Des Moines River
Regulated Flow Frequency Study

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Executive Summary

a. The flow frequency analysis was performed to determine the peak annual maximum regulated flow frequency curves at specific locations on the Des Moines River. The 1day annual maximum regulated frequency curves were obtained by integrating a critical duration unregulated flow volume frequency curve and a regulated versus unregulated relationship at Saylorville and Red Rock dams and at SE6th Street. A drainage area adjustment to the unregulated 1day annual maximum regulated frequency curves combined with reservoir holdouts to obtain regulated frequency curves at Second Avenue,. Ottumwa and Keosauqua. The 1day regulated frequency curves were then converted to a peak curve using a regression relationship between peak and daily flows.

b. The assumptions made that result in limitations in the value of flood frequency analysis are as follows:

- Annual maximum floods result from a stationary random process. Practically speaking, this requires that the influence of climatic variability is small over the planning period relevant to the risk measures estimated by a flood frequency analysis.
- Flow discharges are measured without error.
- The flood record is homogenous, not being influenced by anthropomorphic activities, such as regulation, channel modification and land use change.
- Annual maximum floods can be described using a single flow-frequency distribution irrespective of magnitude.

Research sponsored by the Corps of Engineers for the recent Upper Mississippi Flow Frequency Study (see Corps of Engineers, 2003, Matalas and Olsen, 2001, and Stedinger et al., 2001) indicate that the stationary assumption is a useful approximation for applications of frequency analysis in planning studies.

Measurement error of very large floods is likely to have the greatest influence on frequency curve estimation for infrequent quantiles such as the 100-year flood. A large flow with significant measurement error can be recognized as high outlier by statistical tests used as part of the frequency analysis. If a high outlier is recognized then the flow measurement datum should be reexamined.

The assumption regarding flow homogeneity can be mitigated by either choosing the period of record or by performing analyses to remove the effects of such activities (e.g., regulation). In this study, a significant effort was made to remove the affects of regulation and the period of record was chosen to avoid the influence of land use change.

c. The federal guidelines for performing flow frequency analysis, Bulletin 17B (IACWD, 1982) were applied to estimate the annual maximum volume duration frequency curves. The assumptions described in item (b) dictate that this frequency analysis method, like any other of the well recognized methods, is approximate. A comparison of methods has shown (see Thomas and Eash, 1993, and Corps of Engineers, 2000a) that Bulletin 17B and other methods agree reasonably well on the average when predictions are compared over a large number of gages within the range of empirically estimated flow frequencies (e.g., from plotting positions).

As expected, the difference between individual predictions increases with return interval. The difference between individual predictions is due to both sampling and model errors. Sampling error is due to the limited record lengths available for estimation of flow frequency distribution parameters. Model error results because any selected flow frequency distribution (e.g., log-Pearson III, log-Normal, generalized extreme value, etc.) is at best an approximation because of the assumptions made in performing a frequency analysis. Sampling error is quantifiable by statistical measures error such as confidence limits. However, model error is not quantifiable and becomes increasingly important as the predicted return interval increases. However, Hosking and Wallis (1997) examined the potential for model error of typically available stream flow record lengths by comparing predictions of different flow frequency distributions. They found that model error is increasingly important for predictions exceeding the 100-year return interval. Practically speaking, this means that sampling and model error need to be appreciated when selecting safety factors for flood damage reduction measures.

In this study, the significance of model error was investigated by comparing the predictions of the distribution used in Bulletin 17B, the log-Pearson III (lpiii), and, the prediction obtained using the L-moment estimation procedure developed by Hosking and Wallis (1997). This comparison provides some perspective on the consequences of choosing a particular distribution for estimating flow frequency curves.

d. The unregulated flow period of record that was used in the flow frequency analysis was estimated from the available period of regulated flow using a routing model. Possible errors in estimating these regulated flows were investigated using trend and double mass curve analysis. The trend analysis did not identify any anomalous or unusual patterns in the estimate regulated flow record. Double mass curve analysis is performed to determine if any change in a relationship between gage measurements, such as flow values, has occurred over time.. This analysis, applied to the period of record flows at the Des Moines River locations (both the observed record prior to reservoir construction and estimated unregulated flows after construction) and observed flows on the unregulated Cedar River, demonstrated no change in the relationship between gages. Consequently, the estimated Des Moines River unregulated flows are consistent with the period of record prior to construction of the Saylorville and Red Rock Dams.

e. The volume duration frequency (VDF) curve analysis was performed on unregulated flows for both the Iowa and Des Moines Rivers to obtain an average regional skew estimate needed for application of Bulletin 17B. Table 5.5 shows the estimated exceedance probability for the 1993 flood for various durations from this analysis.

Table 5.5: 1993 Event Exceedance Probability for each duration, average exceedance probability and return interval

	1DAY	15DAY	30DAY	60DAY	90DAY	105DAY	120DAY
SAYLORVILLE	0.0276	0.0108	0.0096	0.0075	0.0089	0.0072	0.0050
SE6th	0.0044	0.0068	0.0079	0.0083	0.0090	0.0084	0.0065
RED ROCK	0.0040	0.0041	0.0047	0.0070	0.0070	0.0069	0.0051
TRACY	0.0050	0.0047	0.0051	0.0074	0.0072	0.0072	0.0052
OTTUMWA	0.0076	0.0037	0.0037	0.0056	0.0053	0.0061	0.0046
KEOSAUQUA	0.0113	0.0043	0.0042	0.0065	0.0067	0.0068	0.0051
CORALVILLE	0.0228	0.0099	0.0058	0.0020	0.0015	0.0018	0.0019
IOWA CITY	0.0185	0.0092	0.0048	0.0015	0.0014	0.0016	0.0016
LONE TREE	0.0070	0.0085	0.0037	0.0014	0.0012	0.0014	0.0014
WAPELLO	0.0267	0.0070	0.0037	0.0017	0.0012	0.0014	0.0015
average	0.0135	0.0069	0.0053	0.0049	0.0049	0.0049	0.0038
[1]average return interval	74	145	188	204	202	205	264

[1]Return interval in years

f. Comparison of Corps' 2002 and present study VDF curve estimates show a significant increase in the frequency of reservoir inflow flood events (see Table 5.9). This results because of the significantly wet period or record that has occurred since 1993 that has increased the mean and standard deviation of the period of record (see Table 5.10). The increase in regulated frequency curve quantiles (e.g., the 100 year flow) is due to this wet period. In particular, the frequency of the 1993 30day inflow volumes increase to almost the 100 year for Saylorville. Combining this result with the observation that the Saylorville 1993 1day maximum inflow and regulated 1day flow are about equal shows that Saylorville dam has almost no impact on the 100 year flood downstream of the dam. The derived regulated frequency curves reflect this significant result.

Table 5.9 Comparison of Current and present estimates of 1993 event return interval

			Corps 2002 Study		Current Study	
	duration	unregulated flow	[2]prob	return interval	prob	return interval
Saylorville regulated 1day	[1]60 day	27350	0.0016	625	0.0075	133
50760	30 day	32850	0.0059	169	0.0096	104
Red Rock regulated 1day	[1]120day	44320	0.0031	323	0.0070	143
71430	30day	71430	0.0020	500	0.0047	213

[1]Critical inflow duration found in the Corps' 2002 study, [2]Exceedance probability

Table 5.10 Period of Record Statistics

Saylorville	*[1]1917-1994*		*1917-2008*		*[2]1995-2008*	
	mean	[3]sdev	mean	sdev	mean	sdev
	8650	5930	9230	6020	12450	6050
Red Rock	*[1]1917-1994*		*1917-2008*		*1995-2008*	
	mean	sdev	mean	sdev	mean	sdev
	17390	10750	19560	12650	25630	15280

[1]Corps' 2002 study, [2]Additional period of record, [3]standard deviation

g. A particular problem with the Bulletin 17B analysis is that the lpiii distribution underestimates the 1993 plotting positions for the Iowa and Des Moines River gages. The censoring threshold was originally developed for application to peak annual stream flows. This under prediction suggested the need to evaluate the Bulletin 17B low-outlier censoring threshold application to volume duration frequency analysis.

h. The Bulletin 17B low outlier censoring threshold was selected by the Water Resources Council (see Thomas, 1985) because it resulted in low prediction bias of peak annual flow frequency curves in both Monte Carlo simulation tests and in application to observed peak annual flows at 50 gages. To evaluate this methodology for VDF curves, the threshold was applied in a comparative study with two other censoring methods for estimating VDF curves at 70 unregulated flow gages within Iowa. The results of the analysis demonstrated that the Bulletin 17B performed as well as the other censoring method as measured by prediction bias.

i. An L-moment regional analysis was performed using Iowa and Des Moines River gages to provide a perspective on model error, i.e., the error made because of the approximation made in assuming any particular flow frequency distribution.

j. Table 7.2 shows the difference between the L-moment regional generalized normal distribution estimates and Bulletin 17B lpiii distribution return interval estimates of the 1993 30day maximum volume. The average return interval difference between the methods of about 20 years does not seem significant given the annual flood return interval is about 200 years. The generalized normal distribution does estimate a more frequent occurrence of the 1993 30day volume than the Bulletin 17B lpiii.

k. The duration of the annual maximum daily frequency duration curve to use in computing the regulated frequency curve depends on the effect of Saylorville and Red Rock Reservoir storage on reducing flood flows at downstream locations. The storage effect is measured by how well the annual maximum unregulated volume for an observed event in the period explains the corresponding observed annual maximum regulated flow for reservoir releases exceeding the objective release (the objective release is typically some measure of channel capacity or flow magnitude that causes initial damaging river stage). Examination of the period of record demonstrated that the 30day annual maximum flow was critical for explaining the regulated flows exceeding channel capacity at Saylorville and Red Rock Dams.

Table 7.2: Comparison of Bulletin 17B lpiii and l-moment Generalized Normal Estimates of 1993 30day Annual Maximum Volume Return Intervals

location	Q_{1993}	gNorm	lpiii	difference
Saylorville	32853	153	104	49
SE6th	56113	167	127	41
Red Rock	71426	179	213	-34
Tracy	71426	174	196	-22
Ottumwa	80385	205	270	-65
Keosauqua	81837	168	238	-70
Coralville	21835	154	172	-18
Iowa City	23520	192	208	-16
Lone Tree	31902	282	270	12
Wapello	81153	166	270	-104
average return interval difference				-23

l. The regulated versus unregulated relationship is used to compute the annual maximum 1day regulated frequency curve from the critical duration annual maximum unregulated volume frequency curve. This relationship is characterized by zones where flows are less than or greater than some measure of channel capacity (i.e., the flow magnitude where significant flooding is possible). Graphical analysis of the observed event is used to describe this relationship for flows less than channel capacity. The description of this relationship is more difficult, and more important, for flows exceeding channel capacity. The difficulty stems from the lack of data. Only two events in the period of record, 1993 and 2008 significantly exceed the channel capacity, giving little information to estimate this relationship. Additionally, this region is critical in estimating the 100-year regulated flow value, which is important for regulatory purposes. More information is obtained for estimating this relationship by simulating ratios of important historical events. In this case, ratios equal to 1.2, 1.5 and 1.7 of the 1993 and 2008 Saylorville and Red Rock Reservoir inflows were simulated. The inflow ratio provides for larger events that consider the importance of inflow magnitude and hydrograph shape on reservoir releases.

m. The integration of the annual maximum unregulated flow frequency curves for the critical duration and the regulated versus unregulated relationships at each location resulted in the 1day annual maximum regulated flow frequency curves shown in the table below. Also shown are estimates from the previous Corps of Engineers (2002) study.

Comparison of estimated and 2002 study 1day annual maximum regulated frequency curve estimates

	Saylorville		**SE 6th Street**		**Red Rock**		**Ottumwa**		**Keosauqua**	
Prob	2002 Study	**Current Study**	2002 Study	**Current Study**	2002 Study	**Current Study**	2002 Study	**Current Study**	2002 Study	**Current Study**
0.5	13000	**12000**	23000	**25300**	26000	**25000**	28000	**27500**	30000	**29400**
0.1	16000	**17000**	37000	**42400**	30000	**30000**	31300	**35900**	55000	**39500**
0.02	27000	**44700**	71000	**80300**	50000	**65500**	58000	**74800**	61000	**80000**
0.01	33000	**52800**	85000	**103600**	69000	**89000**	78000	**99600**	81000	**105400**
0.005	38000	**61200**	100000	**117700**	94000	**130000**	103000	**141900**	106000	**148400**
0.002		**73000**		**136900**		**130000**		**143700**		**151100**
0.001		**82400**		**151900**		**150500**		**165500**		**173600**

n. The 1day annual maximum regulated frequency curves were converted to peak values by developing the regression relationships shown below from gage records. No significant differences between the relationships were found when using data before or after Saylorville or Red Rock Dam construction.

Table 8.12: Peak versus daily annual maximum regression

Location	Usgs ID	[1]POR	[2]a	[2]b	[3]R^2
Saylorville	5486150	1962-2009	1.0073	207.58	0.99
SE6th	5485500	1941-2008	1.0368	80.009	0.99
Red Rock/Tracy	5488500	1920-2008	1.0607	-818.61	0.98
Ottumwa	5489500	1917-2008	1.0116	1484.3	0.97
Keosauqua	5490500	1917-2008	1.0695	498.33	0.95

[1]Period of Record
[2]$Q_p = a + b(Q_d)$, where Q_p = peak annual discharge (cfs),
Q_d = maximum annual daily discharge (cfs)
[3]R^2 = regression coefficient of determination

o. Peak 1day regulated frequency curves shown in Table 8.11were computed by applying the regression equations in Table 8.10 to the 1day annual maximum regulated frequency curves.

Table 8.13: Peak Flow Regulated Frequency Curves

[1]Probability	Saylorville	[2]2nd Avenue	SE6th	Red Rock	Ottumwa	Keosauqua
0.5	**12300**	**13140**	**26300**	**25700**	**29300**	**31900**
0.1	**17300**	**19470**	**44100**	**31000**	**37800**	**42800**
0.02	**45200**	**48510**	**83300**	**68700**	**77100**	**86000**
0.01	**53400**	**57220**	**107500**	**93600**	**102200**	**113200**
0.005	**61900**	**66240**	**122100**	**137100**	**145000**	**159200**
0.002	**73800**	**78900**	**142000**	**137100**	**146800**	**162000**
0.001	**83200**	**88980**	**157600**	**158800**	**168900**	**186100**

[1]Exceedance probability, [2]Based on drainage area ratio with Saylorville

p. Regulated frequency curve estimates sensitivity to inflow critical duration was explored by computing regulated curves for alternative inflow durations: 15day and 60day for Saylorville Reservoir and 15day and 120day for Red Rock Reservoir. The results showed that the variation in 100 year regulated flow at Saylorville was a maximum of about 4% and Red Rock Reservoir was about 11%. Selecting the critical duration adds somewhat to the uncertainty inherent in the regulated frequency curve due to statistical sampling error.

q. The Saylorville and Red Rock pool elevation frequency curves were determined by analyzing two distinct operating regions for reservoir releases: 1) elevations relating to pool surcharging to limit outflows to the objective release (i.e., a downstream control point channel capacity or limiting downstream initial damage stage); and, 2) elevations where flood control operations results in releases greater than the objective release. In region (1), annual maximum pool elevation is not directly related to inflow frequency; and consequently, an inflow frequency curve cannot be used to estimate the pool frequency curve. Instead, plotting positions are used to estimate an empirical annual maximum pool frequency curve. In region (2): inflows are related to both the pool elevation and flood release; and, consequently, an inflow frequency curve can be directly related to both annual maximum pool elevations and regulated outflows. Table 9.1 displays resulting pool elevation frequency curves with comparisons to the previous Corp of Engineers (2002) study.

Table 9.1: Saylorville and Red Rock Reservoir Pool Elevation Frequencies

	Saylorville Elevation (ft)			Red Rock Elevation (ft)	
[1]Probability	Current Study	2002 Study [4]	[1]Probability	Current Study	2002 Study [4]
	[2]regulated flow			[2]regulated flow	
0.001	**896.30**		**0.001**	**785.00**	
0.002	**894.70**		**0.002**	**784.00**	
0.005	**893.20**	889.8	**0.005**	**782.60**	780.9
0.01	**892.10**	889.6	**0.01**	**780.70**	780.2
0.02	**890.60**	888.9	**0.02**	**780.10**	779.1
	[3]plotting position			[3]plotting position	
0.1	**880.55**	881.0	**0.1**	**770.00**	768.0
0.2	**868.95**	868.0	**0.2**	**764.72**	763.5
0.5	**844.80**	842.5	**0.5**	**749.30**	748.2

[1]Exceedance probability
[2]Elevation based on log-Pearson iii 30 day critical duration inflow
[3]Interpolated from median plotting positions
[4]Estimates from previous study (Corps of Engineers, 2002)

Contents

List of Figures

List of Tables

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1 Introduction

1.1 Scope

The purpose of this report is to describe the methods and application used to estimate project regulated peak annual frequency curves at selected locations on the Des Moines River below Saylorville Dam. Section 1.2 provides an area description, including a description of Saylorville and Red Rock dams and the locations where frequency curves were computed.

Section 2 describes the assumptions and limitations common to all methods used in obtaining flow frequency estimates. These limitations dictate that any chosen methodology, such as that described in the federal guidelines used in this study, Bulletin 17B (IACWD, 1982), is approximate; but, provide prediction that are commensurate with those obtained by other acceptable methods.

A key flow frequency analyses assumption is that the period of record flows are statistically stationary. This is an approximation because climate is known to be variable. Section 3 provides a summary of research, performed as part of the Corps' (2002) Upper Mississippi Flow Frequency study, investigating the impact of climate variability and the corresponding impact of the stationary assumption on the value of flow frequency estimates.

The basic methodology used in this study is to integrate a Bulletin 17B estimated annual daily maximum unregulated flow frequency curve and a regulated versus unregulated relationship to obtain a 1day annual maximum regulated frequency curve at each river location of interest. This 1day regulated curve is converted to a peak annual maximum regulated flow frequency curve using a daily to peak flow regression estimate. Section 4 describes the data analysis needed to estimate a period of record of unregulated daily flows from the available regulated flow record which are need to estimate both the unregulated flow frequency curve and the regulated versus unregulated relationship. Unregulated flows were needed for both the Iowa and Des Moines River basins so that an area average regional skew could be computed that is required to obtain Bulletin 17B estimates of the unregulated flow frequency curves.

Section 5 describes the application of Bulletin 17B to estimate annual daily maximum volume duration frequency (VDF) curves ranging in duration from 1day to 120days. The analysis of durations up to 120days was necessary to correctly identify the critical duration for identifying the effects of reservoir storage when estimating regulated frequency curves.

Note that the Bulletin 17B guidelines were developed for estimating peak annual flow frequency curves, not for estimating VDF curves. Regardless of this, the Corps' guidelines for performing VDF analysis (Corps of Engineers, 1993) recommend using Bulletin17B. This is reasonable given that the log-Pearson III distribution (lpiii) is flexible enough to describe the empirical distributions that are obtained from annual maximum flow volumes. However, an aspect of Bulletin17B methodology that deserved critical evaluation is the low-outlier censoring criterion. The low-outlier censoring criterion was developed to censor small-magnitude flow values that have unreasonable influence on the upper tail (i.e., at exceedance probability for large flows). This criterion was empirically determined by the Water Resources Council (see Thomas, 1985) by examining peak flow records from a large number of gages. Section 6 describes a similar effort performed to assess the applicability of the low-outlier criterion to VDF curves estimated from flow records in Iowa.

Bulletin 17B application of the lpiii distribution, although operationally reasonable, is not the only approach to estimating frequency curves. The Upper Mississippi Flood Frequency Study (Corp of

Engineers, 2000a) evaluated potential model error, the error made by using a necessarily approximate frequency distribution, by comparing the regional L-moment approach (Hosking and Wallis, 1997) to the Bulletin17B approach. Section 7, describes a comparison of these methods to show the difference in estimates at each project location.

Section 8 describes: 1) the estimation of the regulated versus unregulated relationships and its integration with unregulated flow frequency curves to obtain the 1day annual maximum regulated frequency curves; and, 2) the estimation of the daily to peak regressions and the regressions application to the 1day regulated frequency curves to obtain peak flow frequency curves. Section 9 describes the estimation of Saylorville and Red Rock annual maximum pool elevation frequency curves.

1.2 Reservoir System Description

Flow frequency analysis estimates were developed for gages on the Iowa River downstream of Coralville Reservoir and Des Moines River below Saylorville Reservoir as shown on figure 1.1 and described in Table 1.1. Analysis of both river systems was performed to obtain reservoir system consistent skew estimates.

The Coralville Lake project is located on the Iowa River upstream from Iowa City in Johnson County and is a part of the general comprehensive plan for flood control and other purposes in the Upper Mississippi River region. Construction began on this project in July 1949, and it was completed and put into operation in October 1958. The dam controls runoff from 3,115 square miles and is operated to provide both protection for reaches immediately downstream and control stages on Mississippi River. The normal conservation pool at the dam is 683.0 feet NGVD with 42,200 acre-feet of storage. The flood control pool (elevation 712.0 feet) provides an additional 419,000 acre-feet of storage.

The Red Rock Dam and the Lake Red Rock Project on the Des Moines River are located chiefly in Marion County, but extend into Jasper, Warren and Polk Counties. The dam is approximately 60 miles downstream from the City of Des Moines.

The drainage area above the dam site is 12,323 square miles. A permanent lake of 265,500 acre feet storage is formed behind the dam. With the flood control pool full (elevation 780.0 feet) the reservoir storage is 1,484,900 acre feet above the conservation pool of 742 feet National Geodetic Vertical Datum (NGVD). Flood protection is provided to 36,000 acres of agricultural lands in the Des Moines River basin and to the Cities and Towns of Ottumwa, Eldon, Eddyville, Keosauqua and Farmington.

In 1958, Congress authorized construction of Saylorville Lake on the Des Moines River about 11 miles upstream from the City of Des Moines. The drainage area above the dam is 5,823 square miles. The principal purpose of the Saylorville Project is to furnish needed additional storage to supplement the flood control capacity of the downstream Red Rock Dam and to provide flood protection to the City of Des Moines. The permanent conservation pool forms a lake with storage of about 90,000 acre-feet and extends some 17 miles upstream from the dam.

The reservoir was constructed in 1977 to have a total capacity of 676,000 acre-feet at full flood control pool elevation 890 feet and covers about 16,700 acres. The conservation pool was raised from 833 to 836 feet in 1983 to provide a water supply for the City of Des Moines and the Iowa Southern Utilities near Ottumwa Iowa. Pneumatic gates were installed on top of Saylorville spillway in 1994. The regulation plan for Saylorville did not change with the installation of the pneumatic gates. The introduction of pneumatic gate operation does not influence the downstream regulated flow frequency. When forecasts indicate the pool is rising above 884 feet the pneumatic gates are raised. This allows the pool to rise to 890 feet without water flowing over the spillway. If the forecast is for the pool to rise above 890 feet then the pneumatic gates are lowered gradually starting when the pool reaches 889 feet and are fully lowered when the pool reaches 890 feet. The outflow remains at 21,000 cfs as the pool rises from 884 feet to 889 feet and from 21,000 cfs to 42,000 cfs as the pool rises from 889 feet to 890 feet. Since the installation of the pneumatic gates all of the 21,000 cfs flow from 884 feet to 889 feet is through the conduit instead of a gradual closing the conduit as the flow increases over the spillway to maintain a constant 21,000 cfs.

FIGURE 1.1: LOCATION DIAGRAM

Table 1.1: Frequency Analysis Gages

Location	River	USGS Gage	DA (sq mi)	Period of Record
Saylorville Dam	Des Moines	05481650	5823	1917-2008
S.E. 6th Street	Des Moines	05485500	9879	1917-2008
Red Rock Dam	Des Moines	05488100	12330	1917-2008
Tracy	Des Moines	05488500	12479	1917-2008
Ottumwa	Des Moines	05489500	13374	1917-2008
Keosauqua	Des Moines	05490500	14038	1917-2008
Coralville	Iowa	05453510	3115	1904-2008
Lone Tree	Iowa	05455700	4293	1904-2008
Wapello	Iowa	05465500	12500	1904-2008

2 Frequency Analysis Assumptions and Limitations

The assumptions made in flood frequency analysis regarding annual maximum flow data are that:

- the flows are stationary;
- flows are measured without error;
- flows are homogenous;
- and the likelihood of relatively small-frequent flows is relevant to estimating the likelihood of infrequent large flows.

The stationary assumption states that the frequency distribution of flows does not change with time. Obviously, this is an approximation given the fact that climate is variable, causing apparent cycles and trends in stream flow data. Research sponsored by the Corps of Engineers for the recent Upper Mississippi Flow Frequency Study (see Corps of Engineers, 2003, Matalas and Olsen, 2001, and Stedinger et al., 2001) indicate that the stationary assumption is a useful approximation for applications of frequency analysis in planning studies (for a further discussion of this point see Section 3).

Measurement error is an often ignored aspect of flood frequency analysis. Stream flow measurement of infrequent large flows are often based on an extrapolation of a rating curve or a slope-area estimate rather than a direct measurement. Quantifying this measurement error is very difficult and is not typically accounted for in stream flow analysis. However, if a flow is identified as a high outlier in the Bulletin 17B methodology then the accuracy of the flow measurement should be examined.

The homogeneity of stream flows is typically considered to be affected by anthropomorphic activities, such as agricultural or urban development causing land use change, river channel modifications, and reservoir regulation. A review of land use change provided in the Upper Mississippi Flood Frequency Study concluded that the most significant affects of land use change due to agricultural development in the mid-west ended by about 1900 (see Corps of Engineers, 2000b). Consequently, the period of record used in this study was chosen to span the period 1898-2008. As in the case of the Upper Mississippi study, the effects of regulation and channel modifications on Des Moines River flows were accounted for in routing studies used to determine the unregulated flow period of record.

Flood frequency curve estimation assumes that the likelihood of relatively small frequent events is relevant to determining the same for large events. The difficulty with this assumption stems from the empirical nature of flood frequency analysis. For example, the success of a three parameter probability distribution, such as the lpiii, is judged by its overall performance in comparison to the empirical distribution obtained from many stream flow gages in a large region (see IACWD, 1982, Appendix 14). The frequency distributions are flexible enough to represent the empirical frequency from a flood record containing a mixture of events (e.g., thunderstorms and hurricanes); or flows contained within river bank and exceeding channel capacity caused by the varying coincidence of meteorologic and hydrologic conditions. However, the important question is whether or not the estimation of the flood frequency distribution can be used to reliably estimate the likelihood of relatively rare events? In other words, are estimates of the 100year or 500year flood reliable given that stream flow record lengths available for flood frequency distribution are typically between 20 and 100 years?

This question really speaks to a need to evaluate the limitations of flood frequency analysis techniques given the assumptions made in frequency curve estimation. The limitations of flood frequency analysis have been addressed both by recognizing that the selection of any flood frequency distribution represent an approximation of the true nature of flood risk; and, that the uncertainty in the flood frequency distribution estimates of flood risk are a function of both sampling and model error. This approximate nature of flood frequency distributions has been described in detail by Stedinger, J. R., Vogel, R. M., and

Foufoula-Georgiou, Efi, (1992) (pg. 18.22), J.R.M. Hosking and J.R. Wallis ("Regional frequency analysis, An Approach Based on L-Moments", Cambridge University Press, 1997, pg. 77) and the Water Resources Council (see IACWD, 1982, Appendix 14).

The value of the flood frequency methods is that even though the chosen probability distribution/estimation methods differ significantly at a particular gage, the estimates are reasonably consistent when averaged over a large number of gages. Consider, for example, the comparison made by Thomas and Eash (1995) between the Bulletin 17B and regional L-moment methods in estimating the return interval for the great flood of 1993 flood peaks in the Upper Mississippi Basin. As can be seen from Figure 2.1, the estimates from each method agree on the average, but the difference between individual prediction increases significantly with return interval.

The differences between method predictions are a function of sampling error and model error. Sampling error is the error due to limited stream flow records at a gage. More importantly sampling error explains some of the difference between return interval or exceedance probability obtained by method predictions and the population or "true" value.

As predictions focus on less frequent occurrences (the 100-year flow or greater) the importance of model error, the error due to selecting a particular flow frequency distributions (e.g., the lpiii) that are necessarily approximate, becomes much more important than sampling error. Identifying the true model error is not possible. It results from the assumptions made in frequency analysis, such as assuming stream flows are statistically stationary. However, Hosking and Wallis (1997, chapters 6 and 7) provide insight into the potential model error in their examination of flood frequency prediction error using the regional L-moment approach. Model error was assessed by examining the difference in predictions obtained by using commonly selected three parameter and a four parameter flood frequency distribution. The four parameter distribution was used to try and capture the variation in flood frequency statistics that would not be expected given the assumptions made in the commonly accepted three parameter probability distributions. Briefly stated, they concluded that model misspecification error is more important than sampling error for exceedance probabilities less than 0.01 (return intervals greater than the 100-year) given typically available stream gage record lengths.

Practically speaking this means that sampling error uncertainty estimates of frequency curve prediction uncertainty, such as quantified by confidence limits, does not tell the whole story. Model error is also important, particularly for estimates of large floods. Consequently, selection of safety factors used in flood damage reduction measures should certainly keep in mind both the potential for sampling and model error in flood frequency estimates.

In summary, the limitations in prediction accuracy for various flood frequency analysis methods, including the methods described in Bulletin 17B, result in predictions errors that are both due to sampling and model errors. The sampling errors, those errors due to limited gage record lengths, result in significant differences between alternative flood frequency methods at a gage, but agreement between the methods on the average. The study performed by Thomas and Eash (1995) demonstrated a small differences on the average between regional L-moment and Bulletin 17B predictions of the peak annual flow return interval for the great flood of 1993 on the Upper Mississippi River. However, at-gage prediction differences increased significantly for return intervals equal or exceeding the 100 year. Model error (the error from incorrectly specifying the flood frequency distribution) is not quantifiable. However, Hosking and Wallis (1997) made a reasonable attempt to approximate this error by comparing the predictive capabilities of three and four parameter probability distributions. They concluded that model error becomes dominant for return intervals exceeding the 100 year for stream gage record lengths typically available. Practically speaking, this means that sampling and model error need to be appreciated when selecting safety factors for flood damage reduction measures.

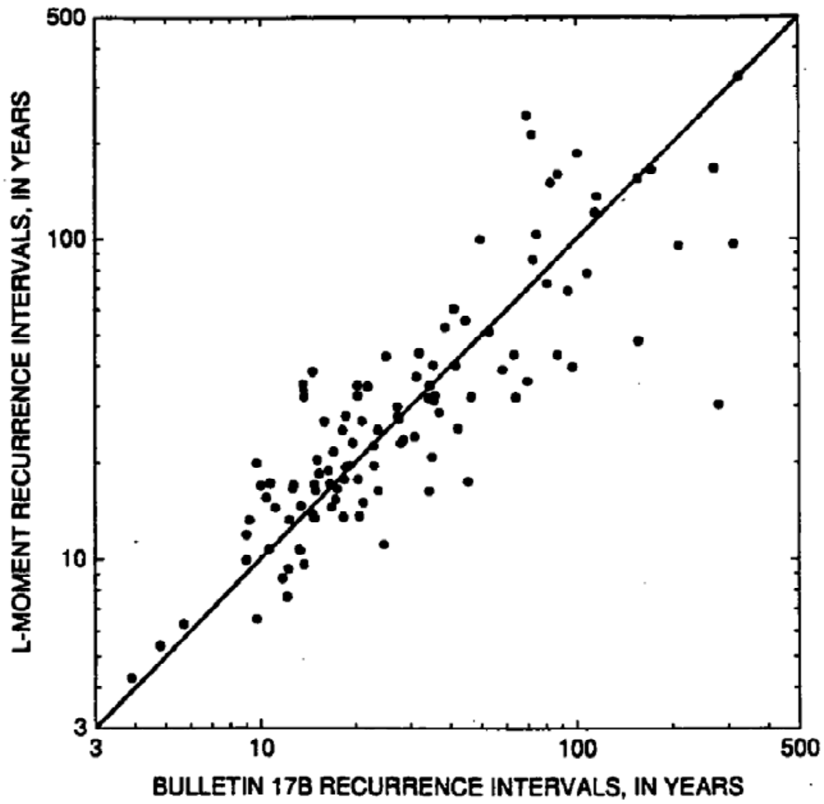

FIGURE 2.1: COMPARISON OF L-MOMENT AND BULLETIN 17B EXCEEDANCE PROBABILITY FOR UPPER MISSISSIPPI 1993 FLOOD PEAKS (FROM THOMAS AND EASH, 1995)

3 Climate Variability

The Corps of Engineers has had, in the past, and continues, to answer the public's questions regarding the impacts of climate variability on flood risk and its consequences for reservoir operations. Irrespective of the state of the science, addressing these issues by continuing to review and participate in research in this subject area is important to assuring the public that flood control agencies are not ignoring the issue. The value of participating in research on this subject was demonstrated in the recent major study of flood risk in the Upper Mississippi River Basin (Corps, 2002). As part of this study, the Corps produced investigations of the potential impact of climate change on flood risk. By doing this, the Corps was able to satisfactorily answer the public's concerns and defend the estimates of flood risk obtained in the study.

