

From: [Burnham, Nicolle](#)
To: [Downtown West Project](#)
Subject: FW: Diridon LG Creek Data
Date: Tuesday, July 21, 2020 1:39:18 PM
Attachments: [VWConcurrenceLetter.pdf](#)
[GuadalupeHydrology_Nov2009_FINAL.pdf](#)
[Impacted Parcels PARCEL COUNT.pdf](#)
[Impacted Parcels AREA Calcs.pdf](#)

From: Caitlin Gilmore [mailto:cgilmore@rewsprojects.com]
Sent: Tuesday, July 21, 2020 10:01 AM
To: Do, Ryan <ryan.do@sanjoseca.gov>; Burnham, Nicolle <nicolle.burnham@sanjoseca.gov>
Cc: Robert Yin <robertyin@google.com>; Bhavesh Parikh <bhaveshp@google.com>; Paul Dunne <pauldunne@google.com>
Subject: Diridon LG Creek Data

[External Email]

Ryan and Nicole,

Per our meeting on Friday, here is the requested data:

1. [Link to existing conditions model](#). Note the total file size is around 11GB. If it does not download, please let me know and I can mail hard drives of the hydraulic model to the necessary recipients if you can provide addresses.
2. Attached is the concurrence letter from VW on the hydraulic model.
3. Attached is the USACE PDF of the hydrology study.
4. Attached are tables of both parcel count and area (Google versus other) associated with project alternatives.

Thank you,

Caitlin Gilmore, PE, QSD, CPSWQ, LEED AP |
Senior Project Manager

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January 28th, 2020

Mr. Bhavesh Parikh
Director, Real Estate Development
Google Inc.

Subject: Concurrence of the Los Gatos Creek Hydraulic Model

Dear Mr. Parikh,

Valley Water has reviewed and determined that the HEC-RAS hydraulic model dated 12/14/2019, along with the accompanying documentation titled *Los Gatos Creek Independent QC and Revision to FINAL HEC-RAS Model*, is a valid representation of Los Gatos Creek in its current condition between I-280 and the Guadalupe River.

Sincerely,

A handwritten signature in blue ink, appearing to read "Liang Xu".

Liang Xu, PhD, PE
Hydraulics, Hydrology, & Geomorphology Unit Manager
Valley Water



**GUADALUPE WATERSHED
HYDROLOGIC ASSESSMENT**

**Prepared by:
U.S. Army Corps of Engineers
San Francisco District**

April 2008

Final Report Update: November 2009

ACKNOWLEDGEMENTS

Special thanks to the members of the Guadalupe Hydrology Technical Working Group:

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GUADALUPE WATERSHED HYDROLOGIC ASSESSMENT

EXECUTIVE SUMMARY

This report presents the updated study methodology and results of the 2009 Guadalupe Watershed hydrologic assessment. This report serves to update the hydrologic assessment submitted in 1977.

From a hydrologic standpoint, the Guadalupe Watershed is a relatively complex area to study. Precipitation, land use, soil types, elevation, and slopes vary widely from the upper reaches of the basin near Mount Umunhum to the lower reaches of the basin where the Guadalupe River reaches the San Francisco Bay via Alviso Slough. Flow throughout the watershed is controlled by five water supply reservoirs which impact the flow regime substantially, particularly Lexington Reservoir on Los Gatos Creek.

The methodology of this study was selected carefully to best capture the unique aspects of this watershed. A multi-agency “Working Group” was created to ensure that all technical decisions would be adequately discussed and documented, that all options would be properly weighed, and that group consensus could be reached, when possible.

The watershed was split up into 42 sub-basins and unique parameters and assumptions were developed for each. The model was designed to simulate representative discharge values after completion of the Upper Guadalupe Project. All design discharges are assumed to remain within the channel. The model was calibrated to three historic events before the synthetic storm series simulations were configured which estimate peak discharge values for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5, and 0.2-percent chance exceedance events. The meteorology input was constructed to take into account the large difference in precipitation patterns throughout the basin. Unique 72-hour precipitation patterns for each statistical event were developed for each sub-basin.

The results of the synthetic storm series simulations are similar to the results of the 1977 hydrologic report. Peak discharge at the San Jose gage (USGS Gage #11169000) for the 1% chance exceedance event was estimated at 17,967 cubic feet per second (cfs). The 1977 study estimated peak 1% flow at this location to be 17,000 cfs.

1. BACKGROUND

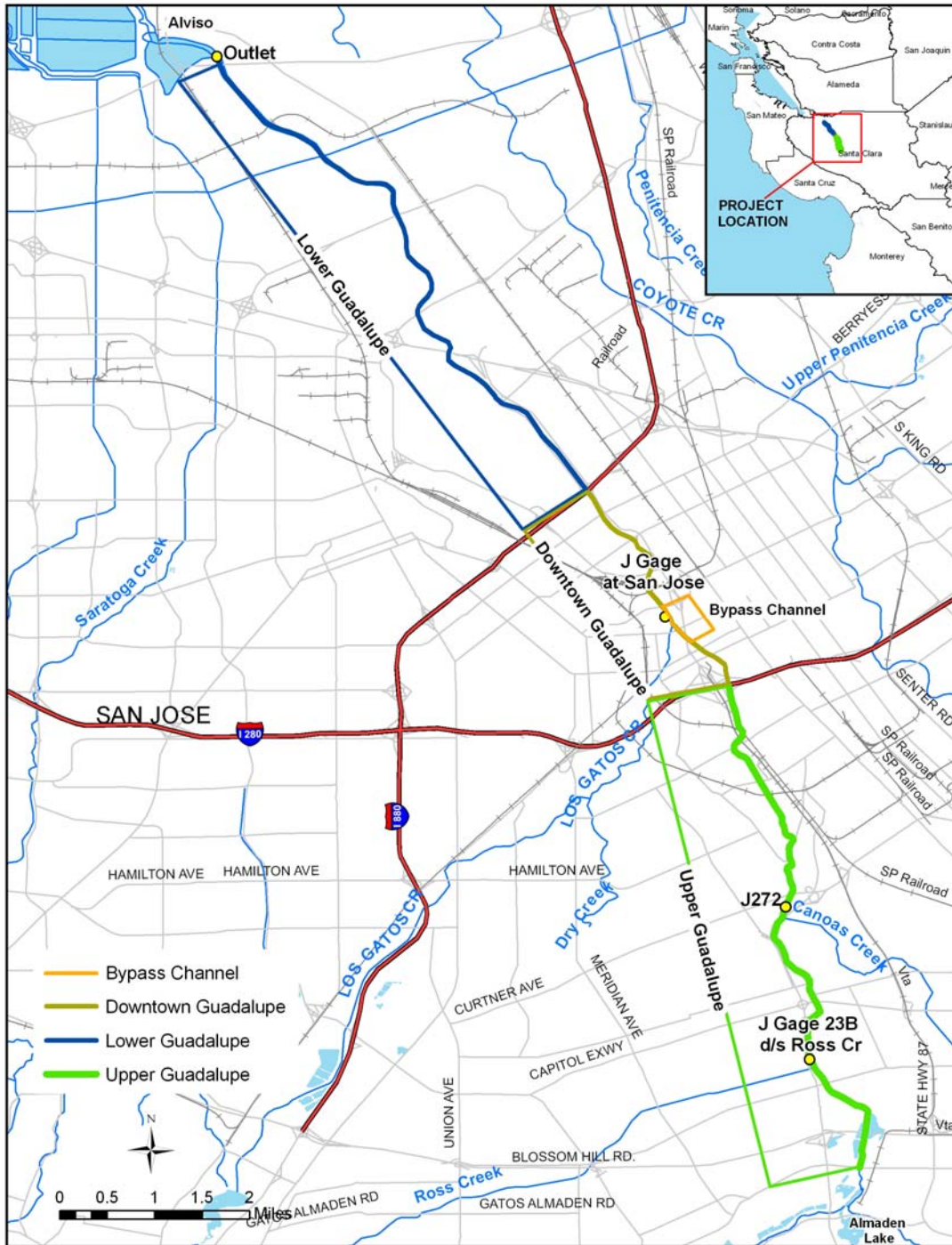
The Guadalupe River watershed is located at the south end of the San Francisco Bay in Santa Clara County, California. Flooding has occurred in the Guadalupe watershed since the early 1900's and an increase in urbanization within the basin has gradually increased flood damages over the years. The Army Corps of Engineers (Corps) has been involved in flood mitigation activities in the Guadalupe watershed since 1945, when a project was authorized to survey all of the streams draining into the San Francisco Bay south of the Dumbarton Bridge. After being suspended and resumed several times, this study was completed in 1958.

The Guadalupe River has been divided into three project study areas: Lower Guadalupe, Downtown Guadalupe, and Upper Guadalupe. The Lower Guadalupe River Project is located between Alviso and I-880, the Downtown Guadalupe River Project runs between I-280 and I-880, and the Upper Guadalupe River Project runs between Almaden Lake and I-280 and also encompasses a small portion of Ross Creek just upstream of the confluence with Guadalupe River (**Figure 1**). The Lower Guadalupe River and Downtown Guadalupe River Projects have been completed¹. The Upper Guadalupe River Project² will be entering the construction phase in the near future.

2. PURPOSE AND SCOPE

This report presents the results of a hydrologic assessment prepared for the entire Guadalupe River watershed. The last hydrologic assessment for the entire watershed was completed in 1977³. In the intervening years, 30 additional years of hydrologic and meteorological information have been collected, several significantly sized flood events have occurred, and additional urbanization in the watershed has taken place. As such, an update of the watershed hydrology was undertaken. Information from this report will be used by the Corps and the Santa Clara Valley Water District (SCVWD) to aid in efforts to reduce flood risk within the basin. This study was initiated in 2007 by two regionally coordinated Corps letters. The first letter was issued 27 April 2007 to the Federal Emergency Management Agency (FEMA), and the second letter was issued 30 April 2007 to the project sponsor, SCVWD, indicating that "The Project Management Plan for the Upper Guadalupe Flood Control project will be amended to incorporate current USACE guidance with updated hydrology and a complete risk and uncertainty analysis for the entire river system". The hydrology of the Guadalupe watershed was updated in two phases. The first phase was initiated in 2007 at the start of the Pre-construction Engineering and Design (PED) phase for the design of the Upper Guadalupe Project. The Upper Guadalupe Hydrologic Assessment focused on the reach of the Guadalupe River upstream of its confluence with Los Gatos Creek and was completed in April 2008. Shortly thereafter, the second phase expanded the hydrologic study to include the entire Guadalupe watershed by adding the Los Gatos Watershed, the Downtown Guadalupe Project and the Lower Guadalupe Project to the hydrologic modeling area. During the course of the expanded study, updates were made at the recommendation of the Working Group (see description in **Section 5**) to the meteorology, hydrologic routing methods, transform methods, and loss rates to reflect new guidance from the hydrologic community on watershed modeling, new computational tools, more detailed modeling techniques, as well as contemporary land use, channel configuration, reservoir operation, meteorological data, and the hydrologic regime of the catchment. Details of the differences between the 1977, 2008, and 2009 reports are presented in **Section 7.2**.

Figure 1 – Guadalupe River Lower, Downtown, and Upper Project Areas



3. DESCRIPTION OF AREA

The Guadalupe River watershed is located south of the Bay Area in Santa Clara County (**Figure 2**). Guadalupe River is the second largest river in Santa Clara County, draining an area of approximately 170 square miles into the South San Francisco Bay at Alviso Slough. The watershed is bounded on the south and southwest by the Santa Cruz Mountains, on the west by the San Tomas Creek and Saratoga Creek Basins, on the north by the San Francisco Bay, and on the east by the Coyote Creek Basin. The Guadalupe River flows in a northwesterly direction and is fed by several major tributaries including Los Gatos, Ross, Canoas, Guadalupe, and Alamos Creeks. Much of the runoff from the mountainous areas is controlled by one of five major reservoirs: Almaden, Calero, Lexington, Guadalupe, and Vasona (see **Figure 3**).

This watershed is unique as it changes drastically between the upper and lower extents. The average annual rainfall in the basin is approximately 15 inches, though it varies greatly throughout, from around 13 inches at the downtown San Jose gage to around 42 inches at the Lexington gage. Elevations within the study area range from sea level at the San Francisco Bay to around 3,800 feet above mean sea level on the top of Loma Prieta. The watershed is composed of very urbanized flat valley and foothill areas, and steep mountain areas with relatively little urban influence.

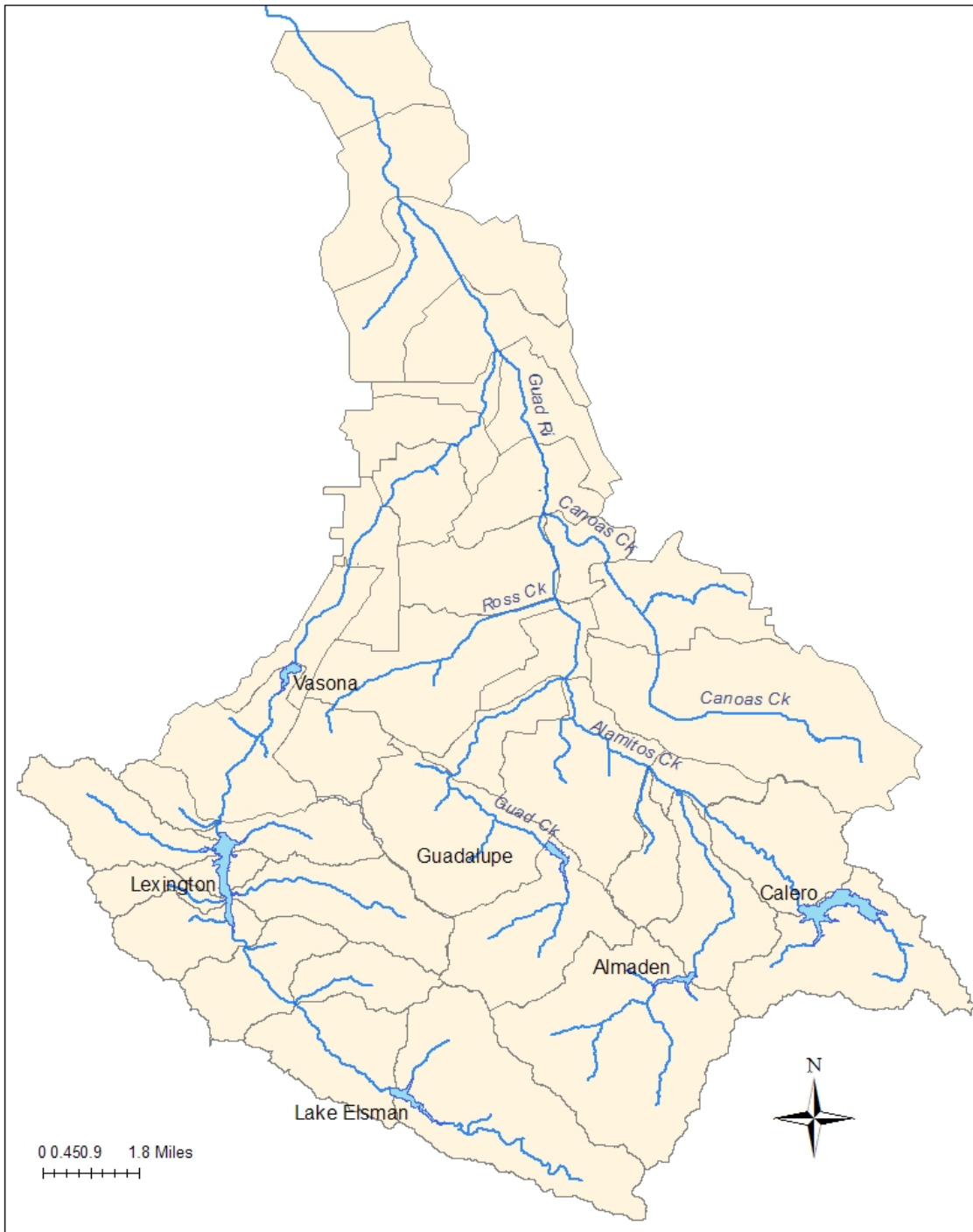
4. PREVIOUS INVESTIGATIONS

The first major hydrologic study on the Guadalupe River and Coyote Creek basins was completed in 1977³. The Guadalupe River and Coyote Creek Hydrologic Investigation conducted by the Corps in June 1977 was used as the basis for the hydrology for the formulation of the Downtown and the Upper Guadalupe River Projects. The 1977 study includes a statistical evaluation of the annual maximum discharge at four of the reservoirs within the Guadalupe basin (Guadalupe, Lexington, Almaden, and Calero). The design 1-percent peak discharges from the 1977 investigation were reviewed in 1991 as part of the General Design Memorandum (GDM) for the downtown reach to consider recent flood events and existing basin conditions, including recent urban development. At that time, it was determined that the hydrology for future conditions in the 1977 report properly reflected recent upstream channel improvements and urbanization, and that the routing method and rainfall loss rates were still valid. In 2007, the 1977 hydrology was reviewed again and the decision was made to perform a new hydrology study of the watershed to take advantage of new data available and to study changes in the system that might impact the discharge values including increases in urbanization and changes in reservoir operation.

Figure 2 – Guadalupe River Basin and Sub-Basin Delineations



Figure 3 – Reservoirs within the Guadalupe Watershed



5. STUDY METHODOLOGY

This study referenced applicable USACE hydrologic modeling guidance when developing the study methodology including:

- EM 1110-2-1415, Hydrologic Frequency Analysis⁴
- EM 1110-2-1417, Flood Runoff Analysis⁵
- EM 1110-2-1419, Hydrologic Engineering Requirements for Flood Damage Reduction Studies⁶
- Bulletin #17b, Guidelines for Determining Floodflow Frequency⁷
- Hydrometeorological Report Number 59 Probable Maximum Precipitation for California Calculation Procedures (HMR 59)⁸.

The study methodology was developed by a multi-agency “Working Group” with participants from the USACE San Francisco (SPN) and Sacramento (SPK) Districts, USACE Hydrologic Engineering Center (HEC), USACE South Pacific Division (SPD), and SCVWD. All technical decisions were discussed thoroughly by the Working Group with the final objective being to choose the most appropriate methodologies and input data for this particular study area.

The Hydrologic Engineering Center’s Hydrologic Modeling System (HEC-HMS version 3.3) was used in this study to simulate the precipitation/runoff process. This model was developed to simulate project design discharges after the completion of the Upper Guadalupe Project. All design discharges are assumed to remain within the channel.

The Guadalupe River watershed was separated into 42 sub-basins (**Figure 2**) and 27 river reaches (**Table 1**). A description of key junctions is presented in **Table 2**. The model was calibrated to peak events in January 1995, March 1995, and February 1998. Model parameters were adjusted accordingly as part of a qualitative calibration process. Stream gages and rain gages throughout the basin were utilized in configuring the models used for calibration. Flood hydrographs for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5, and 0.2- percent chance exceedance events were computed at various locations in the watershed. A comprehensive risk and uncertainty evaluation was also performed.

Table 1 – River Reaches by Sub-Basin in HEC-HMS Model

Sub-Basin	Sub-Basin Description	Reach(s)
Above Ross	Above Ross	R340
Below Almaden	Below Almaden Res	R520
Los Gatos 3	Los Gatos 3	R LG 3
Los Gatos 4	Los Gatos 4	R LG 4
OI	Upstream Guadalupe River on Los Gatos Creek	R LG 2
OJ	Upstream Guadalupe River on Los Gatos Creek	R170
Upper Ross	Upper Ross	NA
W1160	Upstream of Los Gatos Creek on Guadalupe River	R200
W1170	Upstream of Los Gatos Creek on Guadalupe River	R200
W1190	Los Gatos Creek Downstream from Vasona Reservoir	R LG 5
W1190A	Los Gatos Creek Upstream from Vasona Reservoir	R580
W1220	Upstream of Los Gatos Creek on Guadalupe River	R260
W1250	Lower Canoas Creek Basin	R260, R240
W1260	Canoas Creek Basin	R290
W1270	Ross Creek Basin	R Lower Ross
W1300	Upper Canoas Creek Basin	NA
W1350	Upstream Almaden Lake Guadalupe Creek	R490
W1400	Upstream Almaden Lake Alamos Creek	R460
W1480	Alamos Creek	R520
W1540	Downstream from Almaden Reservoir	R800, R530
W1580	Trout Creek Downstream from Lexington Reservoir	NA
W1620	Downstream from Calero Reservoir	R700
W1630	Lexington Reservoir Basin East	NA
W1640	Lexington Reservoir Basin West	NA
W1650	Lexington Reservoir Lake	NA
W1660	Lexington Reservoir Basin East	NA
W1690	Upstream from Calero Reservoir	NA
W1700	Lexington Reservoir Basin West	R760
W1730	Upstream from Guadalupe Reservoir	NA
W1740	Lexington Reservoir Basin West	NA
W1780	Upstream from Lexington Reservoir	R860
W1800	Upstream from Lexington Reservoir	NA
W1850	Upstream from Almaden Reservoir	NA
W1900	Immediately downstream from Lake Elsmar	R910
W1950	Upstream from Lake Elsmar	NA
W1960	Downstream from Guadalupe Reservoir	R550
W1970	Upstream from Guadalupe Reservoir	NA
WN	Upstream of Los Gatos Creek on Guadalupe River	R Hwy 280 to Alma
WP	Downstream from Los Gatos Creek	R 880 to Lgcreek
WQ	Downstream from Los Gatos Creek	R P TO Q
WR	Downstream from Los Gatos Creek	R40
WS	At outlet	R20

Table 2 – Description of Key Junctions in Guadalupe HMS Model

HMS Junction	Description
J gage Below Almaden Res	Almaden Reservoir outlet (gage)
J gage below Guad Res	Guadalupe Reservoir outlet (gage)
J gage below Calero Res	Calero Reservoir outlet (gage)
J gage below Lex Res	Lexington Reservoir outlet (gage)
J380	Vasona Reservoir outlet
J gage Guadalupe Creek	Gage SF43 on Guadalupe Creek
J Guad & Alamitos Cr. Conflu	Confluence of Guadalupe and Alamitos Creeks
J Gage 23B d/s Ross Cr	Gage 23b on Guadalupe River downstream of Ross Creek
J272	Confluence of Guadalupe River and Canoas Creek
J LG cr to Alma	Guadalupe River just upstream of confluence with Los Gatos Creek
J gage Los Gatos u/s Guad	Los Gatos Creek just upstream of confluence with Guadalupe River
J Gage at San Jose	Guadalupe River at downtown San Jose Gage
Outlet	Outlet of Guadalupe River to the Bay

5.1 Guadalupe Watershed Reservoirs

Most of the reservoirs within the study area are small and privately owned. Of the six largest reservoirs in the watershed, five (Almaden, Calero, Lexington, Guadalupe, and Vasona) are operated by SCVWD and one (Lake Elsmar) is operated by San Jose Water Company (SJWC). Details on each of the reservoirs are presented below.

Almaden Dam and Reservoir

Almaden Dam and Reservoir was constructed in 1935. It has a capacity of 1,600 acre-feet of water and surface area of 57 acres⁹. Almaden Reservoir has an outlet pipe/conduit capacity of 190 cfs and is able to discharge up to 56 cfs to the Calero Reservoir via the Almaden-Calero Canal.

Calero Dam and Reservoir

Calero Dam and Reservoir is located on Calero Creek and was constructed in 1934. It has a storage capacity of 9,900 acre-feet and surface area of 349 acres⁹. It has an outlet conduit capacity of 185 cfs and receives up to 56 cfs of flow from the Almaden Reservoir via the Almaden-Calero Canal.

Guadalupe Dam and Reservoir

Guadalupe Dam and Reservoir was approved for construction in 1934. It is located on the Guadalupe River adjacent to Hicks Road. The reservoir has a storage capacity of 3,400 acre-feet and a surface area of 74 acres⁹. Its outlet conduit has a capacity of 235 cfs.

Lexington Reservoir and James J. Lenihan Dam

Lexington Reservoir and James J. Lenihan Dam are located on Los Gatos Creek approximately three miles south of the town of Los Gatos. The reservoir has a capacity of 19,000 acre-feet and has a surface area of 412 acres⁹. Lexington Reservoir has an outlet conduit with a flow capacity of 410 cfs; however, a Division of Safety of Dams (DSOD) restriction enforced in 1997 restricted the discharge to 70 cfs. The restriction will be rescinded when once the replacement outlet conduit construction has been completed in 2009.

Vasona Dam and Reservoir

Vasona Dam and Reservoir was approved for construction in 1934 and completed in 1935. New gates were installed in 1997. It is located on Los Gatos Creek within the town of Los Gatos. Vasona Reservoir has a capacity of 500 acre-feet and has a surface area of 57 acres⁹. It has an outlet conduit capacity of 125 cfs through a 42-inch slide gate and a discharge capacity of 1,050 cfs through the radial gates. The radial gates are typically operated during storm events in the winter months to minimize spillway flows. This serves to minimize stress on the dam and reduce damage to windows in the surrounding area resulting from harmonic vibrations. During large events, water is released through the slide gate and the reservoir is allowed to fill. When flow over the spillway begins to increase, the radial gates are opened to minimize spillway flow. It should be noted that as Vasona Reservoir is relatively small, it provides very little flow attenuation. As a result, most of the peak flows on Los Gatos Creek are attenuated in Lexington Reservoir upstream.

Lake Elsman

Lake Elsman is the only major reservoir in the Guadalupe catchment operated by the SJWC. It is located in the Santa Cruz Mountains upstream of the Lexington Reservoir. The reservoir has a capacity of 6,200 acre-feet and an earthen dam over 900 feet long and 180 feet high. In a wet year, the reservoir provides over 12 percent of the SJWC's total water capacity¹⁰. Lake Elsman was not individually modeled as part of this study as it is very far upstream of Lexington Reservoir and likely does not provide significant flow attenuation.

5.2 Reservoir Operation

The operation rules and policies of the SCVWD reservoirs have changed significantly over the years. Originally, reservoirs were operated solely for the purpose of water supply. In 1997, however the SCVWD implemented new operating strategies for Almaden, Calero, Guadalupe, and Lexington Reservoirs to reduce flood damage while minimizing impact to water supply. It should be noted that the existing operating strategies for these reservoirs are not associated with flood management project design considerations further downstream in the Lower Guadalupe River¹¹.

Storage-Discharge curves for each reservoir were taken from the 2007 SCVWD Reservoir Flood Rule Curve Operations report¹¹ and are presented in **Appendix A**. These storage-discharge curves were calculated by SCVWD and include flow over the spillway and through the outflow conduits. It is assumed that the conduits are closed as soon as water begins to flow over the spillway. In addition to outflow from the spillway and

outflow conduits, the Almaden-Calero Canal allows water to flow from Almaden to Calero. Water is released from Almaden and stored in Calero as Almaden is a small reservoir in a large catchment, while Calero is a large reservoir in a small catchment. As the flow between Almaden and Calero is small (56 cfs), it was assumed to be negligible and was not included in the model.

The initial storage of the reservoirs in the 1995 and 1998 calibration models was calibrated using downstream gage data (**Section 6**) and reservoir stage data. The initial storage at Vasona Reservoir was estimated as no downstream gage data was available. **Table 3** presents the initial storage values used in each of the calibration models.

Table 3 – Initial Reservoir Storage (acre-feet)

	Almaden	Calero	Guadalupe	Lexington	Vasona
Reservoir Capacity	1,586	9,934	3,415	19,044	495
Drainage Area (mi²)	11.9	6.9	6.0	34.6	38.4
January 1995	500	4,700	375	2,800	400
March 1995	800	8,990	3,000	18,000	400
February 1998	914	6,300	1,360	7,828	400

5.3 Precipitation Data

Precipitation data in and around the watershed was available at thirteen rain gages operated by SCVWD (**Table 4, Figure 4**). A gage weight method was used to estimate cumulative precipitation and hyetographs at each of the sub-basins. The Normal Annual Precipitation (NAP) at each sub-basin within the Guadalupe catchment was estimated using an NAP isohyetal map from the National Climatic Data Center based on the most recent statistical precipitation data. The NAP at each gage was also estimated using this map. Thiessen Polygons were used to determine which gages should logically contribute data to each sub-basin, and by how much (weighting factor). Sub-basin NAP, gage NAP, precipitation patterns for each event at each gage, and weighting factors were entered into HEC-HMS, and the estimated cumulative precipitation for each sub-basin was calculated using the following relationship:

$$PPTb = W * (NAPb/NAPg) * PPTg$$

NAPb = Estimated NAP of the sub-basin (isohyetal line average over sub-basin, **Table 5**)

NAPg = Estimated NAP at the gage (isohyetal line average at gage, **Table 4**)

PPTg = Measured cumulative precipitation at the gage (3-day event maximum)

W = Weighting factor (from Thiessen Polygons)

PPTb = Estimated cumulative precipitation for the sub-basin (3-day event maximum)

The rainfall pattern for each sub-basin was assumed to be that of the gage closest to the center of the sub-basin, since averaging several precipitation patterns would unrealistically flatten the main peak of the hyetograph. Precipitation data at gage RF123 for the January 1995 and February 1998 events was only available in units of inches per day. As this did not provide detailed information on the rainfall pattern, rainfall patterns at this gage were not used for this event.

