = \log_{10}(Q^*) + e$$

where Q is the gage estimated flow quantile, Q* is the regression predicted quantile, and e is prediction error. Subtract the log regression prediction from both sides of the equation to obtain:

$$\log_{10}(Q) - \log_{10}(Q^*) = \log_{10}(Q/Q^*) = e$$

which shows the residual as a fraction error. The avp can be used as an average estimate of this error.

This avp error for peak discharges is assumed to quantify the same fraction error as for the prediction of the volume-duration-frequency curve because of the high correlation between peak and n-day volumes. Table 5.8 shows the application of avp to the 1-day regression estimated flow-frequency curve. Note that an average of the avp = 0.18 was obtained from Table 5.1 and applied to obtain the approximate confidence intervals. A comparison of the 1-day frequency curve estimated from the Upper Truckee gage and from the regression equation is shown in Table 5.8 and displayed in Figure 5.2.

Table 5.5. Lake Tahoe Basin Regression Relationships Between Peak Annual Quantile and 1-Day Annual Maximum (based on log-Pearson III estimates from gage analysis, see Table 8.1, SPK, 2005a)

(Regression equations should be limited to open land use drainage areas > 0.5 sq mi where a significant portion of drainage area exceeds 7000 ft msl. They should not be applied to local areas draining to the Upper Truckee River downstream of Meyers at Highway 50, nor to urban areas.)

	0.99	0.95	0.9	0.8	0.5	0.2	0.1	0.04	0.02	0.01	0.002
b	0.958596	0.990323	0.97329	0.979087	0.988666	0.978598	0.972665	0.973848	0.974342	0.978921	0.979076
a	0.048461	-0.015	0.010924	-0.01762	-0.08182	-0.1054	-0.10293	-0.10213	-0.09836	-0.09954	-0.09004
correlation	0.997605	0.998376	0.996842	0.99676	0.995927	0.99353	0.990794	0.986736	0.984563	0.982225	0.980661

[1] $\log_{10}(Q_{1day}) = \mathbf{a} + \mathbf{b}[\log_{10}(Q_{peak})]$, where Q_{1day} is the 1-day duration quantile (e.g., 1-day 0.01 exceedance probability flow (cfs/day)) and Q_{peak} is the quantile for the annual maximum peak flow (cfs)

**Table 5.6. Lake Tahoe Basin Regression Relationships
Between 1-Day Quantile and Other Duration Quantiles
(based on log-Pearson III estimates from gage analysis, see Table 8.2, SPK 2005a)**

(Regression equations should be limited to open land use drainage areas > 0.5 sq mi where a significant portion of drainage area exceeds 7000 ft msl. They should not be applied to local areas draining to the Upper Truckee River downstream of Meyers at Highway 50, nor to urban areas.)

[1]probability	[2]constants/correlation	3-day	7-day	10-day	15-day	30-day
0.99	b	0.993308	0.99025	0.982075	0.96804	0.944451
	a	-0.01257	-0.03303	-0.04046	-0.05078	-0.07836
	correlation	1.000	0.998	0.998	0.997	0.995
0.95	b	0.985982	0.971634	0.962417	0.951393	0.934651
	a	-0.00648	-0.01888	-0.02567	-0.04312	-0.08261
	correlation	1.000	0.998	0.998	0.998	0.997
0.90	b	0.99923	0.987621	0.983363	0.975626	0.961631
	a	-0.03614	-0.06279	-0.07952	-0.10316	-0.14471
	correlation	0.999	0.998	0.998	0.998	0.997
0.80	b	0.996301	0.98171	0.973487	0.982671	0.968724
	a	-0.03775	-0.06061	-0.06864	-0.12775	-0.1647
	correlation	0.999	0.999	0.998	0.998	0.998
0.50	b	0.998265	0.987621	0.981524	0.978761	0.965912
	a	-0.05056	-0.08833	-0.10181	-0.13463	-0.16694
	correlation	1.000	0.999	0.999	0.998	0.998
0.20	b	0.99221	0.975531	0.970496	0.963261	0.950199
	a	-0.04933	-0.07828	-0.09573	-0.11602	-0.14453
	correlation	1.000	0.998	0.998	0.997	0.996
0.10	b	0.983585	0.958233	0.950237	0.939259	0.924215
	a	-0.03924	-0.05574	-0.06823	-0.07869	-0.10142
	correlation	0.999	0.997	0.996	0.995	0.994
0.04	b	0.970159	0.926978	0.911073	0.894943	0.875257
	a	-0.01914	-0.00588	-0.00369	-0.00217	-0.01318
	correlation	0.999	0.995	0.993	0.990	0.988
0.02	b	0.966892	0.906824	0.886033	0.86466	0.841265
	a	-0.012	0.029517	0.04166	0.055717	0.052658
	correlation	0.998	0.992	0.989	0.986	0.982
0.01	b	0.962746	0.884046	0.854176	0.831611	0.803941
	a	-0.00672	0.070869	0.106669	0.122217	0.128375
	correlation	0.998	0.994	0.999	0.999	0.999
0.002	b	0.976006	0.859373	0.819634	0.783623	0.744137
	a	-0.03389	0.118144	0.170341	0.221591	0.261264
	correlation	0.997	0.986	0.978	0.967	0.958