This issue was addressed at length during the Corps Upper Mississippi Flood Frequency Study by a technical advisory group (TAG) and research done both at the Corps Hydrologic Engineering Center and Institute for Water Resources. Summarizing some of the key points in the discussions of the importance of trends in the record of annual flood maximums:

- The recognition that trends exist in the flood record is not new but was addressed by the Water Resources Council circa 1970 in developing the federal guidelines for performing flood frequency analysis, Bulletin 17B. They addressed the problem of non-stationarity (see the federal "Guidelines for Determining Flood Flow Frequency Analysis, Bulletin #17B, Appendix 14, Interagency Committee on Water Data, Reston, Virginia, 1982) when performing the split flow gage record testing used to select a distribution/estimation methodology for modeling flood risk by (pg. 14-2):

  > *…. using odd sequence numbers for one half and even for the other in order to eliminate the effect of any general trend that might possibly exist. This splitting procedure should adequately simulate practical situations as annual events were tested and found independent of each other. ….*

  The Water Resource Council (WRC) obviously did not believe that the appropriate procedure was to consider extrapolating a trend into the future to assess flood risk. Rather, considering the observed variance in the period of record, as modeled by an independent and identically distributed (i.i.d) random variable, was selected as the best means for assessing future flood risk. Here, the WRC is arguing for the variance as the best measure of future flood risk rather than considering using an apparent trend.

- As part of the Upper Mississippi Flood Frequency Study the Institute for Water Resources (Corps of Engineers) examined the importance of trend analysis or persistence in the flood record to the assessment of future flood risk.

  As part of the IWR study, Matalas and Olsen (2001) showed that trends might be manifestations of episodic, slowly varying flood frequencies that can be represented by a stationary persistent process. Stedinger (a member of the TAG) and Crainiceanu (2001) examined the following alternatives to the stationary, i.i.d. model for estimating flood frequencies:

  1. No longer assuming that the variables are identically distributed, that is, there is a trend in the mean or variance over time.
  2. Using only part of the historical record, such as the more recent period, to ensure that the stationary assumption is met.

3. No longer assuming that the variables are statistically independent over time, that is, they are serially correlated with each other.

These researchers identified a number of problems with the first two models. The presence and significance of the trend depends on the period used in the analysis. If a trend exists in the recent period, the analysis would need to determine the "expected" form and duration of the trend given its "expected" time of inception. There is no clear method to determine these values other than by subjective judgment. Another problem is how to extrapolate the trend beyond the period of record.

There are also problems with using only the more recent period of the historical record unless the periods are characterized by a definitive activity such as changes in land cover. There is no clearly defined cause of the apparent trends in the flood record in the Upper Mississippi River basin. The flow frequency study concluded that the cause of the trends was most likely natural climatic variability. Currently there is no skill in predicting climate on an inter-decadal time scale. For the Upper Mississippi River basin, it is problematic to even show in the historic record when past climate shifts occurred. In the absence of a better knowledge of climate it is prudent to use a long period of record so multiple possible climate conditions are included.

The third model assumes that flood risk varies over time. Climate can follow a pattern of episodic wet and dry periods that persist for several years. Sequences of the flood record may show trends, but these episodes will also be manifested as persistence or serial correlation as suggested by Matalas and Olsen (2001). An autoregressive moving average (ARMA) model was used to model the change of flood risk over time. Stedinger and Crainiceanu concluded that the first two models were inappropriate, but the stationary time series (ARMA) model could produce reasonable flood risk estimates. However, when the stationary time series models are used for risk forecasting, the predicted risk returns to the unconditional long-run average as the forecasting horizon increases. Floodplain maps cannot be adjusted on a year-to-year basis, and engineering design has a time horizon of fifty years or longer. Over the forecasting horizon of concern, the use of a stationary time series model with significant long-term persistence will give an almost identical estimate as the traditional method i.i.d assumption adopted by the WRC.

In summary, these researchers concluded that stationary time series models produce a reasonable flood risk forecast and interpretation of historical records. Stationary time series allow risk to vary over time; but, when stationary time series models are used for risk forecasting, the predicted risk returns to the unconditional long-run average as the forecasting horizon increases. The resulting variation in flood risk is likely to affect flood risk management only if decision parameters are adjusted on a year-to-year basis. The results of the flow frequency study are used for engineering design and floodplain delineation for flood insurance purposes. Over the forecasting horizon of concern, the use of a stationary time series model with significant long-term persistence would give an almost identical estimate as the i.i.d. assumption.

4 Data analysis

4.1 Introduction

The purpose of this section is to both describe the estimation of unregulated flows from the period of record; and, assess the suitability of these estimates for flow frequency analysis. The factors that might raise concerns about the flow suitability are: Climate variability, land use change, channel change, gage observation errors and approximations made to account for reservoir hold outs. As discussed in the previous section, the stationary assumption is likely to be reasonable over a planning period despite the existence of climate variability. Consequently, any observed trends in the data will be assumed to be transient aspects of the overall variability in climate.

Land use change was investigated as part of the Upper Mississippi Flow Frequency study (Corps, 2000b). The major land use conversion to agricultural land use occurred prior to the 20th century. Consequently, the expectation is that the influence of land use change on annual maximum flows is minimal.

The primary concern is that gage observation errors or routing approximations used to obtain an unregulated period of record may result in apparent outlying annual maximums; or, that the routing model approximations made resulted in unacceptable errors in the estimated unregulated flows. The Bulletin 17B high outlier test will be applied in the next section on frequency analysis as a standard part of the analysis to identify high outlying values.

In Section 4.2, a description of the routing method used to obtain unregulated flows is described. Trend and double mass curve will be performed in sections 4.3 and 4.4 to examine if gage observation errors or routing approximations produced any anomalous annual maximum events in the routing record for flow frequency analysis. Section 4.5 describes an accuracy analysis of the computed unimpaired maximum 30day inflow hydrographs to Saylorville and Red Rock Reservoirs for major events

4.2 Routing Model

Unregulated flows for the Iowa and Des Moines River basins were estimated by applying channel and reservoir routing models. The Iowa River Basin flow routing was modeled by the Rock Island District using the in-house program Corsim (Corps of Engineer, 1976). Corsim is a daily, period of record program that uses a data base of 105 years of record (1904-2008). The program uses Tatum routing, a coefficient based routing method developed in the Rock Island District for the Des Moines River Basin. From 1904 until Coralville Reservoir was placed in operation in 1959 the unregulated flow records at Coralville Reservoir, Iowa City, Lone Tree, and Wapello were determined by the USGS daily flow record. Wapello record was estimated from 1904 until the gage was established in 1914 using the records of the Iowa River at Iowa City and the Cedar River at Cedar Rapids. Lone Tree was estimated using regression techniques to establish the daily record from 1904 until 1956 when the gage was established. For the period of record after Coralville Reservoir was placed in operation the unregulated daily record was estimated by routing the 1 day, midnight to midnight change in storage (hold out in cfs-days) downstream and adding it to the USGS daily record. The resulting period of record unregulated flows for Coralville Reservoir (inflow), Iowa City, Lone Tree, and Wapello is the base input flow record for Corsim.

The program then follows the regulation plan for Coralville Dam operation and determines what the regulated outflow would be, calculates the holdouts (inflow – outflow), and routs the holdouts downstream, subtracting them from the unregulated flow to determine what the regulated flow would have been under the modeled regulation plan. For this study the current regulation plan, in effect since 2001, was used.

Tatum routing steps are used in both the Corsim and Sayred programs. The Corsim program Tatum routing steps are as follows on the Iowa River: from Coralville 0 steps to Iowa City, 2 steps to Lone Tree and 4 steps to Wapello.

The Sayred program Tatum routing steps are as follows on the Des Moines River: from Saylorville 1 step to SE6th, 2 steps to Tracy, 4 steps to Ottumwa and 6 steps to Keosauqua; and from Red Rock 0 steps to Tracy, 2 steps to Ottumwa and 4 steps to Keosauqua. The coefficients for routing the flow downstream are shown in Table 4.1.

Table 4.1: Tatum Routing Step Coefficients

	Day 1	Day 2	Day 3	Day 4	Day 5	Day 6	Day 7
Iowa River Corsim							
Coralville to Iowa City	1						
Coralville to Lone Tree	1/4	1/2	1/4				
Coralville to Wapello	1/16	1/4	3/8	1/4	1/16		
Des Moines River Sayred							
Saylorville to SE6th	1/2	1/2					
Saylorville to Tracy	1/4	1/2	1/4				
Saylorville to Ottumwa	1/16	1/4	3/8	1/4	1/16		
Saylorville to Keosauqua	1/64	3/32	15/64	5/16	15/64	3/32	1/64
Red Rock to Tracy	1						
Red Rock to Ottumwa	1/4	1/2	1/4				
Red Rock to Keosauqua	1/16	1/4	3/8	1/4	1/16		

The Des Moines Basin flow routing is modeled by the Rock Island District using the in-house program Sayred (Corps of Engineers, 1975). Sayred is a daily, period of record program that utilizes a data base of 92 years of record (1917-2008). The program utilizes Tatum routing, a coefficient based routing method developed in the Rock Island District for the Des Moines River Basin. The unregulated flow record was determined from the USGS daily flow record for the period of time from 1917 until the reservoirs were placed into operation; for Saylorville and Southeast 6th Street (the Des Moines River below Raccoon-SE6th) until 1977 when Saylorville came into operation and for Tracy (used for Red Rock inflow), Ottumwa, and Keosauqua this was until 1969 when Red Rock was placed into operation. From 1917 until 1961 when the Saylorville gage was established the Saylorville record was estimated using the USGS records at Fort Dodge, Boone, and Second Avenue. The SE6th street record from 1917 until 1940 when it was established was estimated using the USGS record at Second Avenue and the Raccoon River at Van Meter. The record at Tracy from 1917 until 1920 when it was established was estimated using the record at Second Avenue and Ottumwa. For the period of record after the reservoirs were placed in operation the unregulated daily record was estimated by routing the one day, midnight to midnight change in storage (hold out in cfs-days) from each reservoir downstream and adding it to the USGS daily record. The resulting period of record unregulated flows for Saylorville Reservoir (inflow), Southeast 6th, Red Rock (inflow), Ottumwa, and Keosauqua is the base input flow record for Sayred.

The program then follows a regulation plan for both Saylorville and Red Rock and determines what the outflow should be, calculates the holdouts (inflow – outflow) and routs the holdouts downstream, subtracting them from the unregulated flow to determine what the regulated flow would have been under the modeled regulation plan. For this study the current regulation plans at Saylorville (2001) and Red Rock (2003) were used.

Pneumatic gates were installed on top of Saylorville spillway in 1994. The regulation plan for Saylorville did not change with the installation of the pneumatic gates. The introduction of pneumatic gate operation does not influence the downstream regulated flow frequency. When forecasts indicate the pool is rising above 884 feet the pneumatic gates are raised. This allows the pool to rise to 890 feet without water flowing over the spillway. If the forecast is for the pool to rise above 890 feet then the pneumatic gates are lowered gradually starting when the pool reaches 889 feet and are fully lowered when the pool reaches 890 feet. The outflow remains at 21,000 cfs as the pool rises from 884 feet to 889 feet and from 21,000 cfs to 42,000 cfs as the pool rises from 889 feet to 890 feet. Since the installation of the pneumatic gates all of the 21,000 cfs flow from 884 feet to 889 feet is through the conduit instead of a gradual closing of the conduit as the flow increases over the spillway to maintain a constant 21,000 cfs.

The value of these coefficient based routing models can be seen by comparison of the routed and observed regulated flows on the Iowa and Des Moines River. Tatum routing model does a reasonable job of reproducing observed flows as can be seen in the comparisons shown in Tables 4.2 – 4.4. In these tables the maximum USGS daily flow for each year area compared to the maximum flow from the Corsim model using the current regulation plan for 1959 through 2008, the years the reservoir has been in operation. The reservoir has been regulated under the current operation plan since 1993, with a few minor changes enacted in 2001.

Iowa City is directly downstream from Coralville Dam, which controls 95% of Iowa City's drainage area. There is very little attenuation of flow from the dam to the gage. Corsim, using Tatum Routing was within 5% of the flow for the peak event 75% of the time and within 10% of the flow over 80% of the time. In 1993, with the flow going over the spillway the Rock Island District was granted a deviation from Coralville's regulation plan from the Mississippi Valley Division for a short period of time at the peak of the flood event to reduce flow through the conduit in a successful attempt to keep the water treatment plant from flooding. This is the reason for the 14% difference between the model and the USGS record for that year.

Lone Tree is about one day travel time downstream of Coralville Dam which controls about 73% of its drainage area. From 1993 through 2008, when the regulation plan was about the same as the current, modeled regulation plan Corsim was within 5% of the USGS record 56% of the time and within 10% of the record over 80% of the time. The 1997 event illustrates one of the problems of trying to duplicate the USGS record with a modeled event. Corsim followed the plan of regulation exactly; drawing the pool elevation down to 679 feet NGVD then reducing outflow to 1,000 cfs and holding Lone Tree to less than 16,000 cfs. Because of concerns for drought conditions, in actuality the Rock Island District did not draw the pool down and when the sudden rise in flows came the District did not cut back releases to avoid the 20,000 cfs flow.

Wapello is just over 2 days travel time downstream from the reservoir and Coralville Dam only controls 25% of its drainage area. Because of the small amount of control the reservoir has the model was within 5% of the USGS peak flows 100% of the time between 1993 and 2008 when the reservoir used the same regulation plan as was modeled despite the longer travel time.

The application of Sayred routing was of similar accuracy to that of Corsim as shown in Tables 4.5-4.8. At SE6th Street, prediction error is within 5% about 63% of the time and 10% about 83% of the time; at Ottumwa prediction error is within 5% about 63% of the time and 28% about 43% of the time; and at Keosaqua prediction error is within 5% about 23% of the time and 10% about 53% of the time.

As in the case of the Des Moines River application, the difference between observed and routed flows is significantly affected by the differences in regulation plan and actual real time operations. For example, consider the routing application at Ottumwa where there is only a 7% increase in drainage from Red Rock

Dam. The difference in estimated Red Rock release according to the regulation plan and actual for 1993 is about 6000 cfs, which is partly due to the difficulty in modeling real time operations and partly due to the deviations from the regulation plan in real time operations. This error propagates to Ottumwa where combined with the approximations made in using a simplified hydrologic routing model results in the 1993 observed flow to be only 85% of predicted flow for the 1993 annual maximum value.

In conclusion, the routing results are reasonably accurate given the unavoidable differences caused variations in real time dam operations from the regulation plan and the simple hydrologic routing methods applied to the available average daily flows comprising the period of record. The reasonable correspondence between observed and predicted values provides good reason to accept the unimpaired daily flows obtained from the routing studies.

Table 4.2: Iowa City Routing Comparisons

	USGS	CORSIM	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/CORSIM
4/14/1959	9250	10781	0.86
4/16/1960	9820	10495	0.94
3/13/1961	9700	10878	0.89
7/14/1962	10200	10422	0.98
3/20/1963	7560	7500	1.01
6/27/1964	3000	7047	0.43
4/24/1965	9900	10841	0.91
2/10/1966	6000	6874	0.87
6/7/1967	4310	6203	0.69
7/24/1968	2920	3880	0.75
7/27/1969	14500	12556	1.15
3/10/1970	8490	10124	0.84
2/26/1971	10200	10755	0.95
7/17/1972	6000	7920	0.76
5/1/1973	10800	10526	1.03
6/9/1974	11200	11319	0.99
4/9/1975	10200	10742	0.95
4/24/1976	6110	10272	0.59
9/18/1977	4710	7148	0.66
3/26/1978	9490	10139	0.94
3/29/1979	10200	10809	0.94
3/19/1980	5090	6076	0.84
6/29/1981	3870	4830	0.80
6/15/1982	8800	10156	0.87
4/26/1983	9450	10532	0.90
3/1/1984	9390	10541	0.89
3/4/1985	9240	10281	0.90
6/30/1986	9180	10590	0.87
4/18/1987	4890	5930	0.82
1/31/1988	3630	3880	0.94
9/9/1989	2670	2360	1.13

Table 4.2: Iowa City Routing Comparisons (continued)

	USGS	CORSIM	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/CORSIM
6/17/1990	10500	9491	1.11
6/13/1991	13000	10848	1.20
4/25/1992	8070	8723	0.93
8/10/1993	26200	30343	0.86
3/10/1994	9080	8768	1.04
4/30/1995	9510	9900	0.96
5/20/1996	6530	7430	0.88
2/21/1997	9980	10470	0.95
4/9/1998	9870	11295	0.87
6/10/1999	7150	7050	1.01
6/12/2000	6480	6170	1.05
3/20/2001	10200	10745	0.95
8/23/2002	4500	4777	0.94
5/15/2003	6180	6190	1.00
6/11/2004	7310	7168	1.02
5/20/2005	6290	6617	0.95
4/8/2006	4980	4891	1.02
3/23/2007	10000	10301	0.97
6/15/2008	40900	42429	0.96

Table 4.3 Lone Tree Routing Comparisons

	USGS	CORSIM	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/CORSIM
3/21/1959	14600	16780	0.87
4/1/1960	27300	30373	0.90
3/15/1961	16200	16231	1.00
3/21/1962	24500	21388	1.15
3/20/1963	10100	9910	1.02
6/24/1964	5550	10523	0.53
9/22/1965	29200	27467	1.06
2/11/1966	14600	13465	1.08
6/11/1967	6710	9930	0.68
11/3/1967	3780	3460	1.09
7/20/1969	21500	19031	1.13
3/5/1970	16700	17750	0.94
2/27/1971	14700	17215	0.85
8/7/1972	13500	15117	0.89
4/22/1973	19300	18495	1.04
5/19/1974	30600	29287	1.04
3/21/1975	15100	19627	0.77
4/26/1976	14700	19152	0.77
8/16/1977	7550	11082	0.68
7/23/1978	11100	12903	0.86
3/20/1979	21300	20756	1.03
6/15/1980	5350	6315	0.85
6/30/1981	6450	7065	0.91
6/16/1982	16700	16718	1.00
4/3/1983	16700	17471	0.96
2/15/1984	13100	15028	0.87
3/5/1985	14300	17387	0.82
5/18/1986	20000	18009	1.11
10/27/1986	7200	8485	0.85
1/21/1988	6000	6388	0.94
9/10/1989	8580	7069	1.21

Table 4.3 Lone Tree Routing Comparisons (continued)

	USGS	CORSIM	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/CORSIM
6/20/1990	25600	24786	1.03
6/3/1991	15300	13984	1.09
7/31/1992	16700	14850	1.12
7/7/1993	55100	56492	0.98
3/6/1994	10400	11025	0.94
4/28/1995	12600	13848	0.91
5/11/1996	26200	25970	1.01
2/21/1997	20100	15785	1.27
4/1/1998	13800	15038	0.92
10/18/1998	11700	11930	0.98
6/14/2000	9190	9807	0.94
3/17/2001	15800	15808	1.00
5/12/2002	7300	7639	0.96
5/10/2003	8800	8647	1.02
3/6/2004	12600	12097	1.04
5/14/2005	7970	9277	0.86
4/7/2006	5930	5839	1.02
4/28/2007	14700	15371	0.96
6/15/2008	53200	63613	0.84

Table 4.4: Wapello Routing Comparisons

	USGS	CORSIM	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/CORSIM
3/22/1959	36100	40144	0.90
4/5/1960	66900	67037	1.00
4/3/1961	66400	66709	1.00
4/6/1962	53100	55914	0.95
3/21/1963	24200	24057	1.01
6/25/1964	11300	16070	0.70
4/13/1965	70300	66941	1.05
5/25/1966	29800	30668	0.97
6/9/1967	21700	21838	0.99
8/10/1968	21100	20648	1.02
7/15/1969	67400	64661	1.04
3/6/1970	33600	35136	0.96
2/28/1971	36200	37891	0.96
8/8/1972	24300	27091	0.90
4/22/1973	84200	83681	1.01
5/19/1974	78900	77633	1.02
3/25/1975	38600	40183	0.96
4/27/1976	36200	40777	0.89
9/20/1977	20600	20861	0.99
3/22/1978	26700	28397	0.94
3/22/1979	62400	61671	1.01
6/8/1980	20900	21021	0.99
7/1/1981	19700	20325	0.97
7/20/1982	47500	46389	1.02
4/4/1983	38400	39677	0.97
2/24/1984	34900	39158	0.89
2/25/1985	42600	44386	0.96
5/19/1986	54700	53152	1.03
8/28/1987	19000	20735	0.92
1/21/1988	11000	11743	0.94
9/10/1989	10900	10039	1.09

Table 4.4: Wapello Routing Comparisons (continued)

	USGS	CORSIM	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/CORSIM
6/19/1990	76000	75067	1.01
5/26/1991	49100	48706	1.01
4/28/1992	31700	36130	0.88
7/8/1993	106000	107496	0.99
3/12/1994	27900	27151	1.03
4/29/1995	36200	37156	0.97
5/12/1996	72000	72059	1.00
2/23/1997	71500	67809	1.05
7/1/1998	48000	48828	0.98
7/29/1999	59900	59032	1.01
7/19/2000	33900	33825	1.00
4/20/2001	42600	41427	1.03
6/7/2002	27500	27669	0.99
5/19/2003	28400	28330	1.00
5/31/2004	63500	65041	0.98
7/6/2005	28900	28994	1.00
4/10/2006	22800	22731	1.00
4/28/2007	35400	36001	0.98
6/15/2008	172000	171541	1.00

Table 4.5: Saylorville Gage Routing Comparisons

DATE	USGS Maximum Daily Flow	SAYRED Pool Routing	Ratio USGS/SAYRED
15/3/1977	937	3570	0.26
7/11/1978	7770	11250	0.69
4/6/1979	18500	22000	0.93
6/19/1980	11700	12000	0.98
6/30/1981	12400	12000	1.03
5/20/1982	11700	13850	0.84
4/29/1983	16800	18000	0.93
6/22/1984	29600	47325	0.63
4/30/1985	9180	9750	0.94
3/23/1986	14700	14383	1.02
4/18/1987	11700	12504	0.94
5/12/1988	4580	5736	0.80
5/27/1989	4670	6290	0.74
6/29/1990	11900	12000	0.99
6/10/1991	25700	34854	0.74
3/14/1992	14000	13760	1.02
7/21/1993	44300	50757	0.87
6/27/1994	12400	13002	0.95
4/26/1995	12700	12346	1.03
6/19/1996	12000	12000	1.00
3/15/1997	13600	12991	1.05
4/11/1998	14800	16000	0.93
4/20/1999	14500	14462	1.00
7/14/2000	9930	11449	0.87
4/16/2001	16900	17000	0.99
10/7/2002	7140	8462	0.84
6/30/2003	12800	12000	1.07
5/31/2004	15300	12000	1.28
5/19/2005	15700	12000	1.31
5/11/2006	13500	12470	1.08
8/27/2007	16600	16000	1.04
6/13/2008	49700	52889	0.94

Table 4.6: SE 6th Street Routing Comparisons

	USGS	SAYRED	Ratio
DATE	Maximum Daily Flow	Tatum, Routing	USGS/SAYRED
8/28/1977	11000	11194	0.98
3/21/1978	20700	18738	1.10
3/21/1979	34800	42616	0.82
6/19/1980	14700	15788	0.93
7/4/1981	14300	14133	1.01
5/21/1982	23900	24767	0.96
7/4/1983	36600	37443	0.98
6/19/1984	56700	64369	0.88
4/30/1985	13800	13956	0.99
7/2/1986	44800	38332	1.17
4/18/1987	22,200	22849	0.97
5/12/1988	6280	6267	1.00
5/26/1989	7380	8274	0.89
6/19/1990	44100	44200	1.00
6/10/1991	44600	48698	0.92
4/24/1992	27400	27073	1.01
7/11/1993	113000	119318	0.95
3/8/1994	19900	20683	0.96
6/1/1995	23800	26063	0.91
6/26/1996	26000	27129	0.96
2/21/1997	20200	20109	1.00
6/16/1998	45000	41426	1.09
5/22/1999	32700	32120	1.02
7/14/2000	11200	11007	1.02
3/25/2001	27500	27163	1.01
5/14/2002	14200	16388	0.87
5/9/2003	28400	27408	1.04
5/25/2004	36800	39703	0.93
5/13/2005	28500	27440	1.04
5/3/2006	21100	20919	1.01
4/27/2007	39600	39852	0.99
6/13/2008	98900	99892	0.99

Table 4.7 Ottumwa Routing Comparisons

	USGS	SAYRED	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/SAYRED
4/17/1969	31100	33232	0.94
5/16/1970	17000	20090	0.85
2/25/1971	31300	32465	0.96
5/8/1972	19500	21050	0.93
5/2/1973	32000	33749	0.95
6/9/1974	26600	24821	1.07
3/22/1975	24100	28510	0.85
4/24/1976	35200	26518	1.33
9/2/1977	15800	21033	0.75
5/13/1978	24500	31932	0.77
4/20/1979	33000	41477	0.80
6/15/1980	23100	30546	0.76
7/4/1981	20900	20722	1.01
7/17/1982	41200	31497	1.31
4/2/1983	35500	36635	0.97
6/27/1984	46800	47398	0.99
2/26/1985	19400	20884	0.93
5/17/1986	31600	28750	1.10
4/18/1987	25400	26582	0.96
4/11/1988	6910	7015	0.99
9/10/1989	13100	8798	1.49
5/25/1990	28300	28409	1.00
4/19/1991	35700	39089	0.91
4/20/1992	28500	32737	0.87
7/12/1993	110000	129507	0.85
3/9/1994	22500	27261	0.83
5/8/1995	28200	32458	0.87
5/27/1996	32700	27262	1.20
5/8/1997	28700	33032	0.87
7/6/1998	40400	32918	1.23
4/28/1999	36400	34807	1.05
6/24/2000	33400	33689	0.99
3/15/2001	31400	33001	0.95
5/11/2002	18700	24444	0.77
5/9/2003	20900	30871	0.68
6/14/2004	23000	44546	0.52
4/13/2005	23800	33366	0.71
5/3/2006	18600	19989	0.93
8/24/2007	33400	47669	0.70
6/17/2008	101000	95492	1.06

Table 4.8: Keosauqua Routing Comparisons

	USGS	SAYRED	Ratio
DATE	Maximum Daily Flow	Tatum Routing	USGS/SAYRED
7/19/1969	33600	36140	0.93
8/5/1970	19900	20862	0.95
2/27/1971	34500	35909	0.96
5/8/1972	20500	21888	0.94
4/21/1973	57100	57834	0.99
5/20/1974	34600	29756	1.16
3/24/1975	25300	30353	0.83
4/24/1976	58000	52949	1.10
8/8/1977	22600	22529	1.00
5/13/1978	32700	34194	0.96
4/20/1979	34800	40503	0.86
6/15/1980	31600	28558	1.11
7/4/1981	34400	29280	1.17
7/17/1982	64900	56332	1.15
4/4/1983	35600	37016	0.96
6/27/1984	43900	49630	0.88
3/4/1985	22000	24511	0.90
5/17/1986	50000	41268	1.21
4/19/1987	25900	28260	0.92
4/12/1988	6550	6465	1.01
9/9/1989	13900	10394	1.34
6/17/1990	44300	38504	1.15
4/19/1991	41900	38978	1.07
4/21/1992	31600	34884	0.91
7/13/1993	108000	124820	0.87
3/10/1994	22000	26000	0.85
5/24/1995	42000	39574	1.06
5/27/1996	50900	46504	1.09
4/16/1997	28200	35159	0.80
7/6/1998	43800	36606	1.20
4/16/1999	39000	36289	1.07
6/24/2000	37600	38179	0.98
3/15/2001	41400	38165	1.08
5/12/2002	30700	31074	0.99
5/10/2003	20700	30046	0.69
5/30/2004	27600	40720	0.68
4/13/2005	26200	31238	0.84
5/3/2006	19100	20428	0.93
8/24/2007	60100	62121	0.97
6/16/2008	105000	85533	1.23

4.3 Trend Analysis

The consistency of the estimated unregulated flows with that of the period of record flows observed prior to reservoir construction was evaluated using trend analysis. The period of record consists of both the USGS flows observed prior to the construction of Saylorville Dam in 1977 and Red Rock in 1969 and the estimated unregulated flows after the reservoir construction. Any inconsistency in the variation of flows, or outlying flows over this period may be revealed in assessing the data trends. Table 4.8 displays the statistical significance measures of the slope term of the trend analysis at these dam locations. As can be seen, the Student's t value for the slope term indicate that the slope term is not statistically significant at the 10% significance level but is at the 1% level. The statistical significance measures are at best approximate because the regression residual errors shown in Figures 4.1-4.2 are not homoscedastic (i.e., the spread of residuals about the regression line vary with time).

More important than the statistical significance measures, the regression analyses for all the gages show that there is a consistent trend of the data despite the need to estimate unregulated flows beginning with the reservoir construction. Apparently, the spread in annual flow values has increased with time over the whole record. Perhaps this is due to some aspect of climate variability. In any case, the estimated unregulated data seems to be consistent with the unregulated flows observed prior to reservoir operations.

Table 4.8: Des Moines River period of record trend analysis statistics for 1day unregulated annual maximum flows.

Location	*t Stat*	*P-value*	*t 10%*	*t1%*
Saylorville	2.86	0.005259	1.98	2.88
Red Rock	3.32	0.001293	1.98	2.88

4.4 Double Mass Curve Analysis

Double mass curve analysis can be used to see if some influence in a watershed (e.g., land use change, channel modifications or reservoir operations) has materially affected flows by comparison with flows in another watershed where conditions are static. The double mass curve is created by comparing cumulative flow values at each gage from a beginning date where watershed conditions are compatible. A change in the double mass curve relationship indicates that there has been some influence that has cause the watershed flow regime to change.

In this application, a double mass curve was developed between the 1day annual maximum unregulated observed and unregulated flows at both Saylorville and Red Rock dams, and the static observations at the Cedar River Rapid City gage. A change in the double mass curve relationship after the construction of the dams would indicate that the estimated unregulated flows are not consistent with the period of record flows prior to dam construction.

Figures 4.3-4.4 show that the estimated unregulated flows maintain the same trend in the double mass curve analysis before and after the dams construction. Consequently, the estimated unregulated flows seem reasonably consistent with the previously observed unregulated flow values.