Table 4 – Rain Gages in and around the Guadalupe Basin, Operated by SCVWD

Rain Gage ID	Gage Name	Gage Location	Index NAP at Gage (inches)
RF 42	Lexington Reservoir	Near Lexington Reservoir.	33
RF 108	West Yard	Santa Clara County West Yard Maintenance facility, off Doyle Road, near Lawrence Expressway.	16
RF1	Alamitos	Right bank of levee road between the Guadalupe River and Alamitos percolation pond.	18
RF123	Guadalupe Watershed	Eastern slope of Sierra Azul, approximately 1,200 ft. south east of Reynolds Road.	30
RF125	Vasona Pump Station	Approximately 1,000 feet northwest of Oka Lane, in the valve yard of the Vasona Pump Station.	18.5
RF128	Calero Reservoir	Approximately 1,000 ft. south of the Calero Ranch entrance, and 400 ft. west of McKean Road.	23
RF 131	San Jose	Near the City of San Jose, Office of Emergency Services, at the northwest corner of N. San Pedro St. and Mission St.	13.5
RF36	Johnson Ranch	On the Johnson Ranch property, approximately 1,000 feet north of Kennedy road, northwest of Fawndale road.	25
RF 37	Laguna Seca	On private property, 1/4 mile southeast of the intersection of Santa Teresa Blvd. and Bailey Rd., in south San Jose.	18
RF4	Almaden Watershed	Northwest of Alamitos Road approximately one half mile southwest of Hicks Road, at Twin Creeks resort	30
RF 44	Loma Prieta	On Loma Prieta Peak, adjacent to the compound entrance for KNTV television channel 11.	38
RF69	Mt. Umunhum	On the western slope of Mt. Umunhum, on the abandoned Almaden Air Force Base.	33
RF79	Rinconata	Above the sludge ponds at the Rinconada Water Treatment Plant, in Los Gatos.	27.35

Figure 4 – Rain Gage Locations in and around the Guadalupe Basin

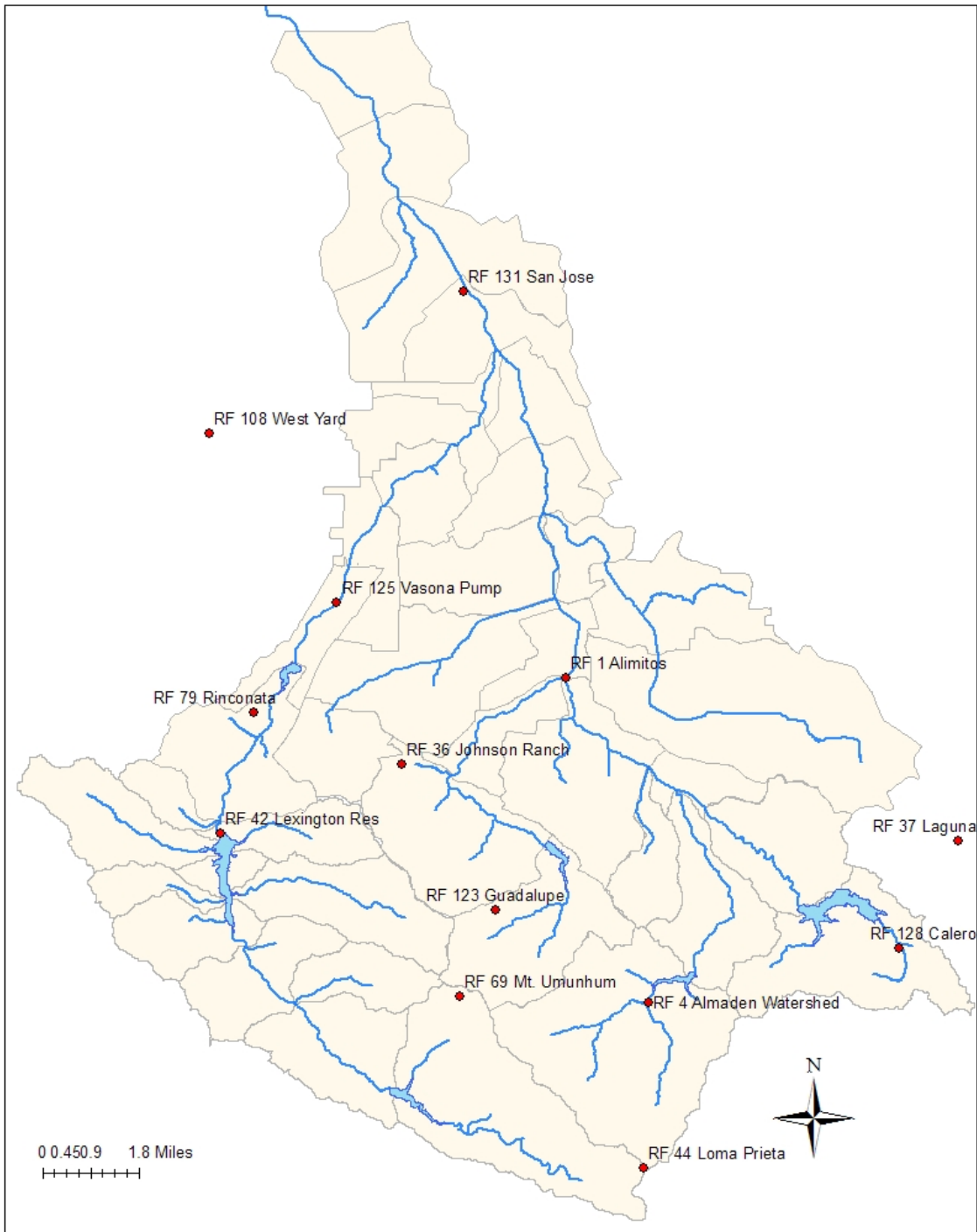


Table 5 – Estimated Normal Annual Precipitation (NAP) by Sub-Basin, (Isohyetal Line Average)

Sub-Basin	Precipitation (inches)
Above Ross	16
Below Almaden Res Basin	19
OI	13.9
OJ	13.9
Upper Ross	25
W Los Gatos 3	14.5
W Los Gatos 4	17
W N	13.4
W P	13.5
W Q	13.3
W R	13
W S	13
W1160	14
W1170	15.9
W1190	22.5
W1190A	22.5
W1220	16
W1250	15
W1260	15.5
W1270	19
W1300	15.5
W1350	21.4
W1400	21.3
W1480	24.1
W1540	24.6
W1580	27.5
W1620	21.5
W1630	27.5
W1640	34.2
W1650	28.3
W1660	27.5
W1690	24
W1700	34.8
W1730	27.5
W1740	34.1
W1780	32.5
W1800	27.8
W1850	28.3
W1900	33.6
W1950	32.9
W1960	26.7
W1970	27.2

Courtesy of the National Climatic Data Center

5.4 Transform Method

Both the “user specified S-graph” and the “Clark unit hydrograph transform method” were used to develop runoff hydrographs in the model. The Clark method was used in most of the predominantly urban sub-basins where runoff into the channel may be limited by interior drainage facilities such as culverts, storm drains, streets, etc. Culverts limit direct flow into Guadalupe River and its tributaries at numerous locations. Streets and interior areas act as detention basins during storm events. Culverts act as the control structures of these storage areas. The Clark unit hydrograph method requires the input of the time of concentration and a storage coefficient. The storage coefficient for individual sub-basins was adjusted to match recorded hydrographs. The HEC-HMS inputs for the Clark transform method are presented in **Table 6**. It should be noted that during some low frequency events the culvert capacities in select sub-basins may be exceeded causing direct runoff into the river. It was determined, however, that this would not be a significant source of error in the model.

The user specified S-graph method was used in the predominantly rural sub-basins as well as two of the very urban sub-basins where flow is limited by pumps (sub-basins WR and WS), and two sub-basins along Guadalupe River that fall in a transition zone between the urban and rural areas (sub-basins W1350 and W1400). The “Del Puerto” S-curve was used in the upper Guadalupe River sub-basins, and the “Walnut Creek Mountain Rural” S-curve was used in the Los Gatos sub-basins. These curves were originally developed by the USACE Los Angeles District for areas deemed similar to the areas within the study area given land use, topography, climate, and soil types. The “LA Valley Urban” S-curve was used for sub-basins WR, WS, W1350, and W1400 as this S-curve was shown during calibration to account for the type of urban runoff observed in these basins. The Del Puerto, Walnut Creek Mountain Rural, and LA Valley Urban S-curves are presented in **Appendix B**. Lag times for each sub-basin using the S-graph method are presented in **Table 7**.

The lag time for the sub-basins using the user specified S-graph method was calculated using the same relationship that was used in the 1977 analysis (see below). Manning’s n was estimated using values presented in the 1977 hydrologic report³. All relevant sub-basin parameters are presented in **Table 8**.

$$\text{Lag (hours)} = 24 * n * ((L * Lca) / (S^{0.5}))^{0.38}$$

Lag = Time from the start of runoff to the point where 50% of the ultimate discharge reaches the downstream end of the basin.

n = Sub-basin average roughness coefficient (Manning’s n), dimensionless.

L = Length of primary watercourse, miles.

Lca = Distance from the centroid of the basin to the downstream end of the basin, measured along the primary watercourse, miles.

S = Average slope of primary watercourse, feet per mile.

Time of concentration for the sub-basins where the Clark transform method are used was calculated using the equations and assumptions presented below¹²:

$$T_c = T_{sheet} + T_{shallow} + T_{channel}$$

T_c = Time of concentration.

T_{sheet} = Sum of travel time in sheet flow segments over the watershed land surface.

$T_{shallow}$ = Sum of travel time in shallow flow segments, down streets, in gutters, or in shallow rills and rivulets.

$T_{channel}$ = Sum of travel time in channel segments through remainder of sub-basin.

$$T_{sheet} = (0.007 * (NL_{sheet})^{0.8}) / ((P_2)^{0.5} * S^{0.4})$$

N = Overland flow roughness coefficient, approximately 0.15.

L_{sheet} = Flow length, assumed to be approximately 300 feet within the study area.

P_2 = 2-year, 24-hour rainfall depth, inches.

S = slope of hydraulic grade line, approximated using average slope of primary watercourse, feet per feet.

$$T_{shallow} = L_{shallow} / V_{shallow}$$

Or

$$T_{shallow} = .75 * L_{shallow} / V1 + .25 * L_{shallow} / V2$$

$V1$ = 1.5 (s=0.002) or 1.8 (s=0.006)

$V2$ = 2.0 ft/s (culvert flow)

The second $T_{shallow}$ equation was used in the more urbanized sub-basins located in and around downtown San Jose where culvert and storm drain flow is a larger component of the shallow segment flow.

$L_{shallow}$ = Flow length for shallow flow segments. Estimated from sub-basin map.

$V_{shallow} = 16.1345 * (S)^{0.5}$ (unpaved, assumed for rural sub-basins)

$20.3282 * (S)^{0.5}$ (paved, assumed for urban sub-basins)

$$T_{channel} = L_{channel} / V_{channel}$$

$L_{channel}$ = Flow length for remaining channel length of flow through sub-basin.

Assumed to be the length of the channel minus $L_{shallow}$, except in urban downstream sub-basins where culvert and storm drain flow occur and distance assumptions vary.

$V_{channel}$ = Calculated using Manning's Equation, output from HEC-RAS model.

The HEC-HMS Technical Reference Manual¹² provides some guidance as to how time of concentration and lag time are related. In general, lag time should be a percentage of the time of concentration. It follows that the final time of concentration numbers ($T_c = T_{sheet} + T_{shallow} + T_{channel}$) were adjusted by a factor of 0.8 in accordance with this guidance to ensure that the time of concentration and lag are in realistic proportions. This may also account for potential assumption errors made in the time of concentration calculations. All parameter assumptions used in the time of concentration calculations for the Clark transform method are presented in **Table 9**.

Table 6 – Clark Unit Hydrograph Transform Method HEC-HMS Inputs

Subbasin	Time of Concentration (HR)	Storage Coefficient (HR)
Above Ross	0.74	1.08
OI	2.12	1.14
OJ	1.78	1.185
Upper Ross	0.44	0.885
W1160	1.90	1.605
W1170	1.72	2.49
W1190	0.78	1.08
W1190A	1.48	1.515
W1220	2.29	1.41
W1250	1.34	0.9
W1260	2.27	1.8
W1270	1.14	1.68
W1300	3.73	3
W Los Gatos 3	1.14	1.02
W Los Gatos 4	1.94	1.02
W N	2.06	1.47
W P	2.13	1.425
W Q	3.19	1.8

Table 7 – User Specified S-Graph Transform Method HEC-HMS Inputs

Subbasin	S-Graph Table	Lag Time (HR)
Below Almaden Res Basin	Del Puerto	0.69
W1350	LA Valley Urban	2.53
W1400	LA Valley Urban	2.43
W1480	Del Puerto	1.41
W1540	Del Puerto	1.48
W1580	Walnut Cr. Mtn Rural	1.48
W1620	Del Puerto	2.59
W1630	Walnut Cr. Mtn Rural	1.79
W1640	Walnut Cr. Mtn Rural	2.92
W1650	Walnut Cr. Mtn Rural	1.17
W1660	Walnut Cr. Mtn Rural	3.13
W1690	Del Puerto	2.63
W1700	Walnut Cr. Mtn Rural	1.86
W1730	Del Puerto	1.48
W1740	Walnut Cr. Mtn Rural	2.23
W1780	Walnut Cr. Mtn Rural	1.74
W1800	Walnut Cr. Mtn Rural	1.81
W1850	Del Puerto	2.49
W1900	Walnut Cr. Mtn Rural	2.19
W1950	Walnut Cr. Mtn Rural	3.84
W1960	Del Puerto	1.89
W1970	Del Puerto	1.84
W R	LA Valley Urban	0.98
W S	LA Valley Urban	1.07

Table 8 – Sub-Basin Parameters

Sub-Basin	Description	Drainage Area	L	L _{ca}	S	Basin n	NAP
		(sq mi)	(mi)	(mi)	(ft/mi)	-	(in)
WN	Immediately upstream of Los Gatos Creek on Guadalupe River	2.9	3.3	1.5	22.0	0.04	13.4
WOI	Upstream Guadalupe River on Los Gatso Creek	2.9	2.5	1.0	22.0	0.04	13.9
WOJ	Upstream Guadalupe River on Los Gatos Creek	1.1	2.8	1.0	22.0	0.04	13.9
Los Gatos 4	Los Gatos 4	2.7	2.0	1.0	25.0	0.04	17.0
Los Gatos 3	Los Gatos 3	2.5	2.0	1.0	25.0	0.04	14.5
WP	Downstream from Los Gatos Creek	4.0	2.9	1.6	23.0	0.04	13.5
WQ	Downstream from Los Gatos Creek	7.3	3.5	2.0	15.0	0.04	13.3
WR	Downstream from Los Gatos Creek	5.1	2.5	1.6	14.0	0.04	13.0
WS	At outlet	4.3	3.1	1.2	8.0	0.04	13.0
W1160	Upstream of Los Gatos Creek on Guadalupe River	1.9	3.6	1.8	22.7	0.04	14.0
W1170	Upstream of Los Gatos Creek on Guadalupe River	1.8	6.0	3.0	17.4	0.04	15.9
W1190	Los Gatos Creek Downstream from Vasona Reservoir	2.4	2.2	1.5	50.0	0.04	22.5
W1190A	Los Gatos Creek Upstream from Vasona Reservoir	5.0	3.5	1.8	30.0	0.04	22.5
W1220	Upstream of Los Gatos Creek on Guadalupe River	4.1	3.2	1.6	30.0	0.04	16.0
W1250	Lower Canoas Creek Basin	1.7	2.0	1.0	12.1	0.03	15.0
W1260	Canoas Creek Basin	4.8	3.7	1.9	8.9	0.04	15.5
W1270	Ross Creek Basin	7.0	4.7	2.4	53.4	0.04	19.0
Upper Ross	Upper Ross	2.9	2.0	1.0	53.0	0.04	25.0
Above Ross	Above Ross	2.1	2.5	1.3	45.0	0.04	16.0
W1290	Canoas Creek Basin	1.4	2.6	1.3	18.0	0.05	14.6
W1300	Upper Canoas Creek Basin	13.3	7.9	3.9	10.6	0.04	15.5
W1350	Upstream Almaden Lake Guadalupe Creek	2.2	4.3	2.1	63.5	0.10	21.4
W1400	Upstream Almaden Lake Alamos Creek	6.5	4.1	2.1	65.9	0.10	21.3
Below Almaden	Below Almaden Res	1.7	2.3	1.2	40.0	0.04	19.0
W1480	Alamos Creek	2.3	3.6	1.8	232.8	0.08	24.1
W1540	Downstream from Almaden Reservoir	4.3	5.1	2.6	55.9	0.05	24.6
W1580	Trout Creek Downstream from Lexington Reservoir	1.2	2.5	1.3	566.6	0.10	27.5
W1620	Downstream from Calero Reservoir	4.7	4.3	2.2	33.1	0.09	21.5
W1630	Lexington Reservoir Basin East	2.6	4.0	2.0	303.4	0.10	27.5
W1640	Lexington Reservoir Basin West	4.1	5.0	2.5	319.2	0.14	34.2
W1650	Lexington Reservoir Lake	0.9	1.5	0.7	306.5	0.14	28.3
W1660	Lexington Reservoir Basin East	4.9	5.8	2.9	402.0	0.14	27.5
W1690	Upstream from Calero Reservoir	6.9	5.6	2.8	84.2	0.09	24.0
W1700	Lexington Reservoir Basin West	1.4	3.1	1.6	517.3	0.14	34.8
W1730	Upstream from Guadalupe Reservoir	4.3	3.1	1.6	782.3	0.12	27.5
W1740	Lexington Reservoir Basin West	2.9	3.6	1.8	358.4	0.14	34.1
W1780	Upstream from Lexington Reservoir	3.6	2.9	1.4	556.2	0.14	32.5
W1800	Upstream from Lexington Reservoir	2.4	3.3	1.7	801.6	0.14	27.8
W1850	Upstream from Almaden Reservoir	11.9	5.1	2.5	439.1	0.13	28.3
W1900	Immediately downstream from Lake Elzman	4.5	3.7	1.8	440.0	0.14	33.6
W1950	Upstream from Lake Elzman	9.7	7.0	3.5	303.5	0.14	32.9
W1960	Downstream from Guadalupe Reservoir	6.7	4.1	2.1	532.2	0.12	26.6
W1970	Upstream from Guadalupe Reservoir	1.6	3.0	1.5	227.6	0.12	27.2

Table 9 – Parameters used in Time of Concentration Calculations

Subbasin	Time of Concentration (hours)	P ₂ (in)	Land Use	Lshallow (mi)	Lchannel (mi)	Vchannel (ft/s)	Tsheet (hours)	Tshallow (hours)	Tchannel (hours)
Above Ross	0.74	1.82	Urban	0.66	1.84	7.00	0.01	0.52	0.39
OI	2.12	1.58	Urban	2.46	2.22	6.0	0.02	2.09	0.54
OJ	1.78	1.58	Urban	1.89	2.42	6.0	0.02	1.61	0.59
Upper Ross	0.44	2.84	Urban	0.38	1.62	9.0	0.01	0.27	0.26
W1160	1.90	1.59	Urban	1.99	3.14	7.0	0.02	1.69	0.66
W1170	1.72	1.80	Urban	1.14	5.52	7.0	0.02	0.98	1.16
W1190	0.78	2.56	Urban	0.47	1.73	4.2	0.01	0.35	0.61
W1190A	1.48	2.55	Semi-Rural	0.76	2.74	3.6	0.01	0.73	1.11
W1220	2.29	1.82	Urban	2.84	2.82	7.0	0.01	2.26	0.59
W1250	1.34	1.70	Urban	0.76	1.24	3.5	0.02	1.14	0.52
W1260	2.27	1.76	Urban	0.95	2.75	3.5	0.02	1.67	1.15
W1270	1.14	2.16	Urban	1.04	3.66	8.0	0.01	0.75	0.67
W1300	3.73	1.76	Urban	1.14	6.72	3.5	0.02	1.83	2.81
W Los Gatos 3	1.14	1.65	Urban	1.42	0.86	6.0	0.02	1.21	0.21
W Los Gatos 4	1.94	1.93	Urban	2.46	1.34	6.0	0.02	2.09	0.33
W N	2.06	1.52	Urban	2.31	2.83	7.0	0.02	1.96	0.59
W P	2.13	1.53	Urban	2.65	2.24	8.5	0.02	2.25	0.39
W Q	3.19	1.51	Urban	2.94	2.55	4.5	0.02	2.69	0.83

5.5 Baseflow

Baseflow represents the remaining runoff from the preceding storm plus subsurface flow from the current storm. Acceptable ranges for baseflow utilized in the calibration process were adopted from the 1977 hydrologic study³, and finalized during the calibration process. In general, the 1977 study suggested appropriate baseflow rates to be between 2-22 cubic feet per second per square mile (csm). Constant monthly baseflow rates were applied to the downstream urban areas (**Table 10**). As this study did not intend to analyze seasonal variations, one average constant monthly baseflow rate was applied over the entire year. A recession method was used in the rural upper basin areas to better match the receding limb of the hydrographs. The initial discharge was assumed to be 1.5 csm, the recession constant was set at 0.45, and the ratio to peak was selected during calibration (**Table 11**).

Table 10 - Constant Monthly Baseflow Rate, by Sub-Basin

Subbasin	Constant Monthly Rate (cfs)
Above Ross	2
OI	5
OJ	2
Upper Ross	2
W1160	6
W1170	6
W1190	11
W1190A	10
W1220	5
W1250	3
W1260	5
W1270	5
W1300	15
W1400	5
W Los Gatos 3	2
W Los Gatos 4	2
W N	9
W P	11
W Q	22
W R	15
W S	11

Table 11 – Baseflow Ratio to Peak by Sub-Basin

Subbasin	Ratio to Peak		
	Jan-95	Mar-95	Feb-98
Below Almaden Res Basin	0.1	0.1	0.1
W1350	0.3	0.3	0.3
W1480	0.1	0.1	0.1
W1540	0.1	0.1	0.1
W1580	0.3	0.3	0.3
W1620	0.1	0.1	0.1
W1630	0.3	0.16	0.3
W1640	0.3	0.16	0.3
W1650	0.3	0.16	0.3
W1660	0.3	0.16	0.3
W1690	0.3	0.3	0.3
W1700	0.3	0.16	0.3
W1730	0.3	0.4	0.3
W1740	0.3	0.16	0.3
W1780	0.3	0.16	0.3
W1800	0.3	0.16	0.3
W1850	0.3	0.4	0.3
W1900	0.3	0.16	0.3
W1950	0.3	0.16	0.3
W1960	0.25	0.25	0.25
W1970	0.3	0.4	0.3

5.6 Loss Rates and Impervious Areas

Data from the SCVWD and 2005 aerial photographs (**Figure 5**) were used to estimate the percentage of each sub-basin that is impervious to rainfall infiltration. Impervious area percentages were calculated by multiplying the estimated percent impervious area using SCVWD GIS maps by a factor to account for the portion of the estimated impervious area that will not contribute to direct runoff. This factor was estimated to be between 0.7 and 0.8, and was selected during calibration. An impervious area reduction factor of 0.8 was applied to the Guadalupe sub-basins and a reduction factor of 0.73 was applied to the Los Gatos sub-basins.

Soil data from the United States Department of Agriculture Natural Resources Conservation Services Soil Survey¹³ were used to create a soil map of the basin to estimate appropriate ranges of loss rates for each sub-basin (**Figure 6**). Soil types within the basin were classified based on infiltration capacity and all were considered either Type B (higher infiltration rate) or Type D (lowest infiltration rate). Soil within the more urban areas of the basin are generally classified as Xerotents characterized by sandy loam (Type B). Most of the upper basin is characterized as Montara-Henneke, Vallejos-Parish-Los Gatos-Graviota, or Maymen-Lompico-Felton-Ben Lomond with higher amounts of clay (Type D). Final loss rates selected during calibration were limited to predefined ranges as deemed appropriate. Soil type assumptions for each sub-basin, and final loss rates and percent impervious area for each calibration event are presented in **Table 12**. A reasonable initial loss rate of 0.4 inches per hour was selected for all of the calibration events, which all had an estimated recurrence frequency of less than 28 years.

Figure 5 – 2005 Aerial Photograph for Land Use Assumptions, Guadalupe Basin

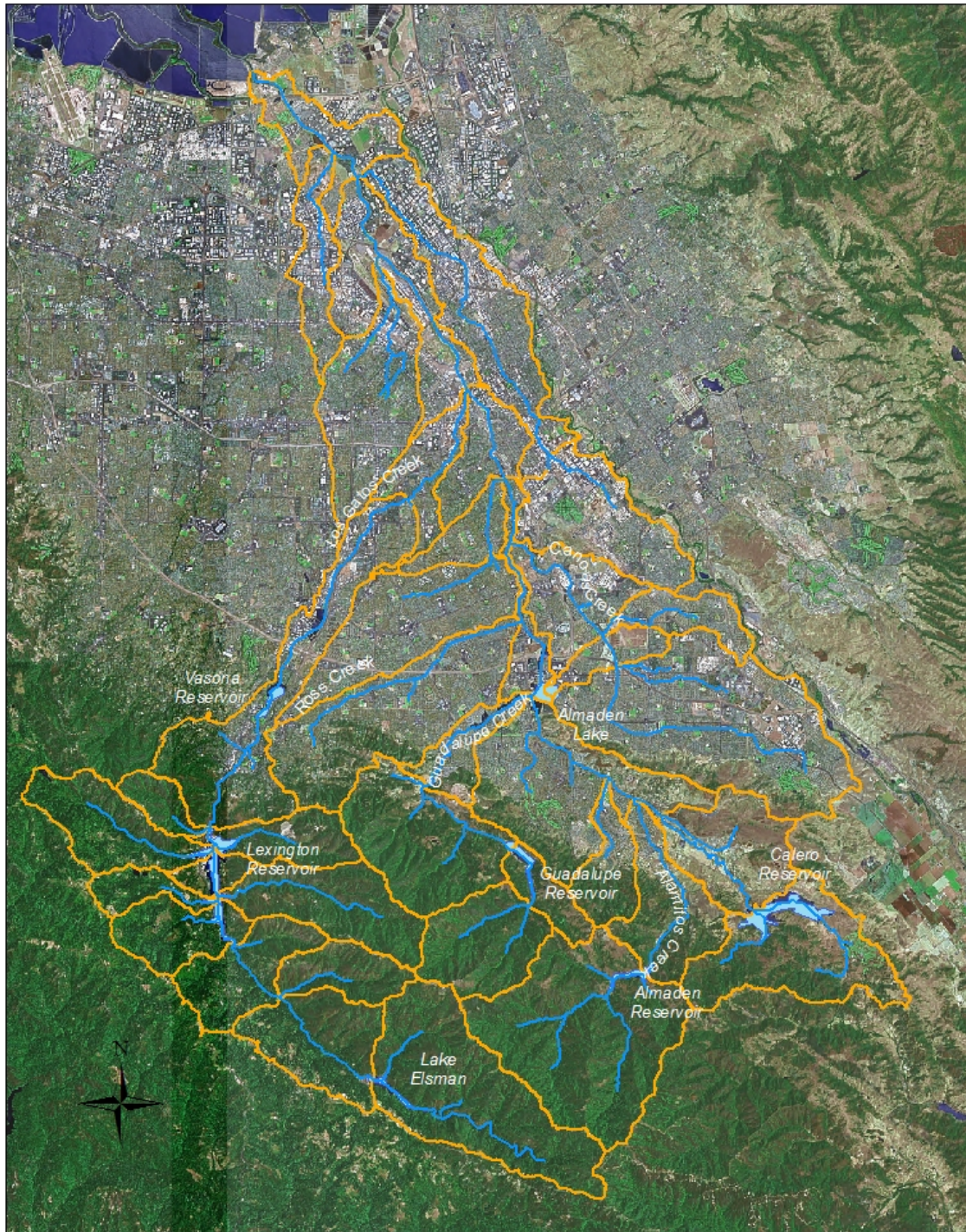


Figure 6 – Soil Map, Guadalupe Basin



Courtesy of the U.S. Department of Agriculture Natural Resources Conservation Services Soil Survey¹³

Table 12 – Final Calibration Soil Type, Loss Rate, and Impervious Area Assumptions by Sub-Basin

Sub-Basin	Soil Type	Constant Loss Rates			% Impervious
		Jan-95	Mar-95	Feb-98	
Above Ross	B	0.3	0.35	0.3	60
Below Almaden Res Basin	10%B 90%D	0.1	0.1	0.1	10
OI	B	0.35	0.35	0.35	55
OJ	B	0.35	0.35	0.35	59
Upper Ross	30%B 70%D	0.22	0.22	0.22	40
W Los Gatos 3	B	0.35	0.35	0.35	51
W Los Gatos 4	B	0.35	0.35	0.35	55
W N	B	0.3	0.3	0.3	60
W P	B	0.3	0.3	0.3	60
W Q	B	0.3	0.3	0.3	64
W R	B	0.3	0.3	0.3	72
W S	B	0.3	0.5	0.3	64
W1160	B	0.3	0.3	0.3	48
W1170	B	0.3	0.3	0.3	48
W1190	B	0.35	0.35	0.35	59
W1190A	55%B 45%D	0.25	0.25	0.25	40
W1220	B	0.3	0.3	0.3	52
W1250	B	0.35	0.3	0.35	48
W1260	B	0.35	0.3	0.35	52
W1270	80%B 20%D	0.28	0.28	0.28	52
W1300	80%B 20%D	0.28	0.28	0.28	48
W1350	70%B 30%D	0.22	0.22	0.22	24
W1400	45%B 55%D	0.16	0.22	0.16	18
W1480	15%B 85%D	0.1	0.1	0.1	15
W1540	10%B 90%D	0.1	0.1	0.1	15
W1580	D	0.22	0.15	0.15	0
W1620	30%B 70%D	0.15	0.15	0.15	5
W1630	D	0.15	0.3	0.2	0
W1640	D	0.15	0.3	0.2	0
W1650	D	0.15	0.3	0.2	0
W1660	D	0.15	0.3	0.2	0
W1690	D	0.11	0.11	0.11	0
W1700	D	0.15	0.2	0.2	0
W1730	D	0.18	0.18	0.18	0
W1740	D	0.15	0.3	0.2	0
W1780	D	0.15	0.3	0.2	0
W1800	D	0.15	0.3	0.2	0
W1850	D	0.15	0.15	0.15	0
W1900	D	0.15	0.3	0.2	0
W1950	D	0.15	0.3	0.2	0
W1960	D	0.24	0.24	0.24	16
W1970	D	0.18	0.18	0.18	0

5.7 Channel Routing

Two routing methods, Modified Puls and Muskingum-Cunge were used to describe flow routing in the model reaches. The channel size and characteristics change drastically from the upper basin to the lower basin areas. **Figures 7 – 10** display pictures of typical channels throughout the basin. In general, the Modified Puls method was used in the more urbanized areas where HEC-RAS model data was available to describe storage-discharge relationships. As a result, most of the reaches in the main stem of Guadalupe River are characterized using Modified Puls. The storage-discharge functions from the HEC-RAS model are presented in **Appendix C**. Routing in the remaining reaches where limited cross-section data was available was characterized using the Muskingum-Cunge method (**Figure 11**). The Muskingum-Cunge method requires assumptions of basic channel geometry (**Table 13**).