[1]Exceedance probability
[2]$\log_{10}(Q_{nday}) = \mathbf{a} + \mathbf{b}[\log_{10}(Q_{1day})]$, where Q_{nday} is the duration quantile (e.g., 3-day 0.01 exceedance probability (cfs/day)), and Q_{1day} is the quantile for the 1-day volume duration frequency curve (cfs/day)

Table 5.7. Regression Computation of Volume Duration Frequency Curves Example

Prob	0.002	0.01	0.02	0.04	0.1	0.2	0.5	0.8	0.9	0.95	0.99
Qpeak	964	**778**	695	610	494	399	269	159	119	93	63
[1]Q1day	679	**531**	469	408	329	276	209	137	107	86	59
a	-0.09004	**-0.09954**	-0.09836	-0.10213	-0.10293	-0.1054	-0.08182	-0.01762	0.010924	-0.015	0.048461
b	0.979076	**0.978921**	0.974342	0.973848	0.972665	0.978598	0.988666	0.979087	0.97329	0.990323	0.958596
[2]Q3day	537	**419**	372	326	273	236	184	124	98	80	56
Q7day	356	305	283	259	227	201	160	109	87	73	53
Q10day	310	275	256	237	211	187	150	103	82	69	50
Q15day	276	247	232	216	193	172	137	94	75	63	46
Q30day	234	211	199	187	168	149	119	81	64	53	39
B											
3day	0.976006	**0.962746**	0.966892	0.970159	0.983585	0.99221	0.998265	0.996301	0.99923	0.985982	0.993308
7day	0.859373	0.884046	0.906824	0.926978	0.958233	0.975531	0.987621	0.98171	0.987621	0.971634	0.99025
10day	0.819634	0.854176	0.886033	0.911073	0.950237	0.970496	0.981524	0.973487	0.983363	0.962417	0.982075
15day	0.783623	0.831611	0.86466	0.894943	0.939259	0.963261	0.978761	0.982671	0.975626	0.951393	0.96804
30day	0.744137	0.803941	0.841265	0.875257	0.924215	0.950199	0.965912	0.968724	0.961631	0.934651	0.944451
A											
3day	-0.03389	**-0.00672**	-0.012	-0.01914	-0.03924	-0.04933	-0.05056	-0.03775	-0.03614	-0.00648	-0.01257
7day	0.118144	0.070869	0.029517	-0.00588	-0.05574	-0.07828	-0.08833	-0.06061	-0.06279	-0.01888	-0.03303
10day	0.170341	0.106669	0.04166	-0.00369	-0.06823	-0.09573	-0.10181	-0.06864	-0.07952	-0.02567	-0.04046
15day	0.221591	0.122217	0.055717	-0.00217	-0.07869	-0.11602	-0.13463	-0.12775	-0.10316	-0.04312	-0.05078
30day	0.261264	0.128375	0.052658	-0.01318	-0.10142	-0.14453	-0.16694	-0.1647	-0.14471	-0.08261	-0.07836

[1]$\log_{10}(Q_{1day}) = \mathbf{a} + \mathbf{b}[\log_{10}(Q_{peak})]$, where Q_{1day} is the 1-day duration quantile (e.g., 1-day 0.01 exceedance probability flow (cfs/day)), and Q_{peak} is the quantile for the annual maximum peak flow (cfs)
[2]$\log_{10}(Q_{nday}) = \mathbf{a} + \mathbf{b}[\log_{10}(Q_{1day})]$, where Q_{nday} is the duration quantile (e.g., 3-day 0.01 exceedance probability, (cfs/day)) , and Q_{1day} is the quantile for the 1-day volume/duration frequency curve (cfs/day)

Table 5.8. Computation of Regression Confidence Limits 1-Day Annual Maximum Flow-Frequency Curve Example