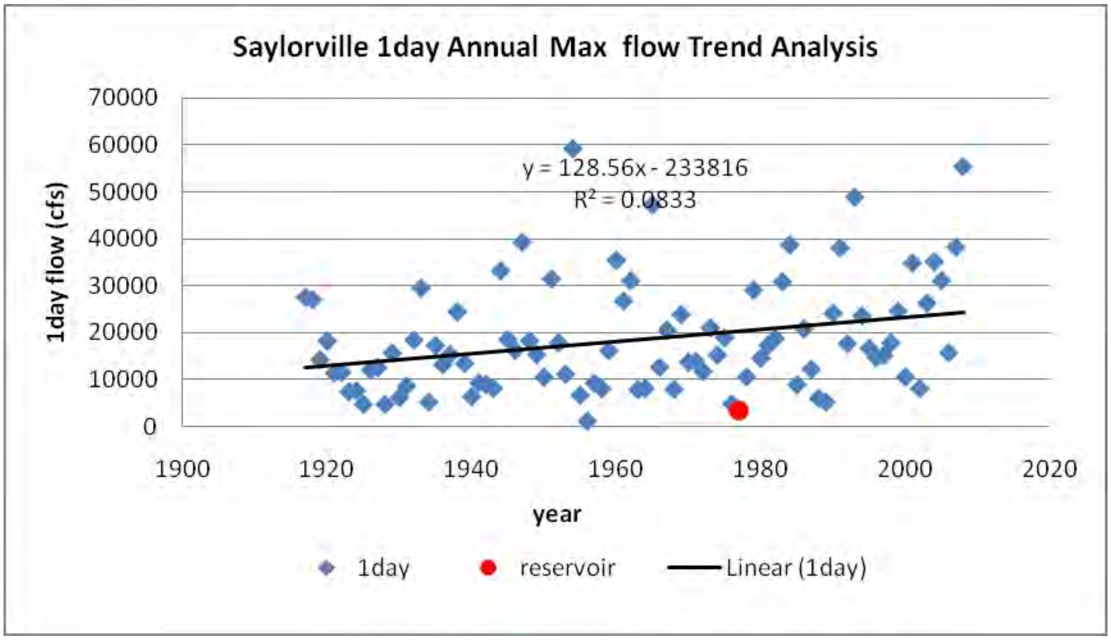

FIGURE 4.1: SAYLORVILLE 1DAY ANNUAL MAXIMUM UNREGULATED FLOW TREND ANALYSIS

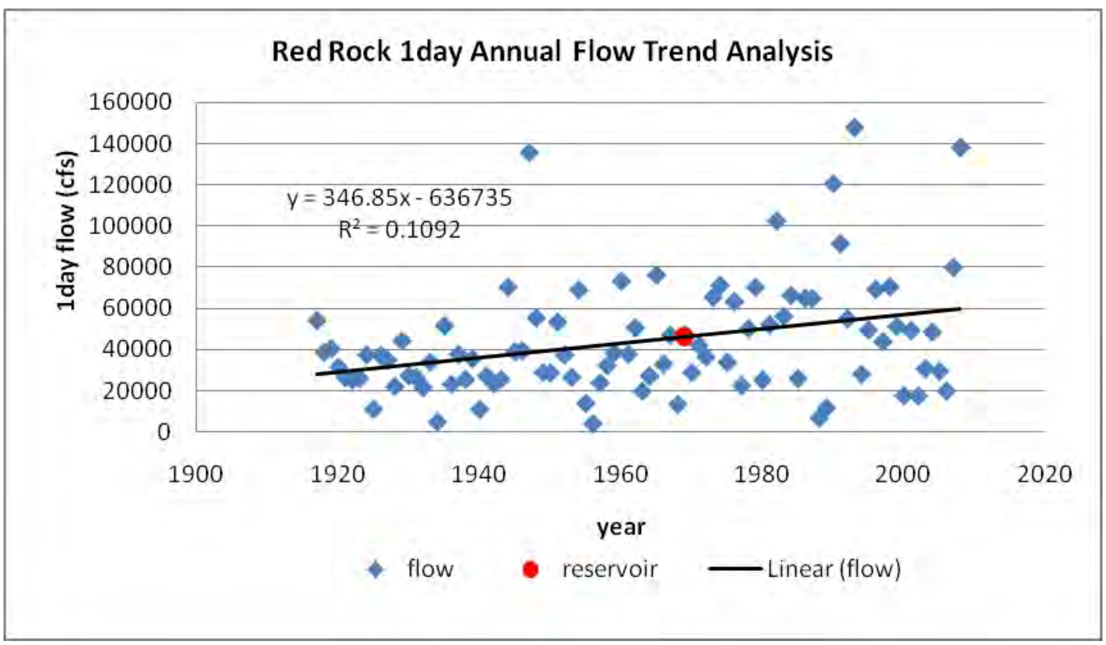

FIGURE 4.2: RED ROCK 1DAY ANNUAL MAXIMUM UNREGULATED FLOW TREND ANALYSIS

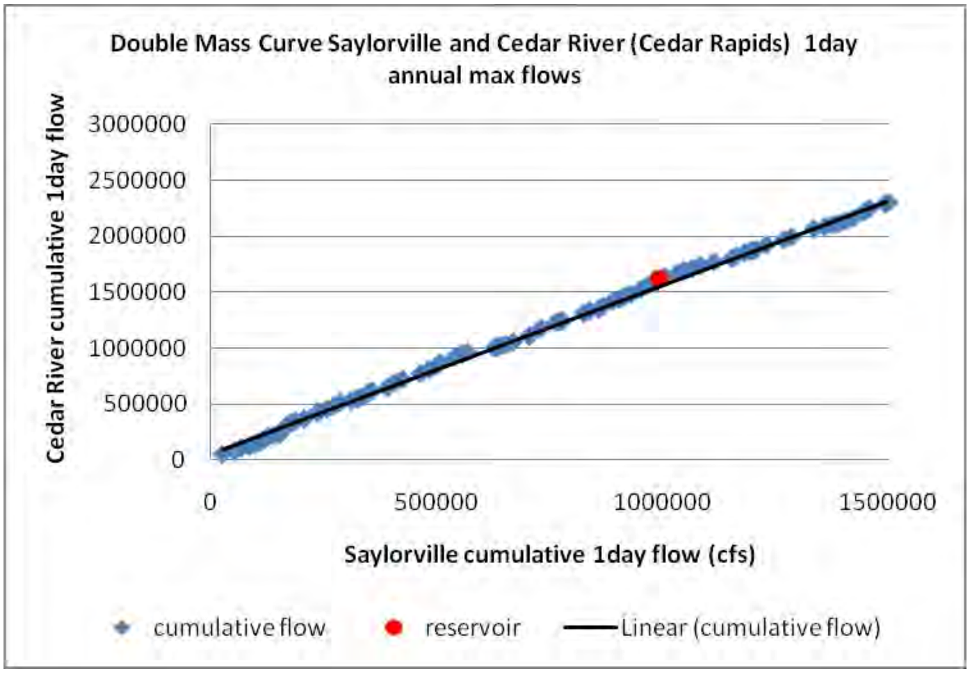

FIGURE 4.3: SAYLORVILLE VS CEDAR RIVER (CEDAR RAPIDS)
1DAY ANNUAL MAXIMUM UNREGULATED FLOW DOUBLE MASS CURVE.

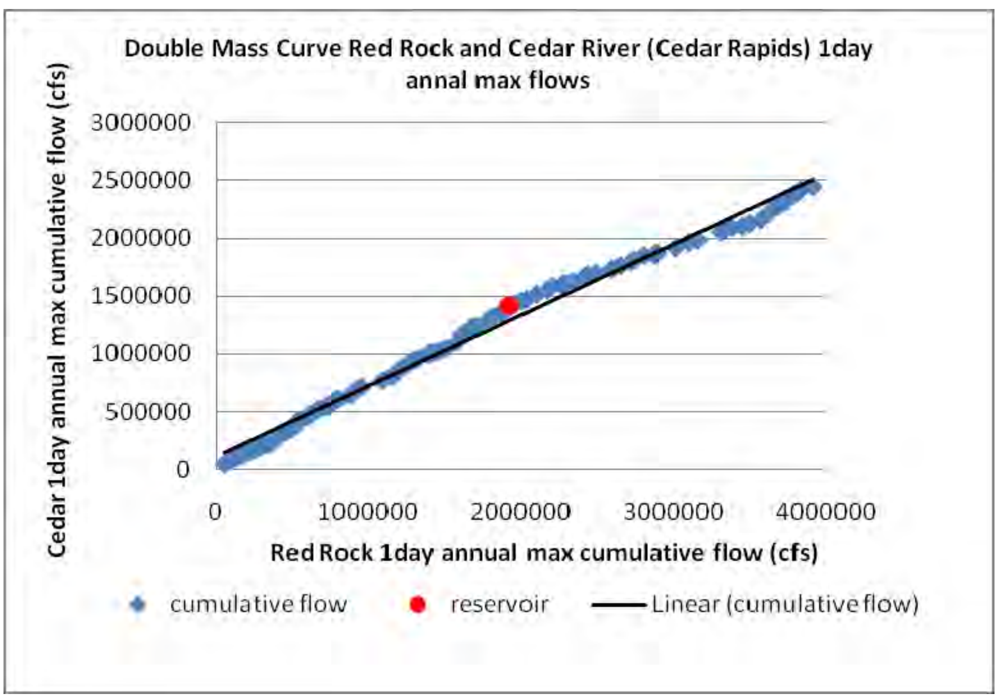

FIGURE 4.4: RED ROCK VS CEDAR RIVER (CEDAR RAPIDS)
1DAY ANNUAL MAXIMUM UNREGULATED FLOW DOUBLE MASS CURVE.

5 Bulletin 17B Volume Duration Frequency Analysis

5.1 Estimated Frequency Curves

The purpose of this section is to provide the results of the VDF analysis of maximum annual unregulated flows for the project area using the Bulletin 17B guidelines (IACWD, 1982). The VDF curves were obtained by using the Bulletin17B procedure for computing gage statistics but deviated from the guidelines in obtaining a regional skew estimate. The Bulletin17B lpiii distribution standard moment estimates of the mean, standard deviation and skew are computed from the period of record of annual maximum flow logarithms$_{(10)}$ for each duration of interest. Second, the low-outlier test is then used to identify, and if necessary, adjust the frequency curve statistics for low-outliers. High outliers are also identified, but none were found in the period of record examined.

The computation procedure deviates from the standard Bulletin 17B application of regional skew because: 1) the interest is on estimating a consistent set of VDF curves rather than a single peak annual frequency curve; and, 2) regional skew values are only available for peak annual frequency curves. The statistics obtained from the period do not ensure that the VDF curve is consistent at a gage because of statistical sampling error. This sampling error can cause intersection of frequency curves for different durations. This problem is avoided by obtaining a regular variation of standard deviation using a regression between the paired mean and standard deviation values for each duration.

In the case of peak annual discharges, the regional skew is obtain from some regional relationship, usually a skew map (see Plate I, IACWD, 1982), and weighted with at-gage skew estimate to obtain an “adopted” skew. However, estimating the regional skew in this manner is not satisfactory, partly because regional skew relationships do not exist for annual maximum flow volume durations; and, partly because the adopted skews would not necessarily result in consistent VDF curves at a gage.

Instead of using an adopted skew value, the decision was made to use an average of the at-gage skews obtained from the gages on the Des Moines and Iowa Rivers as a substitute for the at-gage estimate (as was done in Corps, 2002). The reason for taking this approach is as follows:

- The original guideline development effort by the WRC selected the lpiii distribution; where, period of record log$_{(10)}$ annual peak flows are used in computing using standard moment estimates of the mean and standard deviation, and the regional skew is used in place of the gage skew(not the adopted as is now recommended for peak flow frequency curves). The regional skew is applied in this manner to promote consistency in frequency curve estimates obtained from ever increasing gage record lengths;
- The technical advisory group for the Upper Mississippi Flood Frequency Study (Corps of Engineers, 2000) recommend substituting an average skew because theoretical studies had shown it to be a more accurate approach than using adopted skew when gage record length were on the order of 100 years or greater (see also, Hosking and Wallis, 1997, pg. 148).

Tables 5.1-5.4 show the at-gage statistics, the standard deviation obtained from regression with the mean, the average skew, number of low-outliers, and VDF curves computed for the gages on the Des Moines and Iowa Rivers used in the study. Figures 5.1 and 5.2 provide examples of the difference made by using duration smoothed standard deviation and regional skew versus the at-gage statistics. Figures 5.3-5.7 displays the good correspondence between the at-gage statistic lpiii estimates and the empirical frequency curves estimated from plotting positions for the 1day, 15day and 30 day durations.

Note the small difference in skew values shown for the Des Moines and Iowa River gages shown in Tables 5.1 and 5.2. The differences between the skew values occur because some minor differences occurred between the application of the routing models since the development of frequency curves for the Iowa River (Corps of Engineers, 2009). However, the change in skew values would make very little difference in the Iowa River results.

Tables 5.5 and 5.6 provide estimates of the exceedance probability at each gage, and the average exceedance probability over all gages with corresponding return intervals for the major floods in 1993 and 2008. Interestingly, the likelihood of the 2008 flood volume is less than for 1993 flood for shorter durations, but for volume-durations 30days or greater, has greater likelihood.

Tables 5.7 and 5.8 provide some perspective on the relative magnitude of the 1993 and 2008 events by giving the ratio of the lpiii estimates of the 1000 year flood (0.1% exceedance frequency) to the volume of these events for various durations.

Table 5.1: Des Moines River VDF statistics of log10 annual maximum flows

	duration	[1]mean	[2]StdDev	[3]skew	[4]outlier	[5]$StdDev_{reg}$	[6]$Skew_r$
SAYLORVILLE	1DAY	4.187	0.271	-0.055	1	0.2746	-0.2
	15DAY	3.987	0.289	-0.399	1	0.2837	-0.19
	30DAY	3.878	0.289	-0.37	1	0.2886	-0.18
	60DAY	3.764	0.294	-0.353	1	0.2938	-0.17
	90DAY	3.695	0.297	-0.285	1	0.2969	-0.17
	105DAY	3.668	0.298	-0.275	1	0.2982	-0.17
	120DAY	3.644	0.297	-0.253	1	0.2992	-0.17
SE6th	1DAY	4.44	0.265	-0.082	1	0.2641	-0.2
	15DAY	4.224	0.276	-0.187	1	0.2763	-0.19
	30DAY	4.105	0.281	-0.197	1	0.283	-0.18
	60DAY	3.991	0.289	-0.225	1	0.2894	-0.17
	90DAY	3.922	0.295	-0.13	1	0.2933	-0.17
	105DAY	3.896	0.296	-0.13	1	0.2948	-0.17
	120DAY	3.872	0.295	-0.126	1	0.2961	-0.17
Red Rock/Tracy	1DAY	4.574	0.2536	-0.115	2	0.2498	-0.2
	15DAY	4.332	0.2551	-0.165	2	0.2592	-0.19
	30DAY	4.219	0.2599	-0.098	2	0.2635	-0.18
	60DAY	4.11	0.2667	-0.077	2	0.2678	-0.17
	90DAY	4.04	0.2718	-0.042	2	0.2705	-0.17
	105DAY	4.014	0.2738	-0.065	2	0.2715	-0.17
	120DAY	3.988	0.2738	-0.062	2	0.2725	-0.17
OTTUMWA	1DAY	4.599	0.2459	-0.372	1	0.2442	-0.2
	15DAY	4.376	0.2521	-0.173	2	0.2538	-0.19
	30DAY	4.258	0.2573	-0.112	2	0.2588	-0.18
	60DAY	4.147	0.2627	-0.067	2	0.2636	-0.17
	90DAY	4.078	0.2676	-0.039	2	0.2665	-0.17
	105DAY	4.051	0.2691	-0.051	2	0.2677	-0.17
	120DAY	4.025	0.2687	-0.041	2	0.2688	-0.17
KEOSAUQUA	1DAY	4.614	0.234	-0.223	1	0.2367	-0.2
	15DAY	4.401	0.251	-0.228	2	0.2483	-0.19
	30DAY	4.284	0.257	-0.182	2	0.2546	-0.18
	60DAY	4.171	0.261	-0.135	2	0.2607	-0.17
	90DAY	4.103	0.264	-0.091	2	0.2644	-0.17
	105DAY	4.076	0.266	-0.091	2	0.2659	-0.17
	120DAY	4.05	0.265	-0.073	2	0.2673	-0.17
CORALVILLE	1DAY	4.031	0.285	-0.289	0	0.2782	-0.2
	15DAY	3.833	0.263	-0.095	2	0.2686	-0.19
	30DAY	3.716	0.257	-0.133	2	0.263	-0.18
	60DAY	3.602	0.254	-0.183	2	0.2575	-0.17
	90DAY	3.532	0.254	-0.246	2	0.2541	-0.17
	105DAY	3.505	0.256	-0.274	2	0.2528	-0.17
	120DAY	3.481	0.257	-0.302	2	0.2517	-0.17

Statistics of annual maximum log10(flows) for each duration: [1]mean flow, [2]standard deviation, [3]skew, [4]number of low outliers, [5]standard deviation estimated from regression with mean flow, [6]regional skew

Table 5.2: Iowa River VDF Statistics
(see Corps of Engineers 2009)

	duration	[1]mean	[2]StdDev	[3]skew	[4]outlier	[5]$StdDev_{reg}$	[6]$Skew_r$
CORALVILLE	1DAY	4.031	0.285	-0.289	0	0.2782	-0.20
	15DAY	3.833	0.263	-0.095	2	0.2686	-0.18
	30DAY	3.716	0.257	-0.133	2	0.2630	-0.17
	60DAY	3.602	0.254	-0.183	2	0.2575	-0.17
	90DAY	3.532	0.254	-0.246	2	0.2541	-0.16
	105DAY	3.505	0.256	-0.274	2	0.2528	-0.16
	120DAY	3.481	0.257	-0.302	2	0.2517	-0.16
IOWA CITY	1DAY	4.049	0.284	-0.219	0	0.2770	-0.20
	15DAY	3.845	0.263	-0.055	2	0.2695	-0.18
	30DAY	3.728	0.258	-0.073	2	0.2652	-0.17
	60DAY	3.609	0.265	-0.312	1	0.2608	-0.17
	90DAY	3.544	0.253	-0.119	2	0.2585	-0.16
	105DAY	3.518	0.255	-0.133	2	0.2575	-0.16
	120DAY	3.489	0.267	-0.362	1	0.2564	-0.16
LONE TREE	1DAY	4.153	0.287	0.027	0	0.2851	-0.20
	15DAY	3.937	0.274	-0.116	1	0.2776	-0.18
	30DAY	3.82	0.274	-0.189	1	0.2735	-0.17
	60DAY	3.706	0.271	-0.215	1	0.2696	-0.17
	90DAY	3.637	0.269	-0.27	1	0.2672	-0.16
	105DAY	3.616	0.26	-0.053	2	0.2665	-0.16
	120DAY	3.588	0.27	-0.266	1	0.2655	-0.16
WAPELLO	1DAY	4.568	0.262	-0.321	0	0.2656	-0.20
	15DAY	4.381	0.263	-0.474	0	0.2545	-0.18
	30DAY	4.286	0.247	-0.301	1	0.2488	-0.17
	60DAY	4.187	0.239	-0.182	1	0.2430	-0.17
	90DAY	4.121	0.238	-0.175	1	0.2391	-0.16
	105DAY	4.097	0.239	-0.164	1	0.2377	-0.16
	120DAY	4.076	0.237	-0.179	1	0.2364	-0.16

Statistics of annual maximum log10(flows) for each duration: [1]mean flow, [2]standard deviation, [3]skew, [4]number of low outliers, [5]standard deviation estimated from regression with mean flow, [6]regional skew

Table 5.3: Des Moines River Annual Maximum Flows (cfs/day) versus Exceedance Probability

	duration	**0.5**	**0.1**	**0.02**	**0.01**	**0.005**	**0.002**	**0.001**
SAYLORVILLE	1DAY	15710	34080	52610	60980	69620	81480	90790
	15DAY	9900	22110	34830	40660	46750	55190	61880
	30DAY	7690	17470	27800	32590	37610	44600	50180
	60DAY	5920	13640	21890	25740	29780	35420	39930
	90DAY	5050	11750	19010	22420	26010	31060	35100
	105DAY	4740	11080	17970	21210	24620	29420	33270
	120DAY	4490	10520	17080	20170	23430	28010	31690
SE6th	1DAY	28100	59200	89880	103590	117670	136890	151900
	15DAY	17070	37350	58130	67610	77440	91030	101760
	30DAY	12970	28980	45720	53430	61480	72680	81570
	60DAY	9980	22710	36190	42450	49000	58150	65430
	90DAY	8510	19610	31540	37120	42990	51220	57810
	105DAY	8010	18550	29910	35230	40840	48700	55000
	120DAY	7580	17620	28470	33560	38920	46450	52490
RED ROCK	1DAY	37860	75530	111260	126910	142830	164340	180990
	15DAY	21810	45050	67880	78070	88540	102840	114030
	30DAY	16800	35300	53770	62090	70690	82490	91770
	60DAY	13080	27870	42810	49570	56580	66230	73830
	90DAY	11150	24020	37180	43180	49420	58050	64890
	105DAY	10480	22660	35140	40850	46780	55000	61500
	120DAY	9890	21460	33340	38780	44440	52270	58480
TRACY	1DAY	38220	77370	114880	131400	148250	171070	188780
	15DAY	21870	45610	69080	79600	90430	105250	116860
	30DAY	16840	35630	54480	63000	71800	83910	93440
	60DAY	13110	28060	43200	50070	57180	67000	74730
	90DAY	11150	24090	37360	43420	49730	58450	65360
	105DAY	10500	22760	35350	41110	47100	55400	61970
	120DAY	9890	21500	33450	38920	44610	52500	58750
OTTUMWA	1DAY	40470	80590	118570	135200	152100	174940	192610
	15DAY	24190	49650	74540	85630	97010	112540	124670
	30DAY	18420	38430	58320	67260	76480	89120	99050
	60DAY	14270	30180	46160	53370	60830	71090	79170
	90DAY	12140	25930	39950	46330	52950	62090	69320
	105DAY	11430	24510	37820	43890	50180	58890	65760
	120DAY	10770	23160	35800	41570	47560	55840	62390
KEOSAUQUA	1DAY	41870	81630	118680	134780	151090	173030	189940
	15DAY	25610	51760	77030	88220	99680	115260	127400
	30DAY	19550	40300	60730	69870	79280	92150	102240
	60DAY	15080	31620	48120	55550	63220	73760	82030
	90DAY	12880	27350	41980	48620	55500	64990	72490
	105DAY	12110	25810	39720	46040	52600	61650	68790
	120DAY	11410	24410	37650	43670	49930	58570	65400

Table 5.4: Iowa River Annual Maximum Flows (cfs/day) versus Exceedance Probability (see Corps of Engineers, 2009)

	duration	0.5	0.1	0.02	0.01	0.005	0.002	0.001
CORALVILLE	1DAY	10970	24050	37330	43350	49580	58150	64890
	15DAY	6930	14850	22820	26430	30160	35290	39330
	30DAY	5290	11170	17050	19710	22460	26240	29210
	60DAY	4070	8450	12800	14750	16760	19510	21670
	90DAY	3460	7130	10760	12390	14070	16380	18190
	105DAY	3250	6670	10050	11570	13130	15270	16940
	120DAY	3070	6290	9460	10880	12350	14350	15920
IOWA CITY	1DAY	11430	24980	38700	44920	51340	60170	67110
	15DAY	7130	15300	23560	27290	31160	36480	40670
	30DAY	5440	11550	17710	20490	23380	27340	30470
	60DAY	4130	8670	13200	15240	17340	20230	22510
	90DAY	3560	7420	11280	13030	14830	17300	19250
	105DAY	3350	6970	10580	12210	13890	16190	18010
	120DAY	3130	6500	9850	11350	12910	15050	16720
LONE TREE	1DAY	14540	32490	50990	59440	68200	80300	89850
	15DAY	8820	19360	30200	35140	40280	47380	53000
	30DAY	6730	14630	22720	26420	30260	35560	39760
	60DAY	5170	11120	17170	19920	22770	26710	29810
	90DAY	4410	9430	14540	16870	19280	22610	25250
	105DAY	4200	8970	13810	16010	18300	21450	23950
	120DAY	3940	8380	12890	14940	17060	19990	22310
WAPELLO	1DAY	37740	79830	121500	140150	159310	185490	205950
	15DAY	24470	50330	75650	86930	98520	114330	126690
	30DAY	19640	39810	59430	68160	77110	89330	98870
	60DAY	15630	31160	46090	52690	59450	68630	75780
	90DAY	13410	26490	39020	44560	50230	57940	63940
	105DAY	12690	24960	36690	41870	47160	54350	59950
	120DAY	12090	23690	34750	39630	44610	51370	56630

Table 5.5: 1993 Event Exceedance Probability for each duration average exceedance probability and return interval

	1DAY	15DAY	30DAY	60DAY	90DAY	105DAY	120DAY
SAYLORVILLE	0.0276	0.0108	0.0096	0.0075	0.0089	0.0072	0.0050
SE6th	0.0044	0.0068	0.0079	0.0083	0.0090	0.0084	0.0065
RED ROCK	0.0040	0.0041	0.0047	0.0070	0.0070	0.0069	0.0051
TRACY	0.0050	0.0047	0.0051	0.0074	0.0072	0.0072	0.0052
OTTUMWA	0.0076	0.0037	0.0037	0.0056	0.0053	0.0061	0.0046
KEOSAUQUA	0.0113	0.0043	0.0042	0.0065	0.0067	0.0068	0.0051
CORALVILLE	0.0228	0.0099	0.0058	0.0020	0.0015	0.0018	0.0019
IOWA CITY	0.0185	0.0092	0.0048	0.0015	0.0014	0.0016	0.0016
LONE TREE	0.0070	0.0085	0.0037	0.0014	0.0012	0.0014	0.0014
WAPELLO	0.0267	0.0070	0.0037	0.0017	0.0012	0.0014	0.0015
average	0.0135	0.0069	0.0053	0.0049	0.0049	0.0049	0.0038
[1]average return interval	74	145	188	204	202	205	264

[1]Return interval in years

Table 5.6: 2008 Event Exceedance Probability for each duration average exceedance probability and return interval

	1DAY	15DAY	30DAY	60DAY	90DAY	105DAY	120DAY
SAYLORVILLE	0.0160	0.0162	0.0238	0.0408	0.0456	0.0502	0.0528
SE6th	0.0120	0.0096	0.0155	0.0331	0.0387	0.0441	0.0457
RED ROCK	0.0369	0.0104	0.0115	0.0179	0.0230	0.0254	0.0257
TRACY	0.0075	0.0064	0.0089	0.0150	0.0198	0.0223	0.0226
OTTUMWA	0.0122	0.0055	0.0093	0.0171	0.0232	0.0212	0.0204
KEOSAUQUA	0.0086	0.0056	0.0112	0.0204	0.0251	0.0235	0.0219
CORALVILLE	0.0041	0.0037	0.0059	0.0075	0.0062	0.0049	0.0051
IOWA CITY	0.0050	0.0036	0.0051	0.0071	0.0061	0.0050	0.0047
LONE TREE	0.0044	0.0034	0.0061	0.0071	0.0060	0.0054	0.0050
WAPELLO	0.0017	0.0036	0.0079	0.0089	0.0075	0.0061	0.0064
average	0.0108	0.0068	0.0105	0.0175	0.0201	0.0208	0.0210
[1]average return interval	92	147	95	57	50	48	48

[1]Return interval in years

Table 5.7: Ratio Volume for Duration-Frequency (regional statistics) To 1993 Event Volume

	%prob	1day	15day	30day	60day	90day	105day	120day
Saylorville	0.1	1.86	1.55	1.53	1.46	1.52	1.46	1.35
Se6th		1.26	1.39	1.45	1.48	1.52	1.50	1.42
Red Rock		1.22	1.25	1.28	1.39	1.40	1.40	1.32
Tracy		1.27	1.28	1.31	1.41	1.41	1.41	1.33
Ottumwa		1.36	1.22	1.23	1.33	1.32	1.36	1.29
Keosauqua		1.44	1.25	1.25	1.36	1.38	1.39	1.31
Coralville		1.79	1.48	1.34	1.11	1.06	1.09	1.10
Iowa City		1.70	1.47	1.30	1.06	1.05	1.08	1.07
Lone Tree		1.41	1.46	1.25	1.05	1.03	1.06	1.05
Wapello		1.81	1.36	1.22	1.07	1.03	1.05	1.06

Table 5.8: Ratio Volume for Duration-Frequency (regional statistics) To 2008 Event Volume

	%Prob	1day	15day	30day	60day	90day	105day	120day
Saylorville	0.1	1.64	1.69	1.88	2.20	2.31	2.39	2.43
Se6th		1.52	1.49	1.68	2.06	2.18	2.28	2.31
Red Rock		1.85	1.47	1.52	1.68	1.80	1.85	1.86
Tracy		1.37	1.35	1.45	1.62	1.75	1.80	1.81
Ottumwa		1.48	1.31	1.45	1.66	1.79	1.76	1.75
Keosauqua		1.37	1.30	1.50	1.71	1.82	1.80	1.77
Coralville		1.26	1.24	1.34	1.39	1.34	1.28	1.29
Iowa City		1.31	1.23	1.31	1.39	1.34	1.30	1.28
Lone Tree		1.29	1.23	1.36	1.40	1.35	1.33	1.31
Wapello		1.08	1.22	1.39	1.41	1.36	1.31	1.32