Figure 7 – Typical Channel, Upstream of Ross Creek



Figure 8 – Typical Channel, Guadalupe River Between Ross Creek and Canoas Creek



Figure 9 – Typical Channel, Guadalupe River Downstream of Willow Road, Upstream of Highway 280



Figure 10 – Guadalupe River Downtown Project, Looking Upstream Toward Park Avenue Bridge



Table 13 – Muskingum-Cunge Routing Parameters

Reach	Length (ft)	Slope (ft/ft)	Manning's n	Invert (ft)	Shape	Width (ft)	Side Slope (xH:1V)
R490	17000	0.006	0.07	280	Trapezoid	20	1
R530	3600	0.078	0.08	316	Trapezoid	25	2
R550	12000	0.0067	0.07	425	Trapezoid	13	1
R580	17500	0.0043	0.07	375	Trapezoid	20	2
R700	17000	0.007	0.07	360	Trapezoid	20	2
R760	2300	0.009	0.06	675	Trapezoid	15	1
R800	21000	0.01	0.05	450	Trapezoid	20	1
R860	6600	0.009	0.05	770	Trapezoid	15	1
R910	13200	0.009	0.05	920	Trapezoid	15	1
R LG 4	7500	0.002	0.06	180	Trapezoid	23	1
R LG 5	11000	0.0028	0.06	257	Trapezoid	50	2

Figure 11 – Routing Reaches Characterized by Muskingum-Cunge Routing Parameters



6. MODEL CALIBRATION

The model was calibrated to three events in January 1995, March 1995, and February 1998 to compute hydrographs that matched observed hydrographs in magnitude and timing at several locations. Details on parameter selection are presented in **Section 5**.

Stream gage data was available at twelve locations within the watershed and was used to calibrate the model. A few potential timing problems at select gages and for select events were taken into account when matching up peaks. Overall, the gage data appeared to be reasonable and was very useful in the calibration process. Stream gage locations are presented in **Table 14** and **Figure 12**.

The gage of most interest for this study was stream gage SF23 downstream of the confluence of Guadalupe River and Ross Creek (J Gage 23B d/s Ross Cr), which has a drainage area of approximately 63 square miles. This gage is useful for overall model calibration and is located within the boundaries of a proposed flood control project. A detailed calibration of the hydrograph peak was performed at this location as the gage data is generally reliable, and three of the five reservoirs (Almaden, Calero, and Guadalupe) as well as Alamitos, Guadalupe, and Ross Creeks all feed into this location. Initial and constant loss rates and initial reservoir storage volumes were the most important parameters used in calibrating to this gage.

Gages SF13, SF16, SF17, and SF67 are roughly below Calero, Almaden, Guadalupe, and Lexington Reservoirs, respectively. Reservoir storage volumes and loss rates were selected by calibrating modeled reservoir outflows to data from these gages during the calibration process. Matching up the hydrographs below the reservoirs was difficult, however, as the exact operating schemes at the reservoirs were unknown. As a result, the calibration focused on matching up peak times and water volumes. In general, the amount of water released from the reservoirs during the calibration events was very low (less than 100 cfs in most cases) and therefore likely had only a small impact on model results.

USGS gage data was also available at Guadalupe River downstream of Los Gatos (Gage # 11169000). Several channel locations upstream of this gage are known to overtop during high flow events, most notably at Ross Creek upstream of Guadalupe River, Guadalupe River downstream of Canoas Creek, and Guadalupe River near Highway 280. It is likely that channel overtopping occurred at these locations during the calibration events. As a result, this gage was not used as a main calibration point, since discharge values for the peaks in the gage data will always be lower than the results from the model as a result of the breakaway flow upstream. The HEC-HMS model only approximates full built-out flow where the entire Upper Guadalupe project has been constructed. The model assumes that all of the flow that enters the channel stays in the channel.

Though the SF23 gage downstream of Ross Creek was the main calibration point, an effort was made during calibration to match model results at all locations with reasonable gage data. In some cases, it was necessary to make a compromise since one set of parameters, loss rates for example, may have produced matching results at one gage, and unfavorable results at another. As discussed in **Section 5.6**, model parameters, in particular the constant loss rates, were highly constrained so that they were reasonable given observed soil types, land use, topography, etc. The calculated hydrographs at gage SF23 for the January 1995 and February 1998 events very accurately simulate both the recorded hydrograph shape and peak discharge. The calculated hydrograph at SF23 for

the March 1995 event captures the essential shape of the measured hydrograph but the peak recorded discharge is much lower than what was calculated by the model. Hydrographs upstream of gage SF23 were compared to gage data at Alamitos Creek at Randoff Creek (J gage Alamitos Creek, SF70), Guadalupe Creek at Shannon Creek (J Gage Guadalupe Creek, SF43), and downstream of both Almaden and Guadalupe Reservoirs (SF16 and SF17, respectively). For the March 1995 event, the contributing hydrograph from Alamitos Creek matched up very accurately with the gage data. The modeled hydrograph on Guadalupe Creek also matched up well with the measured data, except around the main peak, where the measured peak spiked considerably, perhaps due to a manually imposed discharge from Guadalupe Reservoir not governed by the storage-discharge curve. As a result, these simulated hydrographs from the final March 1995 calibration model represents a best fit given the parameter constraints. It is possible that some of the gage data could be incorrect or that the manual reservoir operation differed from the SCVWD rule curves, which would lead to this type of inconsistency.

Simulated and measured peaks for the January 1995, March 1995, and February 1998 events are compared in **Table 16**. A comparison of gage and simulated hydrographs at this location for the 1995 and 1998 events are presented in **Figures 13-15**. Estimated return periods of peak flows for each of the calibration events based on both 1977 and 2009 frequency curves are presented in **Table 15**. Overall, the measured and simulated hydrographs match well in shape, peak magnitude, and volume with the exception of the March 1995 hydrograph, as previously described.

Table 14 – Stream Gages in Guadalupe Watershed

Stream Gage	Official Name	Drainage Area (mi2)	Period of Record	Operating Agency
SF51	Ross Creek at Cherry Ave, us of Guadalupe River	9.9	1956 - present	SCVWD
SF73	Canoas Creek us of Guadalupe River	19.8	1977- present	SCVWD
SF50	Los Gatos Creek us of Guadalupe River	52	1955- present	SCVWD
SF16	Downstream from Almaden Reservoir	11.9	1958- present	SCVWD
SF13	Downstream from Calero Reservoir	6.9	1958- present	SCVWD
SF17	Downstream from Guadalupe Reservoir	5.9	1959- present	SCVWD
SF67	Downstream from Lexington Reservoir	34.6	1971- present	SCVWD
SF70	Alamitos at Randoff Creek	30.1	1973- present	SCVWD
SF23	Guadalupe River at Almaden Expressway	62.9	1939- present	SCVWD
SF43	Guadalupe Creek at Shannon Creek	12.7	1960- present	SCVWD
11169000	Guadalupe River at I-280, San Jose Gage	145.6	10/1/29-4/30/03	USGS
11169025	Guadalupe River at US-101	170.0	5/1/02- present	USGS

Figure 12 – Stream Gage Locations, Guadalupe Watershed



Table 15 – Estimated Return Periods for Calibration Events at Gage SF23

	Measured Peak Flow SF23 (cfs)	Return Period (years) 1977 Frequency Curve (SF23)	Return Period (years) 2009 Frequency Curve
January 1995	8,470	45	35
March 1995	5,590	14	10
February 1998	6,780	22	20

Table 16 – Comparison of Simulated and Measured Peaks, SF-23 Gage

	January 1995	March 1995	February 1998
Simulated Peak Flow (cfs)	8,457	7,321	6,226
Measured Peak Flow (cfs)	8,470	5,590	6,780
Difference (cfs)	13	1,730	554

Figure 13 - Hydrograph Comparison, Guadalupe River Downstream of Ross Creek, January 1995

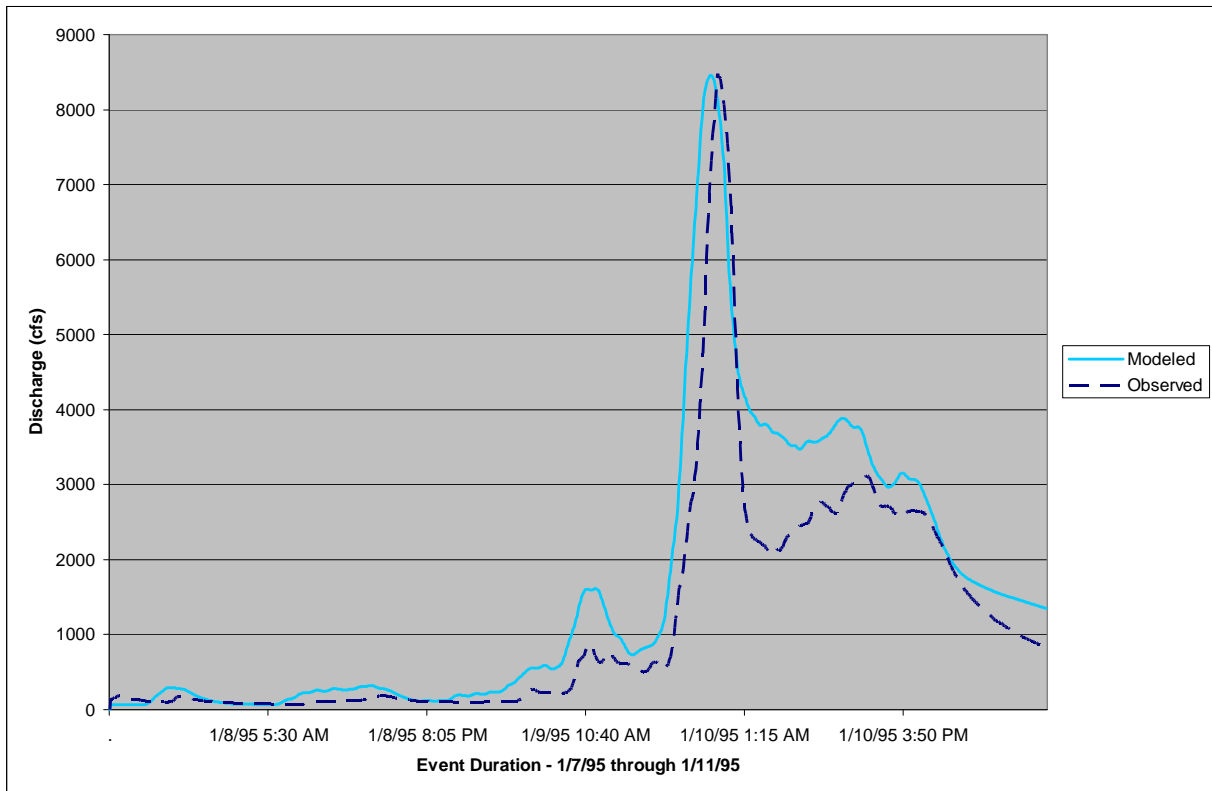


Figure 14 - Hydrograph Comparison, Guadalupe River Downstream of Ross Creek, March 1995

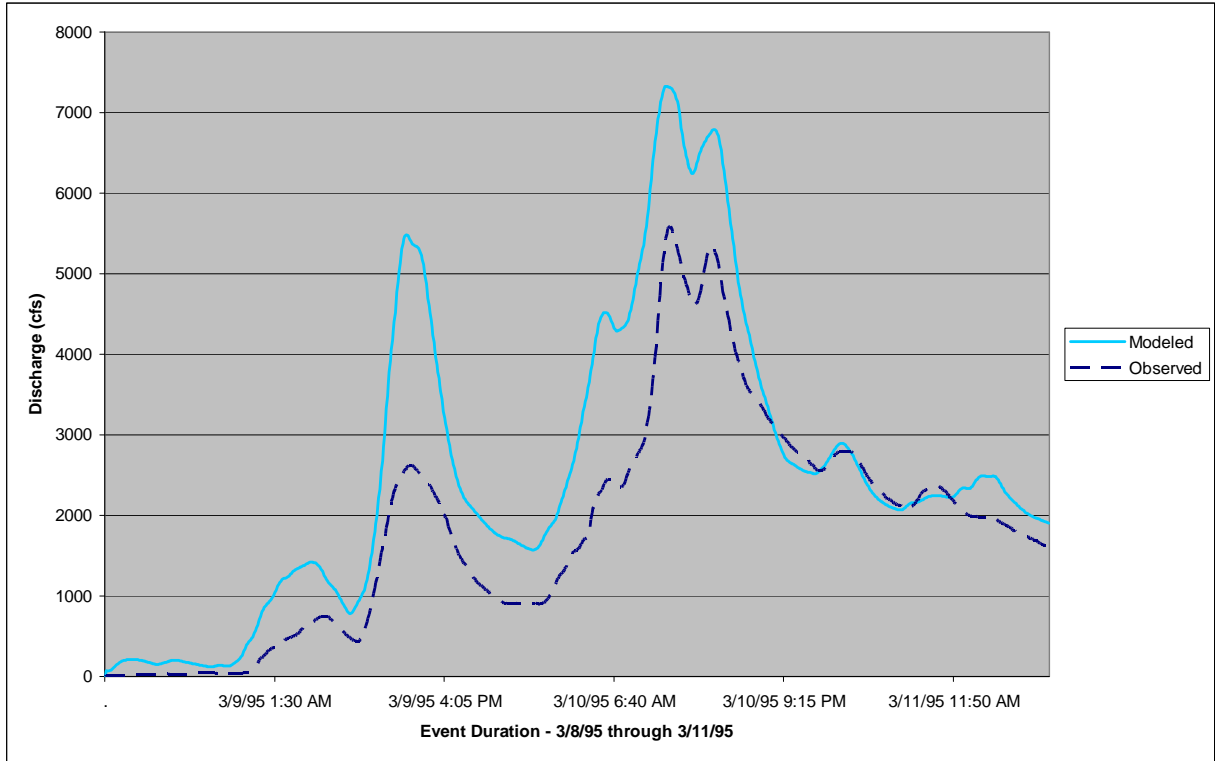
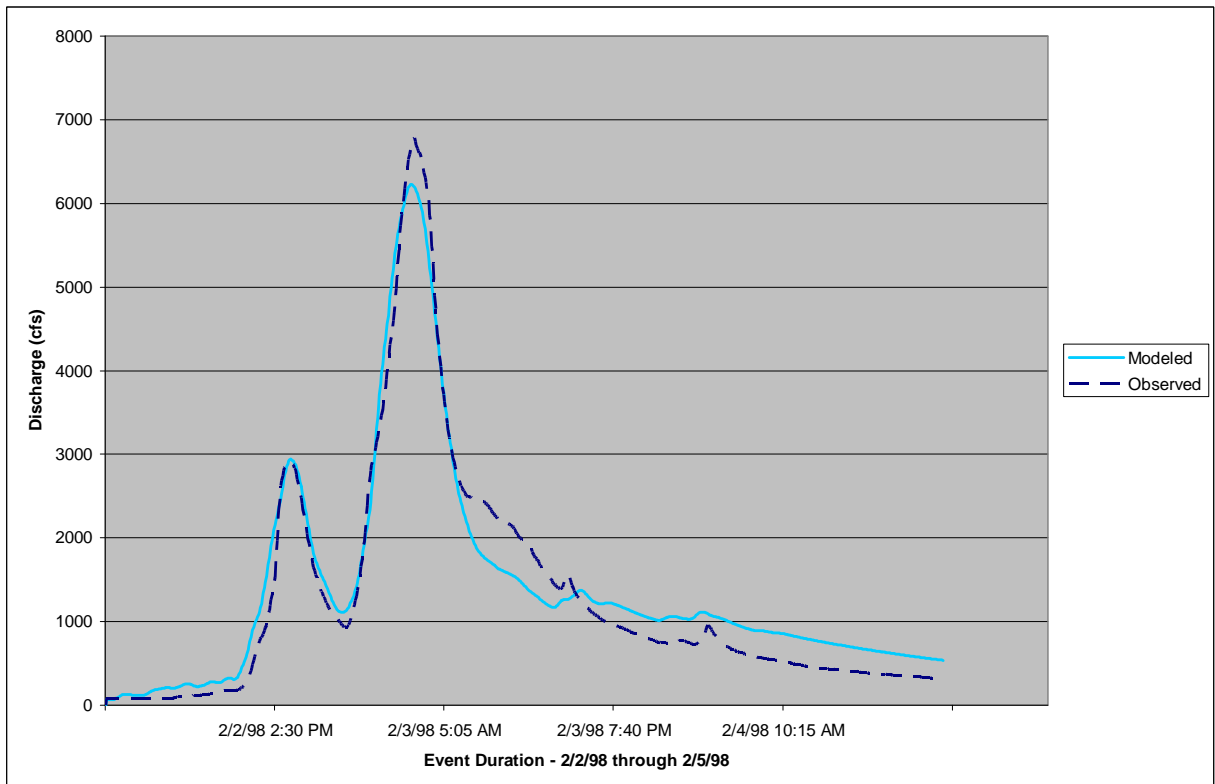


Figure 15 - Hydrograph Comparison, Guadalupe River Downstream of Ross Creek, February 1998



7. SYNTHETIC STORM SERIES

Synthetic storm series simulations were completed for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5, and 0.2-percent chance exceedance events. Initial loss was changed over the range of events as it is assumed that antecedent moisture concentration would vary (**Table 17**). Model parameters for constant loss (**Table 18**), percent impervious (**Table 18**), transform (**Tables 6 and 7**), baseflow (**Tables 10 and 19**), and routing (**Table 13**) were configured as they were for the calibration events.

The initial reservoir storage for Guadalupe, Almaden, Calero, and Vasona Reservoirs was based on an incidental storage analysis performed by SCVWD in 1976 for the 1-percent chance exceedance event. Lexington Reservoir is the largest reservoir in the system and the peak discharge from Los Gatos Creek and the hydrograph timing at the Guadalupe River and Los Gatos Creek confluence is very sensitive to its initial storage, particularly for the lower frequency events. Recorded reservoir storage values at Lexington Reservoir 5 days before peak annual events in which the discharge at the downtown San Jose gage were at least 1,000 cfs were requested for water years 1960 through 2003. The initial storage for the synthetic storm series events at Lexington Reservoir was based on the median storage value of this recorded data, calculated to be approximately 16,000 acre-feet. The initial reservoir storage values for the synthetic storm series events are presented in **Table 20**.

Table 17 – Initial Loss Rates for Synthetic Storm Series

% Chance Exceedance	Initial Loss
50	0.48
20	0.40
10	0.40
4	0.35
2	0.28
1	0.22
0.5	0.18
0.2	0.14

Table 18 – Constant Loss Rate for Synthetic Storm Series

Sub-Basin	Constant Loss Rate	% Impervious
Above Ross	0.3	60
Below Almaden Res Basin	0.1	10.00
OI	0.35	55.00
OJ	0.35	59.00
Upper Ross	0.22	40.00
W Los Gatos 3	0.35	51.00
W Los Gatos 4	0.35	55.00
W N	0.3	60.00
W P	0.3	60.00
W Q	0.3	64.00
W R	0.3	72.00
W S	0.3	64.00
W1160	0.3	48.00
W1170	0.3	48.00
W1190	0.35	59.00
W1190A	0.25	40.00
W1220	0.3	52.00
W1250	0.35	48.00
W1260	0.35	52.00
W1270	0.28	52.00
W1300	0.28	48.00
W1350	0.22	24.00
W1400	0.16	18.00
W1480	0.1	15.00
W1540	0.1	15.00
W1580	0.15	0.00
W1620	0.15	5.00
W1630	0.2	0.00
W1640	0.2	0.00
W1650	0.2	0.00
W1660	0.2	0.00
W1690	0.11	0.00
W1700	0.2	0.00
W1730	0.18	0.00
W1740	0.2	0.00
W1780	0.2	0.00
W1800	0.2	0.00
W1850	0.15	0.00
W1900	0.2	0.00
W1950	0.2	0.00
W1960	0.24	16.00
W1970	0.18	0.00

Table 19 – Baseflow Recession Parameters for Synthetic Storm Series

Subbasin	Ratio to Peak
Below Almaden Res Basin	0.1
W1350	0.3
W1480	0.1
W1540	0.1
W1580	0.3
W1620	0.1
W1630	0.3
W1640	0.3
W1650	0.3
W1660	0.3
W1690	0.3
W1700	0.3
W1730	0.3
W1740	0.3
W1780	0.3
W1800	0.3
W1850	0.3
W1900	0.3
W1950	0.3
W1960	0.25
W1970	0.3

Table 20 – Initial Reservoir Storage for Synthetic Storm Series (acre-feet)

	Almaden	Calero	Guadalupe	Lexington	Vasona
Full Capacity	1,600	9,900	3,400	19,000	500
Initial Storage	800	9,306	2,890	16,000	350

7.1 Meteorology

The meteorology was developed in a way that takes into account the large difference in NAP throughout the basin. The HEC-HMS models created for calibration purposes use historic gage data collected during real storms to estimate rainfall hyetographs throughout the basin. To simulate hypothetical statistical rain events (e.g. an event with a 1-percent chance of occurrence in any given year), synthetic precipitation patterns were created in which hypothetical rainfall is produced based on a statistical analysis of historic gage data. Unique 72-hour rainfall patterns were created for each of the 42 sub-basins to take into account estimated statistical cumulative precipitation for the 50-, 20-, 10-, 4-, 2-, 1-, 0.5, and 0.2-percent chance exceedance events. Statistical rainfall data from the California Department of Water Resources Rainfall Depth-Duration Frequency Analysis for California Rain Gages¹⁴ was used to estimate statistical cumulative precipitation. Rainfall patterns and Depth Area Reduction Factors (DARFs) were adapted from guidance presented in the Hydrometeorological Report Number 59 Probable Maximum Precipitation for California Calculation Procedures⁸ (HMR 59).

The NAP at each sub-basin within the Guadalupe catchment was estimated using an NAP isohyetal map from the National Climatic Data Center based on the most recent statistical precipitation data available (**Table 5**). The NAP values were then adjusted for differences between the NAP estimated on the map and the actual NAP measured at precipitation

gages within the basin. The ratios between gage NAP and map NAP were calculated at 10 precipitation gages within the study region. The ratios were observed to be much higher at gages in the steeper, higher elevation regions. In other words, the difference between the gage NAP and the map NAP was more significant for gages located in higher elevations, likely due to orographic effects. As a result, two different correction factors were developed to adjust NAP values in the lower elevations (under 700 feet NGVD) and in the higher elevations. The NAP values in the lower elevations were increased by 1%; NAP values in the higher elevations were increased by 21%.

Cumulative 24-hour precipitation for each of the 42 sub-basins for each statistical rainfall event was calculated using the relationship between the NAP and 24-hour cumulative precipitation for each statistical event. The Rainfall Depth-Duration Frequency Analysis¹⁴ provides both short and long duration data at most of the rain gages within the basin. Twelve precipitation gages throughout the basin were used to develop a ratio between the cumulative 24-hour precipitation at each gage to the gage NAP for each statistical event. The area weighted average of these ratios for each statistical event was used to estimate the 24-hour precipitation at each sub-basin using the adjusted sub-basin NAP. **Table 21** presents the final area weighted average ratios of the cumulative 24-hour precipitation to gage NAP. An example of the 24-hour precipitation calculations is presented in the following equations:

$$Ratio = (PPT24/NAPg)$$

$PPT24$ = Statistical cumulative 24-hour precipitation at a given rain gage¹⁴.

$NAPg$ = NAP for a given rain gage¹⁴.

$$PPTb = RatioAvg * NAPb * F$$

$PPTb$ = Cumulative 24-hour precipitation for a given sub-basin and statistical event (**Table 22**).

$RatioAvg$ = Area weighted average of ratio calculations at 12 rain gages (**Table 21**).

$NAPb$ = Estimated NAP for a given sub-basin (**Table 5**).

F = NAP correction factor, 1.01 for sub-basins in lower elevations, 1.21 for sub-basins in higher elevations.