[1]Prob	Qg (1)	Q (2)	$\log_{10}$Q (3)	avp (4)	+2avp (5)	-2avp (6)	Qu (7)	Ql (8)
0.002	1185	679	2.831562	0.176	3.183562	2.479562	1527	302
0.01	886	531	2.725001	0.176	3.077001	2.373001	1194	236
0.02	769	469	2.670904	0.176	3.022904	2.318904	1055	208
0.04	657	408	2.610308	0.176	2.962308	2.258308	917	181
0.1	517	329	2.517035	0.176	2.869035	2.165035	740	146
0.2	414	276	2.440336	0.176	2.792336	2.088336	620	123
0.5	273	209	2.320399	0.176	2.672399	1.968399	471	93
0.8	182	137	2.137405	0.176	2.489405	1.785405	309	61

[1]exceedance probability
(1) flow quantile for exceedance probability determined from gage record (2) flow quantile for exceedance probability determined from regression (3) $\log_{10}$ of regression quantile (4) average prediction error (for $\log_{10}$ transformed flow)
(5) regression $\log_{10}$ flow quantile (column (2)) + twice the average prediction error (6) regression $\log_{10}$ flow quantile (column (2)) - twice the average prediction error (7) upper confidence limit on regression flow quantile prediction, $10^{column(5)}$ (8) lower confidence limit on regression flow quantile prediction, $10^{column(6)}$

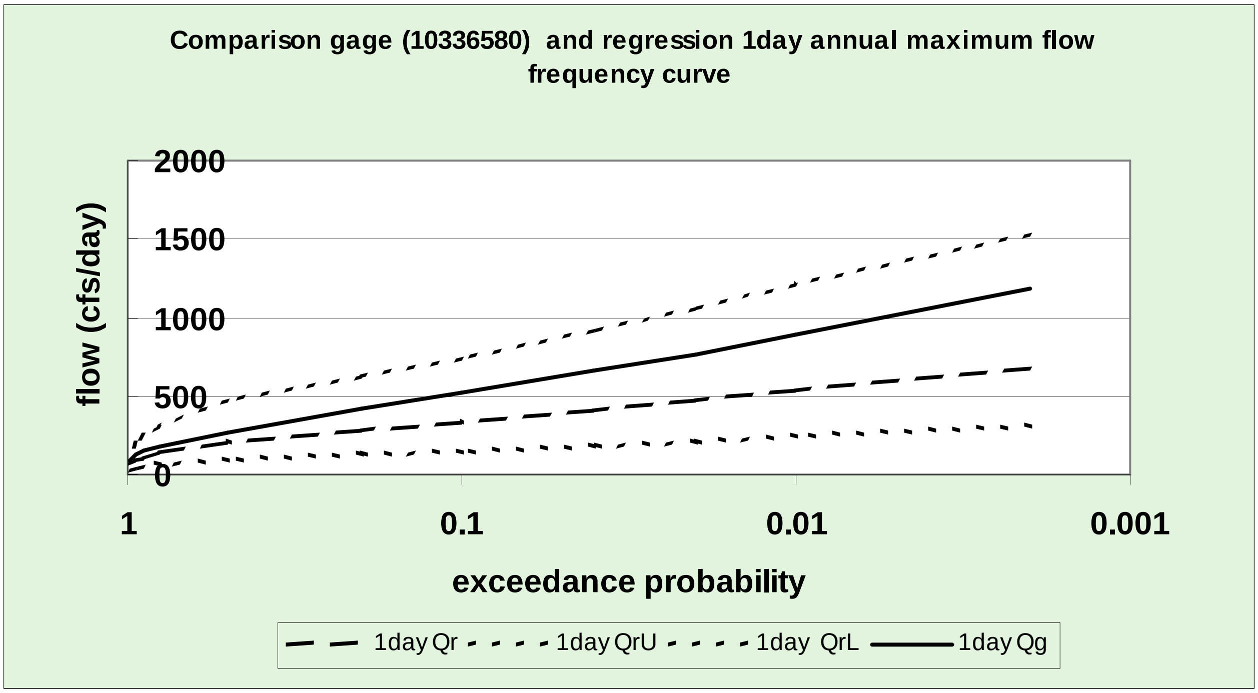

Figure 5.2. Comparison of Regression-Predicted (Qr) 1-Day Volume Flow-Frequency Curve and 2 Average Prediction Error Bounds (QrU and QrL) with Frequency Curve Estimated from Gage Period of Record (Qg)

5.3 7-Day Low-Flow-Frequency Curve

The 7-day flow has some importance to regulatory water quality applications. The annual 7-day low-flow is the smallest 7-day average flow rate occurring during the year (i.e., the lowest consecutive 7-day volume expressed as an average flow rate). The 7-day low-flow-frequency curve provides the likelihood that the 7-day low flow will not be exceeded in a given year.