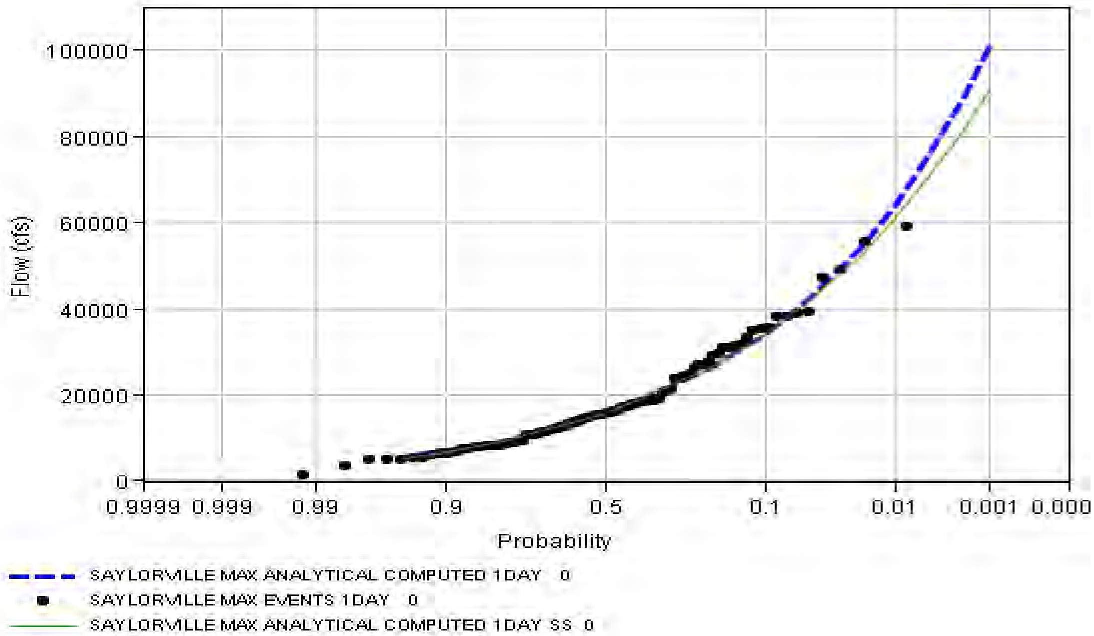

FIGURE 5.1: SAYLORVILLE 1DAY SAMPLE AND SMOOTHED STATISTICS (SS) FREQUENCY CURVES

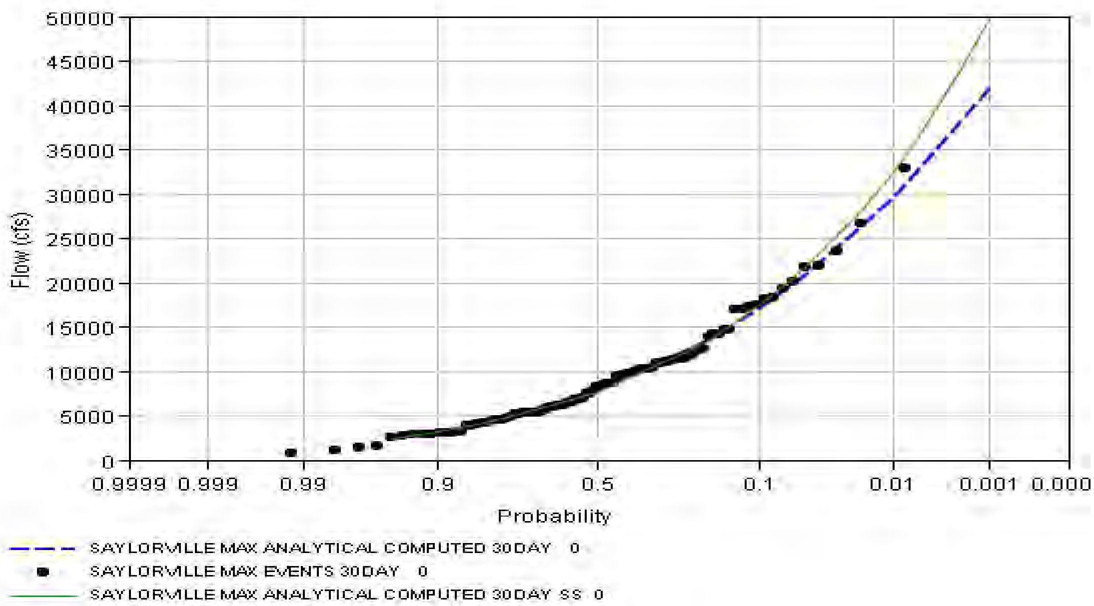

FIGURE 5.2: SAYLORVILLE 30DAY SAMPLE AND SMOOTHED STATISTICS (SS) FREQUENCY CURVES

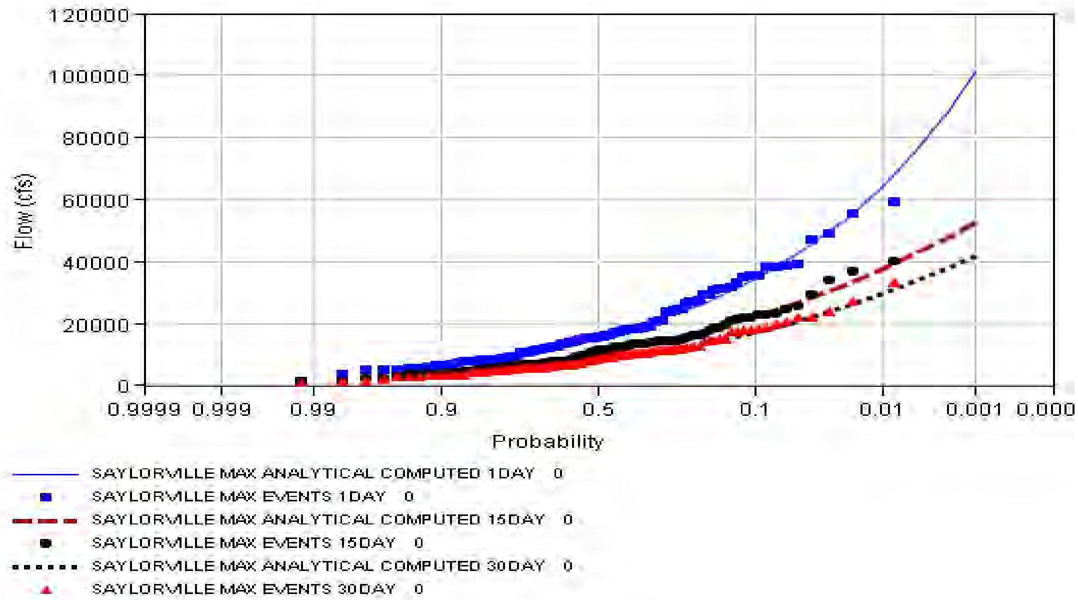

FIGURE 5.3: SAYLORVILLE 1DAY, 15DAY AND 30DAY VDF CURVES

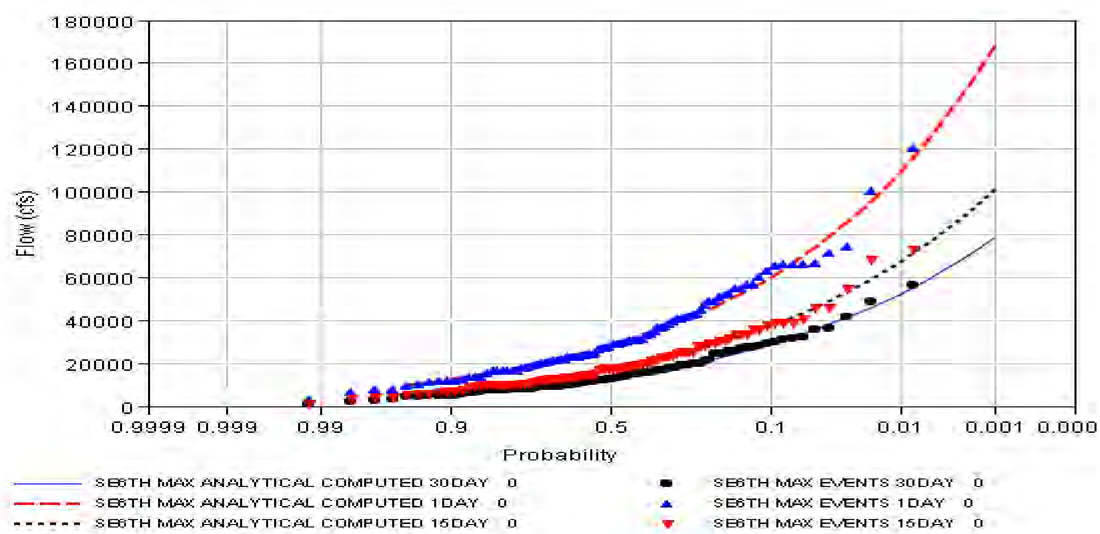

FIGURE 5.4: SE6TH 1DAY, 15DAY AND 30DAY VDF CURVES

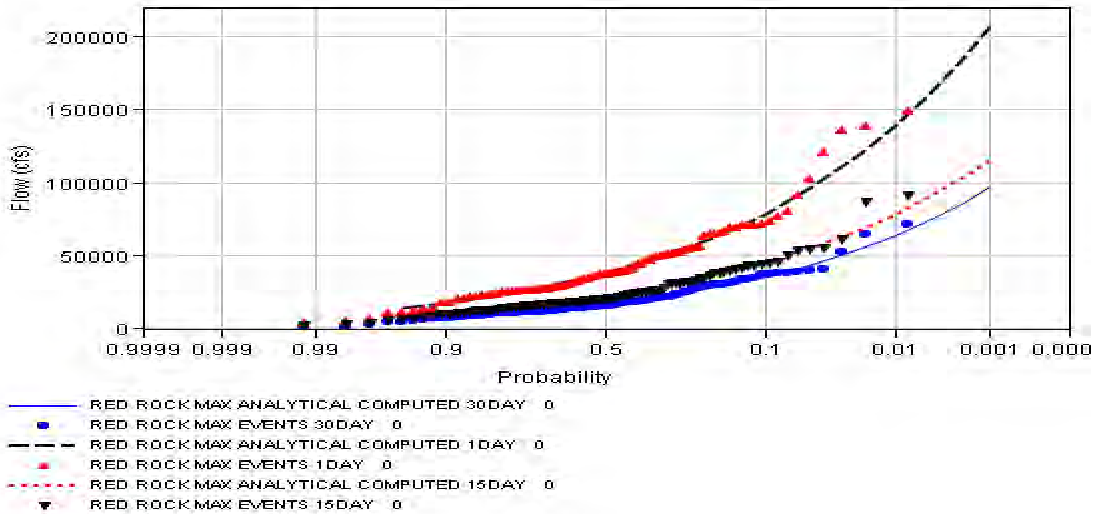

FIGURE 5.5: RED ROCK 1DAY, 15DAY AND 30DAY VDF CURVES

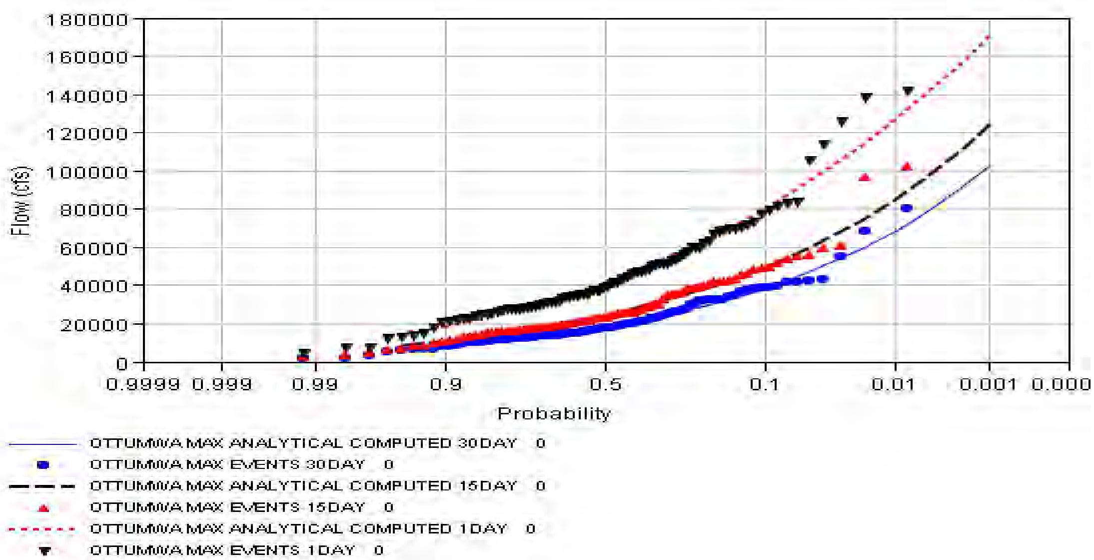

FIGURE 5.6: OTTUMWA 1DAY, 15DAY AND 30DAY VDF CURVES

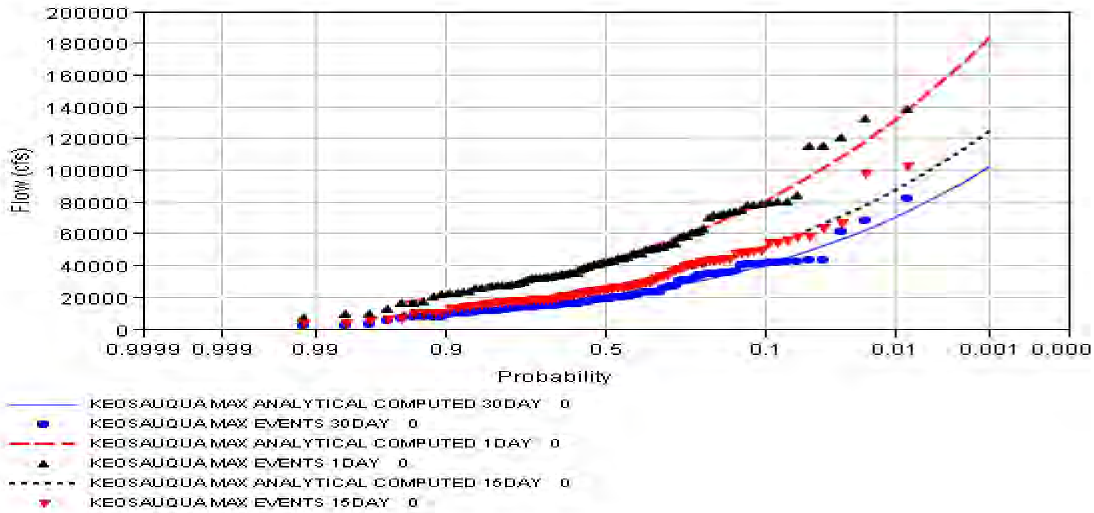

FIGURE 5.7: KEOSAQUA1DAY, 15DAY AND 30DAY VDF CURVES

5.2 Comparison with Corps 2002 study

Apparently, a marked increase in the frequency of high flow volumes into Saylorville and Red Rock Reservoirs has occurred since the completion of the Corps' 2002 study. Table 5.9 compares the exceedance probability and corresponding return intervals for the 1993 event using the frequency analysis from the Corps' 2002 and present study. As can be seen, the estimated frequency of the event volumes has increased significantly. The reason for this increase is readily apparent from the increase in the mean and standard deviation of the period of record flows due to the addition record since 1993 as is shown in Table 5.10. The increase frequency is clearly due to the much wetter period that has occurred since 1993.

The increase in flood frequency has particular importance for Saylorville. As will be discussed, in section 8, the critical duration will be reduced from 60 to 30 days. Consequently, the 1993 regulated outflow will have approximately the same return interval as the 30day inflow. This means that the 100 year flood will correspond to the 1993 regulated maximum 1day flow. This is a significant increase from the 2002 study. Also interesting to note is that 1993 maximum 1day inflow to Saylorville is 48800 cfs/day and the regulated 1day 44300 cfs/day. This means that Saylorville reservoir does not provide any significant reduction in the 100 year event.

Table 5.9 Comparison of Current and present estimates of 1993 event return interval

			Corps 2002 Study		Current Study	
	duration	unregulated flow	[2]prob	return interval	prob	return interval
Saylorville regulated 1day	[1]60 day	27350	0.0016	625	0.0075	133
50760	30 day	32850	0.0059	169	0.0096	104
Red Rock regulated 1day	[1]120day	44320	0.0031	323	0.0070	143
148120	30day	71430	0.0020	500	0.0047	213

[1]Critical inflow duration found in the Corps' 2002 study, [2]Exceedance probability

Table 5.10 Period of Record Statistics

Saylorville	*[1]1917-1994*		*1917-2008*		*[2]1995-2008*	
	mean	[3]sdev	mean	sdev	mean	sdev
	8650	5930	9230	6020	12450	6050
Red Rock	*[1]1917-1994*		*1917-2008*		*1995-2008*	
	mean	sdev	mean	sdev	mean	sdev
	17390	10750	19560	12650	25630	15280

[1]Corps' 2002 study, [2]Additional period of record,
[3]standard deviation

6 Low Outlier censoring Analysis

The intention of the Bulletin17B low outlier censoring threshold is to obtain better correspondence between the top plotting position in the gage record and the estimated lpiii distribution. The purpose of this section is to examine if this low outlier censoring threshold developed for peak annual flows is relevant for volume duration frequency curves. Note that this analysis was originally performed for the Iowa River regulated flow frequency study and the following is taken from the report describing that study (Corps of Engineers, 2009).

The Bulletin 17B low outlier censoring is intended to prevent small frequent flows for having undue influence on the estimates of large, infrequent flows. This is a particular problem when using logarithms of flows with sample standard moments to estimate a frequency distribution. The problem can be seen by considering two observed flows of magnitude 10 and 1000. The $\log_{10}$ values for these two values are 1 and 3, reducing the difference considerably. This reduced difference gives the smaller values much more influence in the computation of the standard deviation, which is proportional to the squared differences with the mean, and the skew, which is proportional to the cubed difference with the mean.

The difficulty of course, is determining the appropriate threshold flow below which low-values should be censored. Thomas (1985, pg.330) describes how this threshold was established by the Bulletin17B work group members:

> *The first step in the analysis of the outlier tests was to apply the outlier test to observed peak discharge data for 50 gaging stations for low outlier detection only. All work group members subjectively identified the number of outliers for each station. On the basis of the number of low outliers identified by each test as compared to the consensus of the work group, 50 outlier test were reduced to 10 tests to detect high outliers as well and apply the test to simulated log-Pearson Type III data. ….The 10 outlier tests were applied to simulated samples and the sample estimates of the 2, 10 and 100 year flood discharges were compared to the true values. On the basis of the bias and root mean square of the 2-, 10- and 100 year flood discharges, the 10 tests were reduced to 6. On the basis of the number of outliers identified by each test as compared to the consensus of the work group, the Grubbs and Beck tests using either zero skew or generalized at a 10% level of significance gave the most reasonable results.*

The Bulletin 17B work group decided to use the zero skew approach because it would not be affected by the various estimates of regional (generalized) skew available from skew maps and zero skew was consistent with the development of the Grubbs and Beck statistic.

Reapplying the entire Bulletin 17B work group procedure for identifying a low-outlier criterion is beyond the scope of this study. Instead, a page will be taken out of the group's procedure by examining the performance of low-outlier censoring threshold for a large group of gages, this time located in Iowa. The focus will be on the bias of lpiii estimates in comparison to the top plotting position in the flow period of record. Invariably, the 1993 or 2008 events are the top ranking floods for all flow durations greater than 1day.

Three different low-outlier censoring levels used were:

- The Bulletin 17B low outlier threshold determined by the Grubbs and Beck statistic at a 10% significance level;
- Censoring at the 0.70 exceedance probability;
- Censoring at the 0.57 exceedance probability

The 0.57 exceedance probability was used to approximate channel capacity while still ensuring enough data points to be able to apply the Bulletin 17B conditional probability adjustment formulas.

Practically speaking, the Bulletin 17B low outlier censoring level does results in a small number of values being censored in the period of record, perhaps one or two (see Tables 6.1 and 6.2). Censoring a few relatively small values is important in that it does not reduce significantly the number of flow magnitudes to estimate flow frequency curve statistics. Note, that censoring does not reduce the effective record length (see discussion of conditional probability adjustment used in the low-outlier analysis in Bulletin 17B, IACWD, 1982, Appendix 5). However, the flow magnitude of the censored values is not used in computing distribution statistics.

The Bulletin 17B work group did not investigate the affect on prediction accuracy of censoring flows by performing a split sample investigation as described in Bulletin 17B (IACWD, 1982, appendix 14). This is not likely to affect prediction accuracy because of the few number of values censored using the Bulletin17B threshold. However, a threshold based on the 0.7 or 0.57 exceedance probability will censor up to almost half the period of record. This has a much larger influence on the information available to estimate flow frequency curve statistics. In the Upper Mississippi Study (Corps of Engineers, 2000, Sections 4 and 5), flow frequency curve distribution/estimation pairings were compared in split sample studies. Pairings using full data sets almost uniformly out performed methods censoring the data below the median (50% exceedance probability). The censored methods used the log-Normal and Gumbel distributions, not the lpiii. Consequently, some caution needs to be used in interpreting the relative value of censoring thresholds because of the potential loss of prediction accuracy as the number of censored values increases. There is a trade-off between censoring values to obtain a better correspondence between the flow frequency distribution and the top plotting position and the potential reduction in prediction accuracy obtained from not using censored flow magnitudes to estimate distribution statistics.

The low outlier censoring tests were applied to the Iowa and Des Moines River gages discussed in the previous section, the Iowa gages described in Table 6.1 and the very large area gages used in the Upper Mississippi study described in Table 6.2.

The comparisons between censoring thresholds was made by computing the average over all gages of the fraction difference between the top plotting position and the lpiii distribution prediction as a measure of bias. The fraction difference is computed as:

fraction difference = (lpiii flow prediction – top plotting position flow)/top plotting position flow

The average fraction difference computed for the Iowa Gages (Table 6.1) resulted in little difference between methods as is shown in Table 6.3. The Bulletin17B threshold was best for the 15day and 30day annual maximum flows, and, the censoring threshold was best for the remaining durations. The range in error was not a function of record length as can be seen from Figure 6.1 for the 1day duration, which is typical of all the other durations. What is interesting from this plot is how the range in error decreases for the exceedance probability thresholds which censor more flows.

The average fraction differences computed for the Iowa gages were compared to Iowa and Des Moines River gages used in this study and the Upper Mississippi River gages in Tables 6.4 and 6.5 for the 1day and 30day duration annual maximum volumes. The difference between the average fractional estimates are typical for other durations. Unlike the Iowa gage comparisons, the exceedance probability thresholds perform somewhat better than the Bulletin17B threshold values. Interestingly, the 0.7 exceedance probability threshold performs somewhat better than the 0.57 exceedance probability for the Iowa and Des Moines River gages.

The average bias does not provide a complete picture of the value of a particular censoring threshold. The distribution of fraction difference shown in Figures 6.2-6.4 for the 1day duration and Figures 6.5-6.7 for the 30day duration reveal that the Bulletin17B threshold provides a more symmetrical distribution about zero difference than the other methods, but a much greater range of errors for the Iowa gages. Conclusion regarding the other gages is more difficult. The same trend in the symmetry of errors about zero seems to be apparent as with the Iowa gages for the different censoring thresholds; but the difference in range between thresholds is more difficult to discern for the Iowa and Des Moines Rivers. Evaluation of the errors for the Iowa and Des Moines River, and, the Mississippi gages is certainly hampered by the small number of gages.

In conclusion, the improvement obtained by increasing the number values censored is not apparent based on the average fraction difference alone. This is particularly true for the large number of Iowa gages available. The advantage of censoring more values comes from the reduction in the range of fractional difference about zero difference.

There is no doubt that censoring more values results in greater correspondence between the top plotting position and the lpiii flow frequency curve predictions. See for example the comparisons shown in Figures 6.8-6.11 comparing plotting positions and lpiii 30day and 120day frequency curve for different censoring thresholds at Coralville Reservoir and Wapello on the Iowa River. The correspondence of plotting position and lpiii prediction is superior when more flows are censored than indicated by the Bulletin17B threshold.

So what censoring level should be selected? Improved accuracy cannot be used as an argument since no split sample testing was performed. The arguments in favor of the Bulletin 17B threshold are that:

- it is the regulatory method;
- it performs as well as the other threshold methods in terms of fractional difference;
- the distribution of differences about zero difference is more symmetrical than the other threshold methods;
- lpiii frequency curve statistics are estimated from more flows, potential resulting in greater prediction accuracy.

The arguments in favor of using a greater censoring level than Bulletin 17B:

- the average fractional difference is comparable to the error obtained from the Bulletin 17B threshold;
- the range in fractional difference is considerably smaller than that from the Bulletin 17B threshold;
- the correspondence between the lpiii predictions and top plotting position for the Iowa; and Des Moines River is better than when using the Bulletin 17B threshold;

Basically, the analysis of all the gages does not present enough evidence to deviate from the regulatory method. Selecting an alternative to Bulletin 17B based on a comparison with only the Iowa River and Des Moines River gages would not be appropriate because of limited number of gages and the potential for large sampling error.

Table 6.1: Additional Iowa Gages Used in Low Censoring Analysis

USGS Gage	Description	Area	Years
5412500	Turkey River at Garber, IA	1545	37
5418500	Maquoketa River near Maquoketa, IA	1553	41
5420560	Wapsipinicon River near Elma, IA	95.2	15
5422000	Wapsipinicon River near De Witt, IA	2336	35
5422470	Crow Creek at Bettendorf, IA	17.8	12
5449000	East Branch Iowa River near Klemme, IA	133	15
5449500	Iowa River near Rowan, IA	429	31
5451500	Iowa River at Marshalltown, IA	1532	37
5451700	Timber Creek near Marshalltown, IA	118	25
5451900	Richland Creek near Haven, IA	56.1	26
5452000	Salt Creek near Elberon, IA	201	25
5452200	Walnut Creek near Hartwick, IA	70.9	23
5453000	Big Bear Creek at Ladora, IA	189	27
5453100	Iowa River at Marengo, IA	2794	22
5454300	Clear Creek near Coralville, IA	98.1	25
5455500	English River at Kalona, IA	574	29
5457700	Cedar River at Charles City, IA	1054	15
5458000	Little Cedar River near Ionia, IA	306	23
5458500	Cedar River at Janesville, IA	1661	34
5458900	West Fork Cedar River at Finchford, IA	846	25
5459500	Winnebago River at Mason City, IA	526	36
5462000	Shell Rock River at Shell Rock, IA	1746	22
5463000	Beaver Creek at New Hartford, IA	347	28
5463500	Black Hawk Creek at Hudson, IA	303	20
5464000	Cedar River at Waterloo, IA	5146	27
5464500	Cedar River at Cedar Rapids, IA	6510	44
5470000	South Skunk River near Ames, IA	315	34
5470500	Squaw Creek at Ames, IA	204	20
5471000	South Skunk River below Squaw Creek near Ames, IA	556	21
5471200	Indian Creek near Mingo, IA	276	18
5471500	South Skunk River near Oskaloosa, IA	1635	29
5472500	North Skunk River near Sigourney, IA	730	26
5473400	Cedar Creek near Oakland Mills, IA	530	10
5412500	Turkey River at Garber, IA	1545	37

Table 6.1: Additional Iowa Gages Used in Low Censoring Analysis (continued)

USGS Gage	Description	Area	Years
5476500	Des Moines River at Estherville, IA	1372	22
5479000	East Fork Des Moines River at Dakota City, IA	1308	29
5480500	Des Moines River at Fort Dodge, IA	4190	33
5481000	Boone River near Webster City, IA	844	33
5481950	Beaver Creek near Grimes, IA	358	21
5482300	North Raccoon River near Sac City, IA	700	18
5482500	North Raccoon River near Jefferson, IA	1619	28
5483450	Middle Raccoon River near Bayard, IA	375	12
5484000	South Raccoon River at Redfield, IA	994	28
5484500	Raccoon River at Van Meter, IA	3441	42
5484800	Walnut Creek at Des Moines, IA	78.4	17
5485640	Fourmile Creek at Des Moines, IA	92.7	17
5486000	North River near Norwalk, IA	349	27
5486490	Middle River near Indianola, IA	489.4	28
5487470	South River near Ackworth, IA	460	26
5487980	White Breast Creek near Dallas, IA	333	21
5489000	Cedar Creek near Bussey, IA	374	28
5494300	Fox River at Bloomfield, IA	87.7	11
6483500	Rock River near Rock Valley, IA	1592	25
6485500	Big Sioux River at Akron, IA	8424	33
6602400	Monona-Harrison Ditch near Turin, IA	900	29
6605000	Ocheyedan River near Spencer, IA	426	13
6605850	Little Sioux River at Linn Grove, IA	1548	14
6606600	Little Sioux River at Correctionville, IA	2500	37
6607200	Maple River at Mapleton, IA	669	31
6607500	Little Sioux River near Turin, IA	3526	32
6608500	Soldier River at Pisgah, IA	407	27
6609500	Boyer River at Logan, IA	871	33
6807410	West Nishnabotna River at Hancock, IA	609	22
6808500	West Nishnabotna River at Randolph, IA	1326	27
6809210	East Nishnabotna River near Atlantic, IA	436	20
6809500	East Nishnabotna River at Red Oak, IA	894	32
6810000	Nishnabotna River above Hamburg, IA	2806	31
6898000	Thompson River at Davis City, IA	701	36
6903400	Chariton River near Chariton, IA	182	16

Table 6.2 Upper Mississippi Gages

Location	Area	Years
Winona	36800	80
Saint Paul	59200	110
Dubuque	82000	71
Clinton	85600	111
Keokuk	119000	100
Hannibal	137000	100

Table 6.3: Comparison of Average fraction bias for Low Outlier Censoring Methods for all USGS gages[1]

	average			maximum			minimum		
censoring	**[2]Bulletin 17B**	**[3]p0.7**	**[4]p0.57**	**Bulletin 17B**	**p0.7**	**p0.57**	**Bulletin 17B**	**p0.7**	**p0.57**
1day	-0.031	0.042	-0.026	0.438	0.249	0.003	-0.406	-0.246	-0.101
15day	-0.014	0.043	-0.021	0.378	0.342	0.018	-0.352	-0.163	-0.092
30 day	-0.017	0.034	-0.021	0.374	0.414	0.024	-0.310	-0.213	-0.074
60 day	-0.034	0.030	-0.021	0.284	0.499	0.014	-0.331	-0.149	-0.080
90 day	-0.045	0.033	-0.021	0.259	0.515	0.013	-0.320	-0.178	-0.088
105 day	-0.045	0.033	-0.021	0.259	0.515	0.013	-0.320	-0.178	-0.088
120day	-0.059	0.029	-0.020	0.309	0.535	0.014	-0.361	-0.183	-0.095

[1]Fraction bias = (lpiii distribution estimated flow – top plotting position)/top plotting position
[2]Bulletin 17B low outlier threshold, [3]censor below 0.7 exceedance probability,[4] censor below 0.57 exceedance probability

Table 6.4: Comparison of Average fraction bias for Low Outlier Censoring Methods 1day duration: USGS, Iowa and Des Moines River and Upper Mississippi River[1]

	average			maximum			minimum		
censoring	[2]Bulletin 17B	[3]p0.7	[4]p0.57	Bulletin 17B	p0.7	p0.57	Bulletin 17B	p0.7	p0.57
usgs	-0.031	0.042	-0.026	0.438	0.249	0.003	-0.406	-0.246	-0.101
I&D[4]	-0.058	0.020	-0.011	0.154	0.118	0.074	-0.266	-0.167	-0.187
Mississippi	-0.076	0.047	-0.006	-0.018	0.149	0.007	-0.115	0.005	-0.014

[1]Fraction bias = (lpiii distribution estimated flow – top plotting position)/top plotting position
[2]Bulletin 17B low outlier threshold, [3]censor below 0.7 exceedance probability, [4]censor below 0.57 exceedance probability, [4]Iowa and Des Moines River Gages

Table 6.5: Comparison of Average fraction bias for Low Outlier Censoring Methods 30day duration: USGS, Iowa and Des Moines River and Upper Mississippi River[1]

	average			maximum			minimum		
censoring	Bulletin 17B	[3]p0.7	[4]p0.57	Bulletin 17B	p0.7	p0.57	Bulletin 17B	p0.7	p0.57
usgs	-0.017	0.034	-0.021	0.374	0.414	0.024	-0.310	-0.213	-0.074
I&D[4]	-0.093	-0.009	-0.051	-0.015	0.052	0.026	-0.199	-0.050	-0.099
Mississippi	-0.091	0.040	-0.002	-0.010	0.088	0.006	-0.205	-0.011	-0.011

[1]Fraction bias = (lpiii distribution estimated flow – top plotting position)/top plotting position
[2]Bulletin 17B low outlier threshold, [3]censor below 0.7 exceedance probability, [4]censor below 0.57 exceedance probability, [4]Iowa and Des Moines River Gages

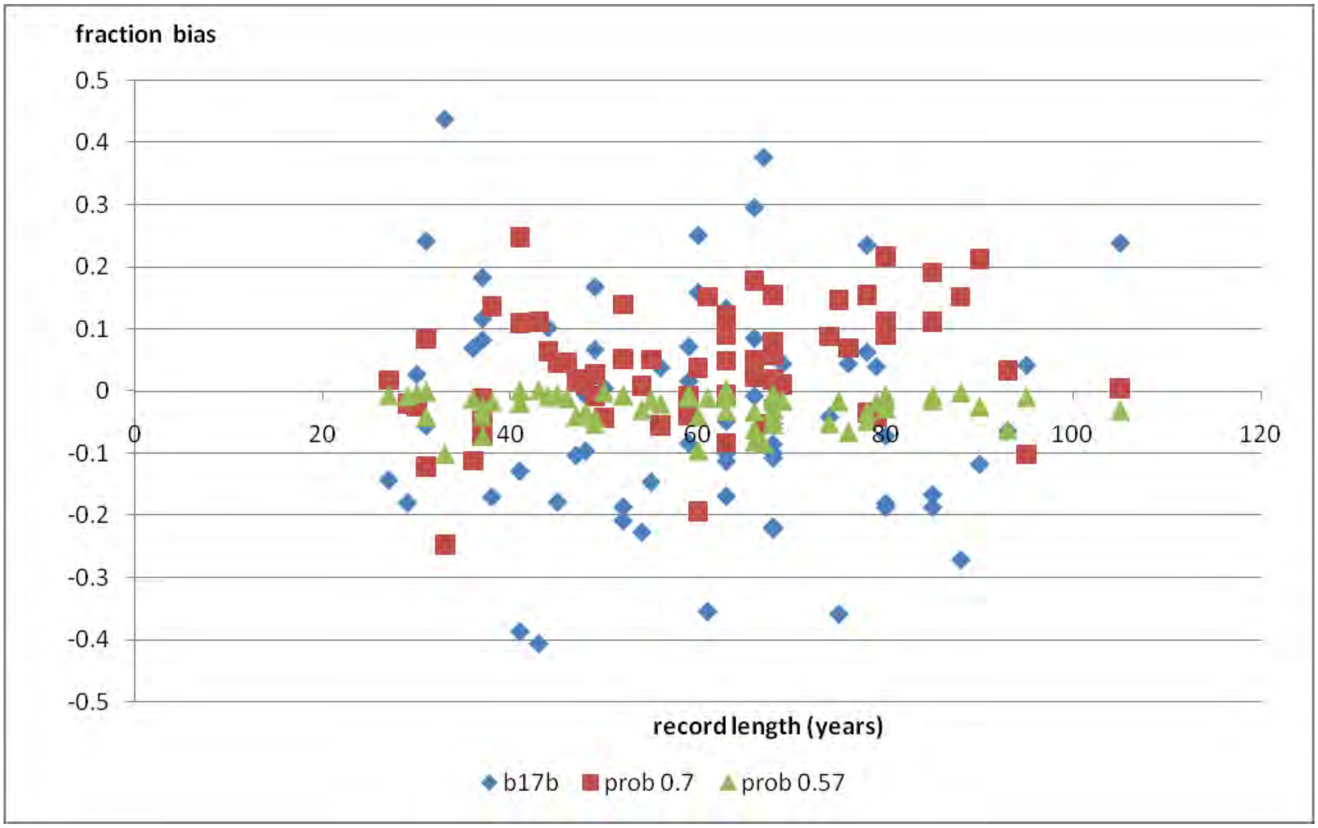

FIGURE 6.1: USGS GAGES FRACTION BIAS FOR BULLETIN 17B CENSORING THRESHOLD
(censoring threshold equal to 0.7 exceedance probability (prob0.7), censoring threshold equal to exceedance probability 0.57
note: fraction bias equals (lpiii estimated flow – plotting position)/plotting position