Table 21 – Area Weighted Average Ratios of the Cumulative 24-Hour Precipitation to Gage NAP

Return Period (years)	Statistical Cumulative Precipitation								NAP	Period of Record (years)
	2	5	10	25	50	100	200	500		
Calero Watershed (RF128)	3.81	5.37	6.56	7.71	8.65	9.55	10.45	11.61	30.68	5
Johnson Ranch (RF36)	2.69	3.79	4.64	5.45	6.11	6.76	7.39	8.21	24.72	17
San Jose, RF131	1.53	2.15	2.57	3.09	3.46	3.83	4.19	4.74	13.74	87
Alamitos (RF1)	2.45	6.46	4.23	4.96	5.57	6.15	6.73	7.47	19.46	7
Almaden Watershed (RF4)	3.39	4.78	5.84	6.85	7.69	8.50	9.29	10.32	32.92	13
Vasona (RF125)	2.33	3.29	4.02	4.72	5.30	5.86	6.40	7.11	20.49	8
Rinconada (RF79)	2.59	3.66	4.47	5.25	5.89	6.51	7.11	4.90	21.69	9
Mt. Um (RF69)	3.60	5.08	6.21	7.29	8.18	9.03	9.88	10.97	37.43	16
Loma Prieta (RF44)	4.95	6.98	8.54	10.02	11.25	12.43	13.59	15.09	31.60	19
West Yard (RF108)	2.13	3.01	3.68	4.32	4.84	5.35	5.85	6.50	16.82	13
Laguna (RF37)	2.22	3.13	3.82	4.49	5.04	5.56	6.08	6.76	20.98	9
Lexington (RF42)	4.61	6.50	7.94	9.33	10.47	11.56	12.64	14.05	42.61	10
Area Weighted Average Ratio to NAP	0.12	0.17	0.20	0.23	0.26	0.29	0.32	0.35		

Table 22 – Cumulative 24-Hour Precipitation by Sub-Basin (inches)

Sub-Basin	50%	20%	10%	4%	2%	1%	0.5%	0.2%
W1730	3.83	5.57	6.55	7.76	8.70	9.62	10.52	11.58
W1970	3.79	5.51	6.47	7.67	8.61	9.51	10.40	11.45
W1480	2.80	4.07	4.79	5.67	6.36	7.03	7.69	8.47
W1850	3.95	5.74	6.75	7.99	8.97	9.91	10.84	11.94
W1540	2.86	4.17	4.89	5.80	6.50	7.19	7.86	8.66
W1950	4.59	6.67	7.84	9.29	10.42	11.52	12.60	13.87
W1400	2.97	4.32	5.07	6.01	6.74	7.45	8.15	8.97
W1960	3.71	5.40	6.34	7.52	8.43	9.32	10.19	11.22
W1800	3.88	5.64	6.63	7.85	8.81	9.73	10.65	11.72
Below Almaden	2.65	3.85	4.52	5.36	6.01	6.65	7.27	8.00
W1660	3.83	5.57	6.55	7.76	8.70	9.62	10.52	11.58
W1620	3.00	4.37	5.13	6.08	6.82	7.53	8.24	9.07
W1350	2.49	3.62	4.25	5.03	5.65	6.24	6.83	7.52
W1690	3.34	4.85	5.70	6.76	7.58	8.38	9.16	10.09
W1900	4.68	6.81	8.00	9.48	10.63	11.75	12.85	14.15
W1780	4.53	6.59	7.74	9.17	10.29	11.37	12.43	13.69
W1630	3.83	5.57	6.55	7.76	8.70	9.62	10.52	11.58
Upper Ross	3.48	5.07	5.95	7.05	7.91	8.74	9.56	10.53
W1270	2.65	3.85	4.52	5.36	6.01	6.65	7.27	8.00
Above Ross	1.86	2.71	3.18	3.77	4.23	4.67	5.11	5.62
W1300	1.81	2.63	3.09	3.66	4.11	4.54	4.96	5.46
W1650	3.95	5.74	6.74	7.99	8.96	9.90	10.83	11.92
W1740	4.75	6.90	8.11	9.61	10.78	11.92	13.03	14.35
W1260	1.80	2.62	3.08	3.65	4.09	4.53	4.95	5.45
W1220	1.86	2.71	3.18	3.77	4.23	4.67	5.11	5.62
W1700	7.05	8.29	9.82	11.02	12.18	13.32	14.66	0.00
W1190A	2.62	3.81	4.47	5.30	5.94	6.57	7.18	7.91
W1250	1.74	2.54	2.98	3.53	3.96	4.38	4.79	5.27
W1190	2.62	3.81	4.48	5.30	5.95	6.58	7.19	7.92
W1580	3.83	5.57	6.55	7.76	8.71	9.62	10.52	11.59
Los Gatos 3	1.98	2.88	3.38	4.00	4.49	4.96	5.43	5.98
W1640	4.77	6.94	8.15	9.66	10.84	11.98	13.10	14.42
Los Gatos 4	1.69	2.45	2.88	3.41	3.83	4.23	4.63	5.10
W1160	1.63	2.37	2.78	3.29	3.70	4.09	4.47	4.92
W1170	1.84	2.68	3.15	3.73	4.19	4.63	5.06	5.57
WOI	1.62	2.36	2.77	3.28	3.68	4.07	4.45	4.90
WN	1.56	2.26	2.66	3.15	3.53	3.91	4.27	4.70
WOJ	1.62	2.36	2.77	3.28	3.68	4.07	4.45	4.90
WP	1.57	2.28	2.68	3.18	3.57	3.94	4.31	4.75
WQ	1.55	2.25	2.64	3.13	3.51	3.88	4.25	4.68
WR	1.51	2.20	2.59	3.07	3.44	3.80	4.16	4.58
WS	1.51	2.20	2.58	3.06	3.43	3.80	4.15	4.57

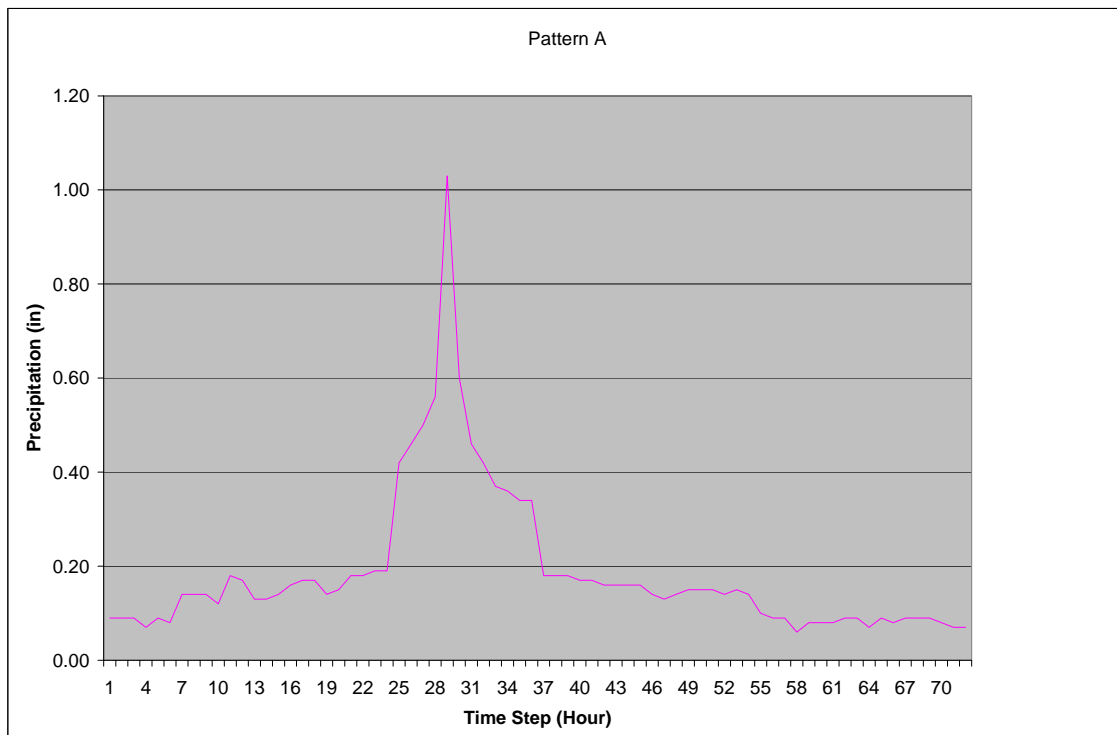
A spreadsheet tool was developed to automate the creation of rainfall hyetographs for each of the 42 sub-basins. **Appendix D** provides a brief documentation of this tool. Two sets of curves presented in HMR 59 were used in developing the meteorology in this study: the “Midcoastal Region All Season Curve” relating the 24-hour point precipitation depth to the 1-, 6-, 12-, 48-, and 72- durations, and the “Midcoastal Region All Season Depth Area Reduction Curves” (**Table 23**). The Area 3 Midcoastal region as defined in HMR 59 spans the California coastline North and South of the Bay Area from approximately 35 N to 40 N Latitude. Special corrections for seasonal differences were not included as the goal of this study is to observe broad annual rainfall patterns and not a particular event that might occur in a particular month of the year.

The cumulative 24-hour precipitation is used to calculate the appropriate statistical 1, 6, 12, 48, and 72-hour precipitation using the statistically derived ratios (**Table 23**) to the 24-hour cumulative rainfall as presented in the HMR 59 “Midcoastal Region All Season Curve”. These ratios were checked using statistical data in and around the study area to make sure they are appropriate to use for all of the recurrence intervals being studied (i.e. 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent chance of occurrence within any given year.) These calculations showed that the ratios between the x-hour cumulative precipitation and the 24-hour precipitation remain very constant over the range of recurrence intervals. In addition, it was found that an average of these ratios calculated for specific events, both large and small, in and around the Guadalupe Watershed study area was very similar to the ratios presented in HMR 59. As a result, it was determined that the ratios presented in HMR 59 are valid for use in this study. The estimated cumulative 24-hour precipitation for each statistical recurrence interval is presented in **Table 22**.

A 72-hour precipitation pattern was chosen out of five patterns based on observed hyetographs presented in HMR 59. The rainfall patterns presented in HMR 59 were compared to rainfall patterns of real events at locations throughout the study region. Ten historical 72-hour storm patterns within the Guadalupe region were analyzed and “Pattern A” was selected as it is the most representative of a typical area storm (see **Figure 16**). Pattern A was found to be very similar to regional patterns with regard to the placement of the maximum 1, 6, 12, and 24, and 48-hour precipitation patterns. A 72-hour precipitation pattern is typical of the large storm events on the coastal range in California. The peak 24-hour cumulative precipitation is typically the most critical duration in that it has the most impact on resulting hydrographs and oftentimes produces the peak in rain events. A 72-hour storm duration allows time to build up to this peak rainfall and to attenuate the peak flow throughout the basin. A 72-hour storm pattern also provides time to account for basin storage, route storm runoff throughout the basin as a whole, and observe the effects of initial and constant loss.

The spreadsheet tool (**Appendix D**) was used to distribute precipitation over the 72-hour rainfall pattern (Pattern A) for each of the 42 sub-basins in the Guadalupe Watershed for each of the statistical rainfall events (i.e. 50-, 20-, 10-, 4-, 2-, 1-, 0.5-, and 0.2-percent chance of occurrence within any given year). The spreadsheet was set up to automatically check the cumulative rainfall for the shorter durations (i.e. 1, 6, 12, 24, and 48-hours) to make sure that they are not too high, and adjusts the 72-hour rainfall pattern slightly, if necessary. This guarantees, for example, that the 24-hour cumulative precipitation within the 1-percent chance exceedance 72-hour rainfall pattern has a recurrence interval of around 100-years, not significantly higher, which could result in inappropriately high discharge values.

Figure 16 – Rainfall “Pattern A”



It is necessary to apply Depth Area Reduction Factors (DARFs) to the precipitation patterns developed for the basin to account for reductions in precipitation depth with distance from the storm center. The DARFs are a function of both distance from the storm center and precipitation duration. A different DARF will be selected for each calculated 1, 6, 12, 24, 28, and 72-hour cumulative precipitation. Depth reduction starts for accumulated drainage areas greater than 10 square miles. Appropriate DARFs are presented in HMR 59 for a range of regions and climates. The DARF curves in HMR 59 were developed specifically for California, making them a better choice than the DARF curves presented in NOAA Atlas 2, for example. The DARFs presented in HMR 59 for “Midcoastal Region, All Seasons” are presented in **Table 23**.

The spreadsheet tool applies the DARFs to calculate both specific and concurrent precipitation depth assuming a given storm centering. The storm center for the Guadalupe River basin was chosen to be in the upper Guadalupe Creek sub-basin, upstream of Guadalupe Reservoir in sub-basin W1730. This is the main region that this study is concerned with, and storms in this region are typically more intense than in the lower elevation areas. A different storm center was chosen for the Los Gatos basin upstream of Lexington Reservoir (sub-basin W1650). In each storm centering scenario, each sub-basin was ordered (1 through 42) in relation to distance from the center of the storm. The cumulative drainage area of each ordered sub-basin was used to estimate drainage area in selecting the DARFs for each sub-basin. The DARFs reduce the cumulative precipitation more the further away a sub-basin is from the storm center. The order of the sub-basins for the Guadalupe and Lexington storm centers are presented in **Table 24**. A dual storm centering was also considered as a possibility given available data from recorded events. It is not uncommon that a storm would have a center over both the Guadalupe and Los Gatos River basins. Several models were configured assuming dual storm centers; however it was determined that this phenomenon does not likely occur often enough to be the condition represented for the purposes of this study.

Table 23 – Depth Area Reduction Factors, California Midcoastal Region, All Season, as Presented in HMR 59⁸

Hours of Cumulative Precipitation	1	6	12	24	48	72
Ratio to Cumulative 24-Hour Precipitation	0.13	0.45	0.74	1.00	1.45	1.70
Drainage Area (square miles)	Depth Area Reduction Factors					
0.1	1	1	1	1	1	1
10	1	1	1	1	1	1
50	0.8750	0.8875	0.9000	0.9100	0.9200	0.9300
100	0.8175	0.8375	0.8550	0.8700	0.8850	0.9000
200	0.7575	0.7825	0.8050	0.8250	0.8450	0.8625
500	0.6750	0.7100	0.7350	0.7600	0.7850	0.8050
1000	0.6075	0.6550	0.6800	0.7050	0.7300	0.7550
2000	0.5300	0.5850	0.6150	0.6400	0.6700	0.7000
5000	0.3800	0.4450	0.4850	0.5200	0.5500	0.5900
10000	0.2500	0.3400	0.3800	0.4200	0.4500	0.4900

Table 24 – Order of Sub-Basins for Guadalupe and Lexington Storm Centers

Sub-Basin Order, Guadalupe Storm Center	Sub-Basin Order, Lexington Storm Center
W1730	W1650
W1970	W1700
W1480	W1630
W1850	W1580
W1540	W1740
W1950	W1640
W1400	W1190A
W1960	W1660
W1800	W1780
Below Almaden	W1800
W1660	Upper Ross
W1620	W1900
W1350	W1960
W1690	W1190
W1900	W1730
W1780	W1270
W1630	W1950
Upper Ross	W1970
W1270	W1350
Above Ross	W1400
W1300	Above Ross
W1650	W Los Gatos 4
W1740	W1850
W1260	W1540
W1220	W1480
W1700	Below Almaden
W1190A	W1220
W1250	W1620
W1190	W1250
W1580	W Los Gatos 3
Los Gatos 3	W1160
W1640	W1170
Los Gatos 4	OI
W1160	W1260
W1170	OJ
WOI	WN
WN	W1690
WOJ	W1300
WP	WP
WQ	WQ
WR	WR
WS	WS

7.2 Synthetic Storm Series Results

The hydrographs resulting from the synthetic storm series at the main calibration location on Guadalupe River, Guadalupe River downstream of Ross Creek (J Gage 23B d/s Ross Cr), and at the second major index point, Guadalupe River downstream of Los Gatos Creek (J Gage at San Jose) are presented in **Figures 17-20**. A summary of peak flow results from eight index locations for all of the eight synthetic storm series events are presented in **Table 25**. These results assume that the Upper Guadalupe Project is complete.

Many parameters were considered in the sensitivity analyses performed for this study. Sensitivity model runs were used to better understand which parameters result in the largest changes to the system so that appropriate final selections could be made. In general terms, sensitivity analyses were performed on:

- Storm Centering - single, dual, Guadalupe Basin, Lexington Basin.
- Precipitation patterns - 1977 pattern, HMR 59 patterns.
- Hydrograph peak timing - Los Gatos and Guadalupe inflow hydrographs at the San Jose Gage.
- Initial reservoir storage
- Depth Area Reduction Factors – NOAA Atlas 2, HMR 59, 1977 Report³ DARFs.
- Routing Methods – urban, rural, Muskingum-Cunge, Modified Puls.
- Loss Rates – based on soil types, urbanization, and antecedent moisture concentration.
- Transform method – user specified s-graphs, San Francisco District s-curve, Clark method.
- Baseflow
- Manning's n

Table 25 – Summary of Peak Flows (cfs), Synthetic Storm Series, With 50% Confidence Limits

% Chance Exceedance	Guad Ri us Ross Ck	Guad Ri ds Ross Ck	Guad Ri ds Canoas Ck	Guad Ri us Los Gatos Ck	Los Gatos Ck us Guad Ri*	Guad Ri ds Los Gatos at San Jose	Guad Ri at J294, near Airport	Guad Ri at Outlet
50	1,382	1,841	2,521	2,670	814	3,317	3,602	3,873
20	3,141	3,910	4,869	4,988	1,469	6,059	6,466	6,530
10	4,095	5,149	6,267	6,398	2,519	7,712	8,196	8,280
4	6,629	7,562	8,865	9,086	4,478	10,463	10,790	11,356
2	8,990	10,181	11,691	11,991	7,086	14,251	14,770	15,221
1	11,165	12,654	14,366	14,700	8,274	17,967	18,597	19,012
0.5	13,298	15,077	16,996	17,381	10,407	22,431	23,152	23,560
0.2	15,676	17,893	19,909	20,349	12,888	27,942	28,769	29,162

* Results from model assuming a storm center over Lexington Reservoir.

Figure 17 – Synthetic Storm Series Hydrographs, Guadalupe River Downstream of Ross Creek (J Gage 23B d/s Ross Cr), 50-, 20-, 10-, and 4-Percent Chance Exceedance Events

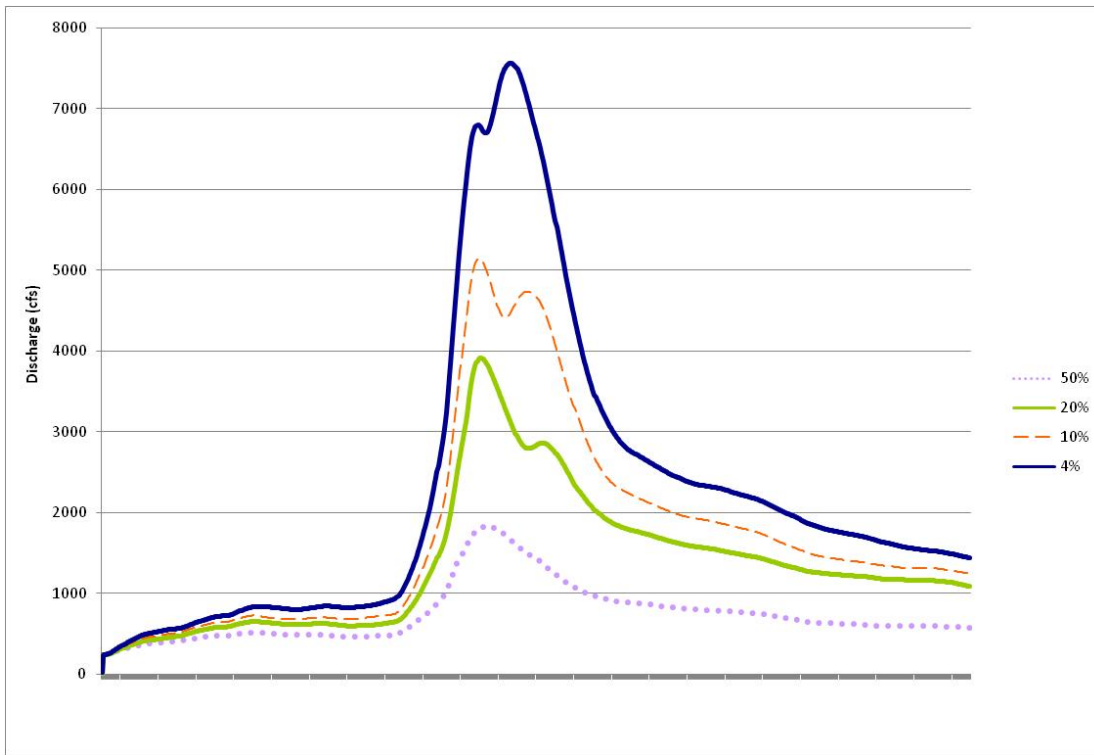


Figure 18 – Synthetic Storm Series Hydrographs, Guadalupe River Downstream of Ross Creek (J Gage 23B d/s Ross Cr), 2-, 1-, 0.5-, and 0.2-Percent Chance Exceedance Events

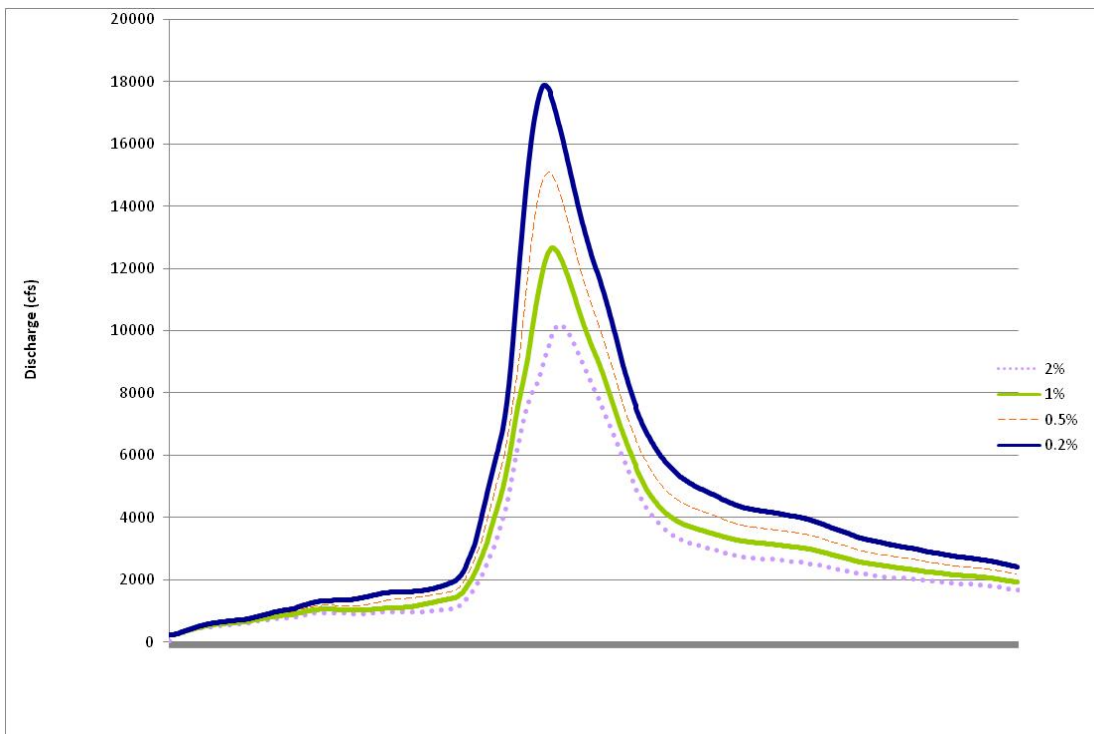


Figure 19 – Synthetic Storm Series Hydrographs, Guadalupe River Downstream of Los Gatos Creek (J Gage at San Jose), 50-, 20-, 10-, and 4-Percent Chance Exceedance Events

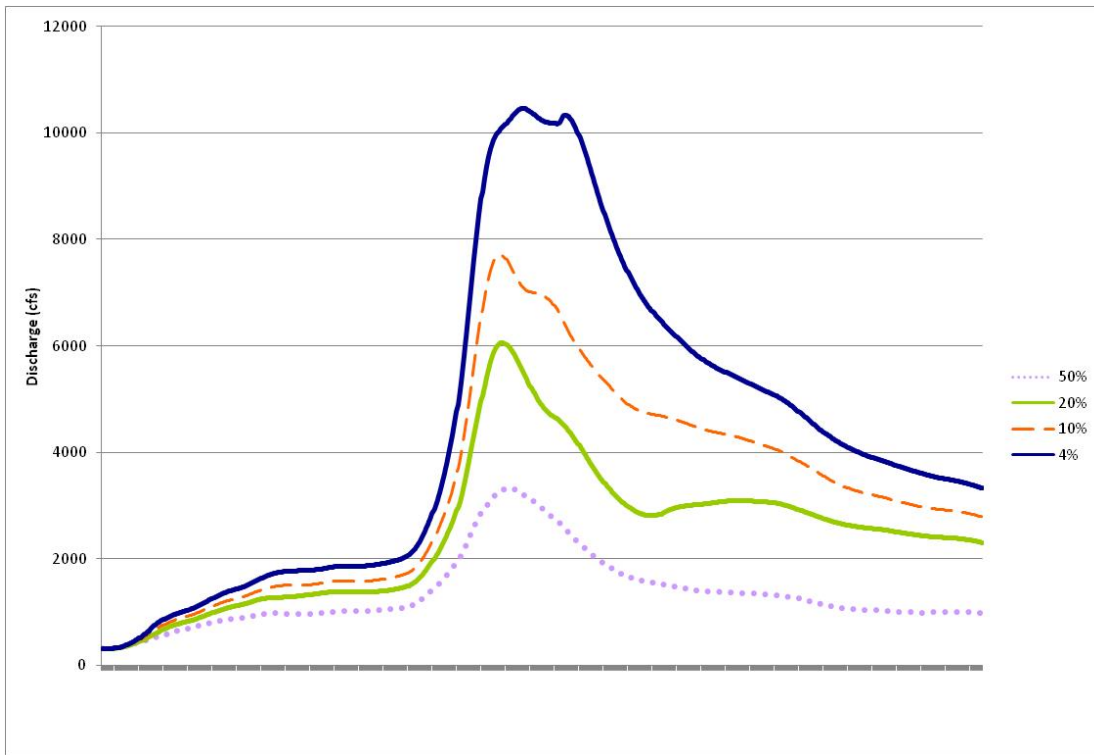
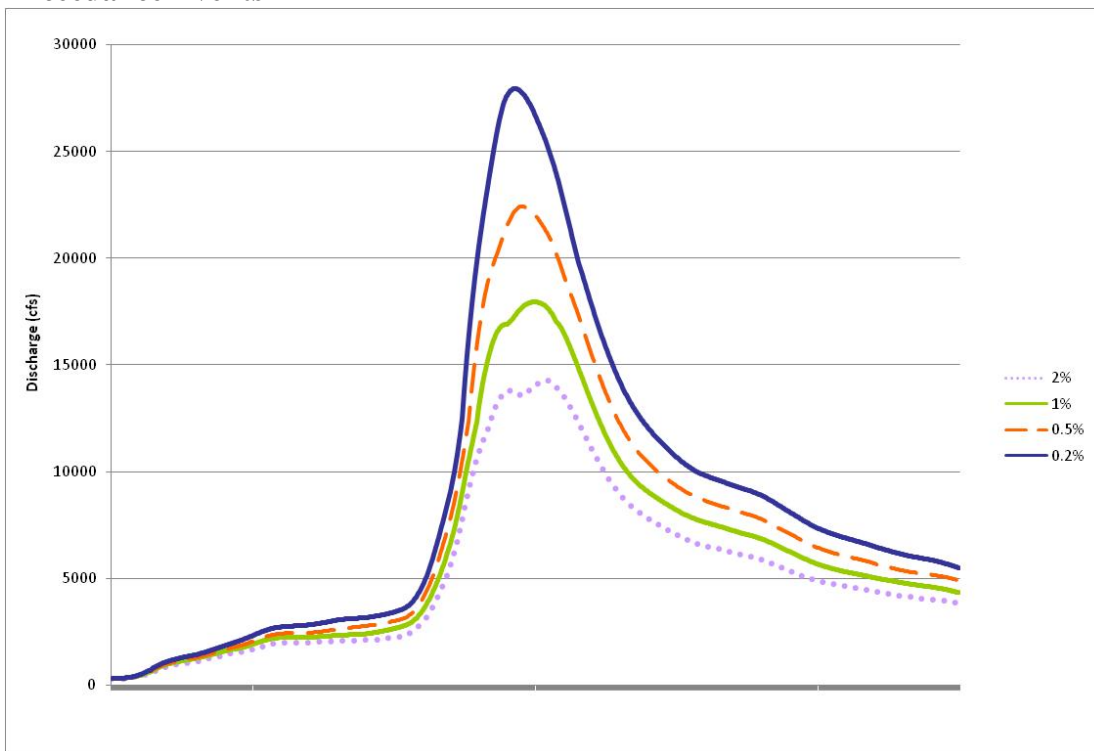


Figure 20 - Synthetic Storm Series Hydrographs, Guadalupe River Downstream of Los Gatos Creek (J Gage at San Jose), 2-, 1-, 0.5-, and 0.2-Percent Chance Exceedance Events



The peak flows for the 1-percent event for the upper portion of Guadalupe River are very similar to the peak flows calculated in the 1977 report³. A comparison of results is presented in **Table 26**.

Table 26 – Comparison of Hydrology Studies, 1-Percent Discharge

Location	Junction	1977 Study	2009 Model	% Difference	SCVWD Design Discharge
Guadalupe ds of Alamitos	J Guad & Alamitos Cr Conflu	11,400	11,165	-2	12,400
Guadalupe ds of Ross	J Gage 23B d/s Ross Cr	12,400	12,654	2	14,600
Guadalupe ds of Canoas	J272	14,600	14,366	-2	NA
Guadalupe us of Los Gatos	J LG to Alma	14,600	14,700	1	NA
Los Gatos us of Guadalupe	J Gage Los Gatos u/s Guad	7,800	8,274	6	NA
Guadalupe ds of Los Gatos	J Gage at San Jose	17,000	17,967	6	NA

Differences between the 1977 and 2009 studies can be attributed to many factors. Changes in methods and assumptions pertaining to meteorology likely had the most profound impact as this fundamentally changed the timing and amount of precipitation input to the system. The 1977 study as well as the previous April 2008 study used only one precipitation pattern for all of the sub-basins. Hourly precipitation was generated randomly, based on statistical 12-hour and 72-hour cumulative precipitation. The depth area reduction assumptions were based on the 1955 storm near Hollister, California. As mentioned, rainfall patterns in the 2009 study were created individually for each sub-basin taking into account statistical precipitation for the 1, 6, 12, 24, and 72 hour durations. Appropriate storm centers were selected and the hyetographs were modified based on updated depth area reduction factors (DARFs) calculated using all available storm data within the California Midcoastal region. In general, the updated HMR 59 DARFs reduced the cumulative precipitation more than the reduction factors used in the 1977 study, particularly with increased distances from the storm center. The HMR 59 storm pattern used in the 2009 study assumes that increases in rainfall over a storm event are more gradual than the rainfall pattern used in the 1977 and 2008 studies. For a given cumulative precipitation, the 2009 storm pattern has higher precipitation within the peak 12, and 24-hour patterns, and less precipitation in the advancing and receding limbs. This causes more precipitation to be lost outside of the 24-hour maximum cumulative precipitation pattern due to initial loss rate assumptions.

Reservoir operation has also changed some since 1977 (**Section 5.2**), which likely has also had some impact on discharge values downstream. In addition, a slight increase in urbanization within the Canoas Creek sub-basin has also had a small impact on the results.

The 2009 hydrology study has made substantial changes to the work submitted in April 2008, in particular related to the meteorology, routing methods, transform methods, and loss rates. Changes to the meteorology are discussed in previous paragraphs. The 2009 study updated the routing method in the model to use Modified Plus in the main branches of the Guadalupe River using updated storage-discharge curves from the HEC-RAS model. The 2008 study used only the Muskingum method to describe routing, which assumes less channel storage. The transform method was also changed in the 2009 study

to use both the user specified S-graph method and the Clark method. The loss rates and percent impervious assumptions were also changed in the 2009 study to represent land use and soil types. Much time was spent during calibration to make sure that the loss rates were realistic given what is seen in the field.

Changes to the routing and transform methods had impacts on hydrograph timing but did not impact the volume of water in the system. The changes to initial and constant loss rates were tempered by the use of impervious areas, particularly in the urban sub-basins, and did not have a large impact on final discharge values. The changes to the meteorology, however, did change water volumes throughout the system as a result of the new storm centering and depth area reduction assumptions.