Table 5.9 provides the recommended regression equations for predicting the 7-day low-flow-frequency curves. As in the case of the peak flow-frequency curves described in section 5.2, including snowfall in the regression improves the prediction error, but not significantly, for the critical, less frequent non-exceedance probabilities, such as the 0.10 (the 7-day/10-year low flow). The recommended regressions result in more consistent frequency curves near the extreme of the range of regression equation applicability. Tables 5.10-5.12 provide an example application of the regression, and Figure 5.3 displays a comparison of the regression equation and confidence limits with the frequency curve obtained from gage estimates. Refer also to Section 5.2.1, where the discussion of regression application to peak annual frequency curves is more detailed and analogous to the application to the 7-day low-flow-frequency curves.

Table 5.9. 7-Day Low-Flow Regional Regression Relationship[1] (see Table 9.4, SPK 2005a)

(Regression equations should be limited to open land use drainage areas > 0.5 sq mi where a significant portion of drainage area exceeds 7000 ft msl. They should not be applied to local areas draining to the Upper Truckee River downstream of Meyers at Highway 50, nor to urban areas.)

[2]Probability	b_0	[3]area (b_1)	[4]snowfall (b_2)	[5]temperature (b_3)	[6]R^2	[7]SE
Recommended regression						
0.01	133.84415	0.68033	0	-83.20121	0.77	0.46
0.05	107.53622	0.58155	0	-66.80492	0.80	0.35
0.10	106.50728	0.57185	0	-66.10442	0.82	0.32
0.20	97.14648	0.54907	0	-60.24327	0.87	0.27
0.50	74.74878	0.50574	0	-46.26403	0.86	0.23
0.80	57.96734	0.47266	0	-35.75592	0.78	0.25
0.90	50.49741	0.45584	0	-31.06690	0.71	0.27
Best regression						
0.20	111.07000	0.68248	-0.86005	-67.65282	0.86	0.26
0.50	92.88154	0.67949	-1.12005	-55.91357	0.90	0.18
0.80	80.95735	0.69295	-1.42008	-47.99028	0.89	0.16
0.90	76.48834	0.70488	-1.60545	-44.89824	0.88	0.16

[1]$\log_{10}(Q_p) = b_0 + b_1(\log_{10}(\text{area})) + b_2(\log_{10}(\text{snowfall})) + b_3(\log_{10}(\text{temperature})$, Q_p is the flow (cfs) for cumulative (non-exceedance probability), see SPK (2005) for example application
[2]cumulative probability (non-exceedance), e.g., 0.10 is the 10-year return interval for the 7-day low flow
[3]regression coefficient for area (square miles)
[4]regression coefficient for watershed average mean annual snowfall (inches)
[5]regression coefficient for watershed average mean annual temperature (^oF)
[6]adjusted multiple coefficient of determination (log units)
[7]standard error (log-unit)

Table 5.10. Example Regression Input Data for 7-Day Low-Flow Regression

USGS ID	Name	area	MAT
10336740	Logan House Ck near Glenbrook	2.09	43.7

Table 5.11. Regression Computation of 7-Day Low-Flow-Frequency Curve Example

[1]Prob	**0.01**	**0.05**	**0.1**	**0.2**	**0.5**	**0.8**	**0.9**
[2]Q	0.004	0.013	0.018	0.031	0.104	0.290	0.477
constant (b_0)	133.8442	107.5362	106.5073	97.14648	74.74878	57.96734	50.49741
area (b_1)	0.68033	0.58155	0.57185	0.549066	0.505742	0.472662	0.45584
Temperature(b_3)	-83.2012	-66.8049	-66.1044	-60.2433	-46.264	-35.7559	-31.0669

[1]cumulative probability (**non-exceedance**), e.g., 0.10 is the 10-year return interval for the 7-day low flow
[2]$\log_{10}(Q_p) = b_0 + b_1(\log_{10}(\text{area})) + b_3(\log_{10}(\text{temperature})$, Q_p is the flow (cfs) for cumulative (non-exceedance probability), see SPK (2005) for example application

Table 5.12. Computation of Regression Confidence Limits
7-Day Low-Flow-Frequency Curve Example

Prob[1]	Qg (1)	Q (2)	$\log_{10}$Q (3)	std err (4)	+2std err (5)	-2std err (6)	Qu (7)	Ql (8)
0.01	0.001	0.004	-2.43	0.46	-4.40	-3.35	0.03	0.00
0.05	0.01	0.013	-1.87	0.35	-3.39	-2.57	0.07	0.00
0.1	0.015	0.018	-1.75	0.32	-3.19	-2.39	0.08	0.00
0.2	0.02	0.031	-1.51	0.27	-2.74	-2.05	0.11	0.01
0.5	0.09	0.104	-0.98	0.23	-1.74	-1.45	0.30	0.04
0.8	0.28	0.290	-0.54	0.25	-0.83	-1.03	0.91	0.09
0.9	0.48	0.477	-0.32	0.27	-0.37	-0.86	1.66	0.14