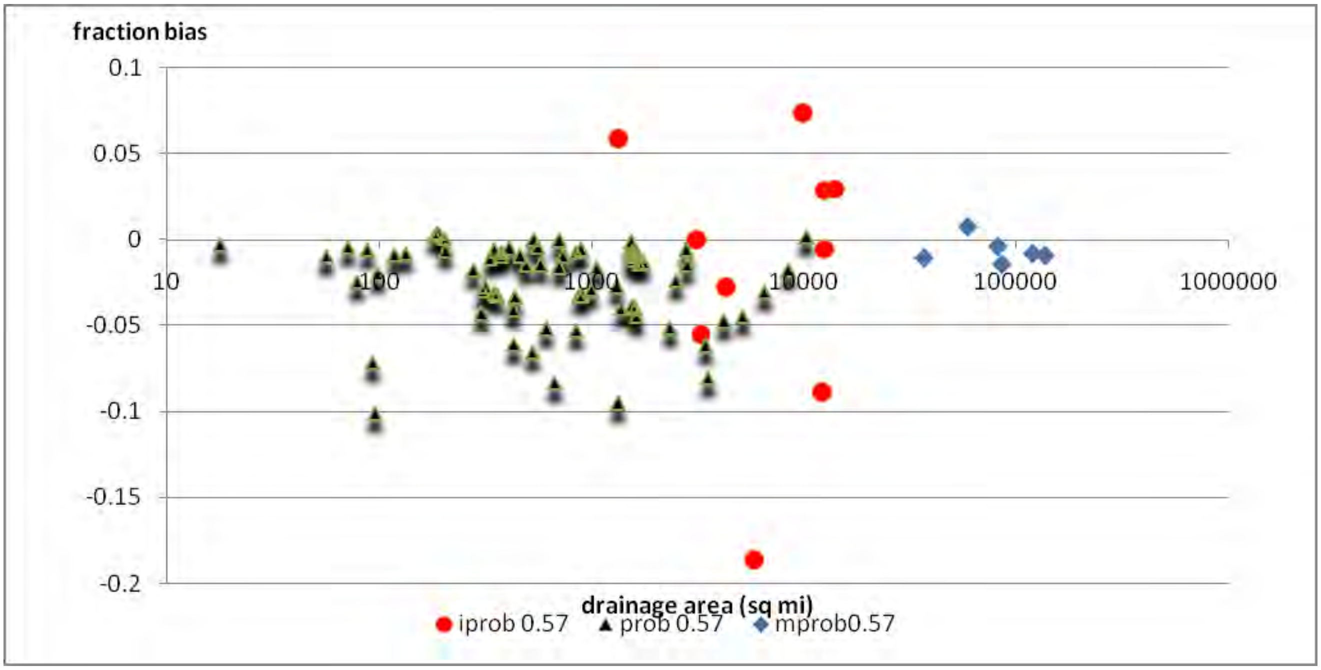

FIGURE 6.2: ALL GAGE 1DAY DURATION FRACTION BIAS COMPARISON
(censoring threshold 0.57 exceedance probability, iprob0.57 – Iowa and Des Moines River gages, prob0.57 – USGS gages, mprob0.57 – Mississippi River gages)

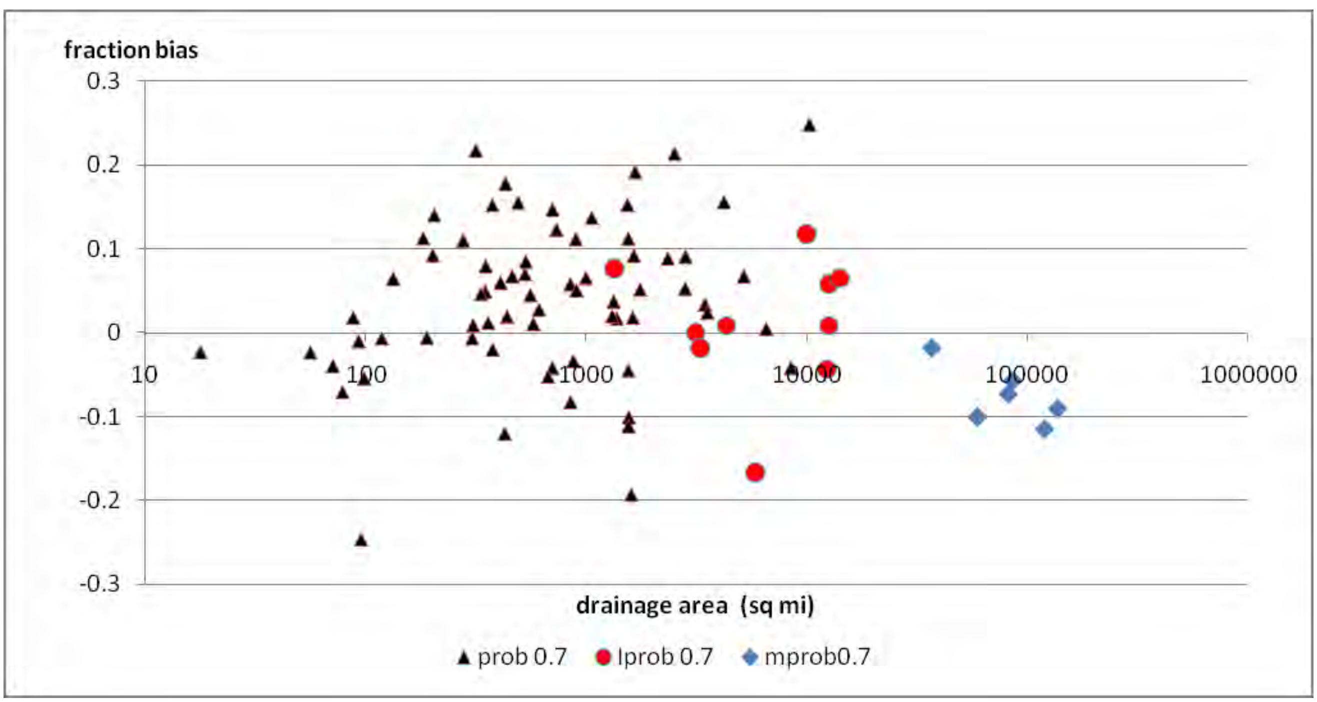

FIGURE 6.3: ALL GAGE 1DAY DURATION FRACTION BIAS COMPARISON
(censoring threshold 0.7 exceedance probability, iprob0. 7 – Iowa and Des Moines River gages, prob0. 7 – USGS gages, mprob0. 7 – Mississippi River gages)

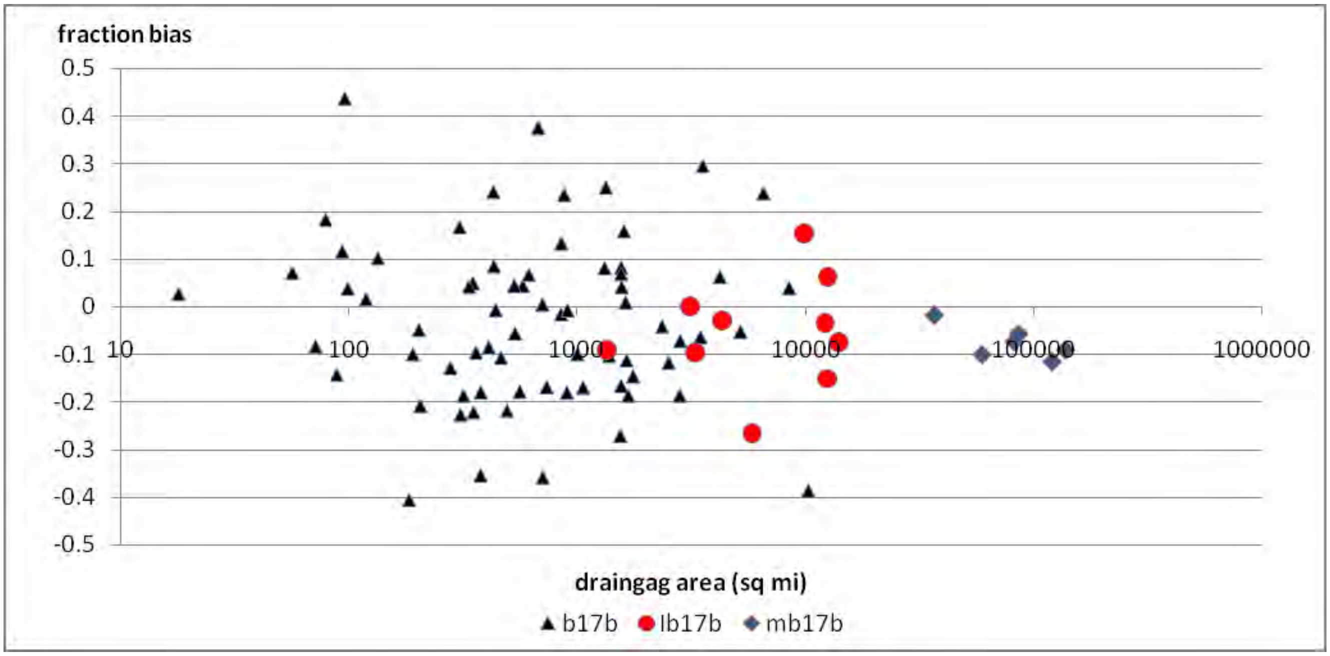

FIGURE 6.4: ALL GAGE 1DAY DURATION FRACTION BIAS COMPARISON
(Bulletin 17B censoring threshold Bulletin 17B. iBulletin 17B – Iowa and Des Moines River gages, mBulletin 17B – USGS gages)

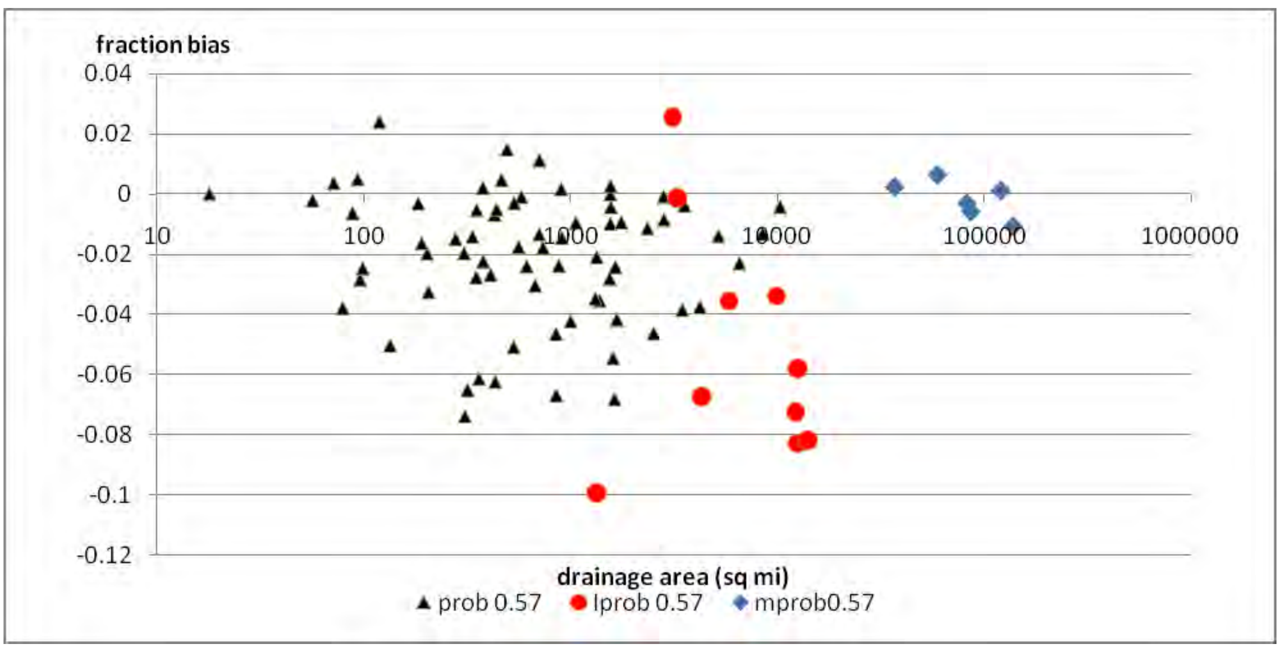

FIGURE 6.5: ALL GAGE 30DAY DURATION FRACTION BIAS COMPARISON
(censoring threshold 0.57 exceedance probability, iprob0.57 – Iowa and Des Moines River gages, prob0.57 – USGS gages, mprob0.57 – Mississippi River gages)

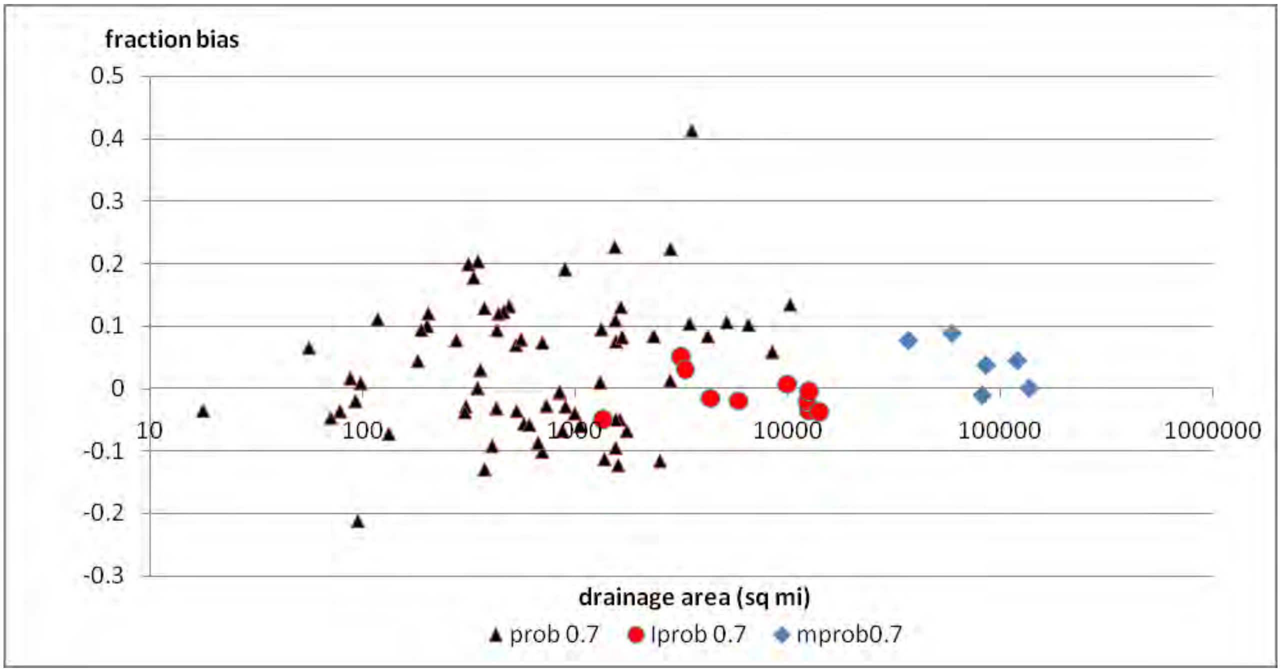

FIGURE 6.6: ALL GAGE 30DAY DURATION FRACTION BIAS COMPARISON
(censoring threshold 0.7 exceedance probability, iprob0. 7 – Iowa and Des Moines River gages, prob0. 7 – USGS gages, mprob0. 7 – Mississippi River gages)

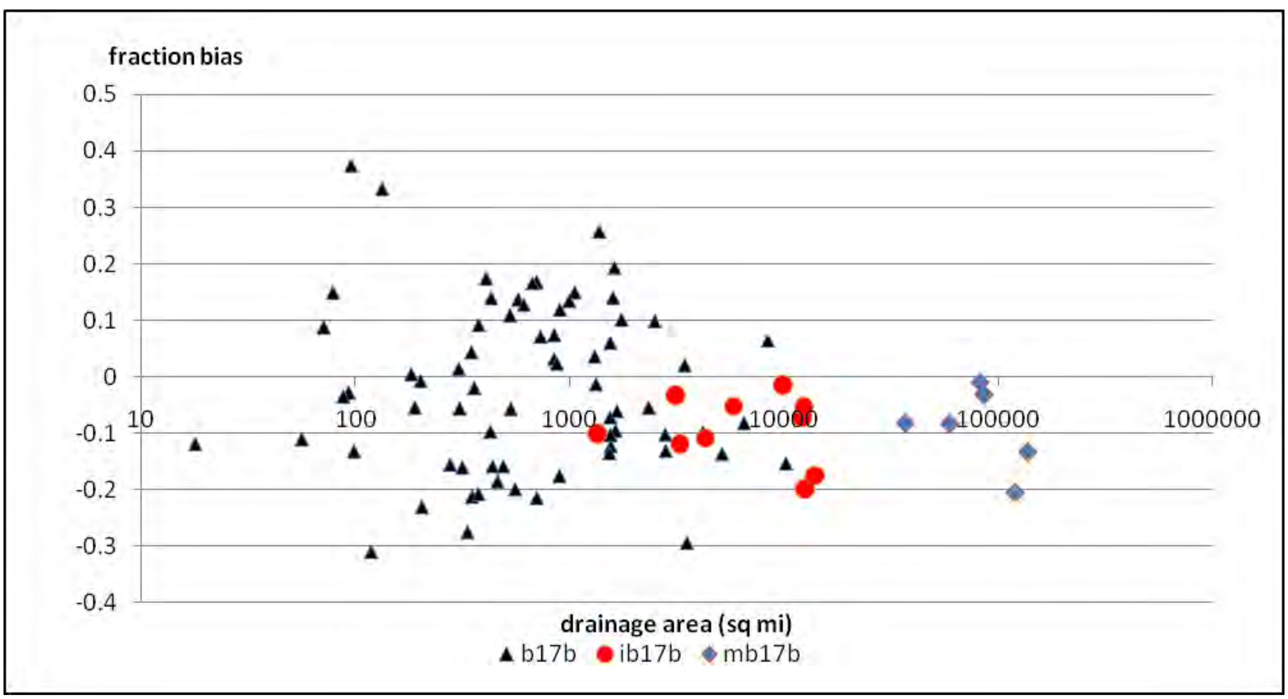

FIGURE 6.7: ALL GAGE 30DAY DURATION FRACTION BIAS COMPARISON
Bulletin 17B censoring threshold Bulletin 17B. iBulletin 17B – Iowa and Des Moines River gages, mBulletin 17B – USGS gages

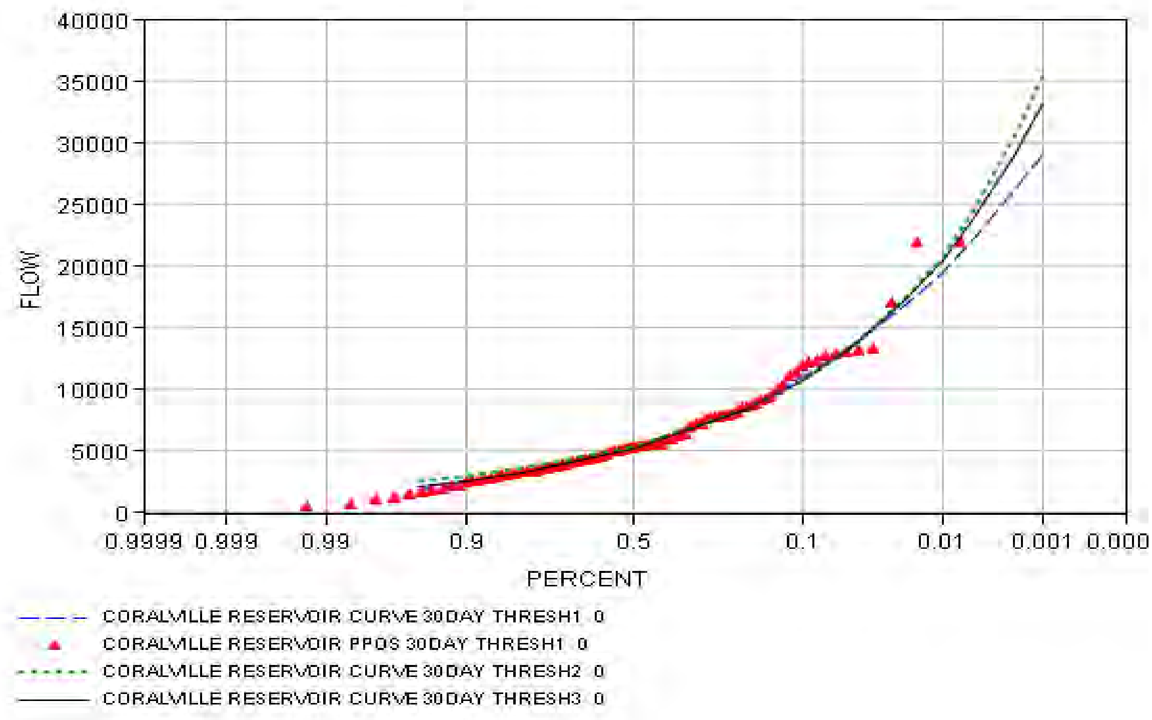

FIGURE 6.8: CORALVILLE RESERVOIR COMPARISON LPIII DISTRIBUTION AND PLOTTING POSITIONS, 30DAY ANNUAL MAXIMUM INFLOW FREQUENCY CURVE

thresh1 – Bulletin 17B censoring, thresh2 – censor at 0.7 exceedance probability, thresh3 – censor at 0.57 exceedance probability

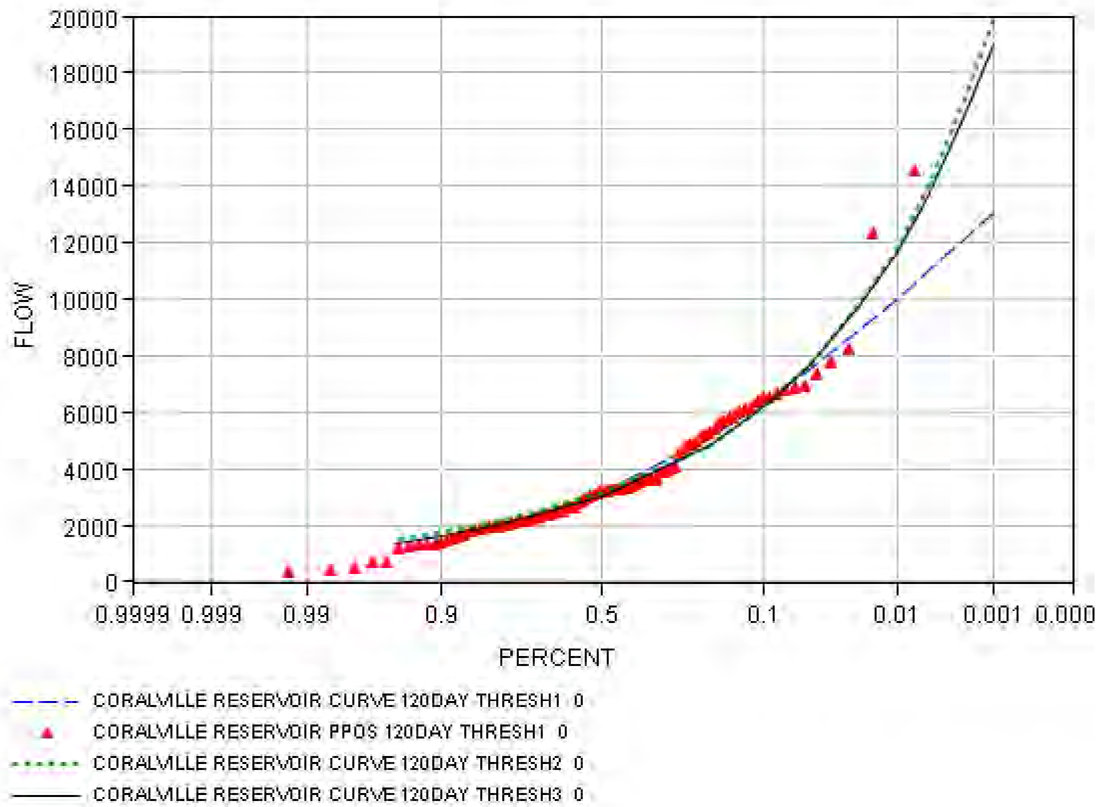

FIGURE 6.9: CORALVILLE RESERVOIR COMPARISON LPIII DISTRIBUTION AND PLOTTING POSITIONS, 120DAY ANNUAL MAXIMUM INFLOW FREQUENCY CURVE
thresh1 – Bulletin 17B censoring, thresh2 – censor at 0.7 exceedance probability, thresh3 – censor at 0.57 exceedance probability

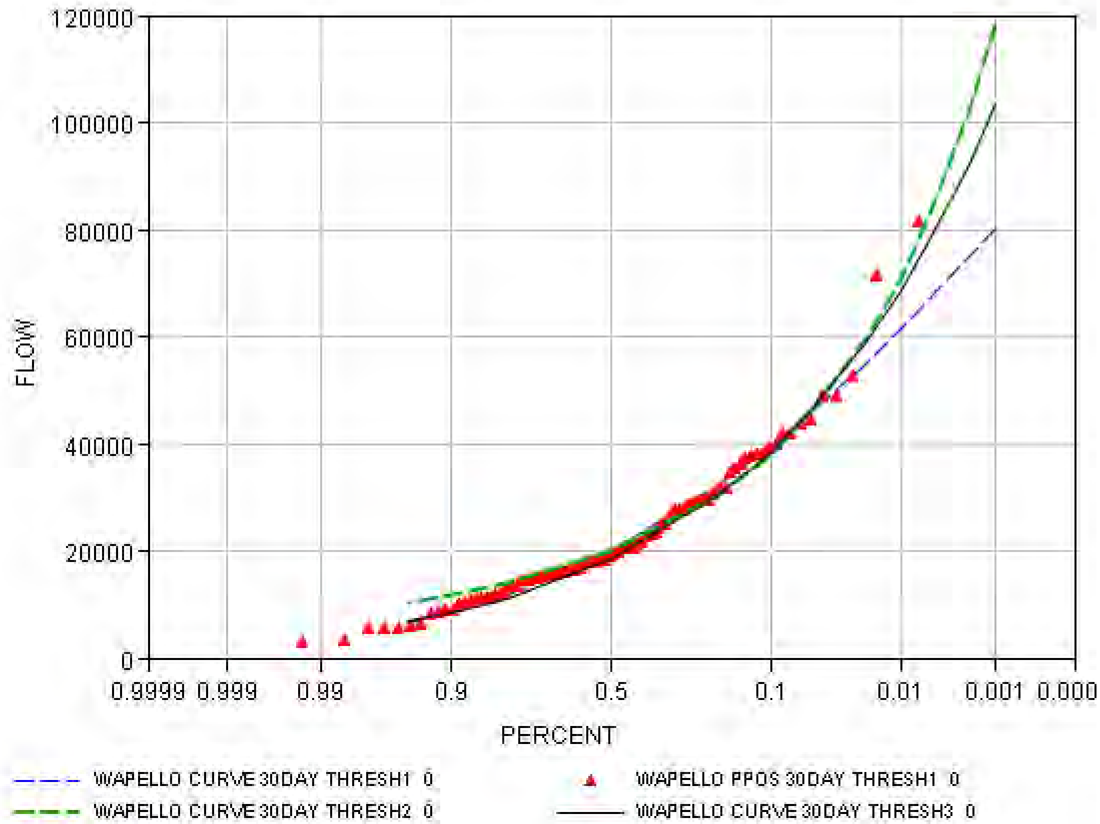

FIGURE 6.10: WAPELLO COMPARISON LPIII DISTRIBUTION AND PLOTTING POSITIONS, 30DAY ANNUAL MAXIMUM FLOW FREQUENCY CURVE
thresh1 – Bulletin 17B censoring, thresh2 – censor at 0.7 exceedance probability, thresh3 – censor at 0.57 exceedance probability

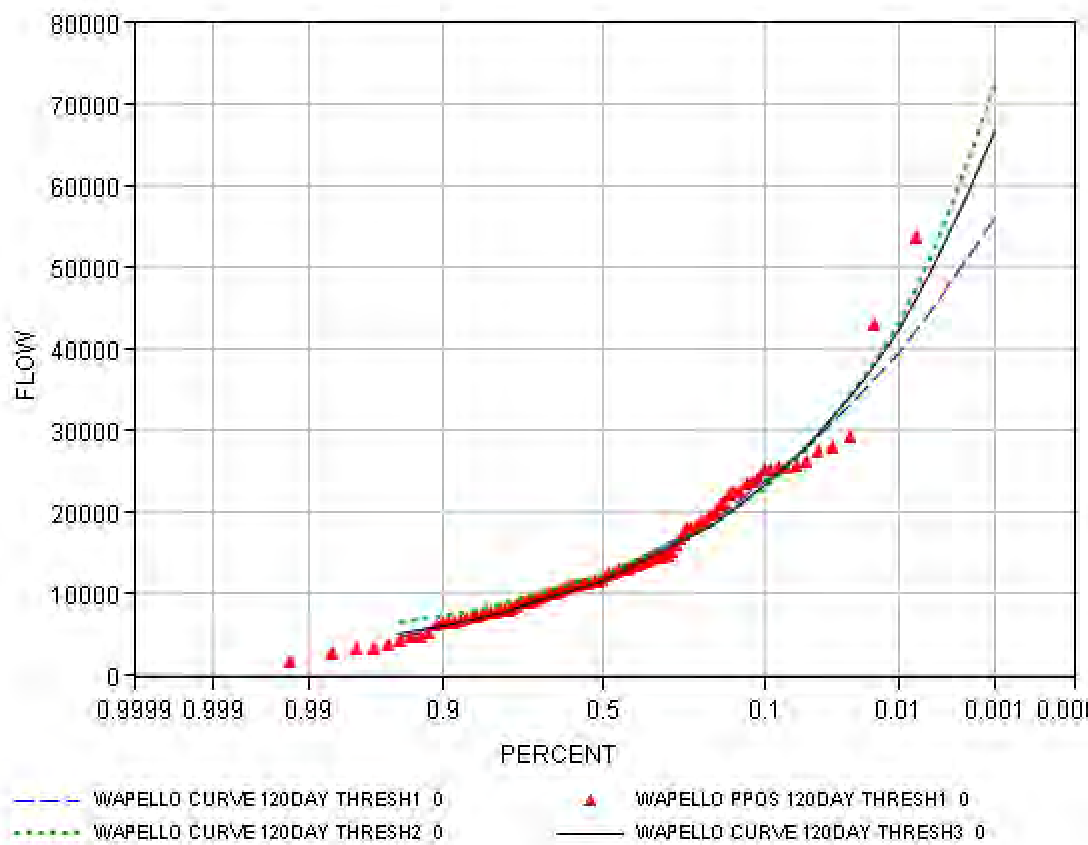

FIGURE 6.11: WAPELLO COMPARISON LPIII DISTRIBUTION AND PLOTTING POSITIONS
120day annual maximum flow frequency curve, thresh1 – Bulletin 17B censoring, thresh2 – censor at 0.7 exceedance probability, thresh3 – censor at 0.57 exceedance probability

7 L-moment Analysis

The purpose of this section is to use L-moment regional analysis procedures (Hosking and Wallis, 1997) to select and estimate flow frequency distributions for the Iowa and Des Moines River gages. Note that this analysis was originally performed for the Iowa River regulated flow frequency study and the following is taken from the report describing that study (Corps of Engineers, 2009).