Many changes have been made to this study and it is difficult to identify and quantify which had the largest impact. Even though in many cases the 2009 study used new methods, the results including both peak discharge values and total water volumes are very similar to those reported in the 1977 study. A comparison of water volumes at key index locations for the 1-percent chance exceedance probability event are presented in **Table 27**.

Table 27 – Volume Comparison for 1-Percent Chance Exceedance Event

Index Point	2009 Volume (AC-FT)	1977 Volume (AC-FT)	% Difference
Guadalupe Ri ds Ross Ck	19,264	19,580	-1.6
Guadalupe Ri ds Canoas Ck	23,249	25,280	-8
Los Gatos Ck us Guadalupe Ck	15,525	13,140	18
Guadalupe Ri ds Los Gatos Ck	37,675	39,600	-5

8. GAGE ANALYSIS

The recent hydrologic studies for the Guadalupe River began with an analysis of stream gage data at SF23 downstream of Ross Creek (J Gage 23b d/s Ross Cr) and the USGS gage #11169000 downstream of Los Gatos Creek (J Gage at San Jose) in Downtown San Jose¹⁵. Since the completion of the 1977 hydrology report, an additional 30 years of stream data has been collected such that there now exists a total of 68 years of data at SF23 and 51 years of data at the USGS gage #11169000. This data was analyzed to determine whether updates could be made to the discharge frequency curves.

Data recorded at SF 23 and the USGS gage #11169000 was graphed and analyzed for various periods of record (e.g. 1940-1977, 1940-2006). As SF23 is downstream from three reservoirs, and gage #11169000 is downstream from six reservoirs, the usual Log Pearson 3 method of analyzing data and plotting a discharge frequency curves is not valid. According to Bulletin 17B, this is due to the effect reservoirs have on both the low end (most frequent events) and the high end of the discharge frequency curve. The most accepted methods of plotting discharge frequency curves for data influenced by upstream reservoirs are graphical analysis and regression analysis.

A regression analysis and a graphical analysis were used to plot discharge frequency curves. The curve drawn in the graphical analysis did not include data points below 1,000 cubic feet per second (cfs) at SF23 and below 1,500 cfs at gage #11169000. This technique of disregarding flows below a given discharge is a method used to adjust for the effect of reservoir operations on a downstream stream gage.

The results of the gage analysis indicate an overall trend of increasing discharge frequency curve values with an increase in period of record. The gage analysis results for the 1-percent event, however, are significantly lower than the results from the 1977 study. This is part of the reason why it was determined that an updated rainfall-runoff model would be necessary to more accurately predict discharge values given the strong influence of reservoirs in the system and the limited amount of available gage data. A comparison of results from the gage analysis and the 1977 and 2009 hydrology studies for the 1-percent event is presented in **Table 28**.

Table 28 – Comparison of 1-Percent Discharge Estimates

Method of Analysis	Location	Period of Record (yrs)	1-Percent Discharge (cfs)
2009 Hydrology Study	SF23	NA	12,654
1977 Hydrology Study	SF23	NA	12,400
Regression Analysis	SF23	68	7,580
Graphical Analysis	SF23	68	8,320
2009 Hydrology Study	11169000	NA	17,967
1977 Hydrology Study	11169000)	NA	17,000
Regression Analysis	11169000	51	13,000
Graphical Analysis	11169000	51	13,600

9. EQUIVALENT LENGTH OF RECORD

The equivalent length of record was estimated to be 30 years based on the USACE Risk-Based Analysis for Flood Damage Reduction¹⁶ guidelines as presented in **Table 29**. The model is calibrated to three peak events and the results suggest that the model is able to accurately predict flood peaks in the upper reaches of the Guadalupe River basin. The length of record was used along with the peak flows (**Table 25**) to calculate confidence limits for the discharge frequency curve using HEC’s Flood Damage Analysis (HEC-FDA) software. A graphical method was used to generate the error bands surrounding the 50-, 20-, 10-, 4-, 2-, 1-, 0.5, and 0.2-percent events. The error band for the higher frequency events was interpolated using the analytical method in FDA to estimate the discharge for the 99.9-percent event. Uncertainty surrounding reservoir operation and initial reservoir storage was not directly considered during the uncertainty analysis. This would have only impacted the error bands, not the peak event discharge values. The results of this analysis including the final discharge-frequency curves at five index locations are presented in **Appendix E**. A summary of the error bands calculated for the 1-percent chance exceedance event are presented in **Table 30**.

Table 29 - Equivalent Record Length Guidelines¹⁶

Method of Frequency Function Estimation	Equivalent Record Length*
Analytical distribution fitted with long-period gauged record available at site.	Systematic record length.
Estimated from analytical distribution fitted for long-period gauge on the same stream, with upstream drainage area within 20% of that point of interest.	90% to 100% of record length of gaged location.
Estimated from analytical distribution fitted for long-period gage within same watershed.	50% to 90% of record length.
Estimated with regional discharge-probability function parameters.	Average length of record used in regional study.
Estimated with rainfall-runoff-routing model calibrated to several events recorded at short-interval event gauge in watershed.	20 to 30 years.
Estimated with rainfall-runoff-routing model with regional model parameters (no rainfall-runoff-routing model calibration).	10 to 30 years.
Estimated with rainfall-runoff-routing model with handbook or textbook model parameters.	10 to 15 years.

* Based on judgment to account for the quality of any data used in the analysis, for the degree of confidence in models, and for previous experience with similar studies.

Table 30 – 1% Chance Exceedance Event Peak Flow With Confidence Limits (cfs)

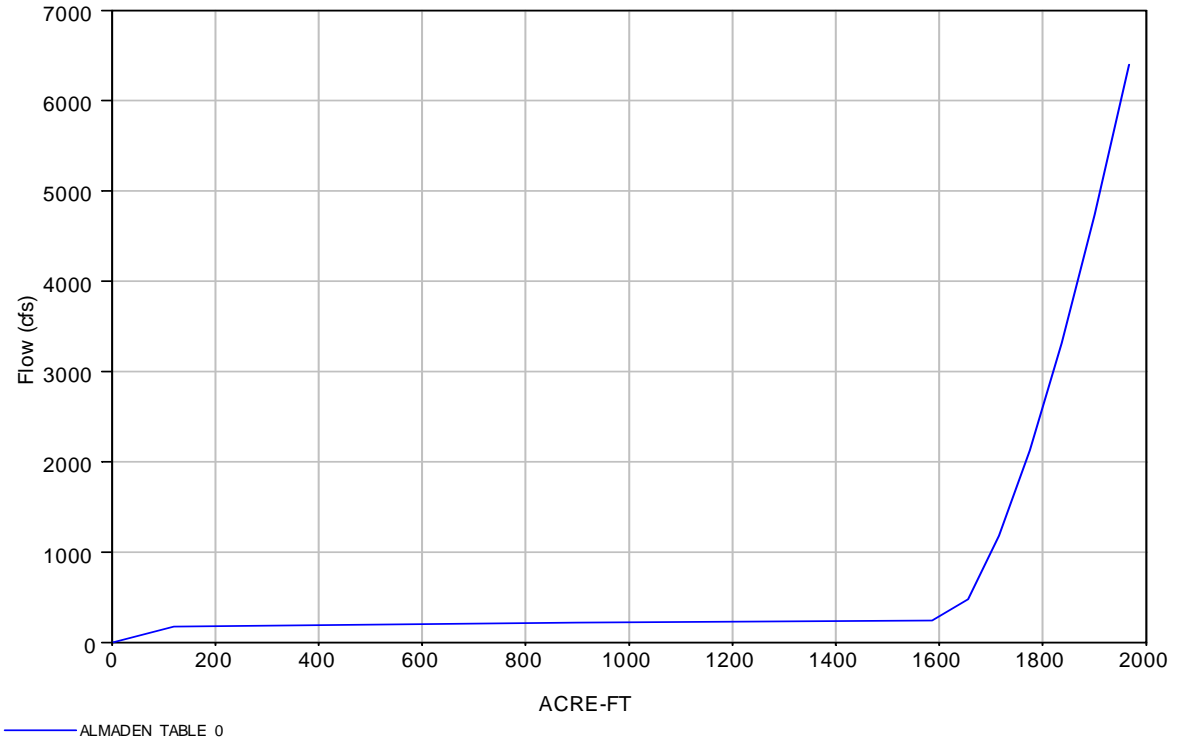
Location	Junction	1-Percent Chance Exceedance Event	5%	95%
Guadalupe ds of Alamos	J Guad & Alamos Cr Conflu	11,165	5,948	20,956
Guadalupe ds of Ross	J Gage 23B d/s Ross Cr	12,654	7,334	21,832
Guadalupe ds of Canoas	J272	14,366	8,636	23,899
Los Gatos us of Guadalupe	J Gage Los Gatos u/s Guad	8,274	2,940	23,284
Guadalupe ds of Los Gatos	J Gage at San Jose	17,967	10,800	29,890

10. REFERENCES

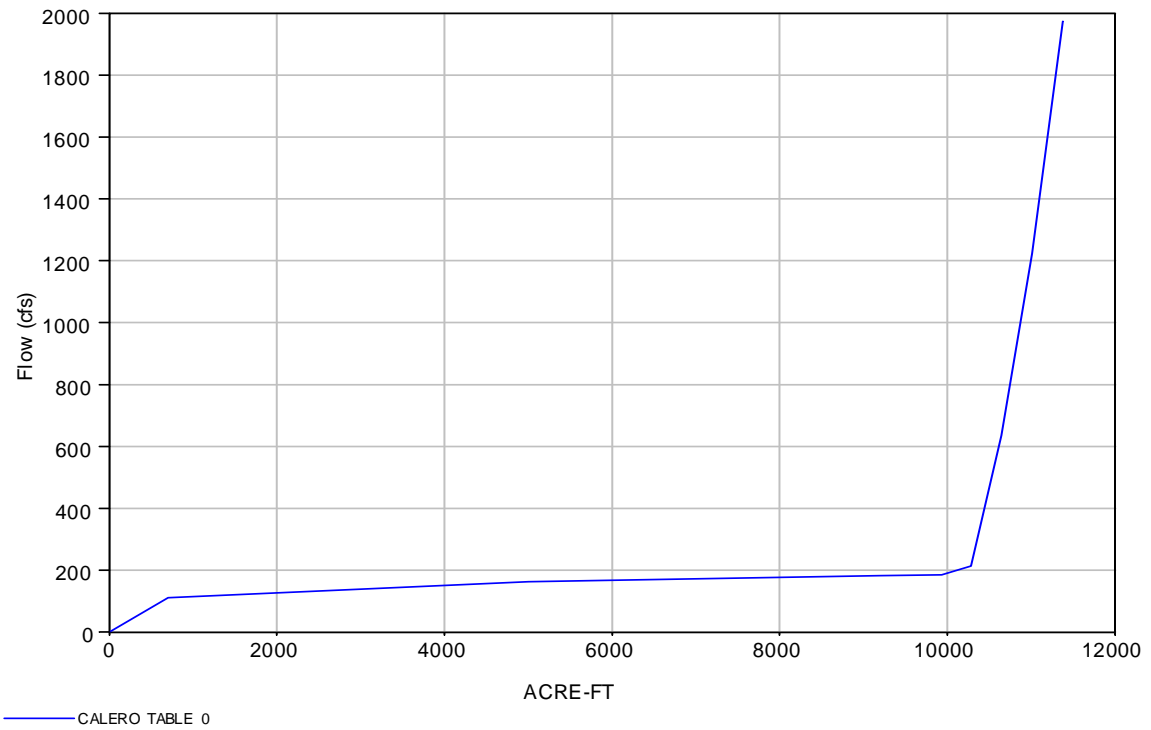
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- ¹ USACE. Guadalupe River Project, Final Operation, Maintenance, Repair, Replacement, & Rehabilitation Manual, San Jose, Santa Clara County, California. 2004.
- ² USACE. Upper Guadalupe River Final Feasibility Report. January 1998.
- ³ USACE, San Francisco District. Guadalupe River and Coyote Creek Hydrologic Study; Santa Clara County, California. Hydrologic Engineering Office. June 1977.
- ⁴ USACE. EM 1110-2-1415, Hydrologic Frequency Analysis. March 1993.
- ⁵ USACE. EM 1110-2-417, Flood-Runoff Analysis. August 1994.
- ⁶ USACE. EM 1110-2-1419, Hydrologic Engineering Requirements for Flood Damage Reduction Studies. January 1995.
- ⁷ U. S. Water Resources Council, Hydrology Committee. Guidelines for Determining Flood Flow Frequency (Bulletin 17B). 1981.
- ⁸ United(States Department of Commerce. Hydrometeorological Report No. 59, Probable Maximum Precipitation for California – Calculation Procedures. Silver Spring MD. October 1998.
- ⁹ Santa Clara Valley Water District Website. *Reservoirs*. http://www.valleywater.org/Water/Where_Your_Water_Comes_From/Local_Water/Reservoirs/index.shtml. Accessed October 2007.
- ¹⁰ Willey, Neil. *Lake Elzman*. Mountain Network News, internet resource. http://www.mnn.net/lake_elsman.htm. Accessed October 2007.
- ¹¹ Santa Clara Valley Water District. Reservoir Flood Rule Curve Operations. Document Number W91761001.010403, Revision G. Effective Date: 7/5/07.
- ¹² USACE. Hydrologic Modeling System HEC-HMS Technical Reference Manual. March 2000. Pages 58-60.
- ¹³ United States Department of Agriculture, Natural Resources Conservation Service. Web Soil Survey (WSS). <http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm>. Accessed April 2009.
- ¹⁴ Goodridge, Jim. Rainfall Depth-Duration Frequency Analysis for California Rain Gages. Data Compiled with support from the California Department of Water Resources. 2000.
- ¹⁵ USACE. Memorandum for the Record, Guadalupe River Hydrology Review – Stream Gage Data Collected Since 1977. Revised April 2008.
- ¹⁶ USACE. Risk-Based Analysis for Flood Damage Reduction Studies, EM-1110-2-1619. August 1996.

Appendix A

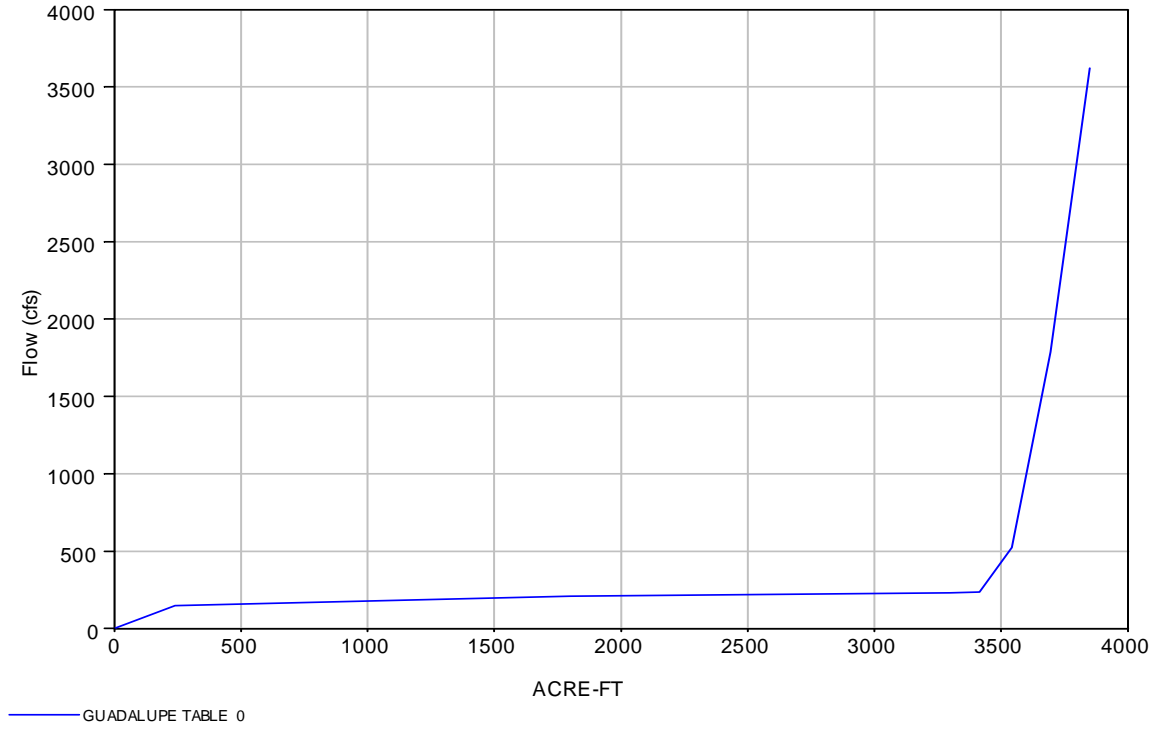
Storage-Discharge Curve for Almaden Reservoir



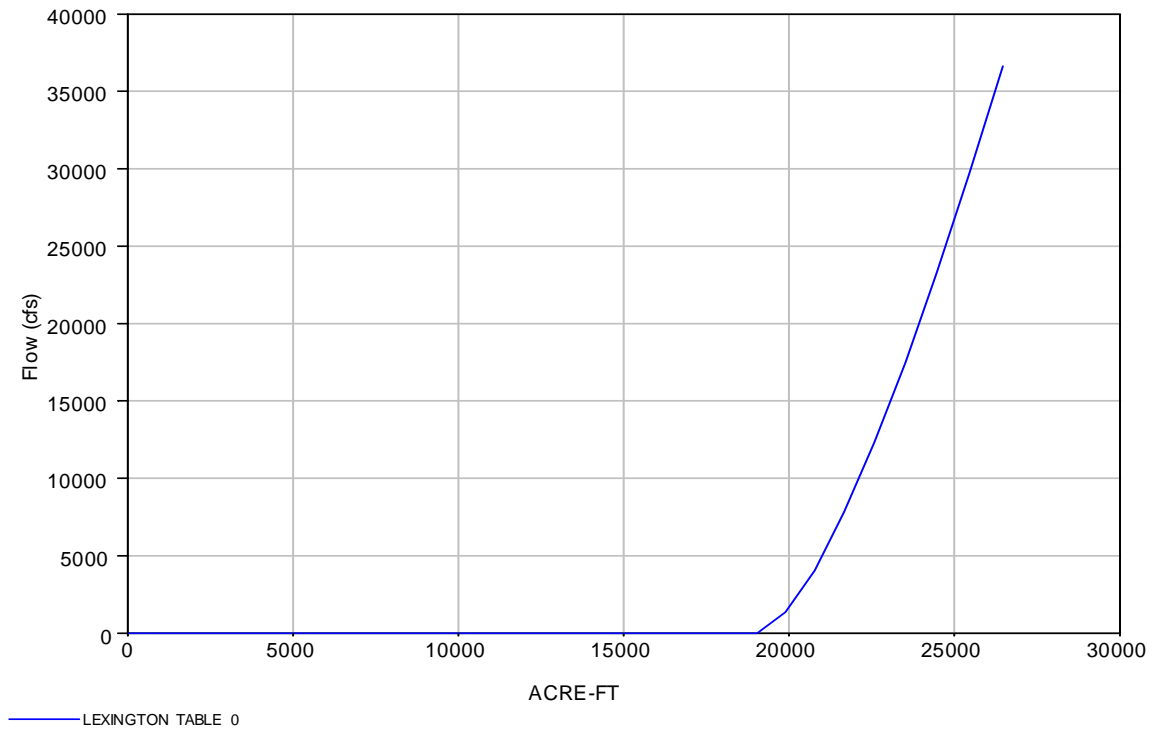
Storage-Discharge Curve for Calero Reservoir



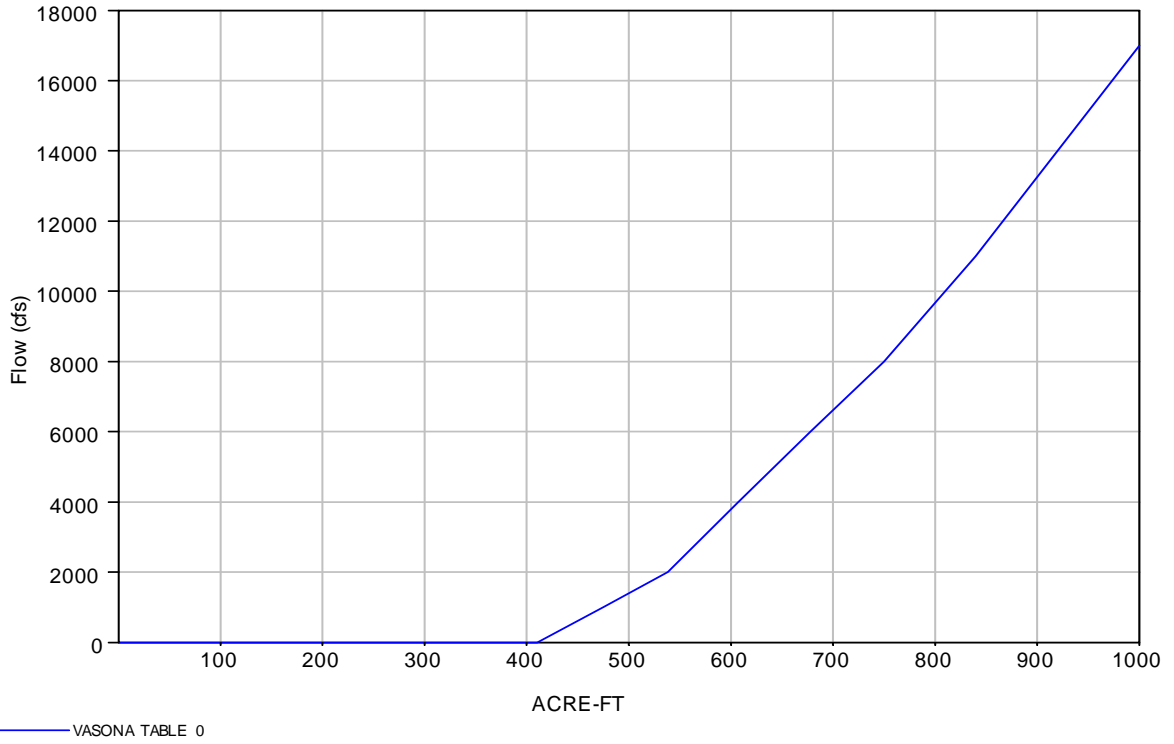
Storage-Discharge Curve for Guadalupe Reservoir



Storage-Discharge Curve for Lexington Reservoir



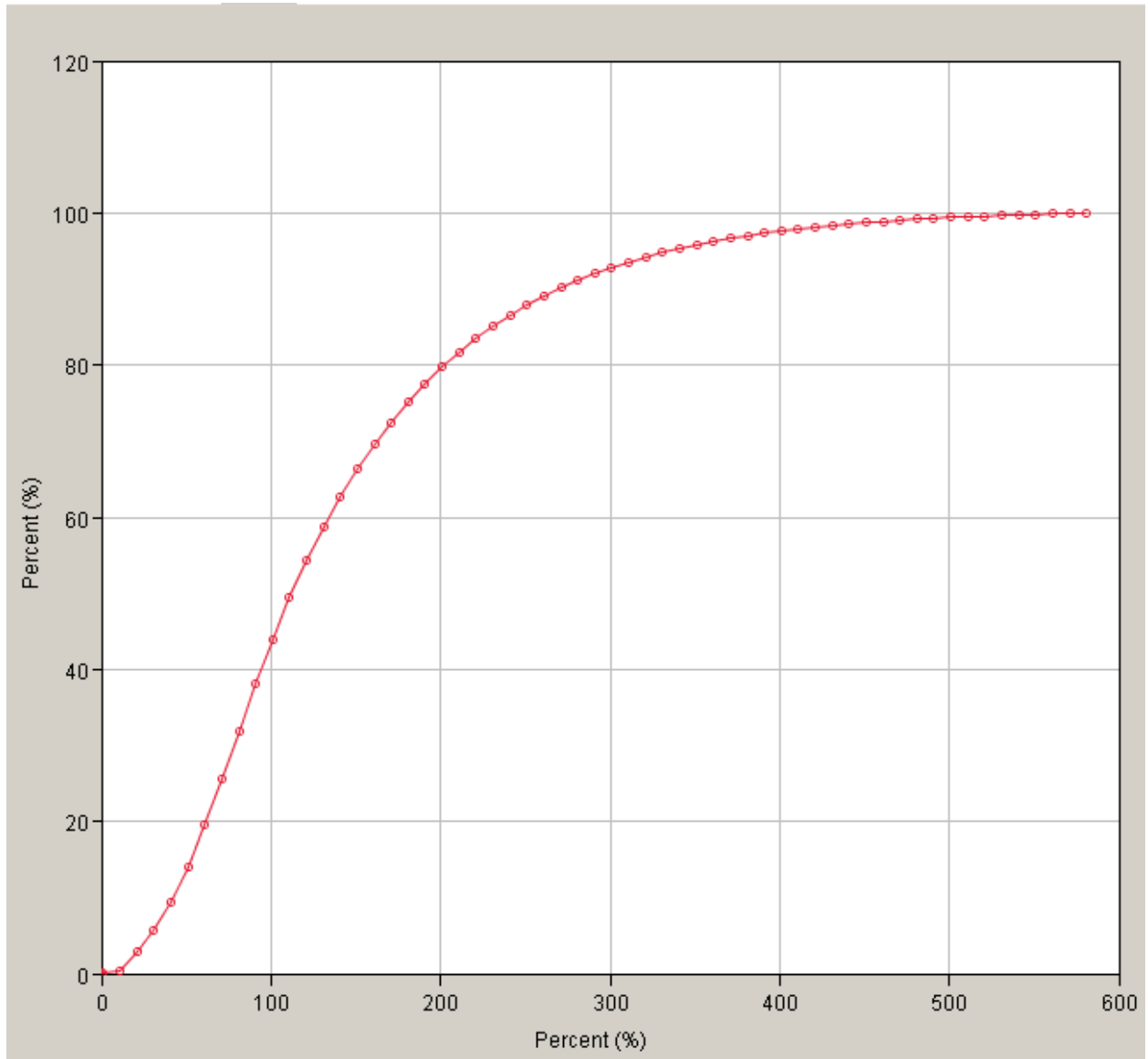
Storage-Discharge Curve for Vasona Reservoir