[1]**non**-exceedance probability
(1) flow quantile for exceedance probability determined from gage record
(2) flow quantile for exceedance probability determined from regression
(3) $\log_{10}$ of regression quantile
(4) average prediction error (for $\log_{10}$ transformed flow)
(5) regression $\log_{10}$ flow quantile (column (2)) + twice the average prediction error
(6) regression $\log_{10}$ flow quantile (column (2)) - twice the average prediction error
(7) upper confidence limit on regression flow quantile prediction, $10^{\text{column(5)}}$
(8) lower confidence limit on regression flow quantile prediction, $10^{\text{column(6)}}$

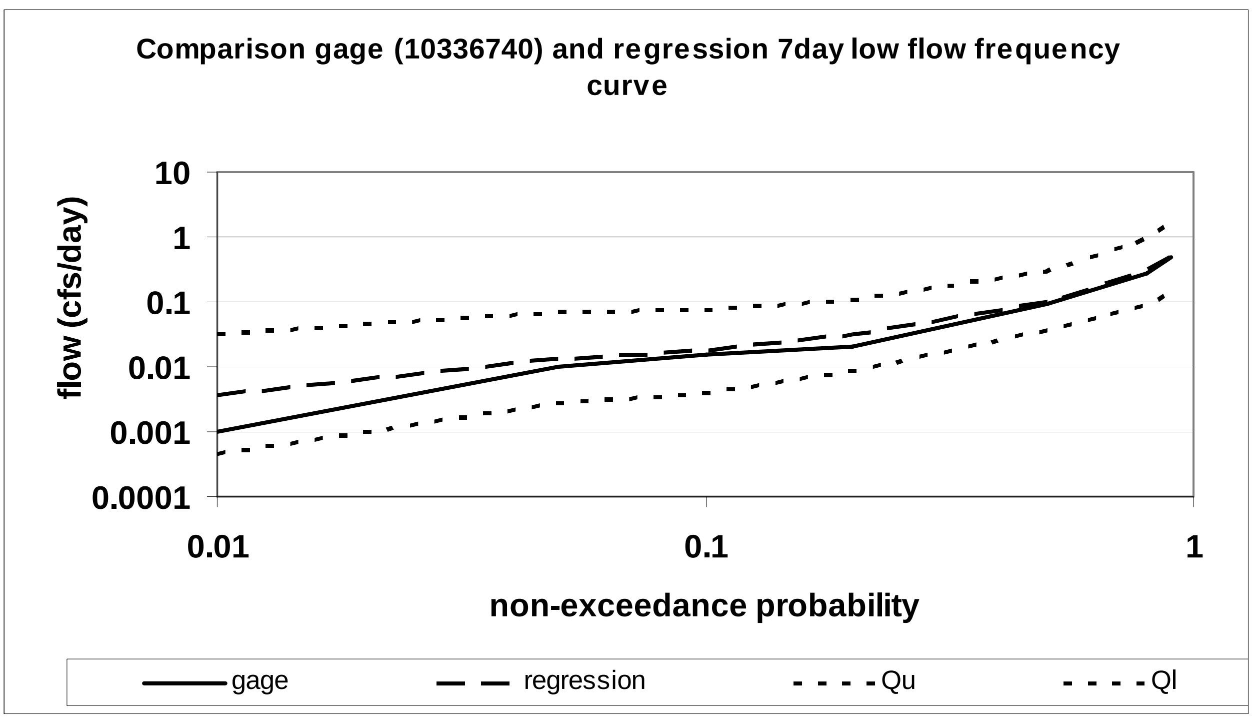

Figure 5.3. Comparison of Regression-Predicted (regression) 7-Day Volume Low-Flow-Frequency Curve and 2 Average Prediction Error Bounds (Qu and Ql) with Frequency Curve Estimated from Gage Period of Record (gage)

5.4 Flow Duration Curves

Tables 5.13 and 5.14 provide the regression equations for the daily annual flow-duration curve. The recommended regression for the 50% exceedance (or equivalently the fraction exceeded 50% of the time) uses mean annual precipitation (MAP) rather than mean annual temperature (MAT), even though the regression with MAT gives a slight improvement in accuracy. Using MAP results in more consistent predictions for applications at the extreme of the regression range of applicability. Tables 5.15-5.17 provide an example application and Figure 5.4 provides a comparison of the regression-predicted curve and confidence limits with the gage-estimated curve.