The value of this analysis is: 1) examining the effect of potential model error (i.e., the error in selecting a frequency distribution that is necessarily approximate given the assumptions made in the analysis); and, 2) using an estimation procedure (L-moment) that is insensitive to nor considers low-outliers. As was discussed in section 2, model error is rarely considered in examining the potential prediction errors of an estimated flow frequency curve. Comparing an alternative L-moment estimated flow frequency distribution to the Bulletin 17B lpiii flow frequency curve will provide some perspective on this model prediction error (e.g., see Figure 2.1). In addition, outlier analysis is not as important in this approach because: 1) generally the log transform is not taken with L-moments; and, 2) the L-moment estimates are linear in deviations from the mean, making the estimates less sensitive to outlying flows.

Note: Terming this as a “regional analysis” is not really a correct description of the analysis provided. A true L-moment regional analysis would involve aggregating gages based on gage watershed hydrologic and meteorologic characteristics, testing if the aggregation passes the statistical criterion for a region, and then identifying an index flood distribution. Consequently, the analysis in this section has a much more limited purpose, which is to examine potential model error.

Table 7.1 provides the Hosking and Wallis statistical measures of the region defined by the Iowa and Des Moines River gages. The H(*) statistics measure the homogeneity of the region and the D indicates the number of discordant gages (gages that do not belong in the region). One gage was found to be discordant, but this is acceptable for the purposes of defining the distribution for this region. The H(*) statistics are all negative. Hosking and Wallis (1997, pg. 75) not that this occurs because there is likely to be a high degree of correlation between gage flows. The practical aspect of this is that the regional analysis will not reduce the sampling error in comparison to that obtained with a single gage analysis. This means that the pooled number of years of record for the Iowa and Des Moines River gages provides no more information about flood frequency than a single gage. However, the regional analysis will provide a reasonable measure of an average or index distribution for the region for comparisons with Bulletin 17B estimates.

Application of the regional analysis goodness of fit statistic indicated that the generalized normal distribution (GNO) was most acceptable (other distributions tested were generalized extreme value, generalized logistic, generalized Pareto, and Pearson Type iii). Table 7.2 and Figure 7.1 show that the GNO predicts smaller return intervals for the 1993 30day annual maximum flows than for the Bulletin 17B lpiii estimates (the 30day was investigated given the importance of longer durations to the estimate of regulated frequency curves for Saylorville and Red Rock dams).

The importance of these results is in providing some awareness about the existence of model error and the need for a degree of safety given the potential errors made in assuming a particular distribution. In this case, the L-moment analysis indicates the potential for more frequent flooding than would be expect from the Bulletin 17B estimate.

Table 7.1: L-moment Region Statistics

duration	H(1)	H(2)	H(3)	D
30day duration	-1.94	-3.14	-3.15	1

Table 7.2: Comparison of Bulletin 17B lpiii and L-moment Generalized Normal Estimates of 1993 30day Annual Maximum Volume Return Intervals

location	Q_{1993}	gNorm	lpiii	difference
Saylorville	32853	153	104	49
SE6th	56113	167	127	41
Red Rock	71426	179	213	-34
Tracy	71426	174	196	-22
Ottumwa	80385	205	270	-65
Keosauqua	81837	168	238	-70
Coralville	21835	154	172	-18
Iowa City	23520	192	208	-16
Lone Tree	31902	282	270	12
Wapello	81153	166	270	-104
average return interval difference				-23

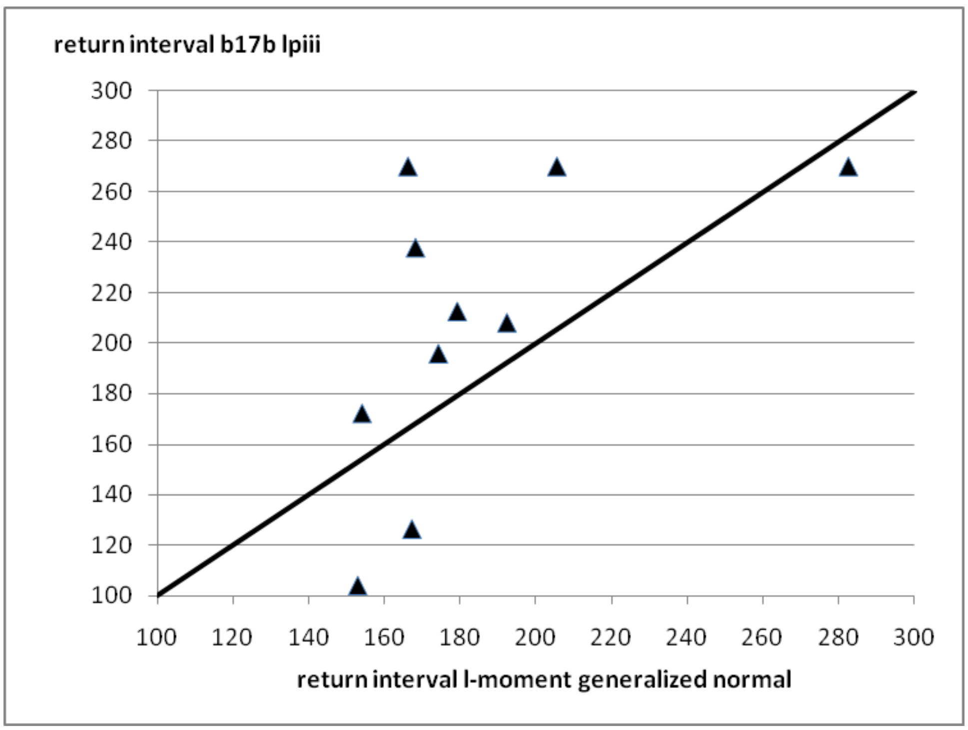

FIGURE 7.1: BULLETIN 17B LPIII AND L-MOMENT GENERALIZED NORMAL FLOW FREQUENCY CURVE RETURN INTERVAL ESTIMATES OF THE 1993 30DAY ANNUAL MAXIMUM VOLUME

8 Regulated Frequency Curves

8.1 Introduction

The purpose of this section is to describe the calculation of the annual maximum regulated peak flow frequency curves at the gages of interest. The daily annual maximum flow frequency curves at Saylorville, Red Rock Dams and SE6th Street were estimated by integrating an annual maximum volume duration frequency curve with a regulated versus unregulated relationship. Section 8.2 describes the selection of the appropriate duration to be used; and, section 8.3 the estimation of the regulated versus unregulated relationships. Section 8.4 provides the results of integrating the frequency curve and unregulated versus regulated relationship. The use of the unregulated versus regulated relationships was not as useful for computing the regulated frequency curve at Ottumwa and Keosauqua as at upstream locations. Section 8.4 also describes an area adjustment methodology used to compute regulated frequency curves at these locations. The estimated annual maximum daily regulated frequency curve is converted to a peak flow frequency curve using regression relationships between peak and daily flows. Section 8.5 describes the estimation of these regression relationships from gage data. Section 8.6 provides the computation of the final annual peak flow regulated frequency using these regression relationships.

8.2 Duration Selection

The duration of the annual maximum daily frequency curve (see section 5) to use in computing the regulated frequency curve depends on the relative effect of Saylorville and Red Rock Reservoir storage on reducing flood flows at downstream locations. The storage effect is measured by how well the annual maximum unregulated volume for an annual maximum event in the period explains the corresponding regulated event for reservoir releases exceeding the objective release (the objective release is typically some measure of channel capacity or flow magnitude that causes initial damaging river stage).

The methodology focuses on releases greater than the objective release because there is a direct relationship with inflow volume for these releases. The more frequent releases less than the objective release occur when inflows are being well controlled and the reservoir is surcharging (inflows are being stored to prevent any flood damage). This typically occurs for flow frequencies less than the 10 year flood. The return interval for these flows is computed using plotting position formula.

Table 8.1 show the relationship between annual maximum daily unregulated flow volumes for corresponding top ranked annual maximum 1day regulated flows that exceed channel capacity in the period of record. As can be seen, the 30day duration inflow volume to Saylorville and Red Rock Reservoir most nearly produces the ranking of corresponding events as occurred for the regulated outflows. Neither the 1day, 7day or 15day does as well. The 60day duration does almost as well as the 30day at Saylorville. However, the 30day provides the proper ranking for the 1947 flood, which along with 1954 flood were the two design event considered when developing regulation plans for the Des Moines River.

The appropriate duration to use at locations downstream of the dams is less clear. As is described in Section 8.4, the drainage area increase from Saylorville to SE6th Street is about double. Considering this area increase, averaging the affects of the uncontrolled drainage area and Saylorville on the regulated frequency curve seems logical. The 1day and 30 day frequency curves were averaged because these duration represent best the contributions of the uncontrolled and regulated drainage areas to flood frequencies at SE6th Street.

The rankings for Ottumwa and Keosauqua do not give a clear indication of the appropriate duration to be used. Furthermore, very inconsistent results are obtained when using a regulated versus unregulated relationship based on various durations to estimate the regulated frequency curve at these locations. Because of this inconsistency, a drainage area adjustment method was used to compute the regulated frequency curves at Ottumwa and Keosauqua.

Table 8.1: Unregulated flow duration selection for computation of regulated frequency curves

(years ranked from smallest to largest annual maximum volumes)

Saylorville			**Unregulated**			
1day	3day	7day	15day	30day	60day	regulated 1day
1947			1951	1991	2001	1991
1965			1984	1984	1984	1965
1993			1965	1965	2008	1984
2008			2008	2008	1991	1993
1954			1993	1993	1993	2008
SE6th			**Unregulated**			
1day	3day	7day	15day	30day	60day	regulated 1day
1990	1947	1979	1947	1965	2007	1979
1954	1954	1991	1991	1947	1979	1990
2007	1990	1990	1965	1979	1983	1991
1984	1965	1965	1979	1991	2008	1947
1947	1984	1984	1984	1984	1991	1984
2008	2008	2008	2008	2008	1984	2008
1993	1993	1993	1993	1993	1993	1993
Red Rock			**Unregulated**			
1day	3day	7day	15day	30day	60day	regulated 1day
			1965	1984	1947	1991
			1990	1991	1984	1984
			1947	1947	1991	1947
			2008	2008	2008	2008
			1993	1993	1993	1993
Ottumwa			**Unregulated**			
1day	3day	7day	15day	30day	60day	regulated 1day
2004	2004	1998	2007	1983	1983	2004
1982	1982	1960	1984	1998	1947	1947
1990	1947	1965	1990	1984	1991	1984
1947	1990	1990	1947	1947	1984	2007
2008	1993	1993	2008	2008	2008	2008
1993	2008	2008	1993	1993	1993	1993
Keosauqua			**Unregulated**			
1day	3day	7day	15day	30day	60day	regulated 1day
2007	2007	1960	1965	1998	1973	1982
1990	1947	1947	2007	1984	1947	1947
1947	1982	1965	1990	1991	1984	1973
1982	1990	1990	1947	1947	1991	2007
1993	1993	1993	2008	2008	2008	2008
2008	2008	2008	1993	1993	1993	1993

8.3 Regulated versus Unregulated Relationship

The regulated versus unregulated relationship is used to compute the annual maximum 1day regulated frequency curve from the critical duration annual maximum unregulated volume frequency curve. The purpose of this section is to describe the development of this relationship for Saylorville and Red Rock Dams, and, SE6th Street. As is described in Section 8.4, a drainage area adjustment method was used to compute the regulated frequency curve at Ottumwa and Keosauqua as an alternative to employing a regulated versus unregulated relationship.

The regulated versus unregulated relationship is characterized by zones where flows are less than or greater than the objective release. The objective releases usually correspond to some measure of channel capacity (i.e., the flow magnitude where flood damage is significant).

Graphical analysis of the observed events is used to describe this relationship for flows less than channel capacity. The description of this relationship is more difficult, and more important, for flows exceeding channel capacity. The difficulty stems from the lack of data. Only two events in the period of record, 1993 and 2008, significantly exceed the channel capacity, giving little information to estimate this relationship. Additionally, this region is critical in estimating the 100-year regulated flow value, which is important for regulatory purposes.

More information was obtained for estimating this relationship by simulating ratios of important historical events. In this case, ratio factors equal to 1.2, 1.5 and 1.7 of the 1993 and 2008 Saylorville and Red Rock reservoir inflows were simulated to extend the regulated versus unregulated relationship to values needed in computing regulated inflows less frequent than 0.01 exceedance probability. The inflow ratio applied to two different major events provide information on the importance of inflow hydrograph shape on regulated peak flow releases.

Figures 8.1 and 8.2 show the estimated regulated versus unregulated relationship for Saylorville and Red Rock dams. Regression and interpolated relationships used to model the regulated versus unregulated relationship for flows exceeding channel capacity are shown on these figures. The regression or interpolated estimates balance the scatter of either simulated or factored flows. The regulated versus unregulated flow pairs shown result from an application of the Sayred model simulations(see Section 4.2) and the observed flows recorded by the U.S. Geological Survey (USGS) for the 1993 and 2008 events. The Sayred model was used to simulate both the regulated flows for the period of record and the factored flows (the 1993 and 2008 floods increased by factors of 1.2, 1.5 and 1.7). The USGS observations were used because of the difficulty in simulating the actual dam operations for these events using Sayred daily computation interval.

In the case of Saylorville Dam, the USGS recorded 1993 and 2008 events agree reasonably well with the Sayred simulated values. A regression estimate was used to balance the factored events and the recorded USGS 1993&2008 events in obtaining the regulated versus regulated relationship for 30day inflows exceeding 30,000 cfs/day. Simple linear interpolation was used to obtain the relationship for lower flows. The interpolated lines were obtained by connecting end point regulated versus regulated values. Note that the 1947 USGS recorded event (an event with significantly smaller releases than 1993 or 2008) was also used to better define the regulated versus unregulated relationship for 30day inflows less than 30,000 cfs/day.

The estimation of the Red Rock dam regulated versus unregulated relationship followed the same procedure as for Saylorville: except that endpoint interpolation was used to define the relationship (instead of regression) for the large factored flows; the USGS recorded 1947 event was not used (it was not useful); and the USGS recorded 1993 and 2008 events were significantly different than the Sayred simulated values. The USGS recorded events were used to estimate the regulated versus unregulated relationship. The use of these recorded events instead of the simulated makes a significant difference in the estimate of the 0.01 exceedance probability (100 year) regulated discharge.

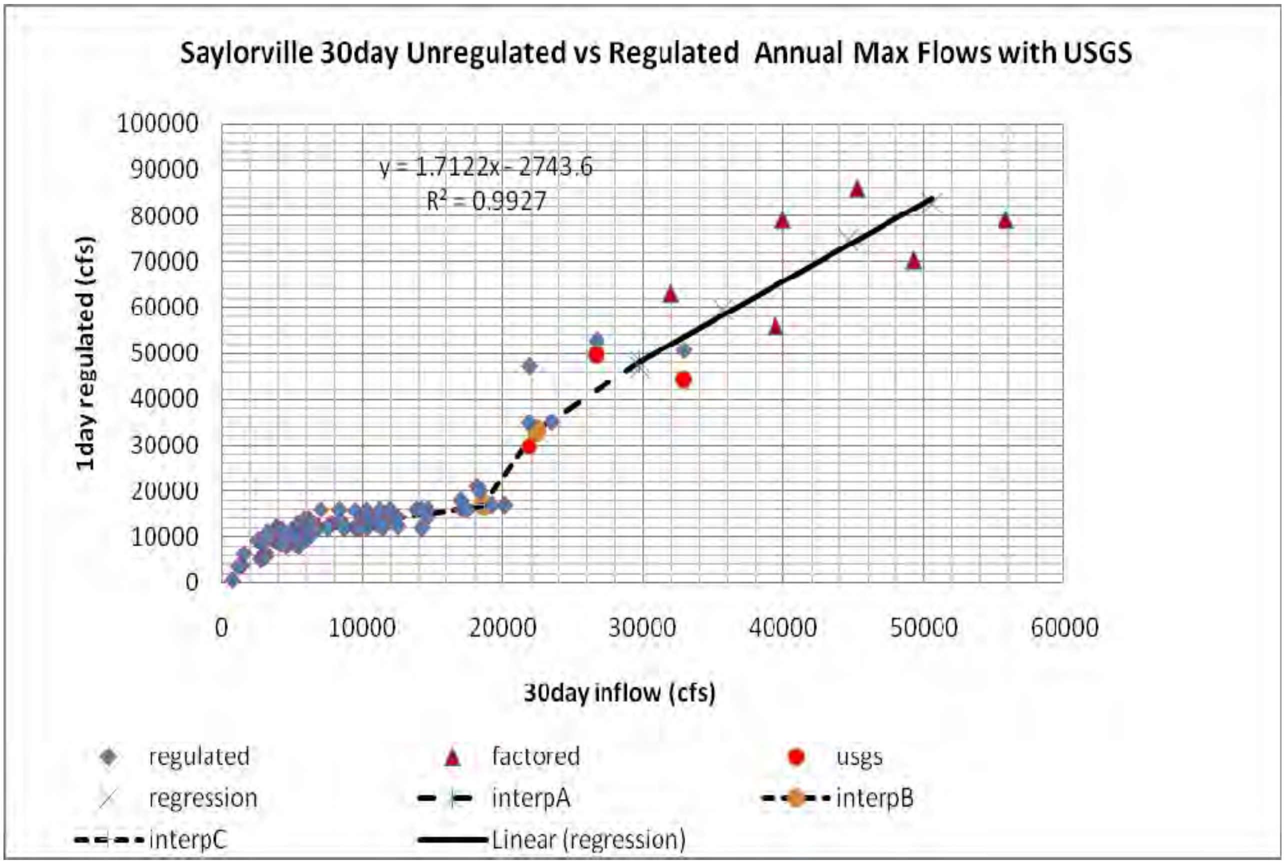

FIGURE 8.1: SAYLORVILLE DAM 30DAY REGULATED VERSUS UNREGULATED RELATIONSHIP
[regulated = Sayred simulated events, factored = (1.2, 1.5 and 1.7 factored 1993 and 2008 events), usgs = USGS recorded 1947, 1993 and 2008 regulated flows, regression = regression points, interpA = estimated relationship, interpB = estimated relationship, interpC = estimated relationship, linear(regression) = estimated relationship]

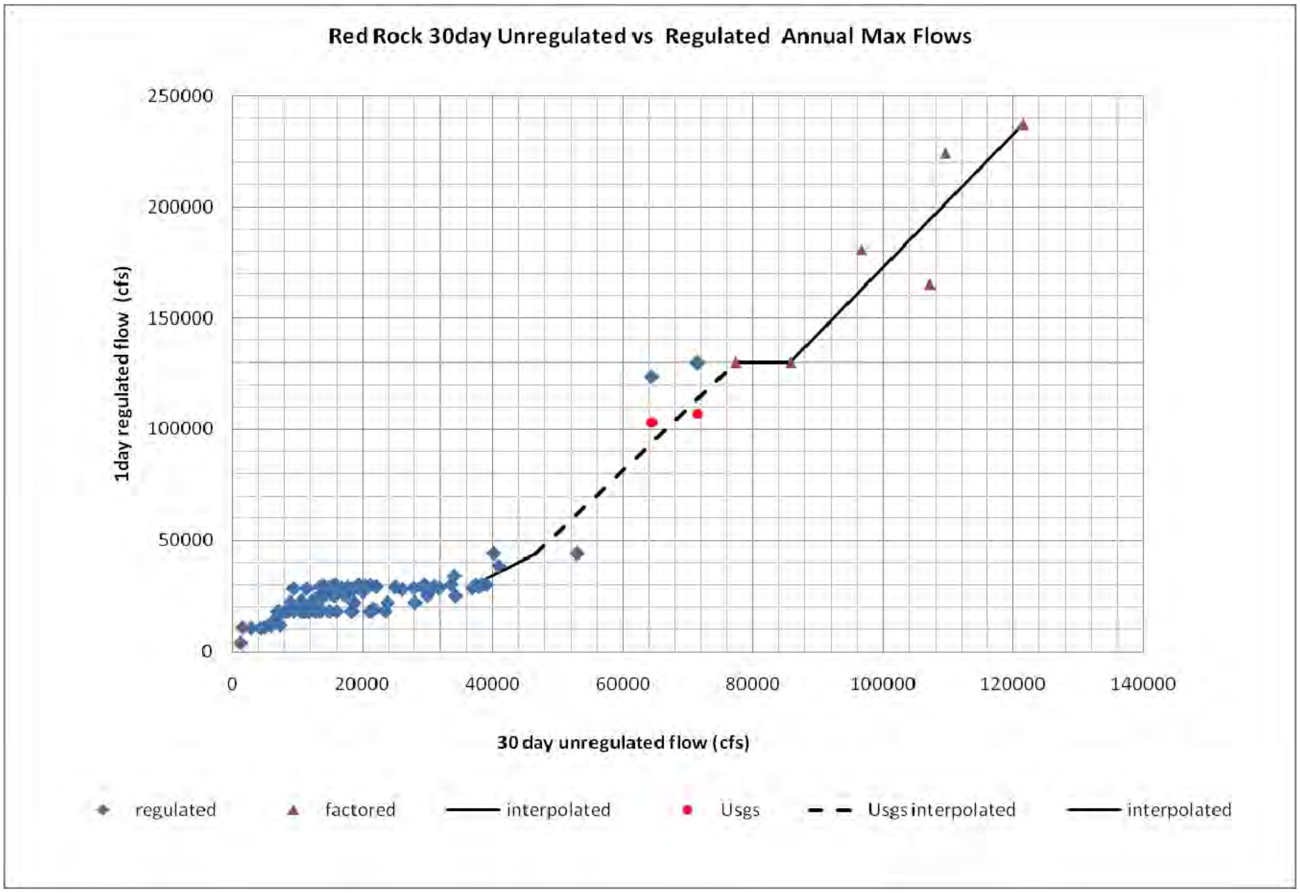

FIGURE 8.2: RED ROCK DAM 30DAY UNREGULATED VERSUS REGULATED RELATIONSHIP
[regulated = Sayred simulated events, factored = (1.2, 1.5 and 1.7 factored 1993 and 2008 events), interpolated = interpolation for factored events, usgs = USGS recorded 1993 and 2008 regulated flows, usgs interpolated = interpolation using USGS recorded events]

As discussed previously in this section, the regulated frequency at SE6th Street will be an average of the 1day and 30day regulated frequency curves. Figures 8.3 and 8.4 show the regulated versus unregulated relationships that are needed for this averaging. As can be seen from the figures, the plotted flows are much more regularly varying than for Saylorville and Red Rock dams. This results because half of the drainage area contributing to stream flow at this location is uncontrolled.

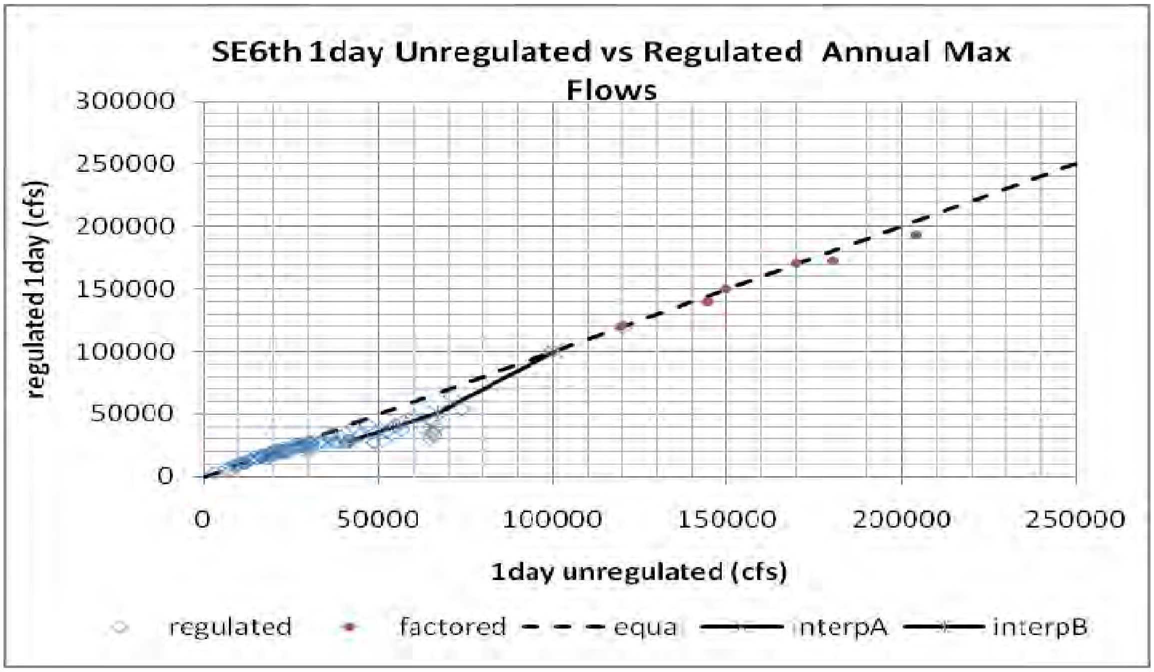

FIGURE 8.3: SE 6TH STREET 1DAY UNREGULATED VERSUS REGULATED RELATIONSHIP
[regulated = Sayred simulated events, factored = (1.2, 1.5 and 1.7 factored 1993 and 2008 events), interpA and interpB = interpolation for simulated events]

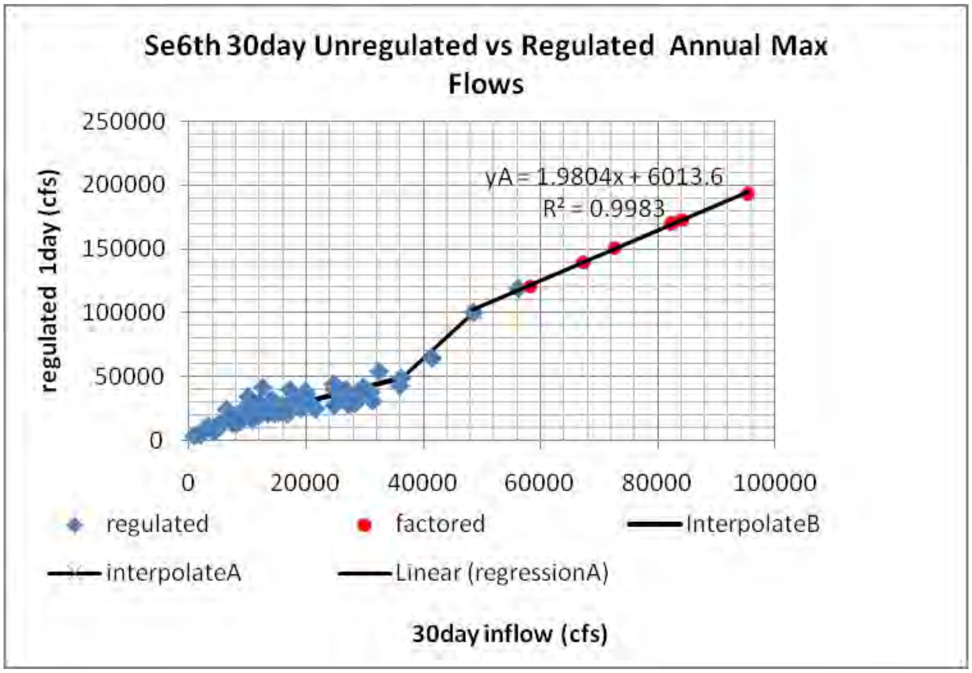

FIGURE 8.4: SE 6TH STREET 30DAY UNREGULATED VERSUS REGULATED RELATIONSHIP
[regulated = Sayred simulated events, factored = (1.2, 1.5 and 1.7 factored 1993 and 2008 events), linear (regression) = regression for factored events and 1993&2008) interpA and interpB = interpolation for simulated events]

8.4 Regulated Frequency Curve Estimates

The regulated frequency curves were estimated using different methods depending on location. As described in Section 8.4.1, the estimates at Saylorville and Red Rock dams were obtained by integrating regulated versus unregulated relationship with the 30day unregulated VDF curve. Section 8.4.2 details the estimation of the regulated frequency curve at SE6th Street as an average of regulated frequency curves derived from unregulated and regulated relationships using 1day and 30day volumes. As is described in Section 8.4.3, a drainage area adjustment method was used to compute the regulated frequency curve at Ottumwa and Keosauqua as an alternative to employing a regulated versus unregulated relationship.

8.4.1 Saylorville and Red Rock Regulated Frequency curves

The Saylorville and Red Rock Dam regulated frequency curves were computed by integrating the 30day volume duration frequency curve (see Table 5.3) and regulated versus unregulated relationship (see figures 8.1 and 8.2). Table 8.2 provides a comparison of the current estimates with that provided in the previous Corps of Engineers (2002) study. There is a significant increase in regulated flow quantiles beginning at 0.02 exceedance probability at Saylorville and Red Rock Dams. Table 8.4 reveals the effects of regulation by comparing the 1day unregulated and regulated frequency curves at both reservoirs. As can be seen, Saylorville dam has a small regulation effect on the 0.01 exceedance flood, reducing the annual 1day from 61000 to 52800 cfs/day. Figure 8.5 and 8.6 compare the computed regulated frequency curves with the plotting positions of the simulated 1day annual maximum period of record flows. Figure 8.6 also provides the plotting positions of the USGS recorded estimates of the 1993 and 2008 events at Red Rock Dam. These recorded estimates were used in developing the unregulated versus unregulated relationship for this dam.