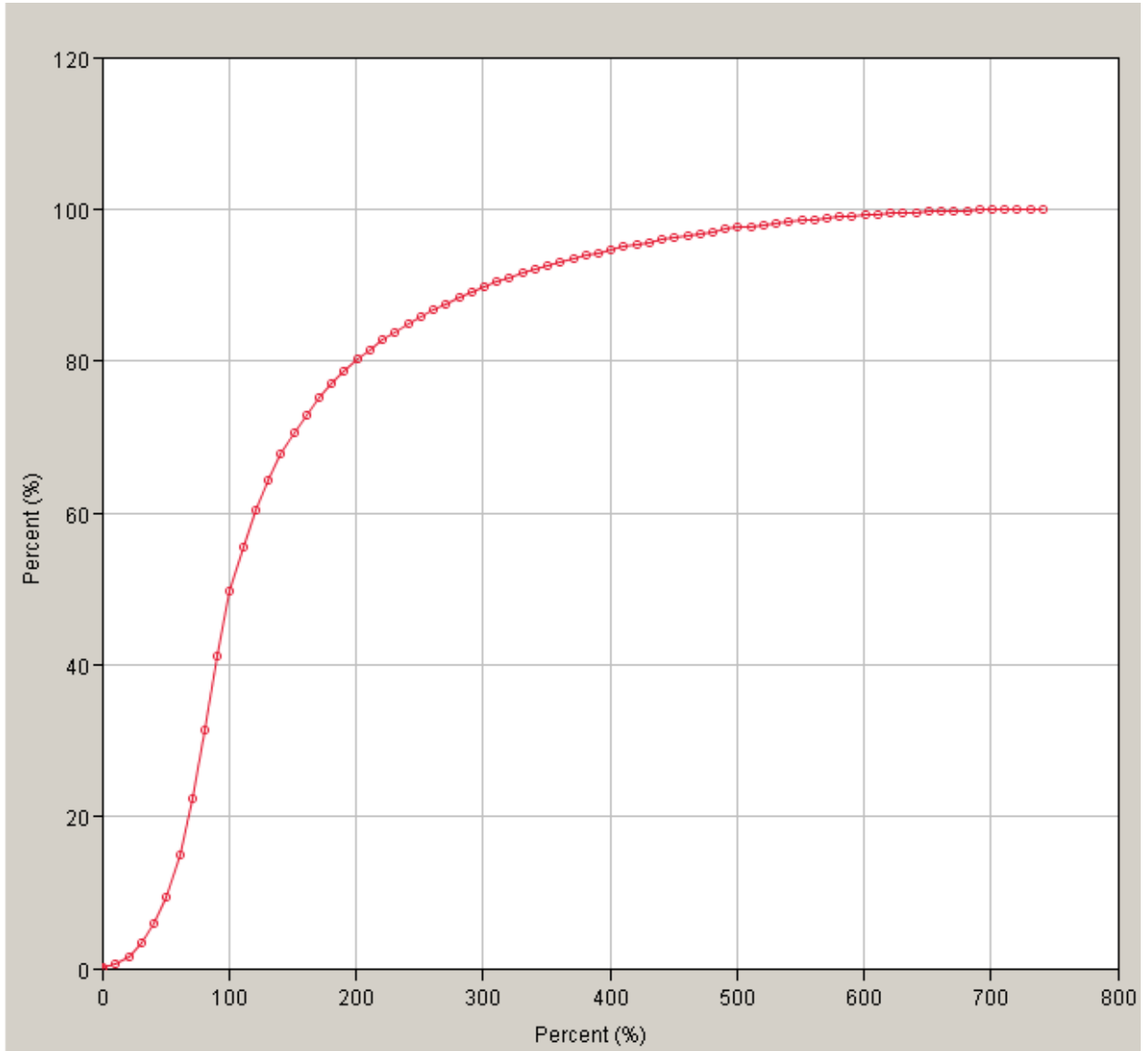
Appendix B

S-Curves

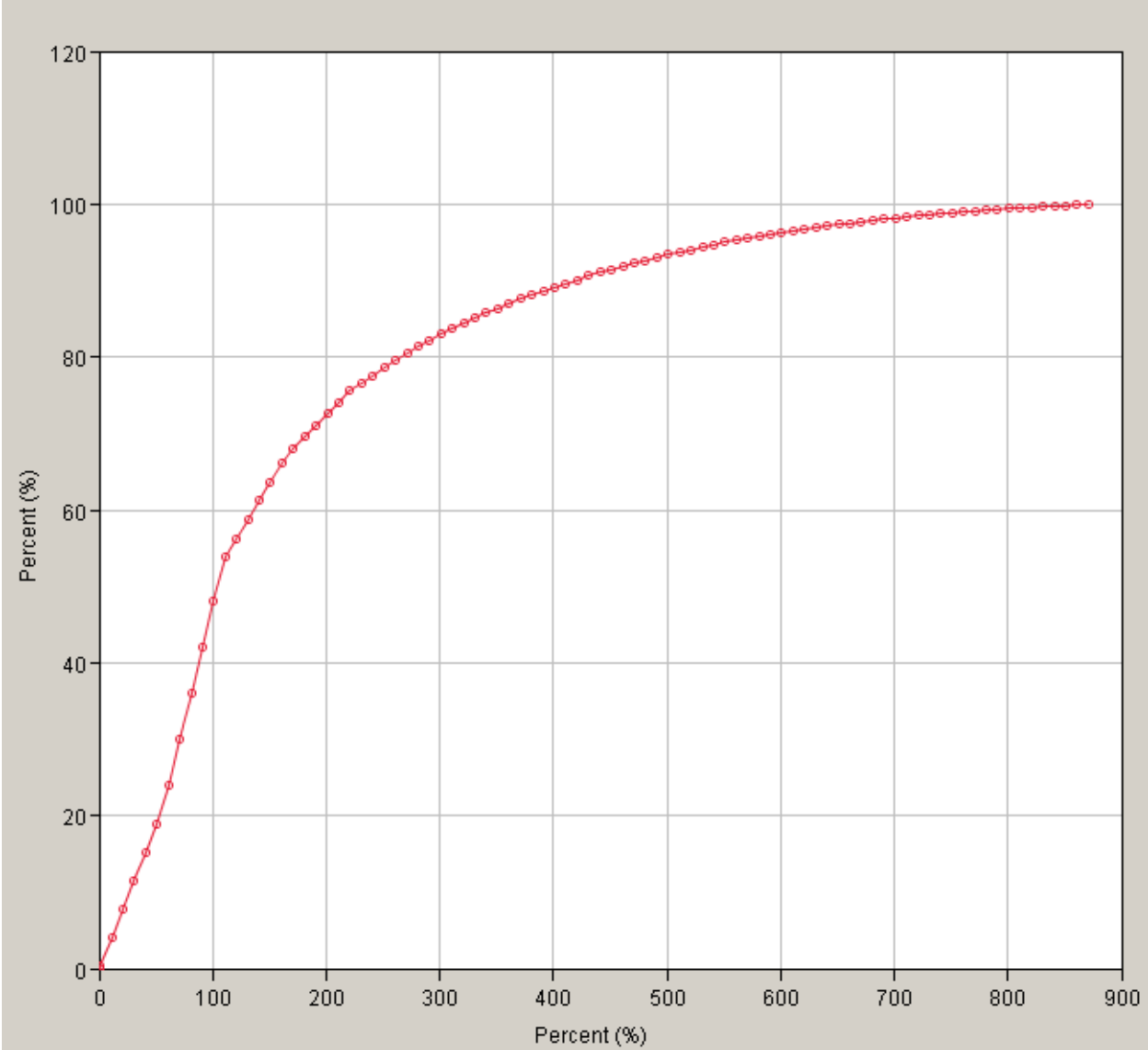
Del Puerto



LA Valley Urban



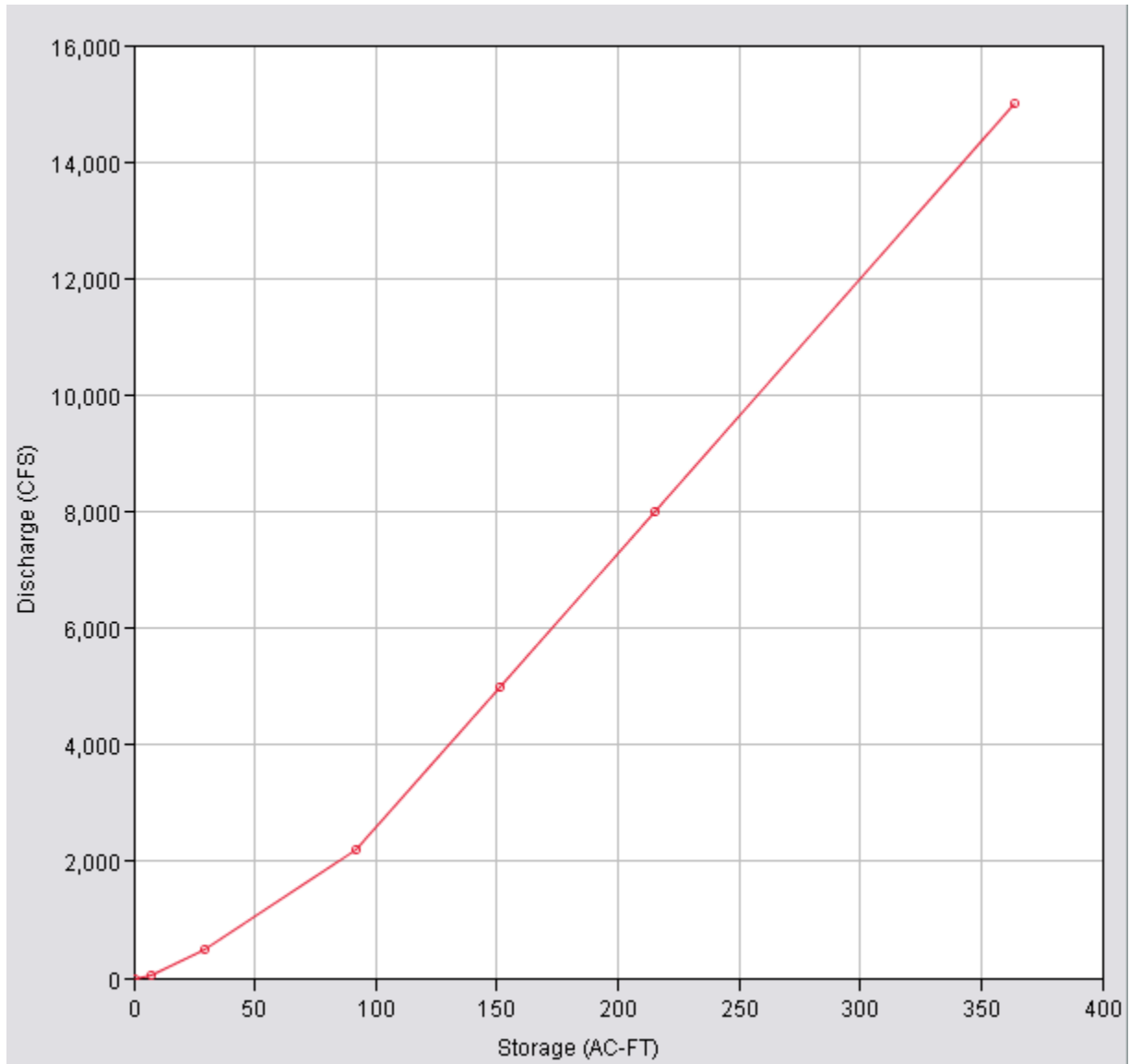
Walnut Creek Mountain Rural



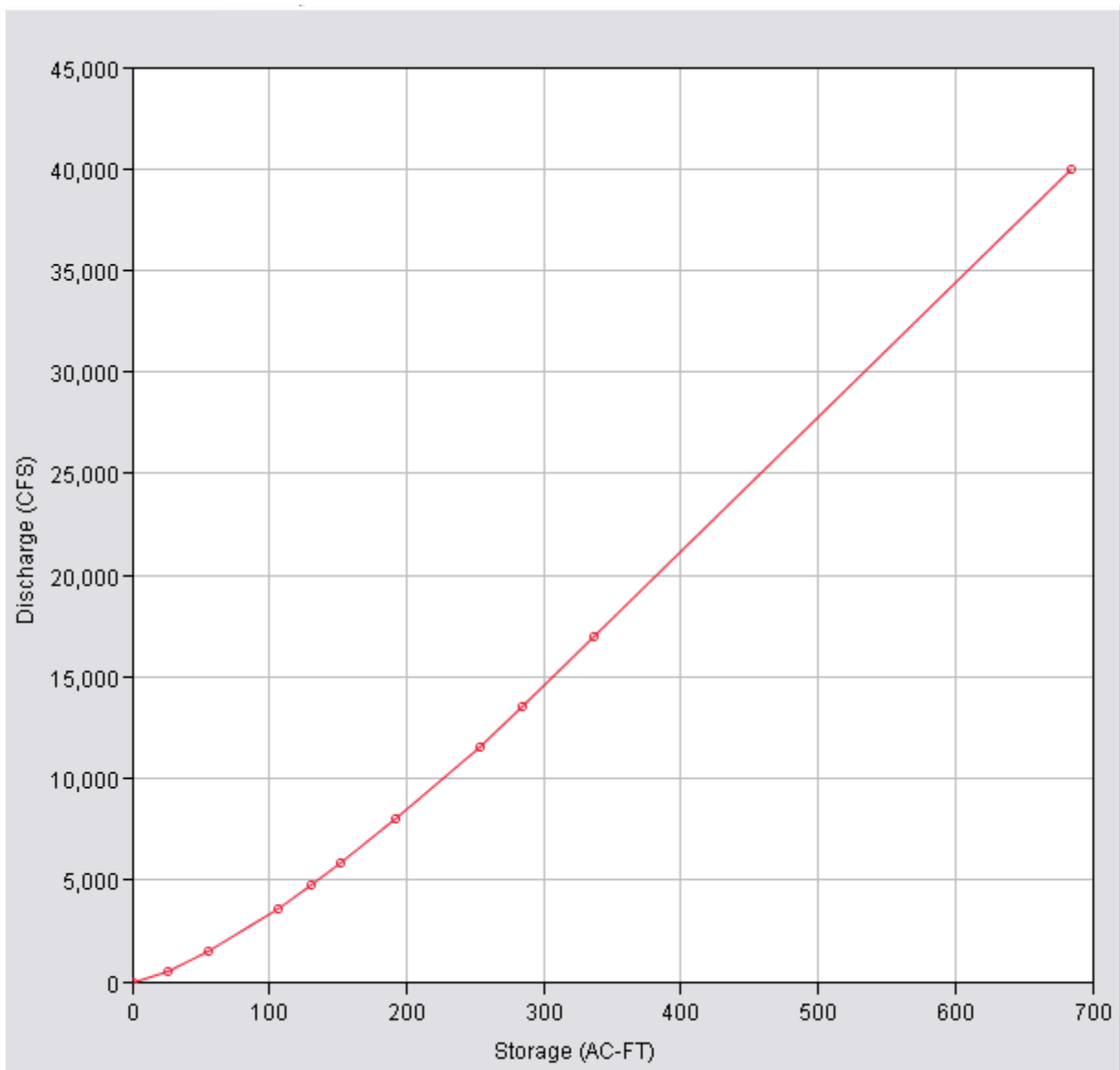
Appendix C

Storage-Discharge Functions for Modified Puls Routing

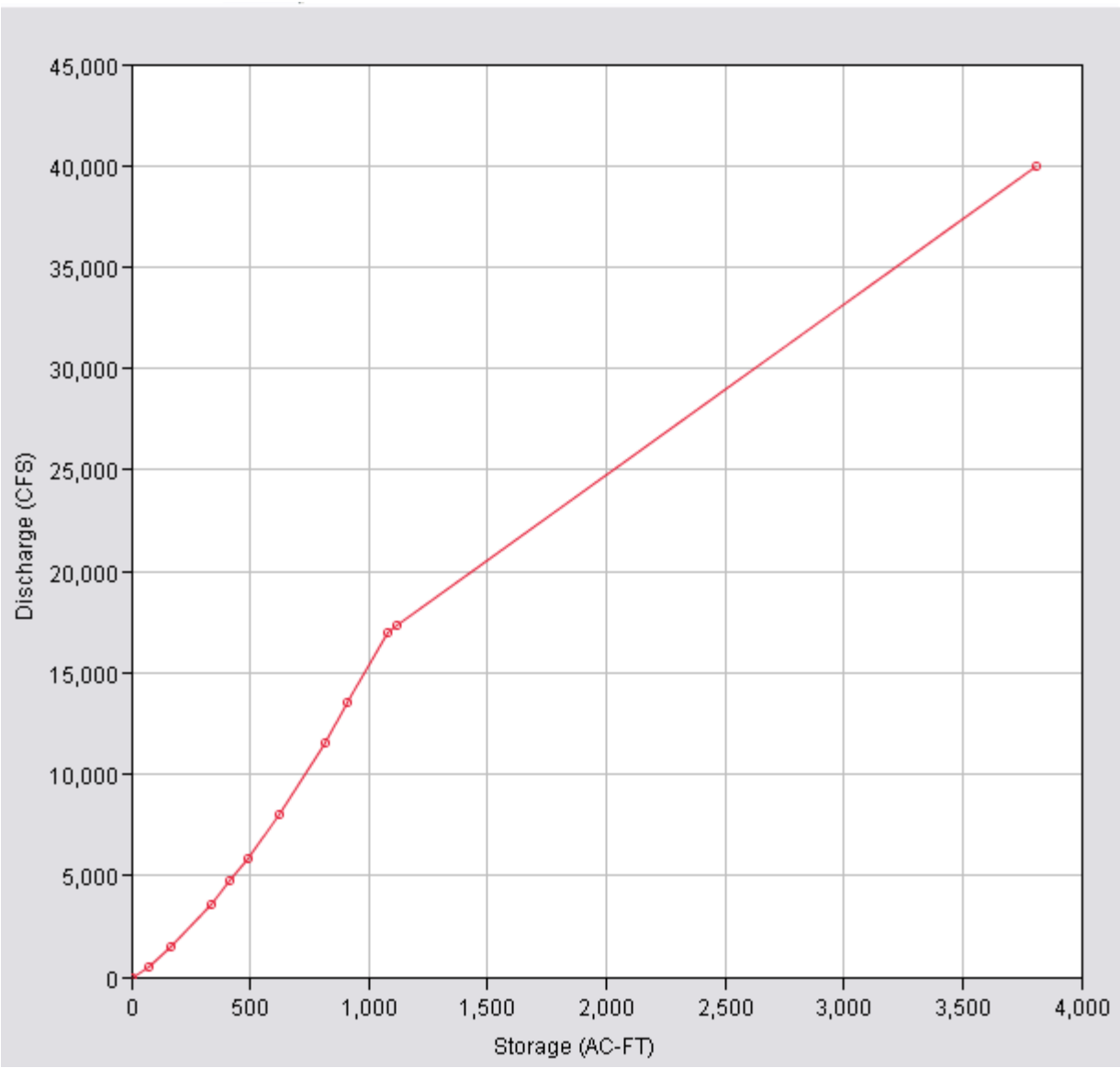
R170 Mod Puls for Reach R170



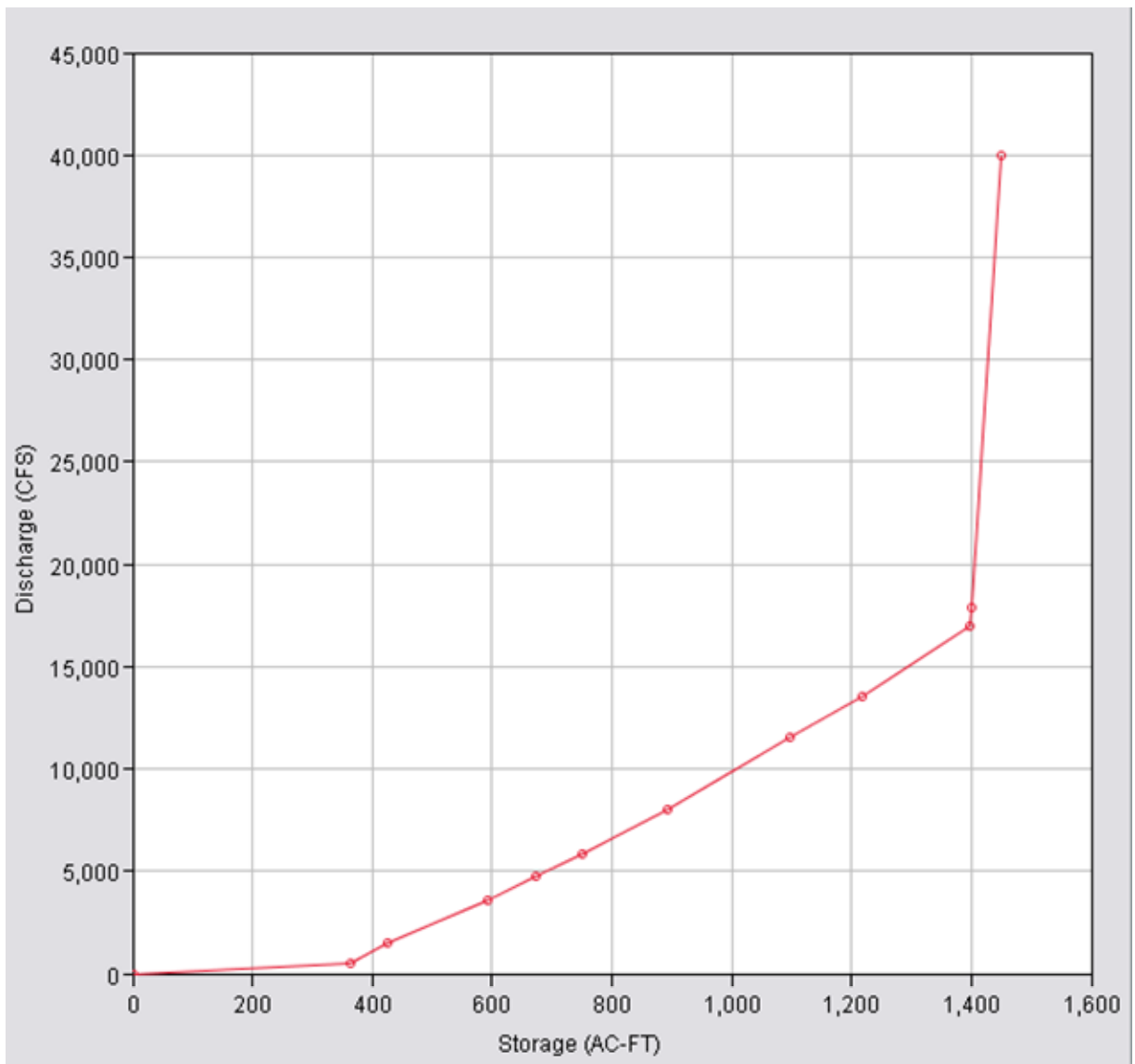
R P TO Q Basin Q R PtoQ Mod puls



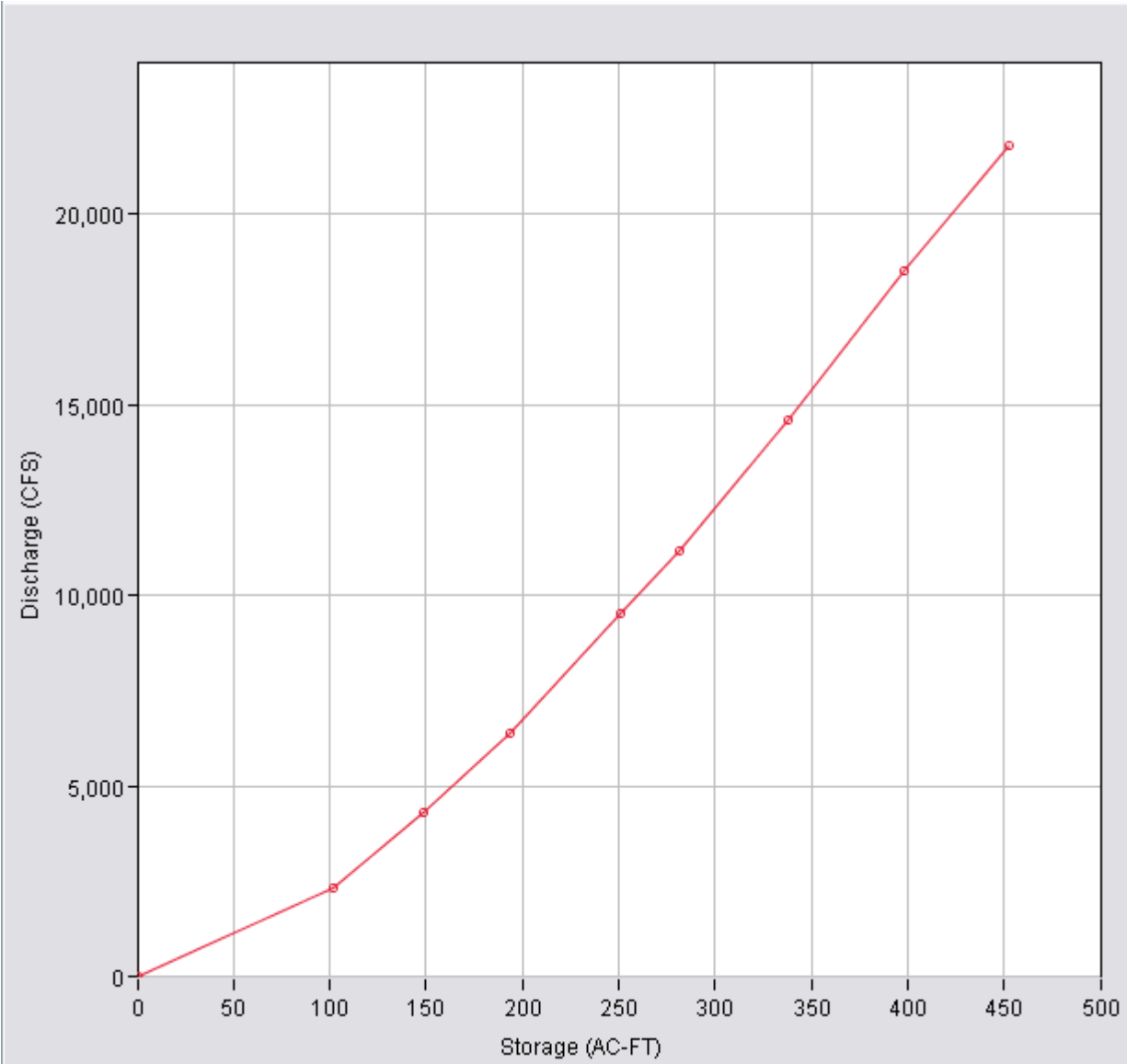
R40 Basin R R40 mod puls



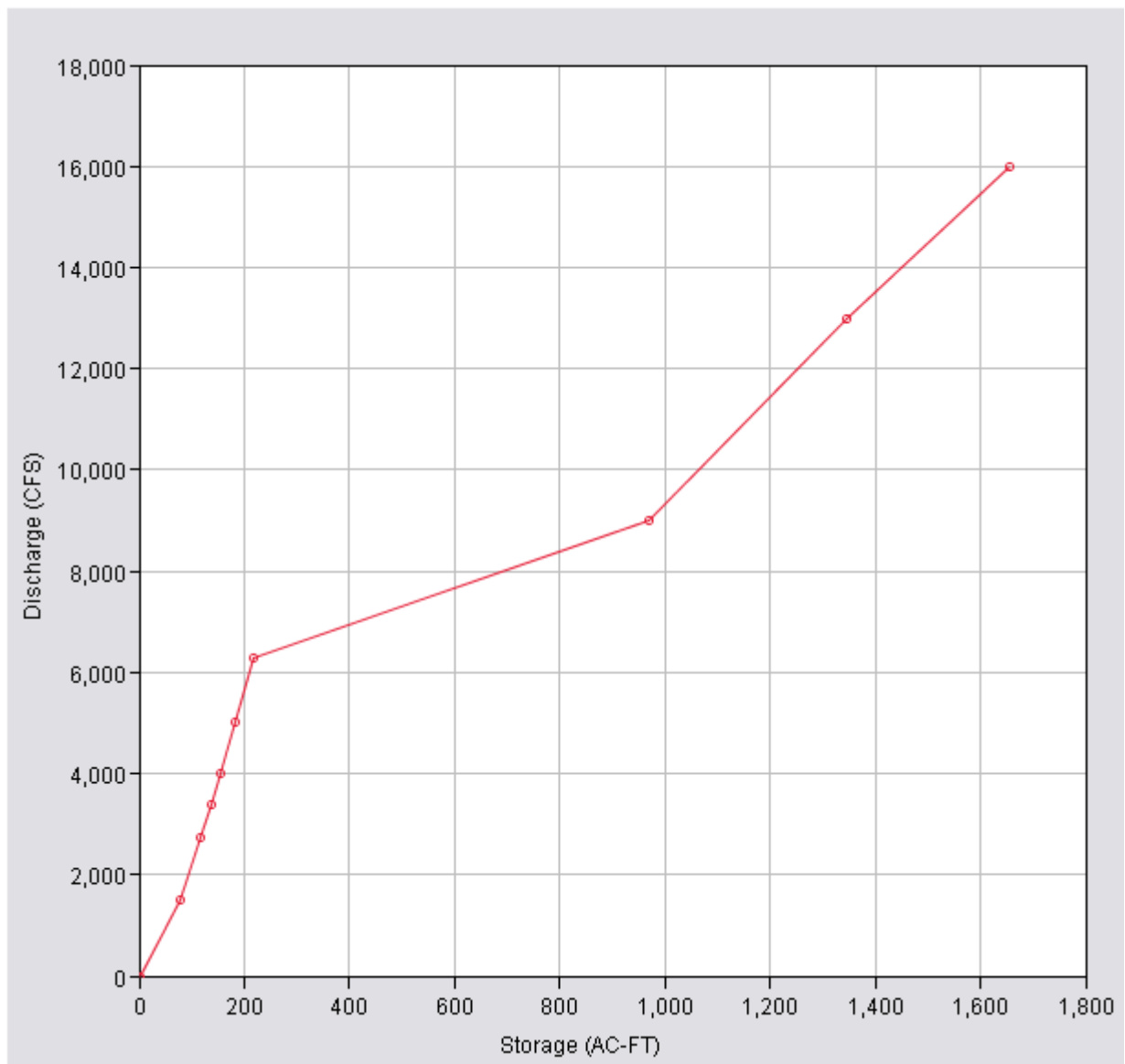
R20 Basin S R 20 Mod puls



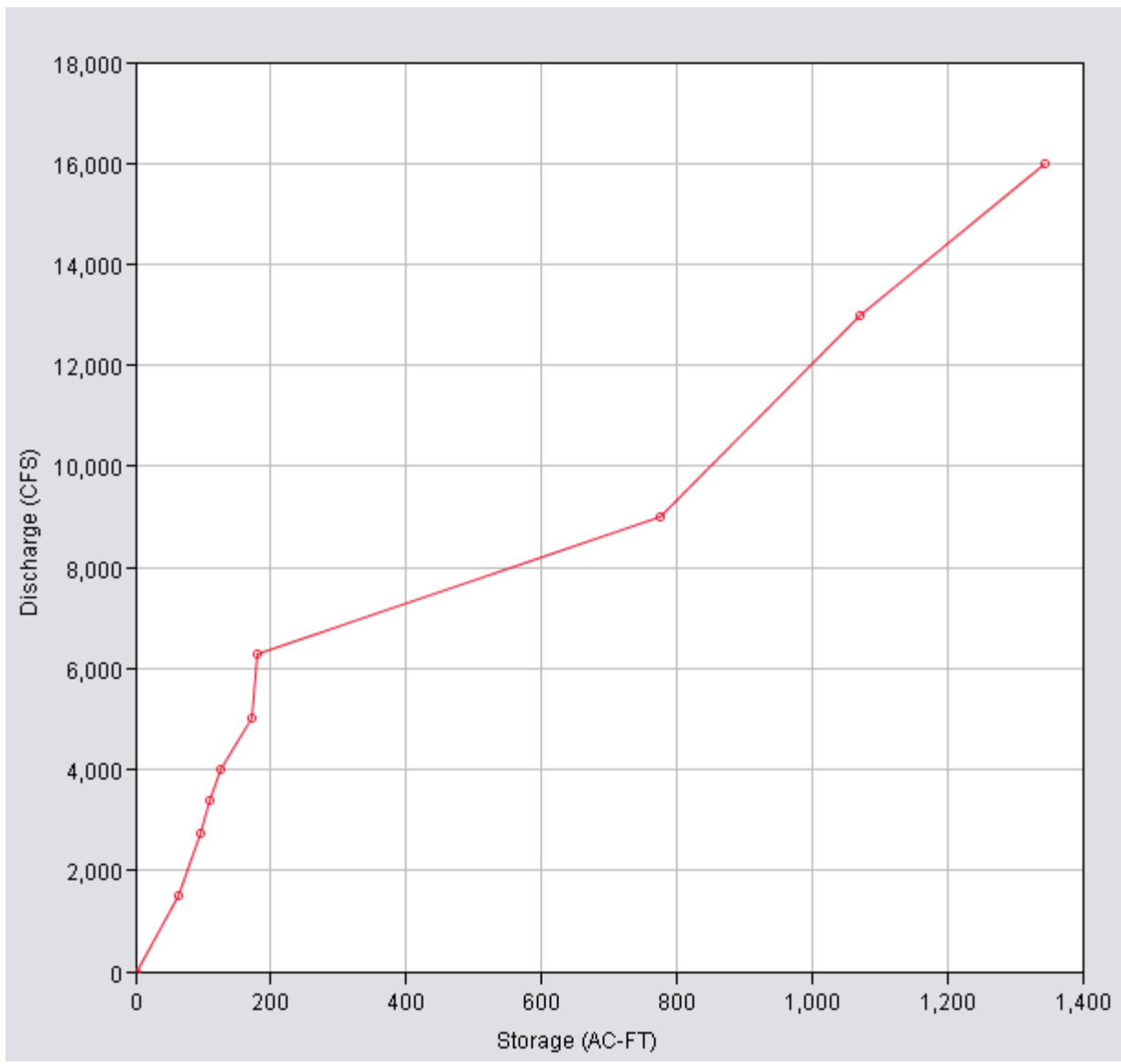
R200 Channel alma to canoas proj



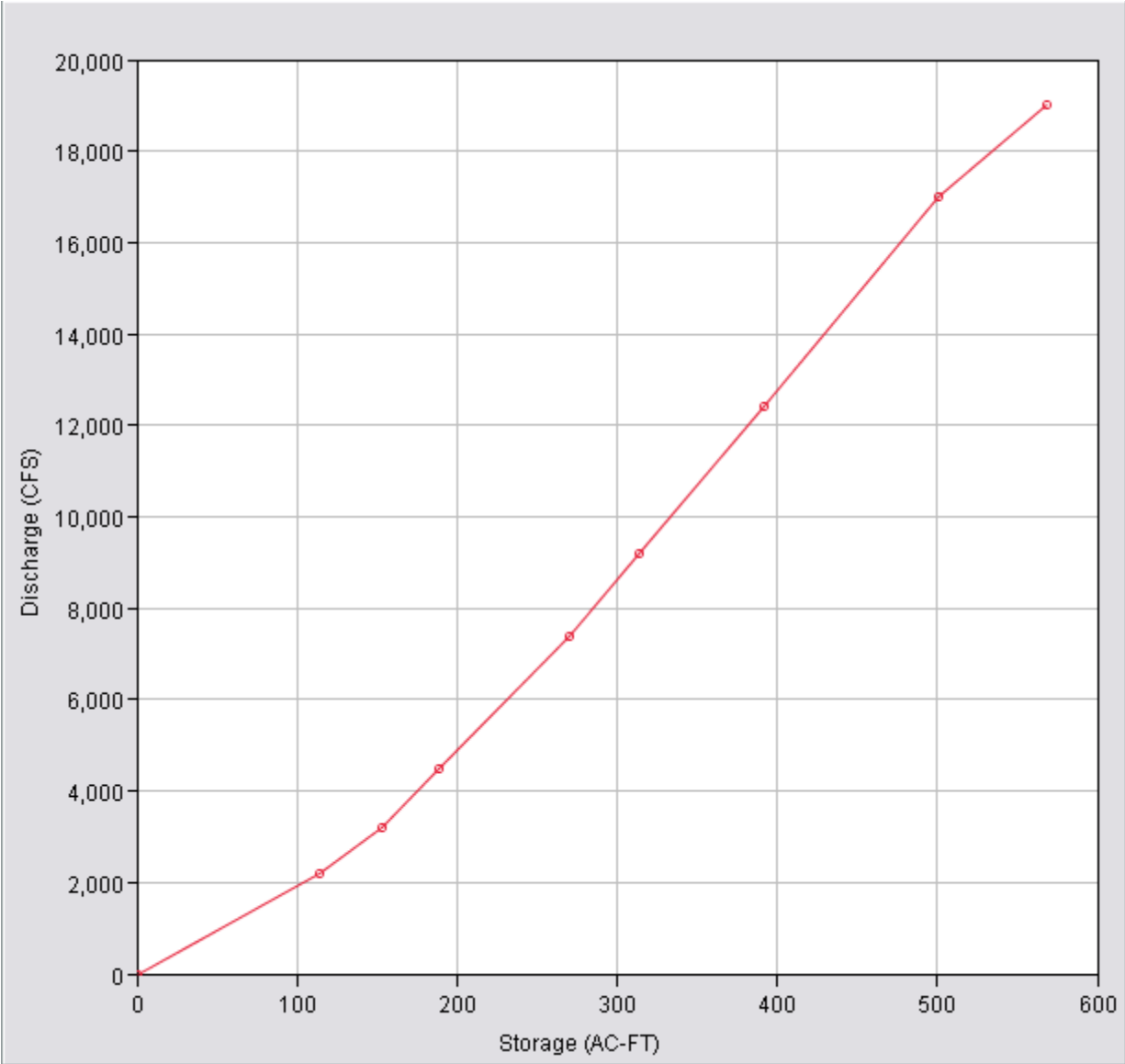
R Hwy 280 to Alma Channel basin N proj



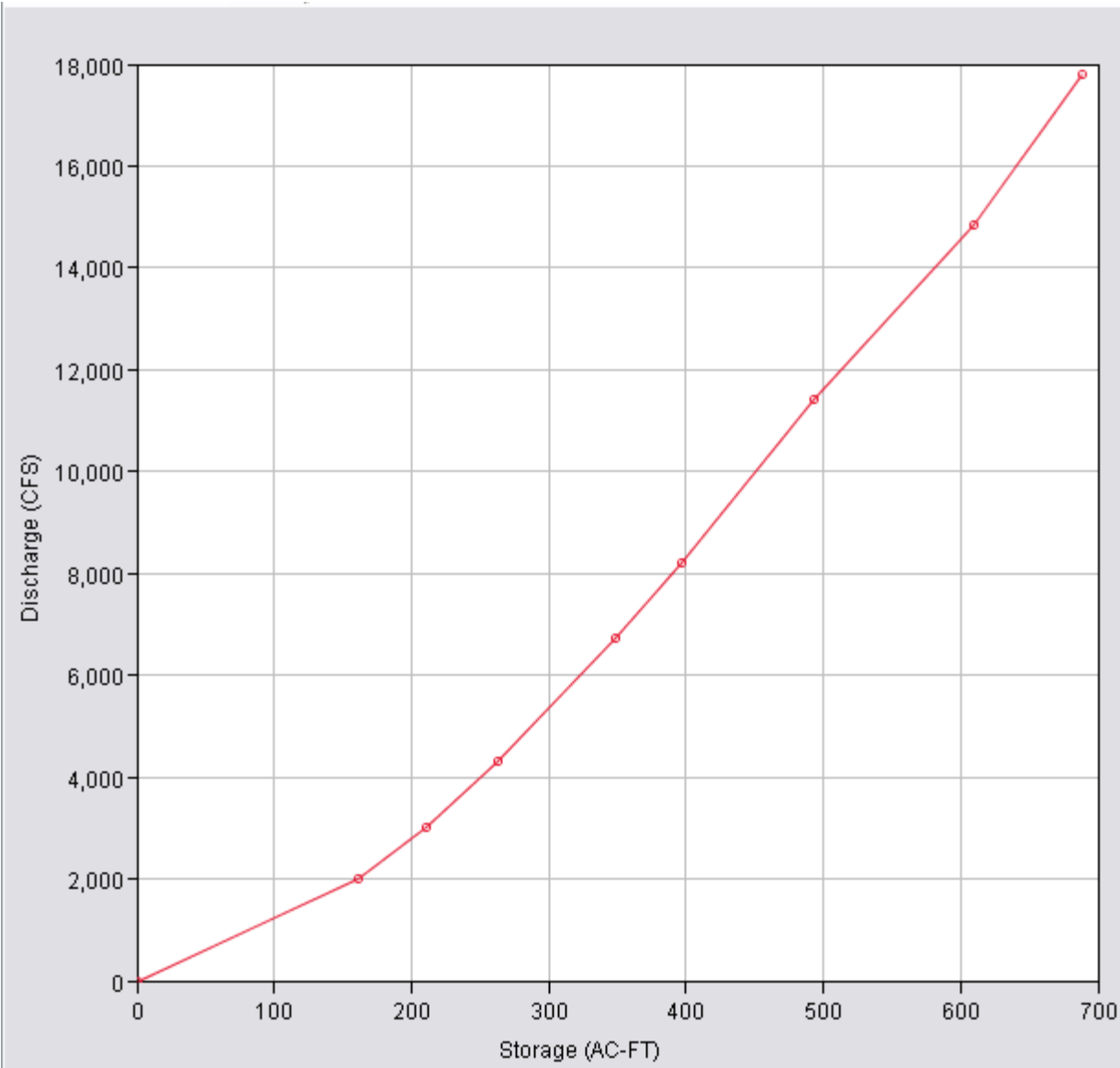
R 880 to LGcreek Channel basin P proj



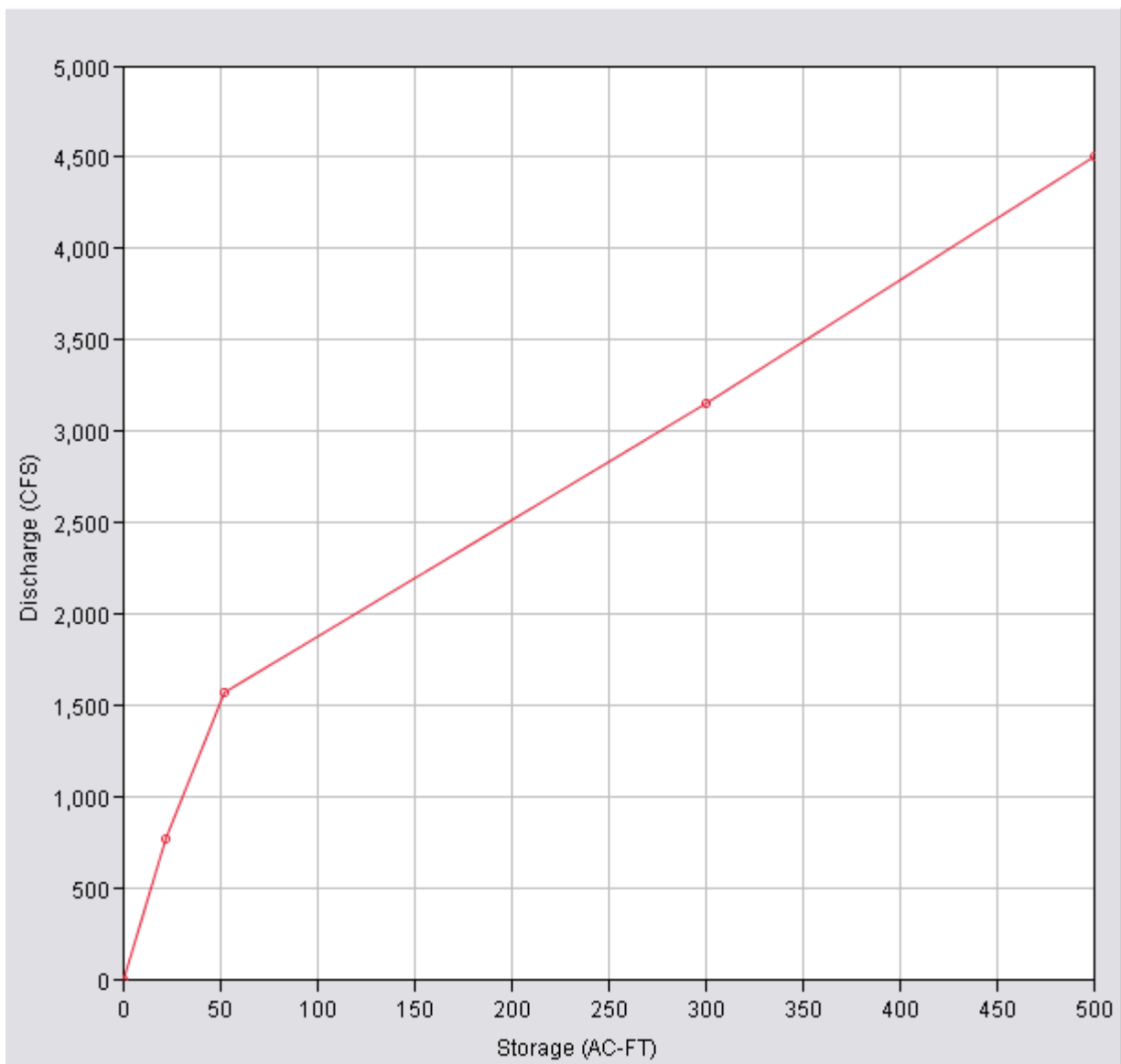
R260 Channel canoas to ross proj



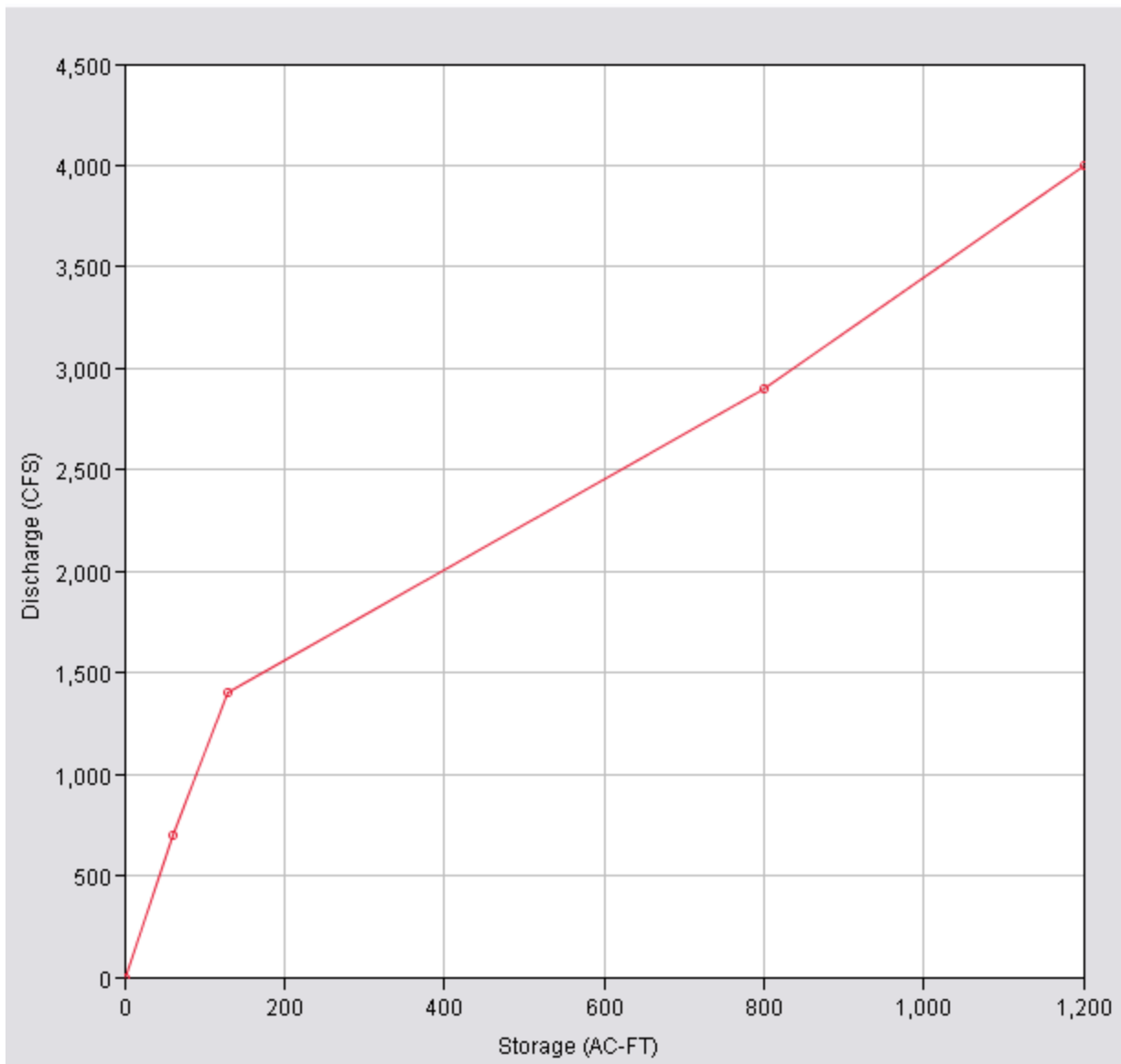
R340 Channel ross to alamos prj



R240 Lower Canoas prj route

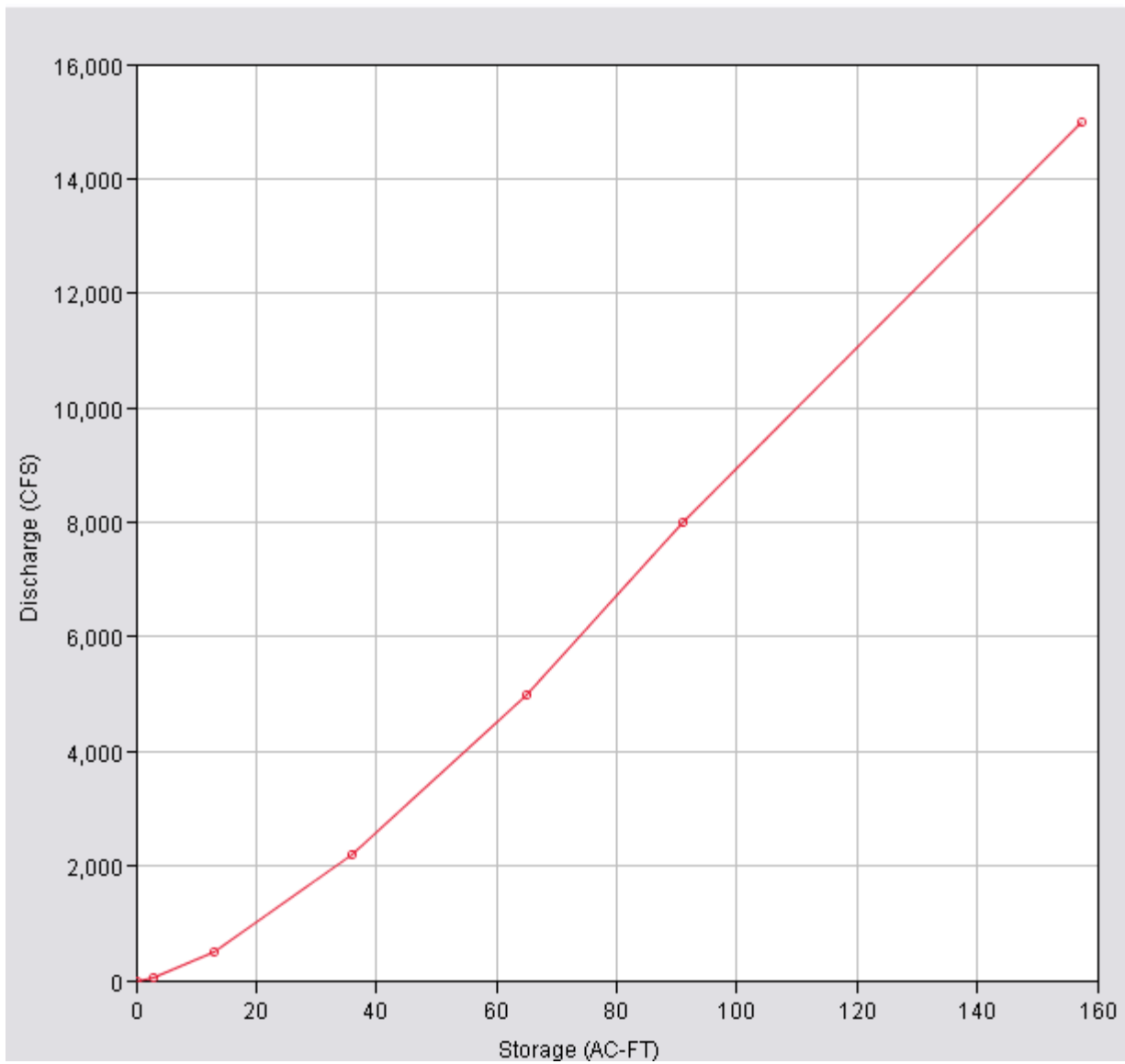


R290 mid canoas project route



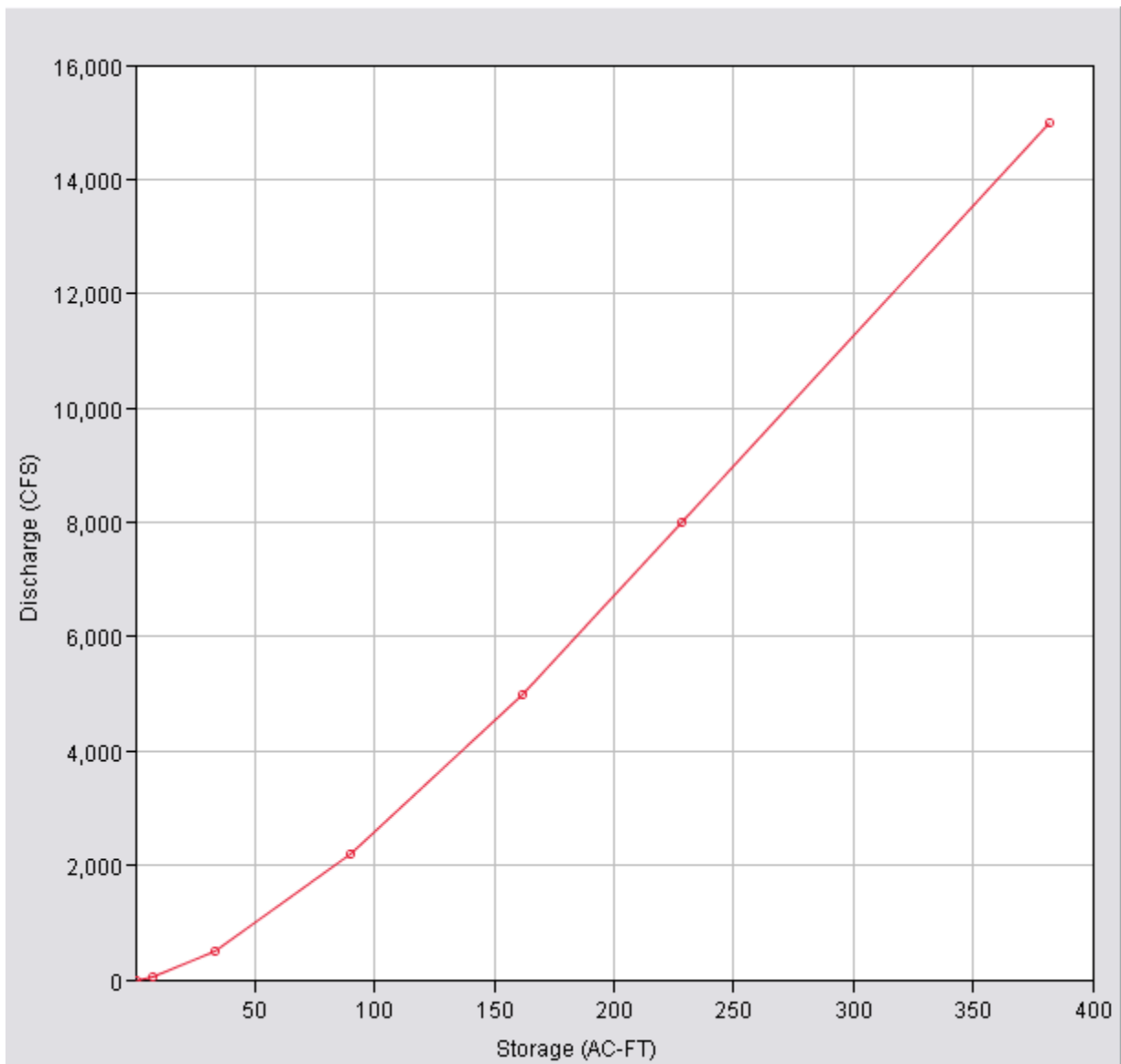
R LG 2

R LG 2 mod puls

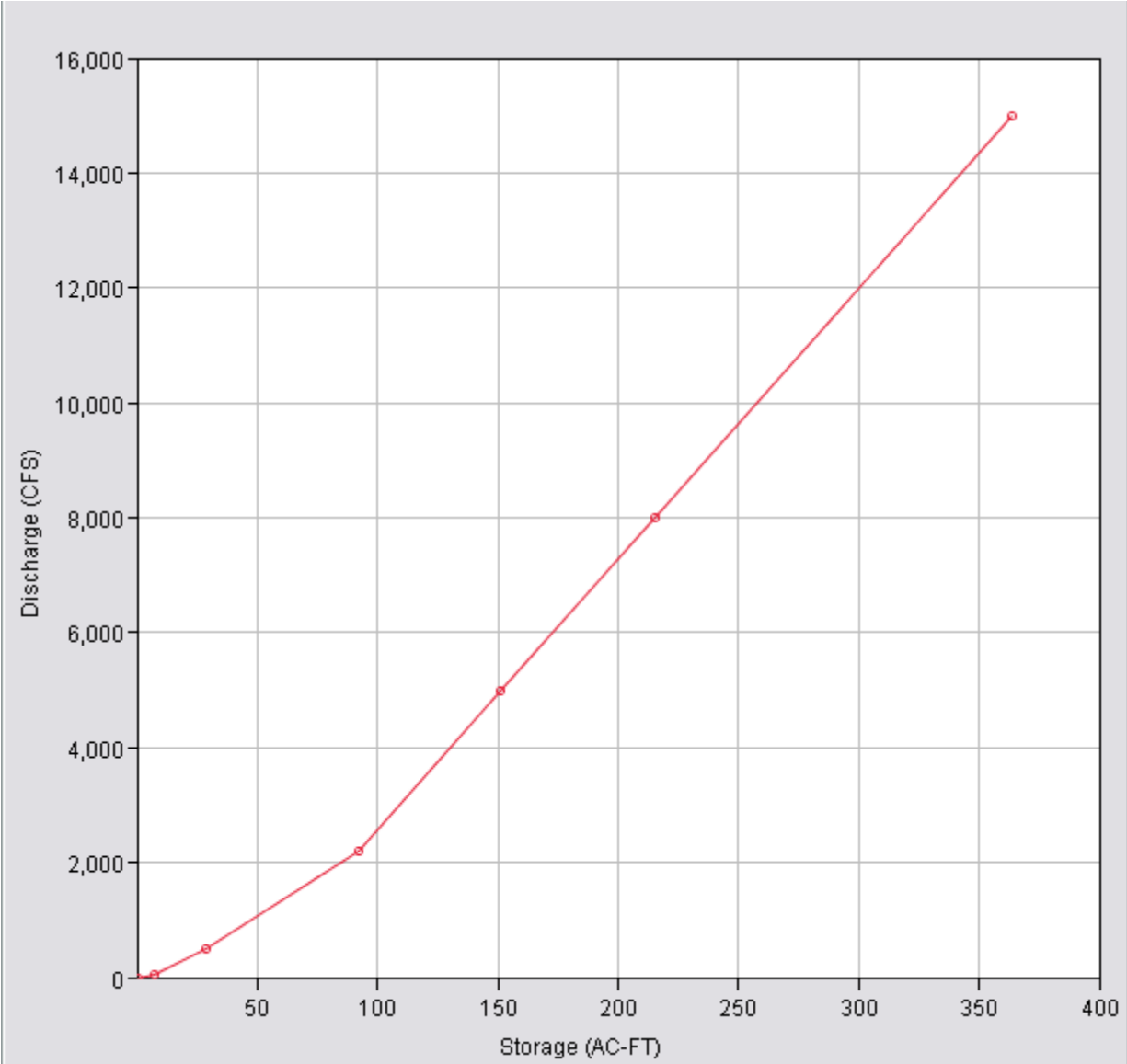


R LG 3

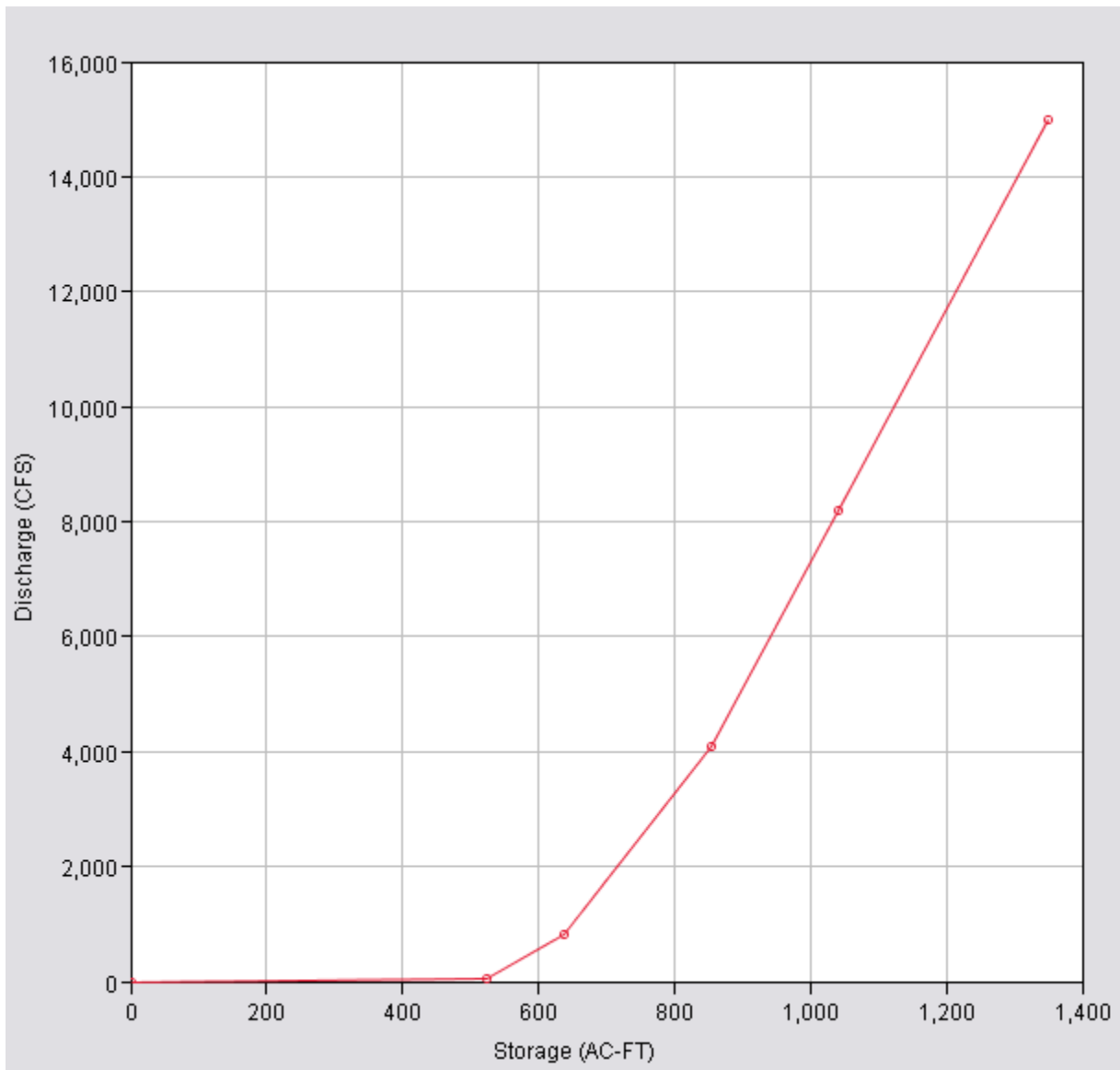
R LG 3 mod puls



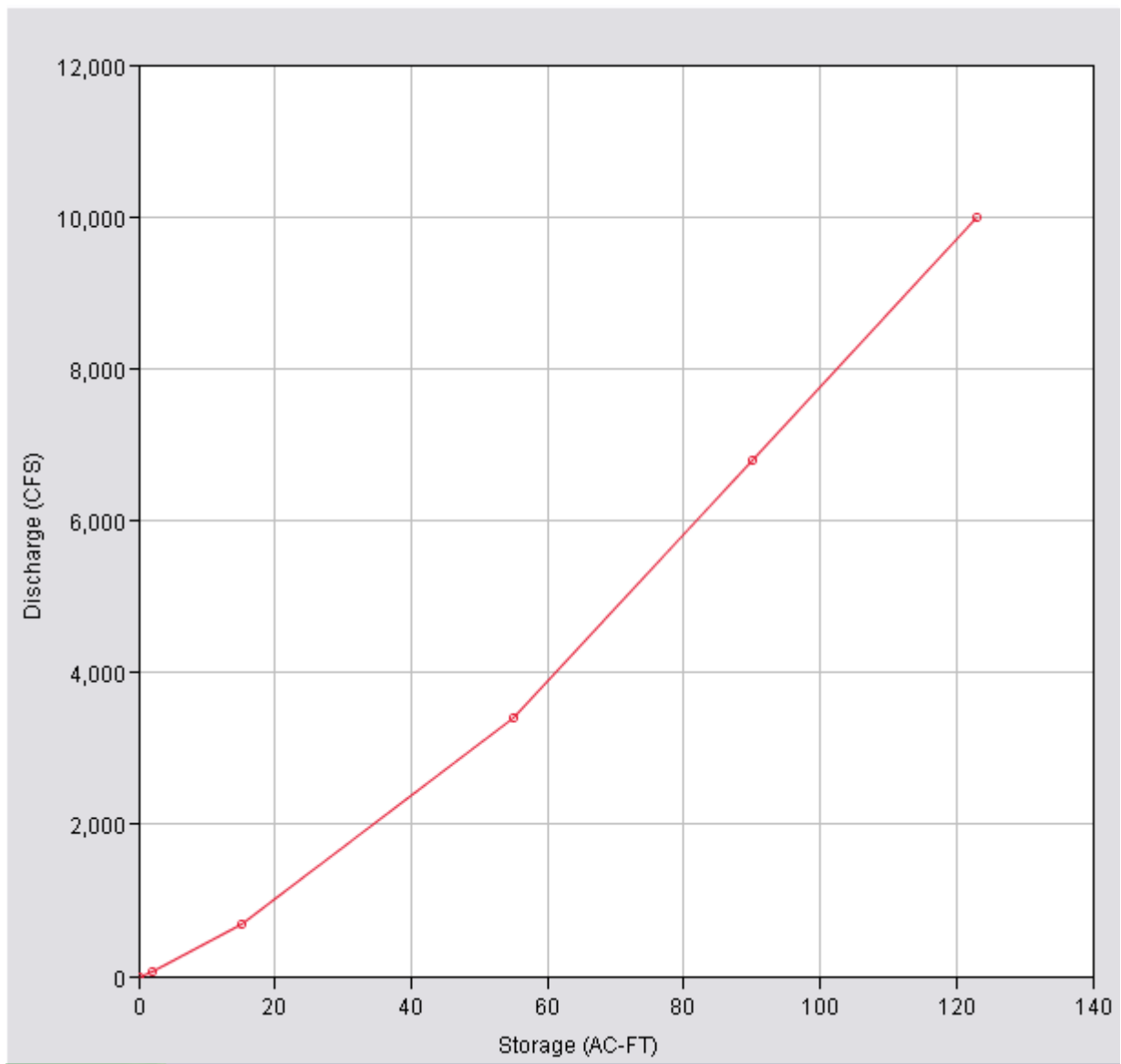
R170 R170 Mod puls



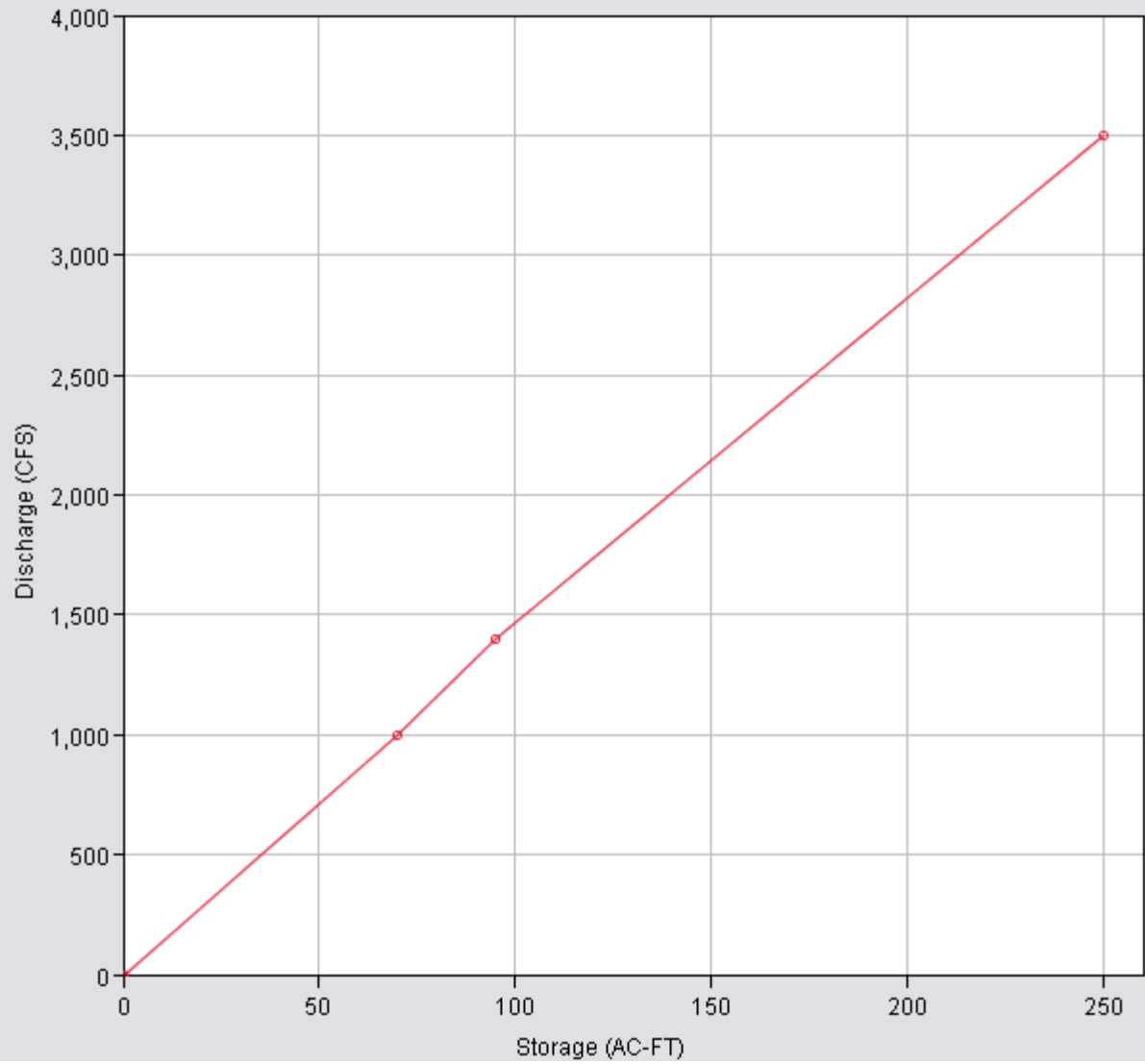
R460 R460 Alamitos Mod Puls



R520 R520 Alamitos Mod Puls



R Lower Ross lower creek project



Appendix D

Documentation of Meteorology Calculations

The purpose of this document is to outline how the 3-day statistical precipitation patterns for the sub-basins were calculated. The following will go through the calculations performed in the Excel spreadsheet created by the Sacramento District which linearly interpolates between the Depth Area Reduction Factors (DARFs) as presented in HMR 59 for various drainage areas, computes the statistical 1, 6, 12, 24, 48, and 72-hour cumulative precipitation for each sub-basin, and creates custom hyetographs.

Depth Area Reduction Factors (DARFs)

	A	B	C	D	E	F	G	H	I	J	K	L	M	
1	CALIFORNIA AREA 3 MIDCOAST MTN, CA --UPDATED MARCH 2006													
2	CHECK NEEDED													
3	HMR RATIOS AND DAD CURVES FROM HMR 58													
4	CALIFORNIA AREA 3 UPDATED August 1999 BY RBC													
5		MIDCOAST MTN, CA					ALL SEASON							
6	HRS	1		6		12		24		48		72		
7	24RATIO	0.130	0.2	0.450	0.49	0.740	0.7	1.000	1.1	1.450	1.6	1.700	0	
8	DA													
9	0.1	1	0	1	0	1	0	1	0	1	0	1	0	
10	10	1	-0.00313	1	-0.00281	1	-0.00250	1	-0.00225	1	-0.00200	1	-0.00175	
11	50	0.875	-0.00115	0.888	-0.00100	0.900	-0.00090	0.910	-0.00080	0.920	-0.00070	0.930	-0.00060	
12	100	0.818	-0.00080	0.838	-0.00055	0.855	-0.00050	0.870	-0.00045	0.885	-0.00040	0.900	-0.00038	
13	200	0.758	-0.00028	0.783	-0.00024	0.805	-0.00023	0.825	-0.00022	0.845	-0.00020	0.863	-0.00019	
14	500	0.675	-0.00014	0.710	-0.00011	0.735	-0.00011	0.760	-0.00011	0.785	-0.00011	0.805	-0.00010	
15	1000	0.608	-0.00008	0.655	-0.00007	0.680	-0.00007	0.705	-0.00006	0.730	-0.00006	0.755	-0.00006	
16	2000	0.530	-0.00005	0.585	-0.00005	0.615	-0.00004	0.640	-0.00004	0.670	-0.00004	0.700	-0.00004	
17	5000	0.380	-0.00003	0.445	-0.00002	0.485	-0.00002	0.520	-0.00002	0.550	-0.00002	0.590	-0.00002	
18	10000	0.250	0	0.340	0	0.380	0	0.420	0	0.450	0	0.490	0	

The Depth Area Reduction Factors (DARFs) from HMR 59 are represented in the Excel spreadsheet as shown above. Columns B, D, F, H, J, and L contain the DARFs for the 1, 6, 12, 24, 48, and 72-hour durations, respectively. The columns in-between (Columns C, E, G, I, K, and M) calculate the incremental change in DARF for a given unit of area (1 square mile). These numbers are used to linearly interpolate between DARF values for drainage areas in-between those specified in column A.

The row labeled “24RATIO” (Row 7) refers to the ratio to 24-hour cumulative precipitation as derived in HMR 59. These ratios are used to convert the 24-hour precipitation (main input parameter) to 1, 6, 12, 48, and 72-hour precipitation for each sub-basin.

For the purpose of this study, the California Area 3 Midcoast Mountain, California DARFs were selected, as noted in Row 1.

Cumulative Precipitation Calculations

1	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ	AK	AL	AM
2	Curve					ENTER		Enter data as indicated below									
3	No.	HMR 58 SIERRA MTN SUBAREA 5	Month diff from all-season (0 to 5)-->			0		<----- Enter data here									
4	Use	TO USE	Off season month ----> (Name, e.g. APR)			All Season		<----- Enter data here									
5	0-5		Seasonal Ratio from fig 13.X, or 2			0		<----- Enter Basion Ratio or enter 2 for Vering basion ratios									
6	0	All season						Other data entry columns are shaded Blue									
7	1	Offset +/- 1 Month For April or October				HMR 58 SIERRA MTN SUBAREA 5		<div style="border: 1px solid black; padding: 2px;"> If the off season month subbasin ratios vary from basin to basin enter in this column. </div> <div style="border: 1px solid black; padding: 2px; margin-top: 5px;"> If all subbasins have the same ratio value in cell 'AB5' is used. </div>									