Table 5.13. Lake Tahoe Watersheds Daily Flow Duration Regression Relationship Parameters (see Table 10.2, SPK 2005a)

(Regression equations should be limited to open land use drainage areas > 0.5 sq mi where a significant portion of drainage area exceeds 7000 ft msl. They should not be applied to local areas draining to the Upper Truckee River downstream of Meyers at Highway 50, nor to urban areas.)

[5]Frequency exceeded (f)	b_0	[1]Area (b_1)	[2]Elevation (b_2)	[3]MAT (b_3)	[4]MAP (b_4)
99%	-43.8641	0.927195	11.04962	0	0
95%	-38.8409	0.945971	9.789445	0	0
90%	-32.7125	0.970529	8.235106	0	0
50%	32.85813	0.80133	0	-20.24583805	0
[7]50%	-1.64067	0.89692	0	0	0.942848
10%	-4.21429	0.85337	0	0	3.011556
5%	-4.11273	0.889998	0	0	3.038292
1%	-3.97303	0.965017	0	0	3.042417

[1]drainage area (square miles)
[2]mean basin elevation (feet msl)
[3] watershed average mean annual temperature (°F)
[4]watershed average mean annual precipitation
[5]annual frequency daily flow level (cfs/day) exceeded
[6]Flow duration curve regression, $\log_{10}(Q_f) = b_0 + b_1\log_{10}(\text{area})\ b_2\log_{10}(\text{elevation}) + b_3\log_{10}(\text{MAT}) + b_4\log_{10}(\text{MAP})$
[7]Recommend regression for 50% frequency exceeded flow, although slightly better R^2 using MAT rather than MAP

Table 5.14. Lake Tahoe Watersheds Daily Flow Duration Regression Goodness-of-Fit and Prediction Error (see Table 10.3, SPK 2005a)

[1]Frequency exceeded	[2]Adjusted R^2	[3] Standard error
99%	0.86	0.18
95%	0.87	0.18
90%	0.90	0.15
50%	0.91	0.15
[4]50%	0.87	0.18
10%	0.96	0.13
5%	0.96	0.13
1%	0.95	0.15

[1] annual frequency daily flow level (cfs/day) exceeded
[2]log regression multiple coefficient of determination (adjusted for degrees of freedom)
[3] standard error $\log_{10}$ units
[4]Recommend regression for 50% frequency exceeded flow, although slightly better R^2 using MAT rather than MAP

Table 5.15. Example Regression Input Data for Annual Daily Flow-Duration Curve

USGS ID	Name	Area	Elevation	MAP
103367592	Eagle Rock Ck NR Stateline, NV	0.63	8286.3	31.1

Table 5.16. Regression Computation of Annual Flow-Duration Frequency Curve

[1]Prob	99%	95%	90%	50%	10%	5%	1%
[2]Q_f	0.176288	0.212734	0.229537	0.385756	1.283652	1.748052	2.362417
constant (b_0)	-43.8641	-38.8409	-32.7125	-1.64067	-4.21429	-4.11273	-3.97303
area (b_1)	0.927195	0.945971	0.970529	0.89692	0.85337	0.889998	0.965017
elevation (b_2)	11.04962	9.789445	8.235106	0	0	0	0
MAP (b_4)	0	0	0	0.942848	3.011556	3.038292	3.042417

[1]exceedance probability (or fraction exceeded)
[2]$\log_{10}(Q_f) = b_0 + b_1\log_{10}(\text{area}) + b_2\log_{10}(\text{elevation}) + b_4\log_{10}(\text{MAP})$

Table 5.17. Computation of Regression Confidence Limits Annual Flow-Duration Frequency Curve

Prob	Qg (1)	Q_f (2)	$\log_{10}Q$ (3)	std err (4)	+2std err (5)	-2std err (6)	Qu (7)	Ql (8)
0.99	0.31	0.18	-0.75	0.18	-0.39	-1.11	0.40	0.08
0.95	0.36	0.21	-0.67	0.18	-0.31	-1.03	0.49	0.09
0.9	0.39	0.23	-0.64	0.15	-0.34	-0.94	0.46	0.12
0.5	0.66	0.39	-0.41	0.18	-0.05	-0.77	0.88	0.17
0.1	1.36	1.28	0.11	0.13	0.37	-0.15	2.34	0.71
0.05	1.45	1.75	0.24	0.13	0.50	-0.02	3.18	0.96
0.01	1.65	2.36	0.37	0.15	0.67	0.07	4.71	1.18