Table 8.2: Comparison of Saylorville and Red Rock regulated frequency curves with previous study estimates (Corps, 2002)

	Saylorville		Red Rock	
[1]Probability	[2]Current Study	[3]2002 study	[2]Current Study	[3]2002 Study
0.5	12000	13000	25000	26000
0.1	17000	16000	30000	30000
0.02	44700	27000	65500	50000
0.01	52800	33000	89000	69000
0.005	61200	38000	130000	94000
0.002	73000		130000	
0.001	82400		150500	

[1]Exceedance probability,
[2]Based on 30day inflow frequency curve
[3]Corps of Engineers (2002)

Table 8.3: Saylorville and Red Rock Unregulated and Regulated1day annual maximum frequency curves

	Saylorville		Red Rock	
[1]Probability	unregulated	regulated	unregulated	regulated
0.5	15700	12000	38200	25000
0.1	34100	17000	77300	30000
0.02	52600	44700	114800	65500
0.01	61000	52800	131300	89000
0.005	69600	61200	148100	130000
0.002	81500	73000	170900	130000
0.001	90800	82400	188500	150500

[1]Exceedance probability

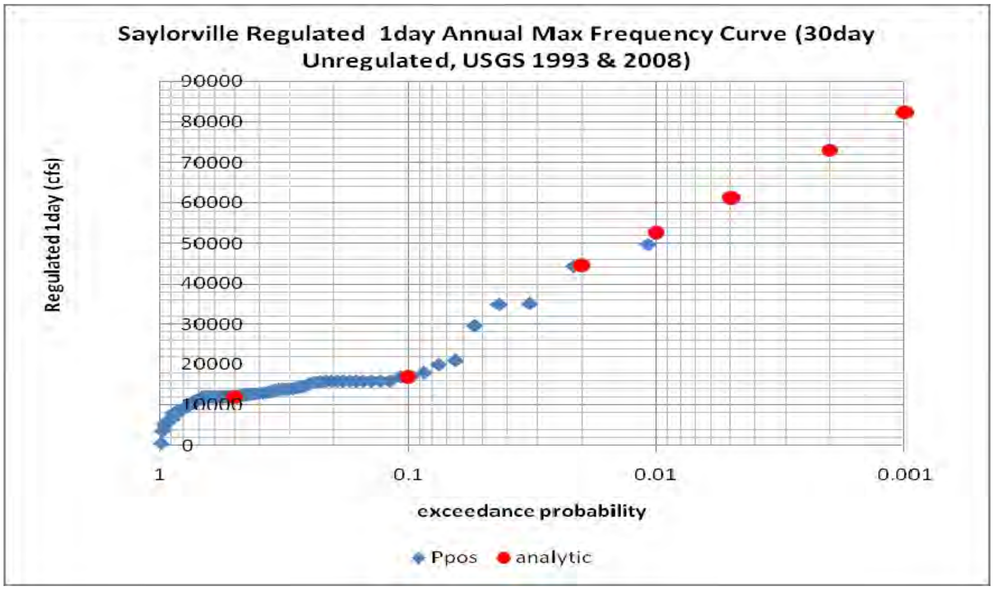

FIGURE 8.5: SAYLORVILLE REGULATED FREQUENCY CURVE
(Ppos = plotting positions of simulated events, analytic = estimated by integrating regulated versus unregulated relationship and log-Pearson iii inflow frequency curves)

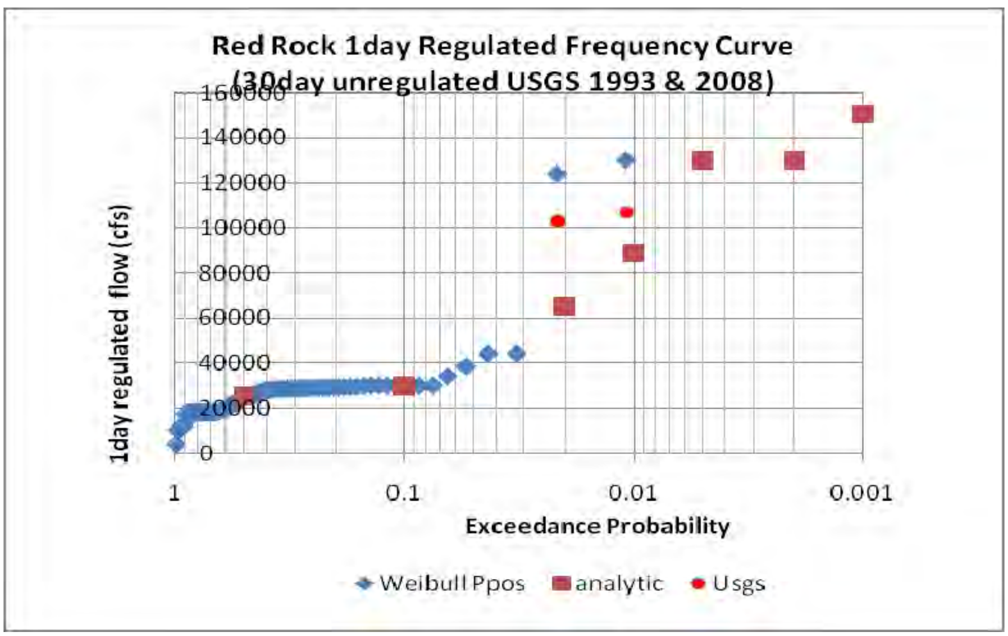

FIGURE 8.6: RED ROCK REGULATED FREQUENCY CURVE
(regulated = plotting positions of simulated events, analytic = estimated by integrating regulated versus unregulated relationship and log-Pearson iii inflow frequency curves, Usgs = plotting position of USGS observed 1993 and 2008 events)

8.4.2 SE6th Street Regulated Frequency Curves

The critical duration to use in estimating the regulated versus regulated relationship at SE6th street was not entirely clear from the ranking analysis performed in Section 8.2 (see Table 8.1). The problem certainly stems from the significant contribution of runoff from the uncontrolled drainage are below Saylorville Dam (the drainage area at Saylorville Dam is 5823 square miles and at SE6th street is 9879 square miles). Consequently, regulated frequency curves using both 1day and 30day unregulated versus regulated relationships were averaged to give equal weighting to the controlled and uncontrolled areas. As can be seen from Table 8.4, there is a significant increase in quantiles from the previous study beginning with the 0.02 exceedance probability. Figure 8.7 compares the computed regulated frequency curve with plotting positions for the simulate period of regulated flows. Table 8.5 shows that the regulation provided by Saylorville has no influence because the unregulated and regulated frequency curves are virtually equal, at least within the error one would expect in estimating the effects of regulation using the simple hydrologic routing used in the Sayred model.

Table 8.4: Comparison of SE6th Street regulated frequency curves with previous study estimates (Corps, 2002)

[1]Probability	1day	30day	average	2002 study
0.5	28100	22500	25300	23000
0.1	44200	40600	42400	37000
0.02	72500	88100	80300	71000
0.01	103600	111300	107500	85000
0.005	117700	127000	122400	100000
0.002	136900	148800	142900	
0.001	151900	166100	159000	

[1]Exceedance probability

Table 8.5: SE6th Street regulated versus unregulated 1day annual maximum regulated frequency curves

[1]Probability	Unregulated	Regulated	Adopted
0.5	28100	25300	**25300**
0.1	59200	42400	**42400**
0.02	89900	80300	**80300**
0.01	103600	107500	**103600**
0.005	117700	122400	**117700**
0.002	136900	142900	**136900**
0.001	151900	159000	**151900**

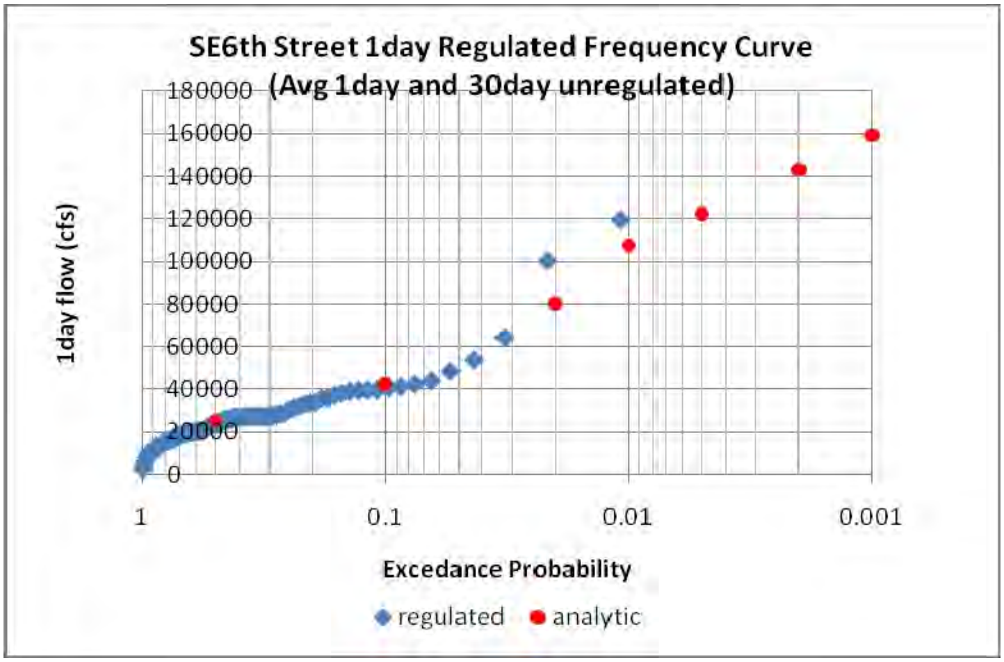

FIGURE 8.7: SE6TH STREET REGULATED FREQUENCY CURVE
(regulated = plotting positions of simulated events, analytic = estimated by integrating regulated versus unregulated relationship and log-Pearson iii inflow frequency curves)

8.4.3 Ottumwa and Keosauqua Regulated Frequency Curves

An area adjustment methodology was employed rather than a regulated versus unregulated relationship to obtain the 1day annual maximum regulated frequency curves at Ottumwa and Keosauqua. Regulated versus unregulated relationships were not used because the ranking methodology describe in section 8.2 did not definitively define a critical unregulated flow duration. Exploration of different durations to use in estimating a regulated versus unregulated relationship resulted in very inconsistent regulated frequency curves both with duration at a particular location and in comparisons between locations. This inconsistency led to the adoption of the drainage area adjustment method as an alternative.

The drainage area ratio adjustment typically has the form $(A/A_r)^x$, where A and A_r are the ratio of the drainage areas where respectively flows are unknown and known, and, x is an exponent. The drainage area and quantiles shown in Table 8.6 were used to obtain the exponents shown in Table 8.7. Apparently, an exponent of 1.0 is appropriate for the similar magnitude Ottumwa and Keosauqua drainage areas.

Table 8.6: 1day unregulated frequency curves

	[1]Area (sq mi)	50-yr	100-yr	200-yr	500-yr
Saylorville	5823	52600	61000	69600	81500
SE 6th	9879	89900	103600	117700	136900
Red Rock	12323	114800	131300	148100	170900

[1]Drainage Area

Table 8.7: Exponent for area ratio

		Exponent for drainage area			
	A/Ar	50-yr	100-yr	200-yr	500-yr
Saylorville scaling to SE 6th	1.70	1.01	1.00	0.99	0.98
Saylorville scaling to Red Rock	2.12	1.04	1.02	1.01	0.99
SE 6th scaling to Red Rock	1.25	1.11	1.07	1.04	1.00

These use of the drainage area ratio benefits from the fact that the drainage increase from Red Rock to Keosauqua is relatively small (Red Rock DA = 12323 square miles, Ottumwa = 13374 square miles, Keosauqua = 14038 square miles).

The regulated frequency curve using the drainage area adjustment methodology is computed as follows:

a. Compute the unregulated 1day annual maximum quantile runoff per unit area at the location of interest. (Ottumwa example: 0.01 quantile = 135190 cfs, quantile/area = 135190(cfs)/13374(square miles) = 10.11 (cfs/square mile).
b. Compute the area adjusted contribution of unregulated flows at the Red Rock Dam location to the locations of interest as the quantile runoff/area in step a by the drainage at Red Rock Dam [Ottumwa example: 10.11(cfs/sq mi)(drainage area at Red Rock) = 10.11(cfs/square miles)(12330(square miles)) = 124642(cfs)].
c. The difference between the area adjusted unregulated contribution from the Red Rock Dam location and the unregulated quantile estimate at the location of interest gives the contribution from the unregulated area below the dam (Ottumwa example: unregulated area contribution = [1]135190 (cfs) – [2]124642(cfs) = 10554(cfs) [[1]step a, [2]step b].
d. The regulated quantile at the location of interest is the sum of the regulated quantile at Red Rock and the unregulated area contribution (step c) (Ottumwa example: regulated 0.01 quantile at Red Rock = 89000 cfs (Table 8.2), regulated 0.01 exceedance probability flow at Ottumwa = 89000(cfs) + [1]10554(cfs) = 99554 cfs [[1]step c].

Application of this simple drainage adjustment resulted in the Ottumwa and Keosauqua regulated flow frequency curves which are shown in comparison to the previous Corps of Engineers (2002) study estimates in Table 8.8 the regulated flow quantiles have increased significantly from the previous study beginning with the 0.02 exceedance probability events. Table 8.9 compares unregulated and regulated flow frequency curves and figures 8.8 and 8.9 compare the curve estimates to the plotting positions of the simulated 1day maximum events for the period of record.

Table 8.8: Comparison of Ottumwa and Keosauqua regulated frequency curves with previous study estimates (Corps, 2002)

	Ottumwa		Keosauqua	
[1]Probabilty	Current Study	2002 Study	Current Study	2002 Study
0.5	27500	28000	29400	30000
0.1	35900	31300	39500	55000
0.02	74800	58000	80000	61000
0.01	99600	78000	105400	81000
0.005	141900	103000	148400	106000
0.002	143700		151100	
0.001	165500		173600	

[1]Exceedance probability

Table 8.9: Ottumwa and Keosauqua unregulated and regulated 1day annual maximum frequency curves

	Ottumwa		Keosauqua	
[1]Probability	unregulated	regulated	unregulated	regulated
0.5	40500	27500	41900	29400
0.1	80600	35900	81600	39500
0.02	118600	74800	118700	80000
0.01	135200	99600	134800	105400
0.005	152100	141900	151100	148400
0.002	174900	143700	173000	151100
0.001	192600	165500	189900	173600

[1]Exceedance probability

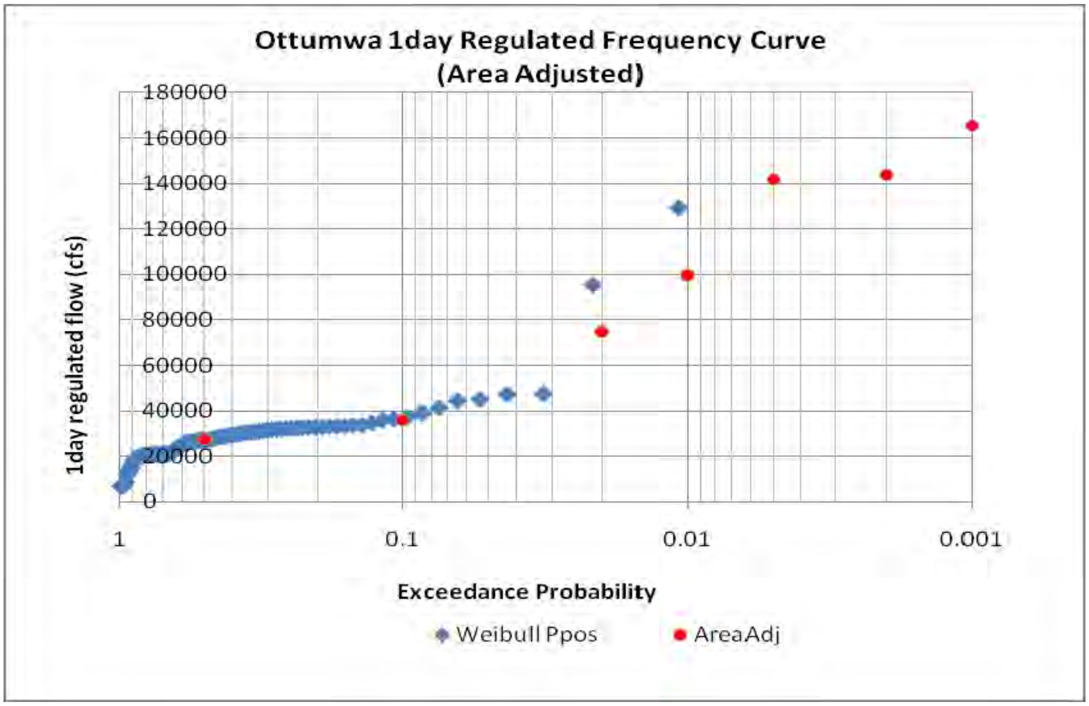

FIGURE 8.8: OTTUMWA REGULATED FREQUENCY CURVE

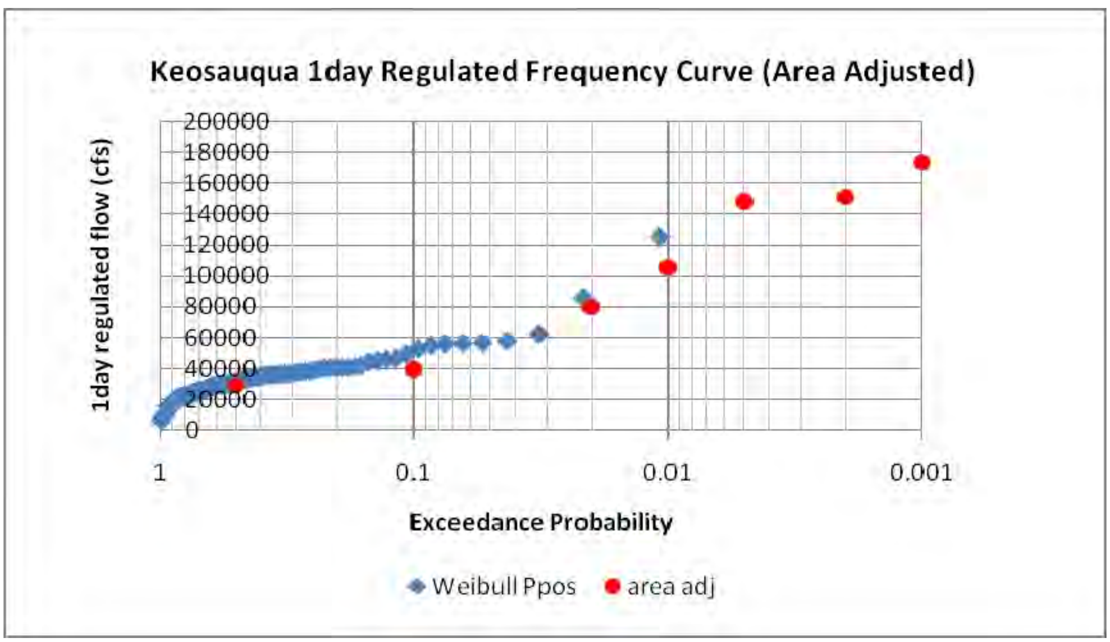

FIGURE 8.9: KEOSAUQUA REGULATED FREQUENCY CURVE

8.5 Sensitivity Analysis

The purpose of this section is to examine the sensitivity of the Saylorville and Red Rock estimated 1day regulated frequency curve to the selected critical inflow volume duration. The durations 15day for both reservoirs and the 60day for Saylorville and 120day Red Rock were selected to bound the 30day critical duration. The 60 day used for Saylorville and the 120day for Red Rock to correspond to the critical durations selected in the Corps' 2002 study.

The results of the analysis described in Tables 8.10 and 8.11 show that the derived regulated frequency curves are not sensitive to duration for Saylorville Dam but are for Red Rock Dam, particularly for exceedance probabilities less than or equal to 0.01. See Figures 8.10-8.17 for the regulated versus unregulated relationship and resulting regulated frequency curves used to obtain the tables.

The Saylorville lack of sensitivity to inflow duration is most likely due to the lack of effectiveness in controlling large inflows. The 0.01 year maximum 1day inflow and regulated flows are about equal for this dam. The sensitivity of the estimated 0.01 regulated inflow is a maximum of about 4%. The Red Rock results are more sensitive to inflow duration, being a maximum of about 11% at the 0.01 exceedance probability.

The regulated outflow is a function of the hydrograph inflow shape, reservoir storage characteristics and dam operations. The critical duration selection results from the inter-play of all these factors. As in any analysis, the results are approximate, and, this contributes to the uncertainty along with statistical sampling error to the uncertainty in estimated regulated frequency curve.

Table 8.10: Sensitivity Analysis Saylorville 1day Annual Maximum Regulated Frequency Curve

Probability	30dayinflow	15day inflow	60day inflow
0.5	12000	12000	12400
0.1	17000	17000	16100
0.02	44700	40600	44500
0.01	52800	49900	54900
0.005	61200	57900	63900
0.002	73000	68900	76600
0.001	82400	77600	86700

Table: 8.11: Sensitivity Analysis Red Rock1day Annual Maximum Regulated Frequency Curve

Probability	30dayinflow	15day inflow	120day inflow
0.5	25000	25000	22800
0.1	30000	30000	35300
0.02	65500	58600	67100
0.01	89000	79700	99700
0.005	130000	101300	125100
0.002	130000	130000	160100
0.001	150500	181300	187800

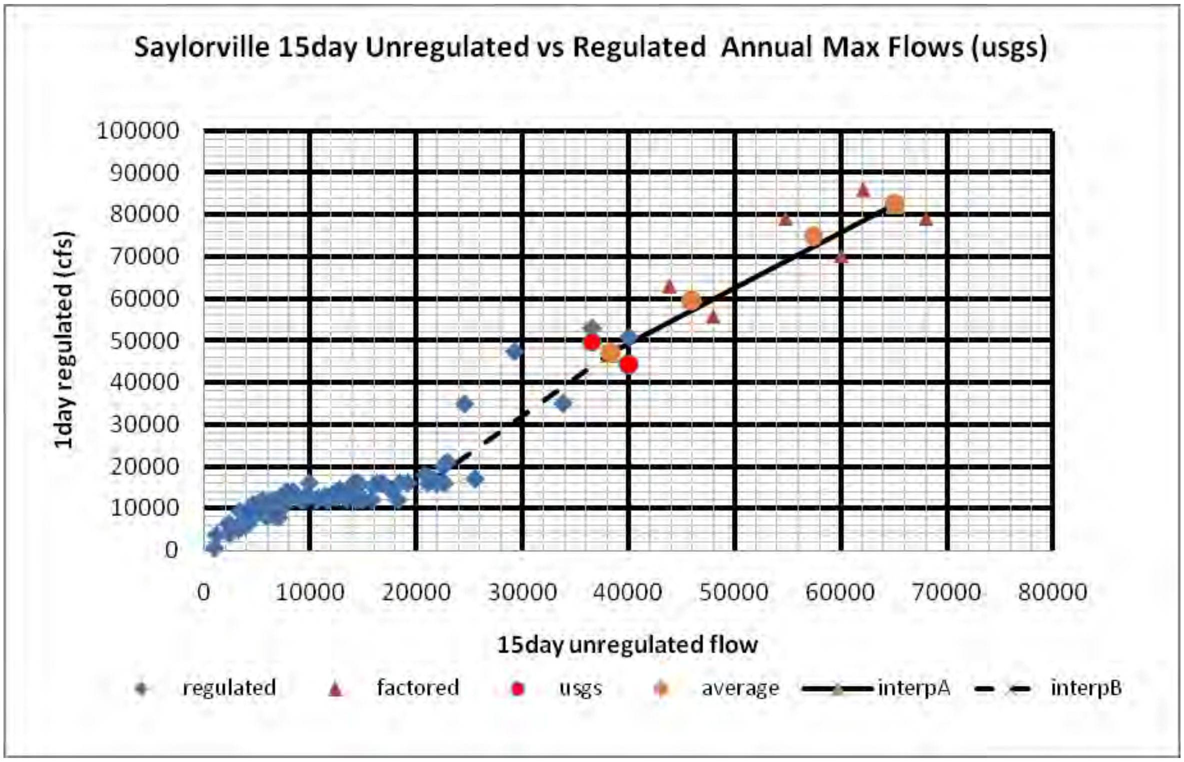

FIGURE 8.10: SAYLORVILLE 15DAY UNREGULATED VS REGULATED ANNUAL MAX FLOWS

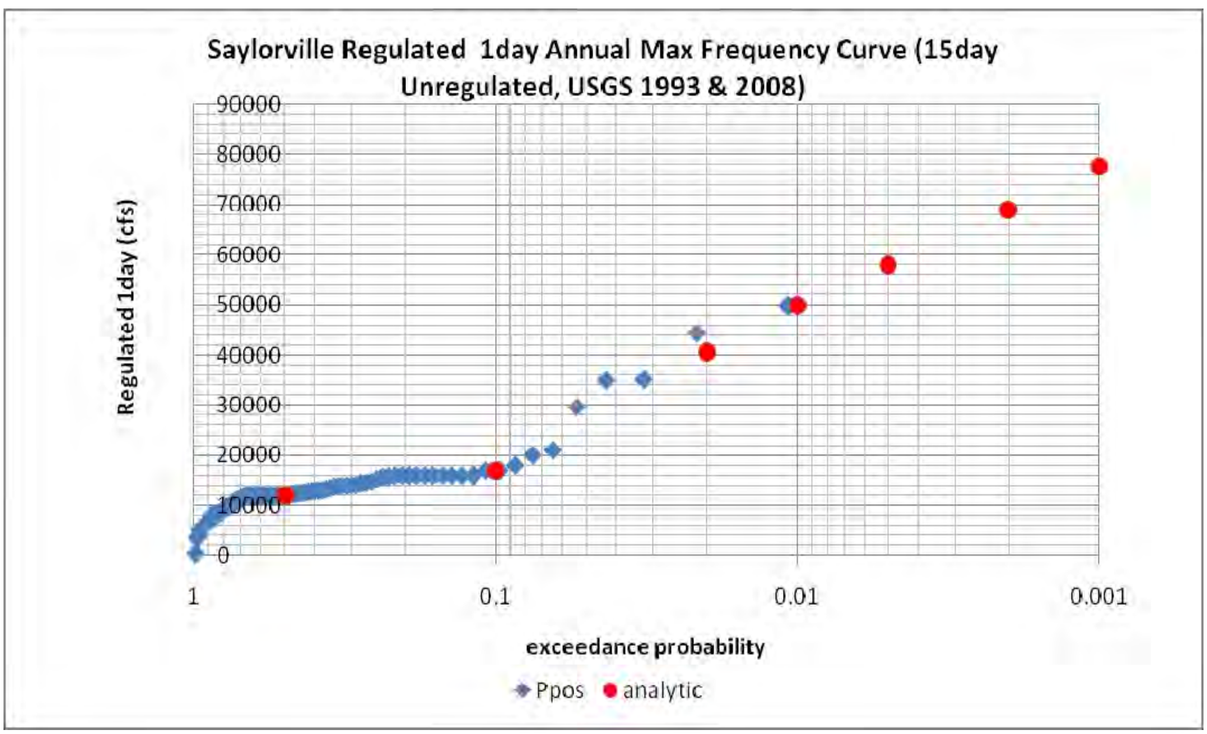

FIGURE 8.11: SAYLORVILLE REGULATED 1DAY ANNUAL MAX FREQUENCY CURVE (15DAY INFLOW)

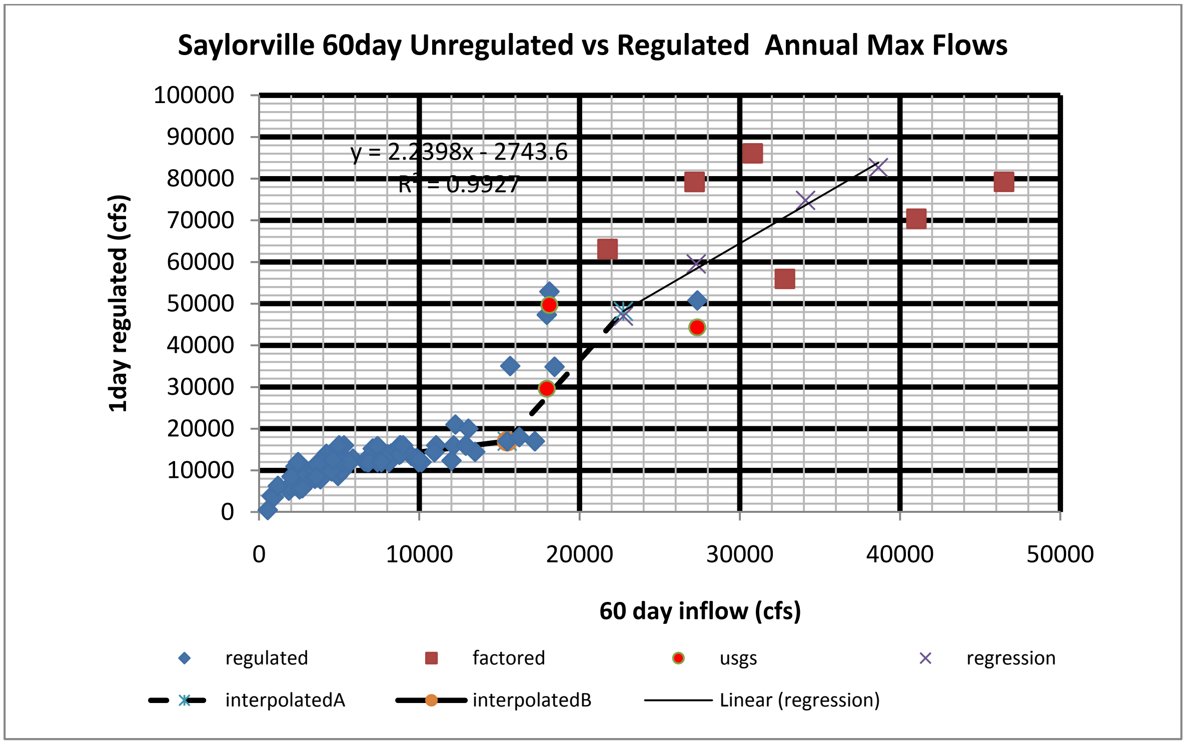

FIGURE 8.12: SAYLORVILLE 60 DAY UNREGULATED VS REGULATED ANNUAL MAX FLOWS

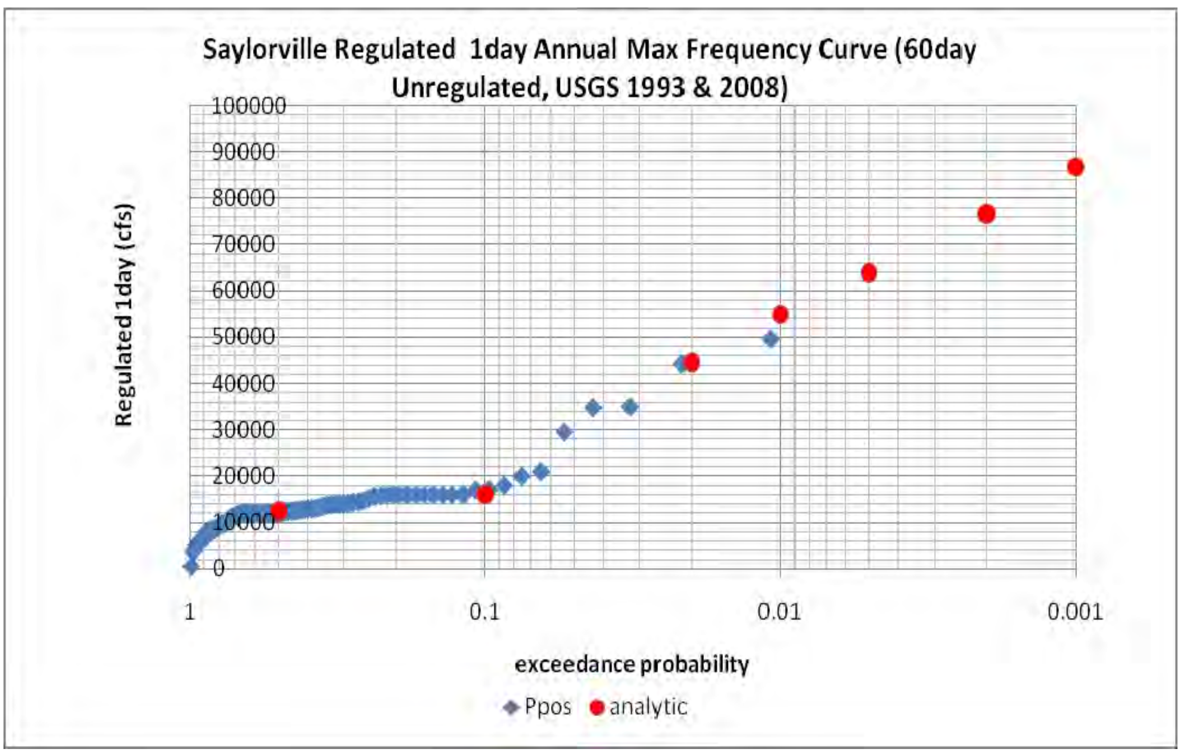

FIGURE 8.13: SAYLORVILLE REGULATED 1DAY ANNUAL MAX FREQUENCY CURVE (60DAY INFLOW)