8	2	Offset +/-2 Months For May Or September				Guadalupe River											
9	3	Offset +/- 3 Months For June Or August				All Season											
10	4	Offset +/- 4 Month July				All season											
11	5	Offset + or - 5 months None in Sierra Mtn. Regon				Upper Guadalupe Centering											
12	6	Multi Region Curves															
13							24-HR			24-HR		24-HR	24-HR	24-HR	1-HR	1-HR	1-HR
14	RANK		Station ID			Drainage	Off-	Off-	Accum-	24-HR	24-HR	24-HR	24-HR	24-HR	1-HR	1-HR	1-HR
15	NO.					Area	Season	Season	ulated	SINGLE	Accum-	Average	Ppt	Ppt	Ppt	Adjusted	
16		Guadalupe River				Ppt	Variable	Month	Ppt	Drainage	BASIN	Depth,	DA			concurrent	
17		Upper Guadalupe Centering					Ratio	1 for all		Area (DA)	VOLUME	Accum-				DA	
18							1 for all	season				ulated					
19								1				BASIN			Ratio for 1HR		
20												VOLUME			0.130		
21															Adjusted for DA		
22			A=Part	B=PART													
23		Sub-Basin Description			sq.mi.	in.				in.	sq.mi.			in.	in.	in.	
24	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
25																	
26	1	Upstream from Guadalupe Reservoir	W1730		4.3	9.58	1	1.00	9.58	4.3	41.50	41.50	9.58		1.24	1.24	1.24
27	2	Upstream from Guadalupe Reservoir	W1970		1.6	9.47	1	1.00	9.47	6.0	15.32	58.82	9.55		1.23	1.24	1.23
28	3	Alamitos Creek	W1480		2.3	7.00	1	1.00	7.00	8.3	16.10	72.92	8.84		0.91	1.15	0.91
29	4	Upstream from Almaden Reservoir	W1850		11.9	9.87	1	1.00	9.87	20.1	117.11	190.03	9.45		1.28	1.19	1.22
30	5	Downstream from Almaden Reservoir	W1540		4.3	7.16	1	1.00	7.16	24.5	31.07	221.10	9.04		0.93	1.12	0.81
31	6	Upstream from Lake Elzman	W1950		9.7	11.47	1	1.00	11.47	34.2	111.80	332.90	9.73		1.49	1.17	1.29
32	7	Upstream Almaden Lake Alamitos Creek	W1400		6.5	7.42	1	1.00	7.42	40.7	48.48	381.36	9.36		0.96	1.10	0.74
33	8	Downstream from Guadalupe Reservoir	W1960		6.7	9.28	1	1.00	9.28	47.5	62.50	443.86	9.36		1.21	1.07	0.91
34	9	Upstream from Lexington Reservoir	W1800		2.4	9.69	1	1.00	9.69	49.9	23.47	467.33	9.37		1.26	1.07	0.92
35	10	Below Almaden Res	Below Almaden		1.7	6.62	1	1.00	6.62	51.6	11.05	478.38	9.28		0.86	1.05	0.67

Fields shaded in light blue represent cells in which user input is possible or required.

Row 3, Column AB is populated with a “0” to represent “all-season” conditions. Seasonal variations in precipitation patterns are not accounted for in this study.

Row 26 and below:

- Column W is populated with the sub-basin “rank number”. Each sub-basin is assigned a rank, starting with the storm center and ending at the mouth of the basin.
- Column X is populated with a description of each sub-basin.

- Column Z includes the sub-basin ID, as used in the HMS model.
- Column AB includes the individual drainage area of each sub-basin.
- Column AC is populated with the calculated 24-hour cumulative precipitation for each sub-basin given a particular event frequency (i.e. 2,5,10,25, 50, 100, 200 or 500-year).
- Columns AD and AE are set at 1, for “all seasons.”
- The remaining Columns (not shaded in light blue) conduct various calculations on the inputted data.
 - Column AF remains the same as Column AC for “all seasons.”
 - Column AG calculates the cumulative drainage area. This is done by summing up all of the drainage areas at a given sub-basin (rank x) from rank 1 to rank x. This number is used to estimate the drainage area upstream of a particular sub-basin. This number is used in selecting an appropriate set of DARFs for a given sub-basin. For example, for sub-basin W1850 with a “rank number” of 4, the cumulative drainage area is 20.1. The DARFs are then calculated using the table previously presented using a linear interpolation between the DARFs for 10 square miles and the DARFs for 50 square miles.
 - Column AH calculates the volume of water allotted to a given sub-basin during the maximum 24-hour duration.
 - Column AI calculates the cumulative volume of water at a given sub-basin (rank x) from rank 1 to rank x. This is used to make sure that the volume of water over the entire basin is conserved.
 - Column AJ calculates the average depth of water for the cumulative drainage area above a given sub-basin. This is calculated by dividing the cumulative water volume (Column AI) by the cumulative drainage area (Column AG). This number is used in subsequent calculations to estimate the specific and concurrent precipitation at a given index location (at the outlet of a given sub-basin) with respect to the entire drainage area above the index location.
 - Columns AK through AM conduct calculations for the 1-hour duration that are subsequently carried out for the remaining durations (6, 12, 24, 48, and 72-hours) to estimate the “specific” and “concurrent” precipitation:
 - Column AK uses the ratio to 24-hour precipitation, 0.130 (see previous table), to estimate the maximum cumulative 1-hour specific precipitation.