(1) flow quantile for fraction exceeded determined from gage record
(2) flow quantile for fraction exceeded determined from regression
(3) $\log_{10}$ of regression quantile
(4) average prediction error (for $\log_{10}$ transformed flow)
(5) regression $\log_{10}$ flow quantile (column (2)) + twice the average prediction error
(6) regression $\log_{10}$ flow quantile (column (2)) - twice the average prediction error
(7) upper confidence limit on regression flow quantile prediction, $10^{\text{column(5)}}$
(8) lower confidence limit on regression flow quantile prediction, $10^{\text{column(6)}}$

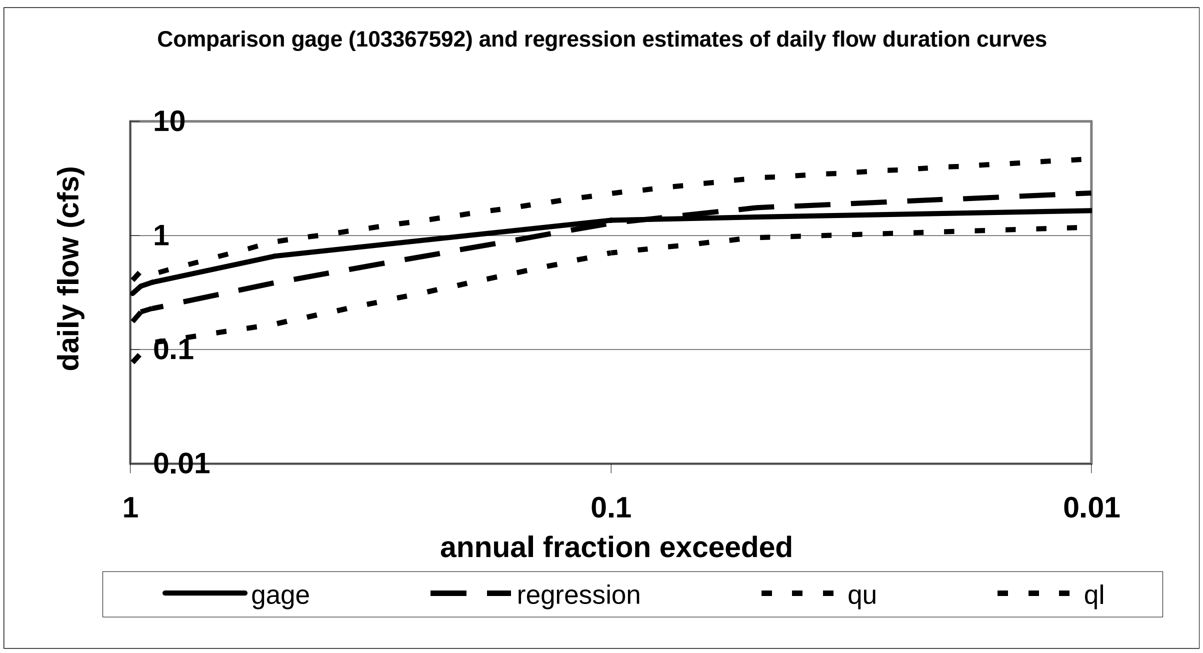

Figure 5.4. Comparison of Regression-Predicted (regression) Daily Annual Flow-Duration Curve and 2 Average Prediction Error Bounds (qu and ql) with Frequency Curve Estimated from Gage Period of Record (gage)

Section 5.5. <u>Comparison of Peak Annual Regression Equation Estimates</u>

The Lake Tahoe peak annual flow regression predictions (Table 5.1) of the 1% peak annual flow were compared with those obtained from the regional gages (see Table 2.1, SPK 2005b), those available from the USGS (see Blakemoore, et al., 1997), and from a study done by HYDMET (see Schivley and Klide, 2004). The USGS regressions used gages for a much larger area than the Tahoe Basin used in this study, covering the southern range of the Sierra Nevada. See SPK (2005a) Table 6.13 for the gages used in the HYDMET study. The GLS regression using regional gages covers an area and number similar to that of the USGS study.

The regression comparisons of the 1% exceedance peak annual discharges demonstrated large difference between the USGS and this study's estimates, while, on the average, agreement is better for the comparison of the HYDMET with this study's regression estimates (see Table 5.18). The differences in predictions with regional gage regression estimates were significantly smaller than the USGS equations, but still significant. The difference with the USGS regression predictions can be explained by the very different sources of data employed in both studies. The same probably can be said for the differences found in comparison with the regional gage regression equations. Although agreement was obtained on the average, there was also a significant east-west location bias in the regression prediction differences with the HYDMET data. A sensitivity analysis of the Eagle Rock Creek gage peak annual frequency curve showed that the HYDMET regressions over-predicted the 1% discharge for the eastern Lake Tahoe gages (see section 6.4, SPK 2005a for a further discussion). The HYDMET equations' smaller predictions compared with those obtained in this study for the western gages is probably due to the lack of western gages used in the HYDMET analysis.