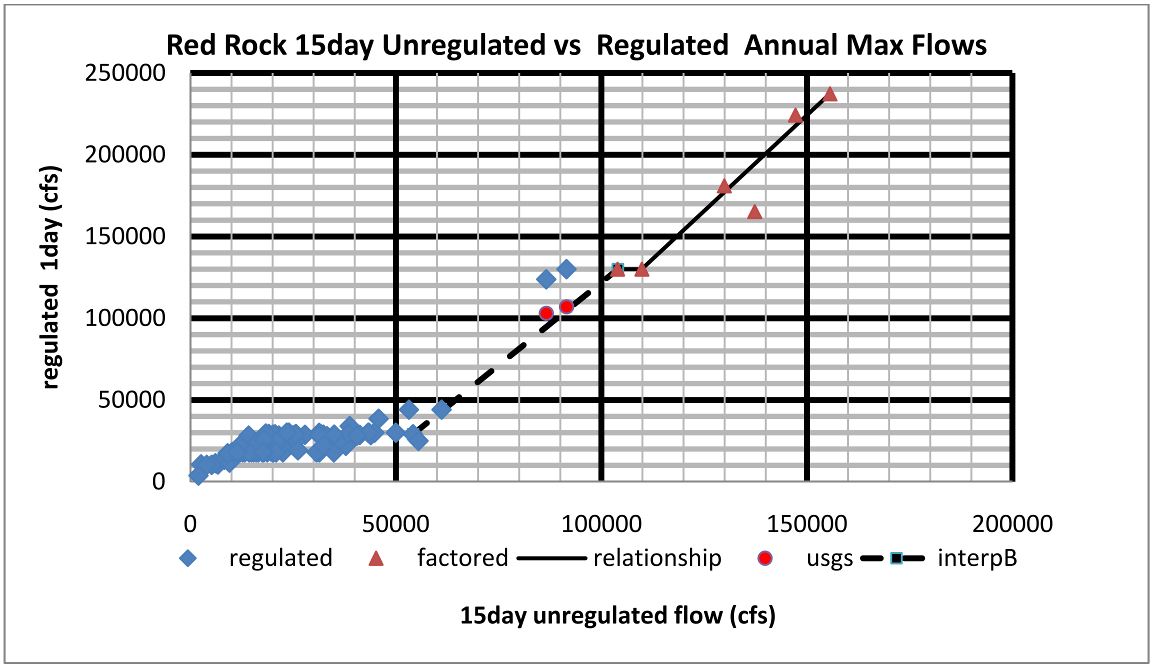

FIGURE 8.14: RED ROCK 15 DAY UNREGULATED VS REGULATED ANNUAL MAX FLOWS

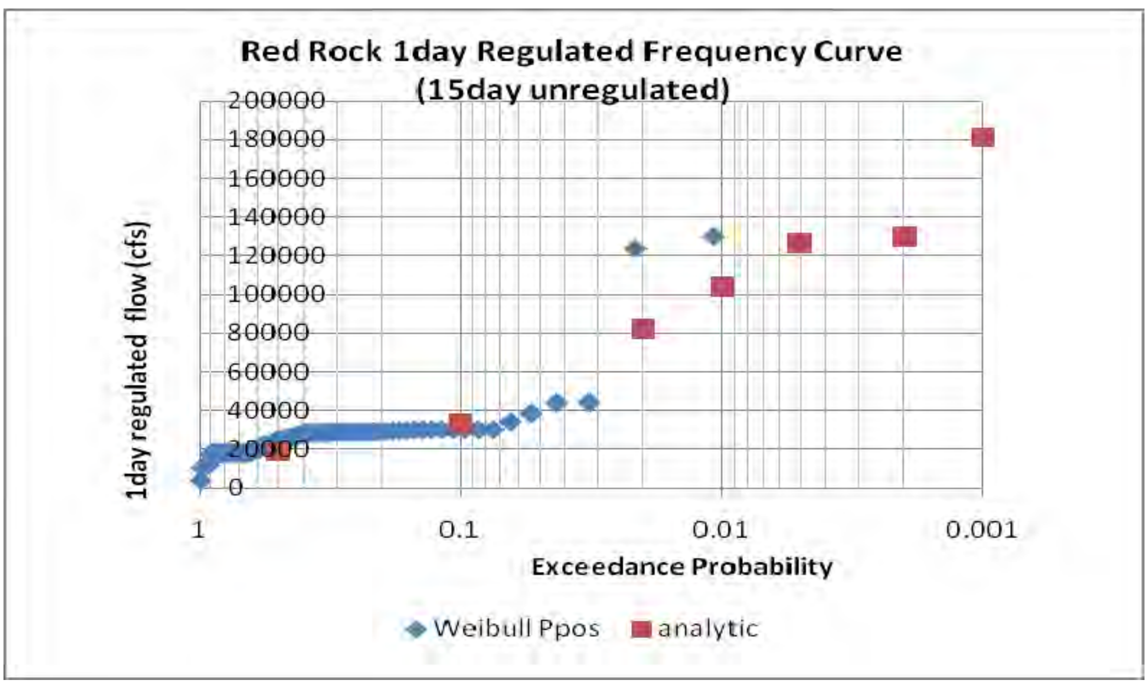

FIGURE 8.15: RED ROCK REGULATED 1DAY ANNUAL MAX FREQUENCY CURVE (15DAY INFLOW)

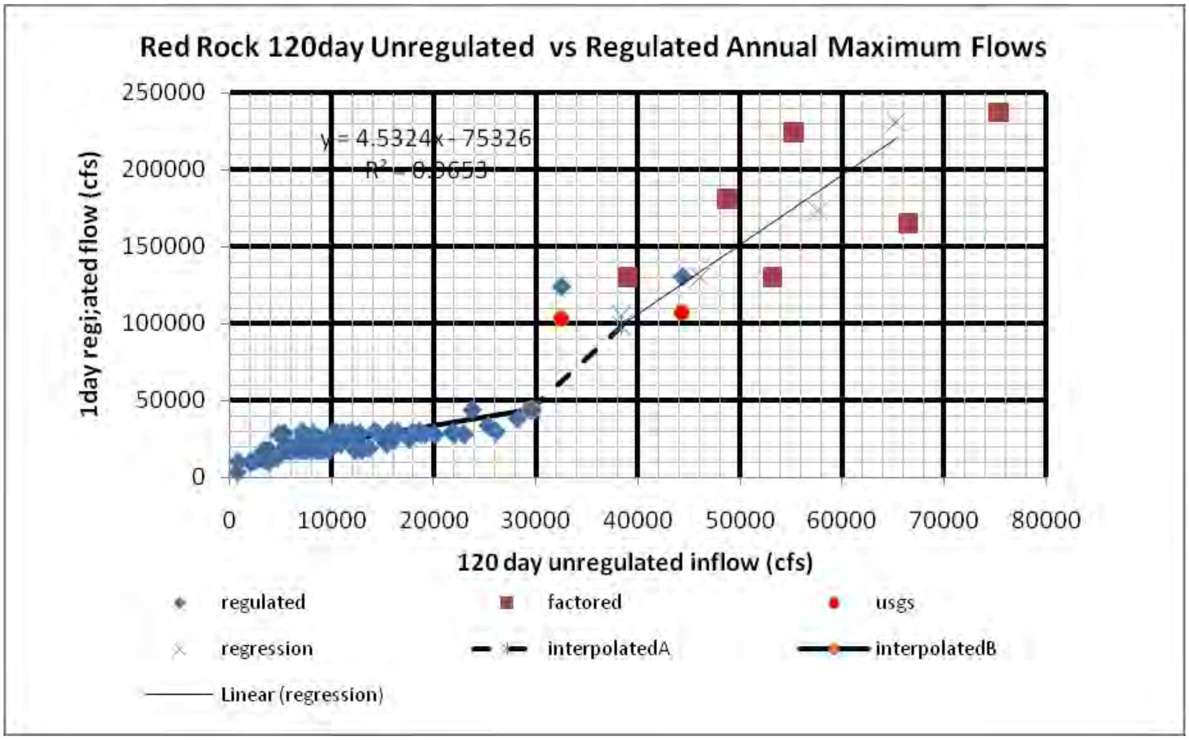

FIGURE 8.16: RED ROCK 120 DAY UNREGULATED VS REGULATED ANNUAL MAX FLOWS

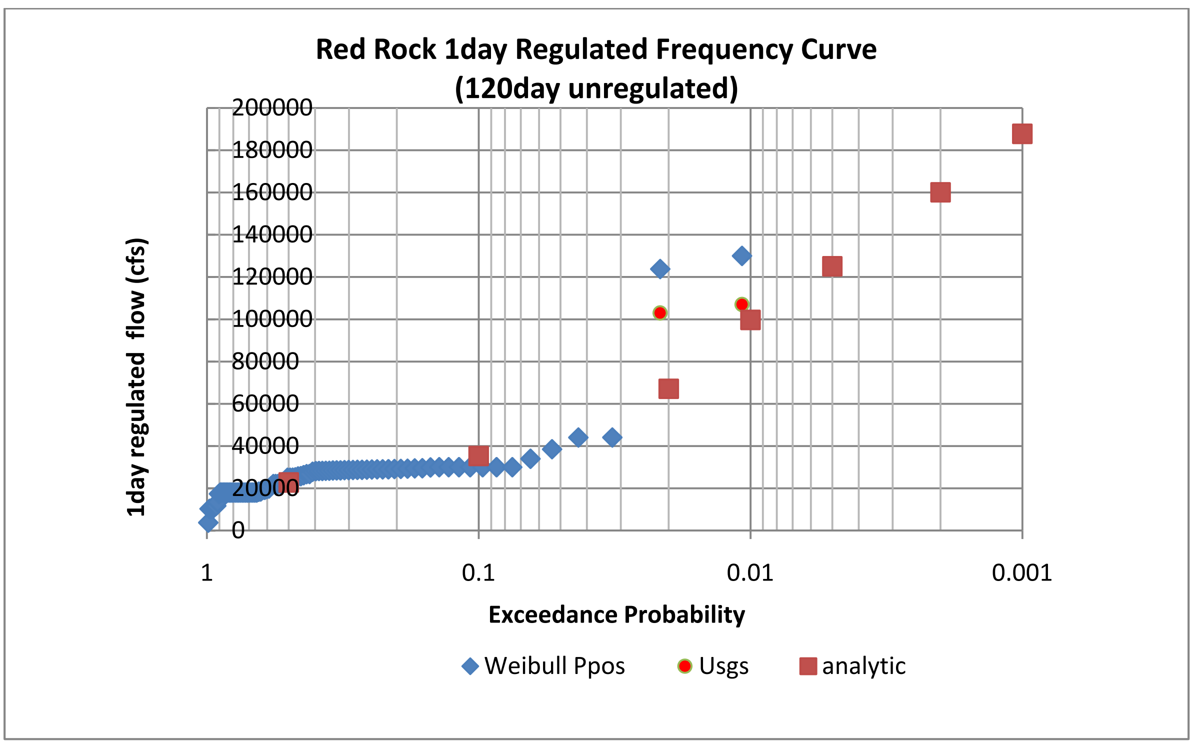

FIGURE 8.17: RED ROCK REGULATED 1DAY ANNUAL MAX FREQUENCY CURVE (15DAY INFLOW)

8.6 Regression between Peak and Daily Flows

The available period of record observed peak and annual maximum flow values at USGS gages are shown in Table 8.12. Annual maximum peak and 1day annual maximum flows that were separated by dates differing by at most 1day were used as the paired data in the regression relationship. The results of the simple linear regression analysis are very good as can be seen from the R^2 values shown in the table and as displayed in Figures 8.18 – 8.22

As can be seen from these figures gage data existed pre and post dam construction. The effect of the regulation on the relationship between peak and daily flow was explored by plotting separately before and after construction values against the derived regression. As can be seen, the regression describes both periods of data equally well.

The excellent fit of the regression perhaps is indicative of the common methodology used by the USGS to compute either peak or daily flows using rating curves and the occasional direct meter measurements. This results in a very strong relationship.

Table 8.12: Peak versus daily annual maximum regression

Location	Usgs ID	[1]POR	[2]a	[2]b	[3]R^2
Saylorville	5486150	1962-2009	1.0073	207.58	0.99
SE6th	5485500	1941-2008	1.0368	80.009	0.99
Red Rock/Tracy	5488500	1920-2008	1.0607	-818.61	0.98
Ottumwa	5489500	1917-2008	1.0116	1484.3	0.97
Keosauqua	5490500	1917-2008	1.0695	498.33	0.95

[1]Period of Record
[2]$Q_p = a + b(Q_d)$, where Q_p = peak annual discharge (cfs),
Q_d = maximum annual daily discharge (cfs)
[3]R^2 = regression coefficient of determination

9 Peak Flow Regulated Flow Frequency Curves

The peak flow regulated flow frequency curve shown in Table 8.13 were computed by applying the regression equations in Table 8.10 to the 1day annual maximum regulated frequency curve shown in Table 8.3, 8.5 and 8.8. Additionally, the peak flow frequency curve at Second Ave was computed based on computing an unregulated curve by a drainage area ratio (see Section 8.4.3) with Saylorville (Saylorville area 5823.0 sq mi, and Second Ave area, 6245.0 sq mi) and then subtracting the Saylorville holdouts from the computed Second Ave unregulated curve (see Table 8.14).

Table 8.13: Peak Flow Regulated Frequency Curves

[1]Probability	Saylorville	[2]Second Avenue	SE6th	Red Rock	Ottumwa	Keosauqua
0.5	12300	13140	26300	25700	29300	31900
0.1	17300	19470	44100	31000	37800	42800
0.02	45200	48510	83300	68700	77100	86000
0.01	53400	57220	107500	93600	102200	113200
0.005	61900	66240	122100	137100	145000	159200
0.002	73800	78900	142000	137100	146800	162000
0.001	83200	88980	157600	158800	168900	186100

[1]Exceedance probability, [2]drainage area ratio with Saylorville

Table 8.14: Computation of Second Ave regulated frequency curve using Saylorville Reservoir Holdouts

	Saylorville			Second Ave	
probability	unregulated	regulated	holdout	[1]unregulated	[2]regulated
0.5	15700	12000	3,700	16838	13138
0.1	34100	17000	17,100	36571	19471
0.02	52600	44700	7,900	56412	48512
0.01	61000	52800	8,200	65421	57221
0.005	69600	61200	8,400	74644	66244
0.002	81500	73000	8,500	87406	78906
0.001	90800	82400	8,400	97380	88980

[1]Unregulated based on drainage ratio with Saylorville
[2]Regulated = unregulated at Second Ave minus holdout at Saylorville

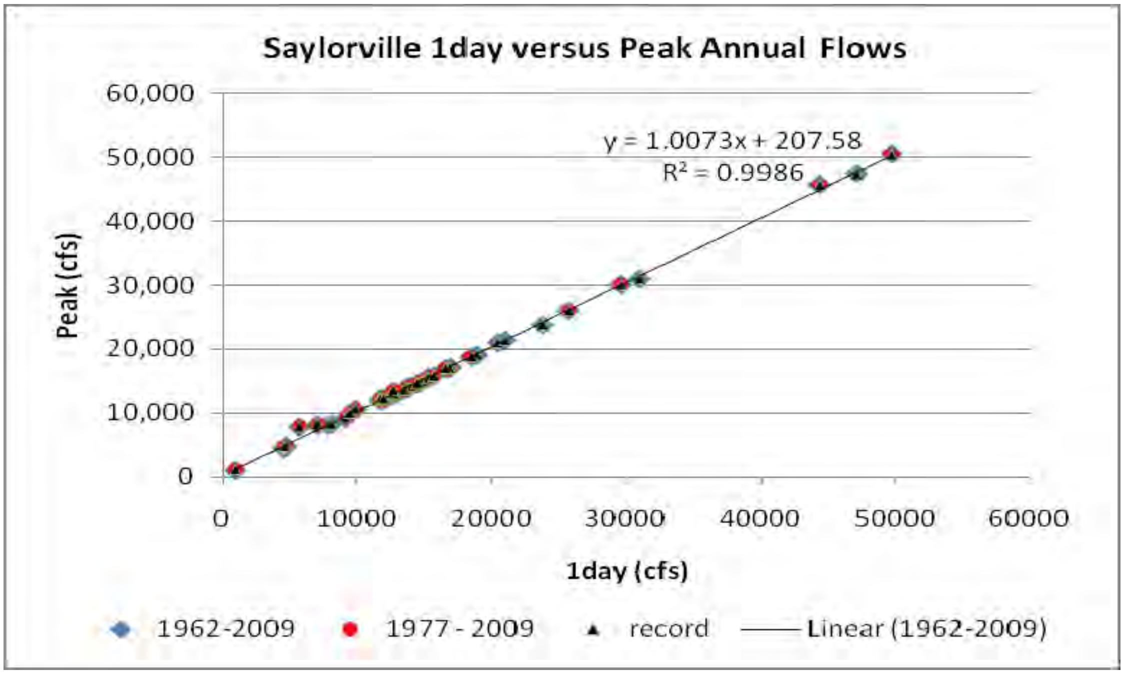

FIGURE 8.18: SAYLORVILLE PEAK VERSUS DAILY REGRESSION

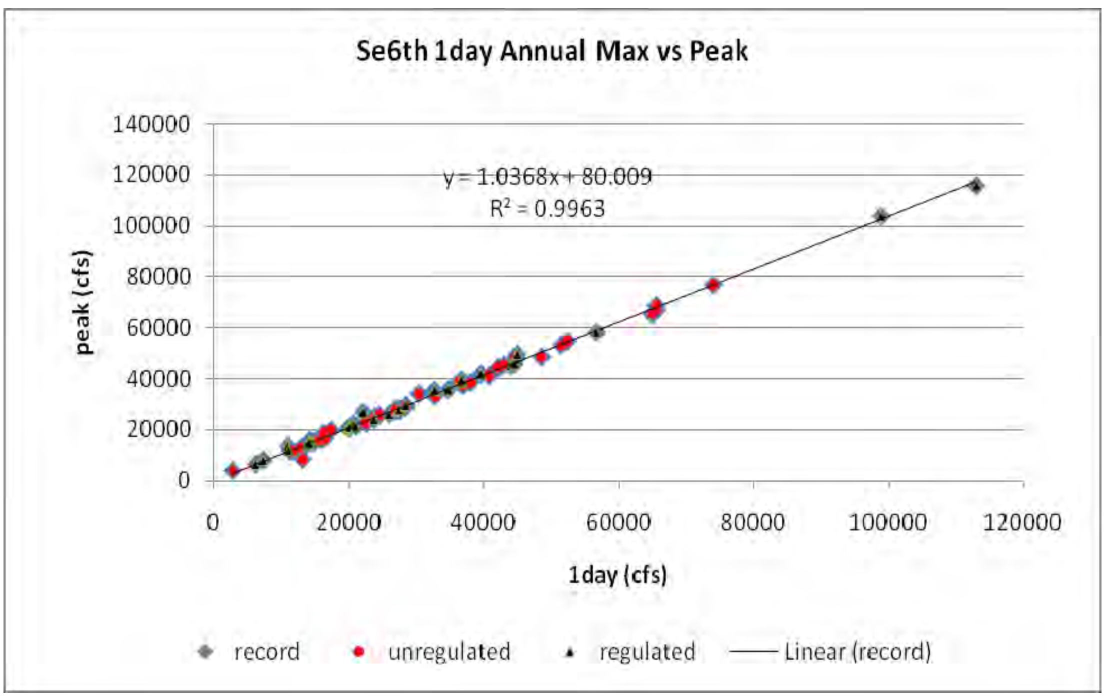

FIGURE 8.19: SE6TH STREET PEAK VERSUS DAILY REGRESSION

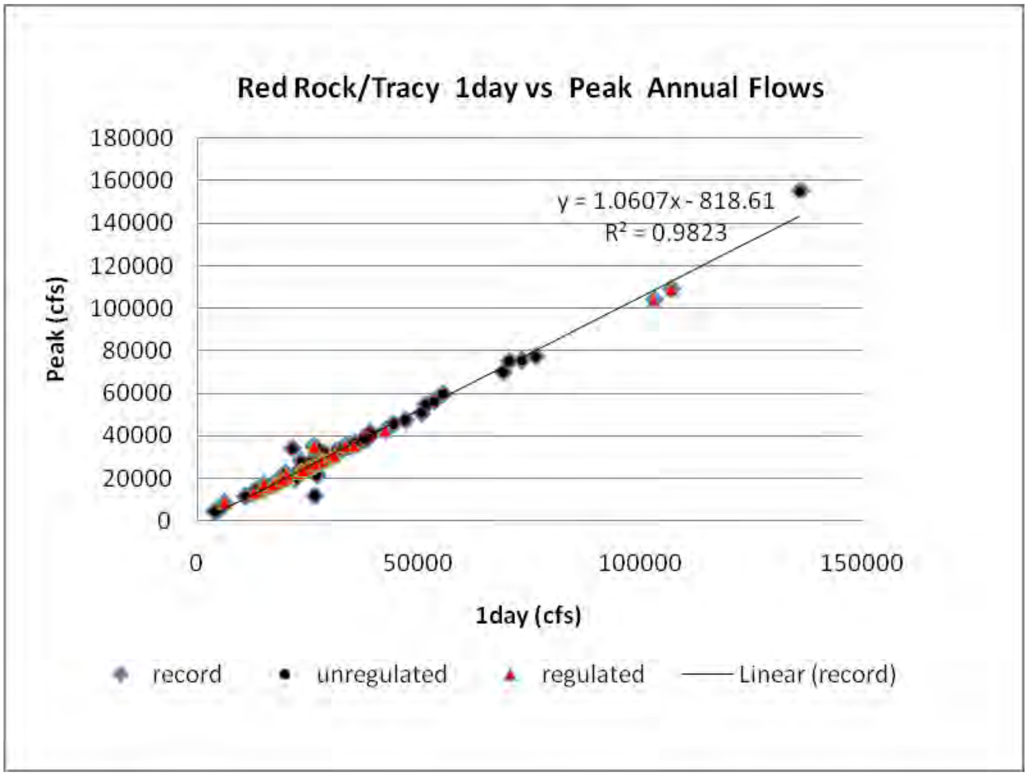

FIGURE 8.20: RED ROCK PEAK VERSUS DAILY REGRESSION

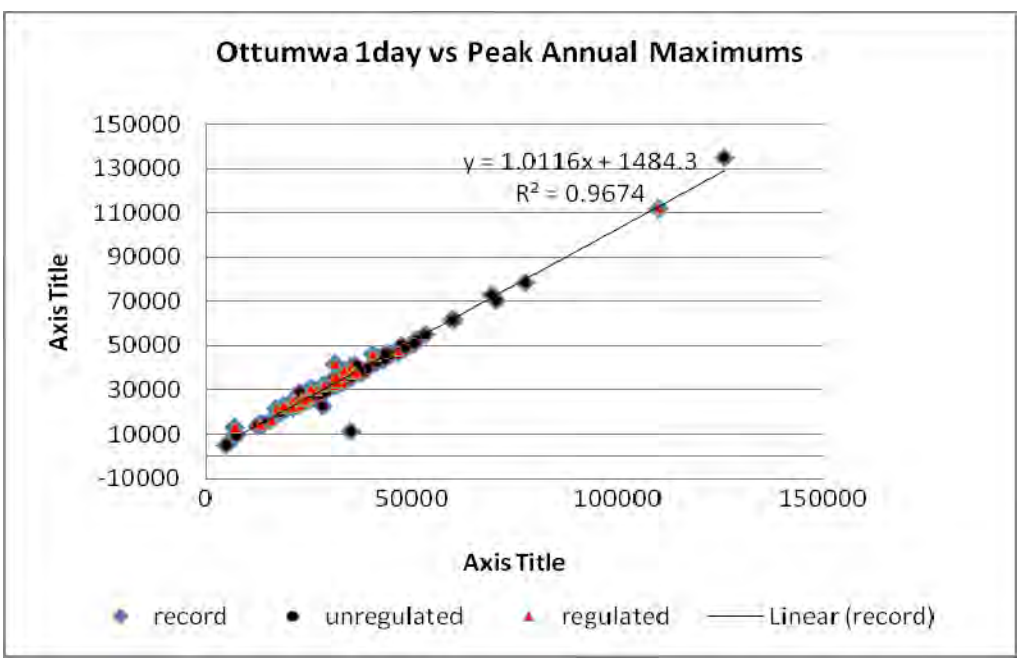

FIGURE 8.21: OTTUMWA PEAK VERSUS DAILY REGRESSION

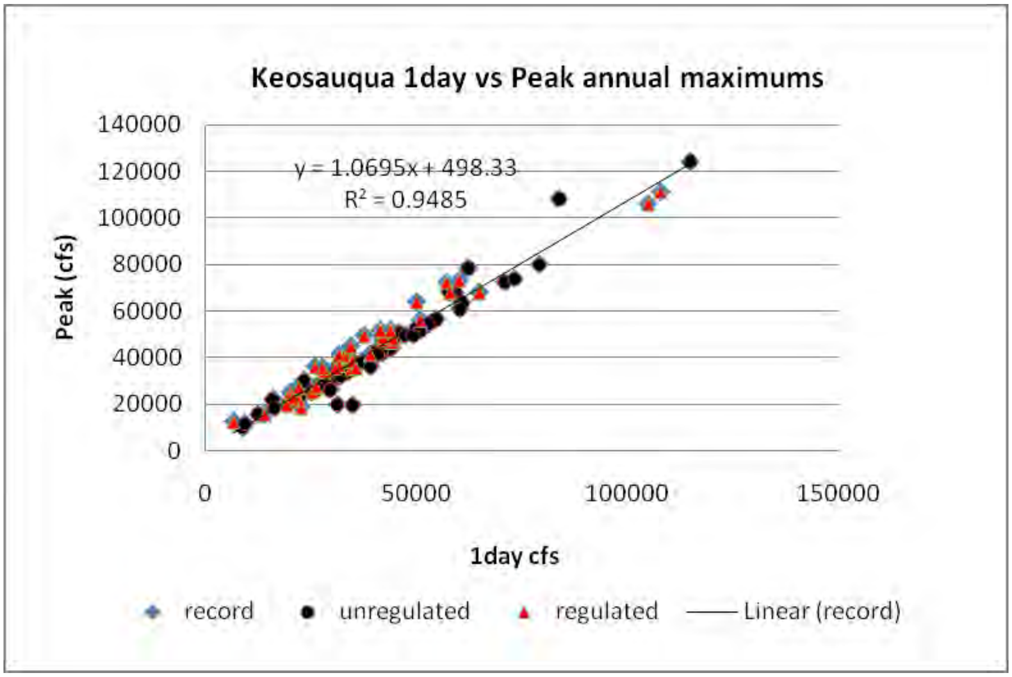

FIGURE 8.22 KEOSAUQUA PEAK VERSUS DAILY REGRESSION

10 Pool Elevation Frequency Analysis

The Saylorville and Red Rock Dam pool elevation frequencies were computed by integrating the outlet works rating and regulated peak flow frequency curve for flow releases exceeding the objective release including simulated factored flows (factors 1.2, 1.5 and 1.7) of the1993 and 2008 events. Plotting position exceedance probability estimates were used when the reservoir surcharges to control flows to the objective release. Figures 9.1 and 9.2 show the resulting pool elevation frequency curves. Table 9.1 compares these pool elevation frequency curves with the estimates obtained from the previous Corp of Engineers (2002) study.

Table 9.1: Saylorville and Red Rock Reservoir Pool Elevation Frequencies

	Saylorville Elevation (ft)			**Red Rock elevation (ft)**	
[1]Probability	**Current Study**	2002 Study [4]		**Current Study**	2002 Study [4]
	[2]regulated flow			**[2]regulated flow**	
0.001	**896.30**		**0.001**	**785.00**	
0.002	**894.70**		**0.002**	**784.00**	
0.005	**893.20**	889.8	**0.005**	**782.60**	780.9
0.01	**892.10**	889.6	**0.01**	**780.70**	780.2
0.02	**890.60**	888.9	**0.02**	**780.10**	779.1
	[3]plotting position			**[3]plotting position**	
0.1	**880.55**	881.0	**0.1**	**770.00**	768.0
0.2	**868.95**	868.0	**0.2**	**764.72**	763.5
0.5	**844.80**	842.5	**0.5**	**749.30**	748.2

[1]Exceedance probability
[2]Elevation based on log-Pearson iii 30 day critical duration inflow
[3]Interpolated from median plotting positions
[4]Estimates from previous study (Corps of Engineers, 2002)

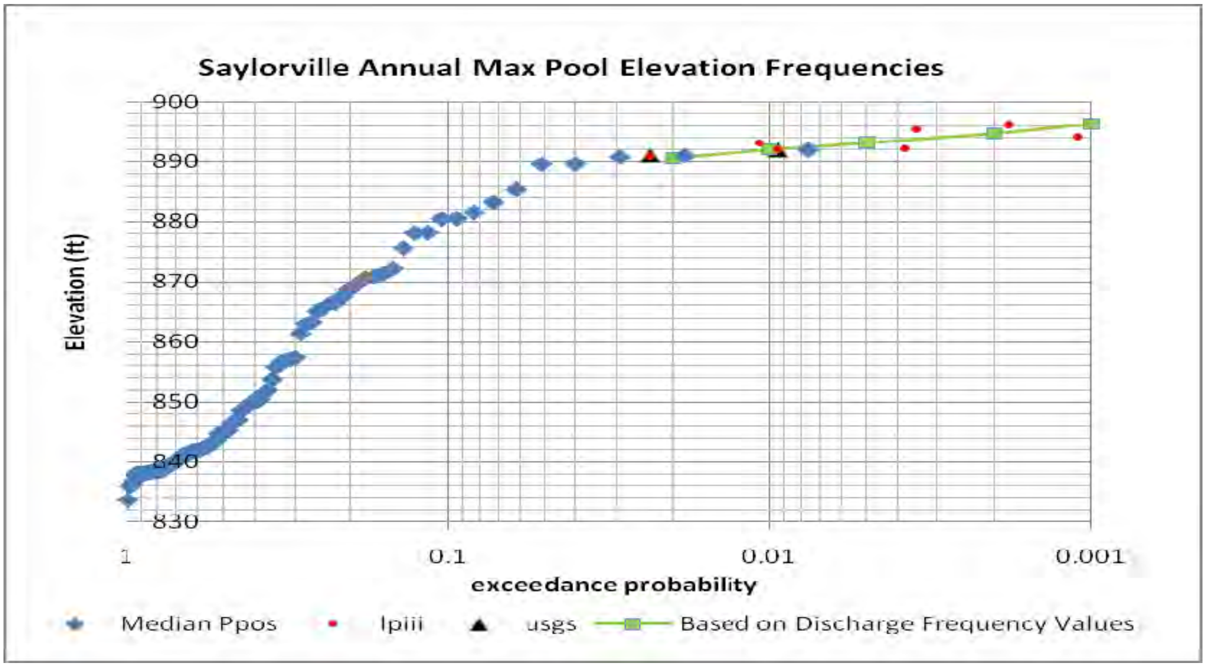

FIGURE 9.1: SAYLORVILLE POOL ELEVATION EXCEEDANCE FREQUENCIES
ppos = median plotting position, lpiii = log Pearson iii exceedance probabilities of releases due to factored inflow, lpiii2008&1993 = log Pearson iii estimates of 2008&1993 elevation exceedances log(lpiii) = regression of top plotting positions

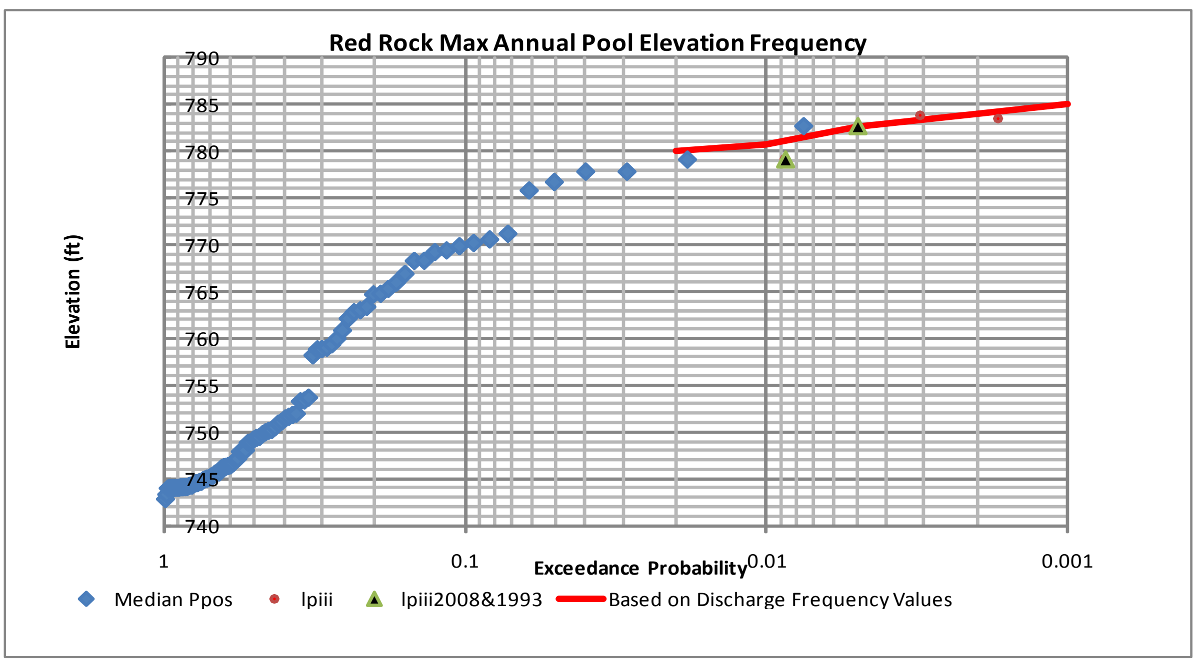

FIGURE 9.2: RED ROCK POOL ELEVATION EXCEEDANCE FREQUENCIES
ppos = median plotting position, lpiii = log Pearson iii exceedance probabilities of releases due to factored inflow, lpiii2008&1993 = log Pearson iii estimates of 2008&1993 elevation exceedances, log(lpiii) = regression of top plotting positions