Table 5.18. Comparison of Regression Equation Estimates

Location	USGS ID	area (sq-mi)	elevation (ft)	latitude (degrees)	[1]MAP (inches)	(1)	(2)	(3)	(4)	(5)	(6)	(7)
UPPER TRUCKEE	10336580	14.09	8258.59	38.79630	51.9	1009.3	422.7	-0.58	1484.7	0.47	790.2	-0.22
UPPER TRUCKEE	10336600	33.1	8042.35	38.84296	50.4	2636.7	993	-0.62	3632.2	0.38	1665.5	-0.37
UPPER TRUCKEE	103366092	34.28	7996.26	38.84852	51.8	3119.0	1028.4	-0.67	4135.1	0.33	1737.4	-0.44
UPPER TRUCKEE	10336610	54.9	7614.23	38.92241	47.0	4825.9	1647	-0.66	6299.6	0.31	2882.5	-0.40
TAYLOR	10336626	16.7	7598.62	38.92157	50.9	1815.4	501	-0.72	2342.1	0.29	1141.2	-0.37
LAKE TAHOE TRIB	10336635	0.64	7106.5	39.01741	44.6	50.4	19.2	-0.62	79.8	0.59	107.3	1.13
GENERAL	10336645	7.44	7196.71	39.05185	48.4	868.1	223.2	-0.74	1125.6	0.30	721.0	-0.17
BLACKWOOD	10336660	11.2	7262.68	39.10741	54.8	1983.0	336	-0.83	2261.9	0.14	995.9	-0.50
WARD	10336674	4.96	7531.76	39.14074	67.6	1411.4	148.8	-0.89	1467.7	0.04	493.3	-0.65
WARD	10336675	8.97	7341.47	39.13685	62.1	2280.9	269.1	-0.88	2381.6	0.04	827.7	-0.64
WARD	10336676	9.7	7288.91	39.13213	60.1	2299.4	291	-0.87	2437.4	0.06	892.1	-0.61
WOOD	10336693	1.69	8198.86	39.26130	41.6	49.7	50.7	0.02	101.0	1.03	185.6	2.74
GLENBROOK	10336730	4.11	7349.24	39.08741	26.6	49.0	123.3	1.52	121.3	1.47	439.5	7.97
LOGAN HOUSE	10336740	2.09	7816.76	39.06657	29.7	24.8	62.7	1.53	63.6	1.56	224.4	8.04
EDGEWOOD	10336756	0.81	7615.31	38.97546	28.3	8.8	24.3	1.77	23.9	1.72	109.0	11.42
EDGEWOOD	103367585	3.13	7529.35	38.96657	29.0	43.6	93.9	1.15	104.8	1.41	320.3	6.35
EAGLE ROCK	103367592	0.63	8286.26	38.95657	31.1	5.8	18.9	2.25	16.9	1.90	73.9	11.72
TROUT	10336770	7.4	8606.66	38.86324	42.4	197.2	222	0.13	391.5	0.98	449.1	1.28
TROUT	10336775	23.7	7820.54	38.90339	40.7	1019.3	711	-0.30	1675.8	0.64	1398.6	0.37
TROUT	10336780	36.7	7931.58	38.91991	38.8	1261.7	1101	-0.13	2171.6	0.72	1922.5	0.52
					average			-0.01		0.72		2.36
					max			2.25		1.90		11.72
					min			0.02		0.04		0.17

[1]Mean annual precipitation

(1) $\log_{10}(Q_{1\%}) = 24.8748 + 1.1256\log_{10}(\text{area}) - 7.2637\log_{10}(\text{elevation}) + 3.3025\log_{10}(\text{MAP})$ (see Table 5.1)

(2) $Q_{1\%} = 30.0\,(\text{area})$ (see Shively and Clyde, 2004)

(3) fraction difference = [(2)-(1)]/(1)

(4) $\log_{10}(Q_{1\%}) = 13.1691 + 1.0121\log_{10}(\text{area}) - 3.9758\log_{10}(\text{elevation}) + 2.5728\log_{10}(\text{MAP})$ (see Table 2.1, SPK 2005b)

(5) fraction difference = [(4)-(1)]/(1)

(6) $\log10(Q_{1\%}) = \log_{10}(7000) + 0.782\log_{10}(\text{area}) - 2.18\log_{10}(\text{elevation}/1000) + 4.6\log_{10}([\text{latitude}-28]/10)$ (Blakemoore, et al., 1997)

(7) fraction difference = [(6)-(1)]/(1)