- Column AL represents an intermediary calculation necessary to calculate the concurrent precipitation. This field multiplies the average depth for the cumulative drainage area above a given sub-basin (Column AJ) by the ratio to 24-hour precipitation (0.130), and applies the appropriate DARF using the aforementioned table.
- Column AM calculates the concurrent cumulative precipitation by subtracting the product of the accumulated drainage area (Column AG) and Column AL from the previous sub-basin (by rank) from the product of Column AG and Column AL of the given sub-basin, and dividing this difference by the drainage area of the given sub-basin. This calculates the precipitation depth for a particular sub-basin while taking into account its placement in the larger basin area with respect to the storm center. In other words:

$$PptCon10 = (CumDA10 * PptSp10 - CumDAR9 * PptSp9) / DA10$$

PptCon10 = Concurrent cumulative precipitation for a sub-basin of a given rank (say, rank = 10).

CumDA10 = Cumulative Drainage Area of the sub-basin (rank = 10)

PptSp10 = Specific cumulative precipitation for the sub-basin, previously calculated using the average rainfall depth over all of the previous sub-basins (ranks 1 through 10).

CumDAR9 = Cumulative drainage area for the previous sub-basin (rank = 9).

PptSp9 = Specific cumulative precipitation for the previous sub-basin (rank = 9), previously calculated.

DA10 = Individual drainage area for the sub-basin (rank = 10).

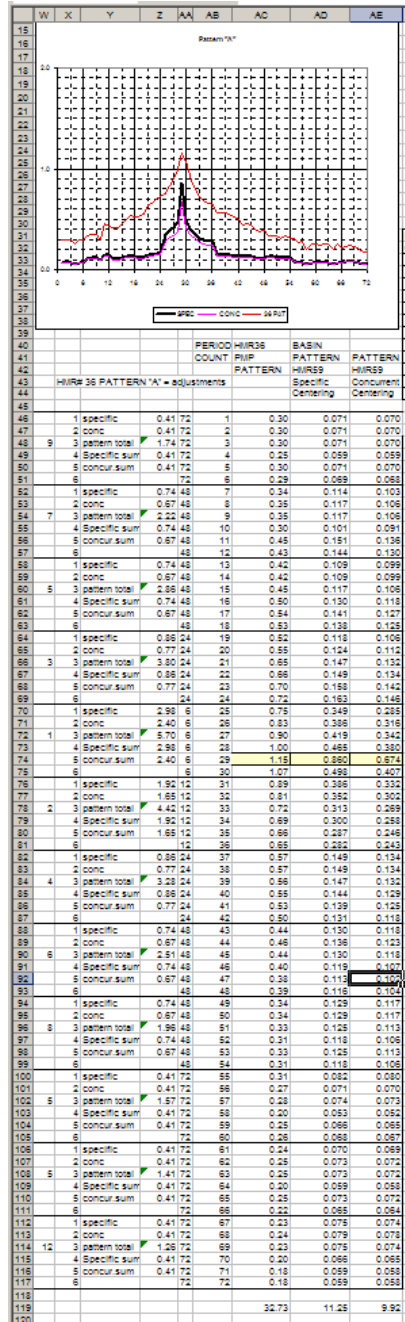
6-Hour Average Precipitation

	BG	BH	BI	BJ	BK
49	Below Almaden Res				
50		HMR59		HMR59	
51	PERIOD	Specific	AVERAGE	Concurrent	AVERAGE
52		Centering	6-HR	centering	6-HR
53			PERIODS		PERIODS
54	1-HR	0.86	5.16	0.67	4.04
55	6HR-1HR	2.12		1.73	
56	6-HR	2.98	2.98	2.40	2.40
57	12HR-6HR	1.92	1.92	1.65	1.65
58	24HR-12HR	1.72	0.86	1.54	0.77
59	48HR-24HR	2.98	0.74	2.69	0.67
60	72HR-48HR	1.65	0.41	1.63	0.41

These calculations will be used in distributing the calculated 1, 6, 12, 24, 48, and 72-hour specific and concurrent rainfall over the chosen precipitation pattern (“Pattern A”).

- Columns BH and BJ present the previously calculated specific and concurrent rainfall depths, and various differences between depths.
- Columns BI and BK calculate the average 6-hour rainfall of a given period (when applicable, no 6-hour average calculated for Row 55).

Creating Precipitation Patterns

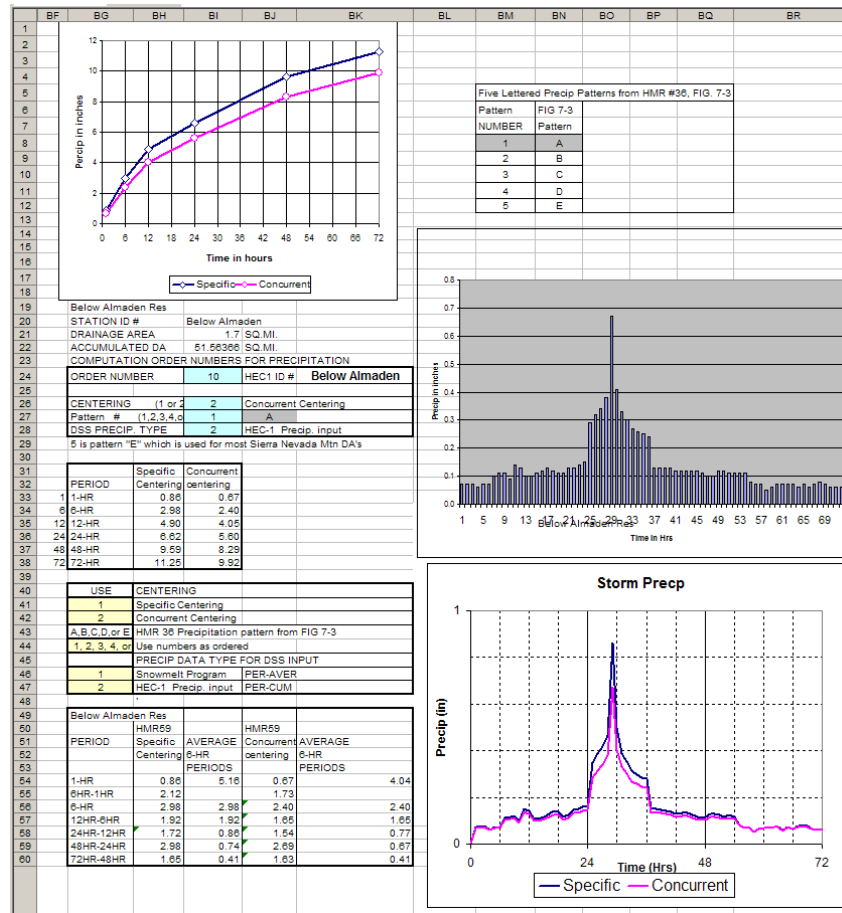


The specific and concurrent precipitation depths for the 1, 6, 12, 24, 48, and 72-hour durations are used to create a unique rainfall pattern for each sub-basin based on a selected pattern. For the purpose of this study, “Pattern A” from HMR 59 was selected, as the critical 1, 12, and 24-hour cumulative precipitation is distributed in a fashion similar to large storms in and around the study area. All of the patterns presented in HMR 59 were taken from recorded storm events throughout California.

The calculations presented in this table distribute the specific and concurrent precipitation for the 1, 6, 12, 24, 48, and 72-hour durations while maintaining the shape of the selected pattern. The rainfall pattern (Pattern A) is broken up into 6-hour increments; the 6-hour average precipitation values previously calculated for various durations are used to estimate the precipitation for each time-step.

- Specific and concurrent rainfall is distributed based average precipitation estimated for a given 6-hour increment. Within a given 6-hour increment, Column Z contains several calculations:
 - Specific and Concurrent: pulls numbers from the 6-Hour Average Precipitation Table (presented previously) depending on which duration is represented in a given 6-hour time increment.
 - Pattern Total, Specific Sum, and Concurrent Sum: Cumulative rainfall for a given 6-hour increment from Columns AC, AD, and AE, respectively.
- Column AA signifies which data from the “6-hour precipitation” table will be used in the calculations for a given 6-hour increment. For example:
 - The beginning of the rainfall pattern (Rows 46 and 51) is characterized by the rainfall between the maximum 48-hour and 72-hour precipitation pattern. As a result, the values in Row 60 of the “6-hour precipitation” table (72HR-48HR) are used to estimate the average “specific” and “concurrent” precipitation.
 - The portion of the hyetograph between Rows 70-75 represents the maximum 6-hour pattern. As a result, the values in Row 56 of the “6-hour precipitation” table (6-HR) are used to estimate the average specific and concurrent precipitation.
- Column AC presents the selected hyetograph (Pattern A).
- Column AD displays the calculated specific hyetograph for a given sub-basin.
- Column AE displays the calculated concurrent hydrograph for a given sub-basin.
- The hyetograph displayed in Column AE is the primary output from this spreadsheet. This hyetograph can be copied and pasted directly into the meteorologic model in HMS for a particular sub-basin. In this example the output for sub-basin “Below Almaden”, rank = 10, is displayed.

Spreadsheet Output



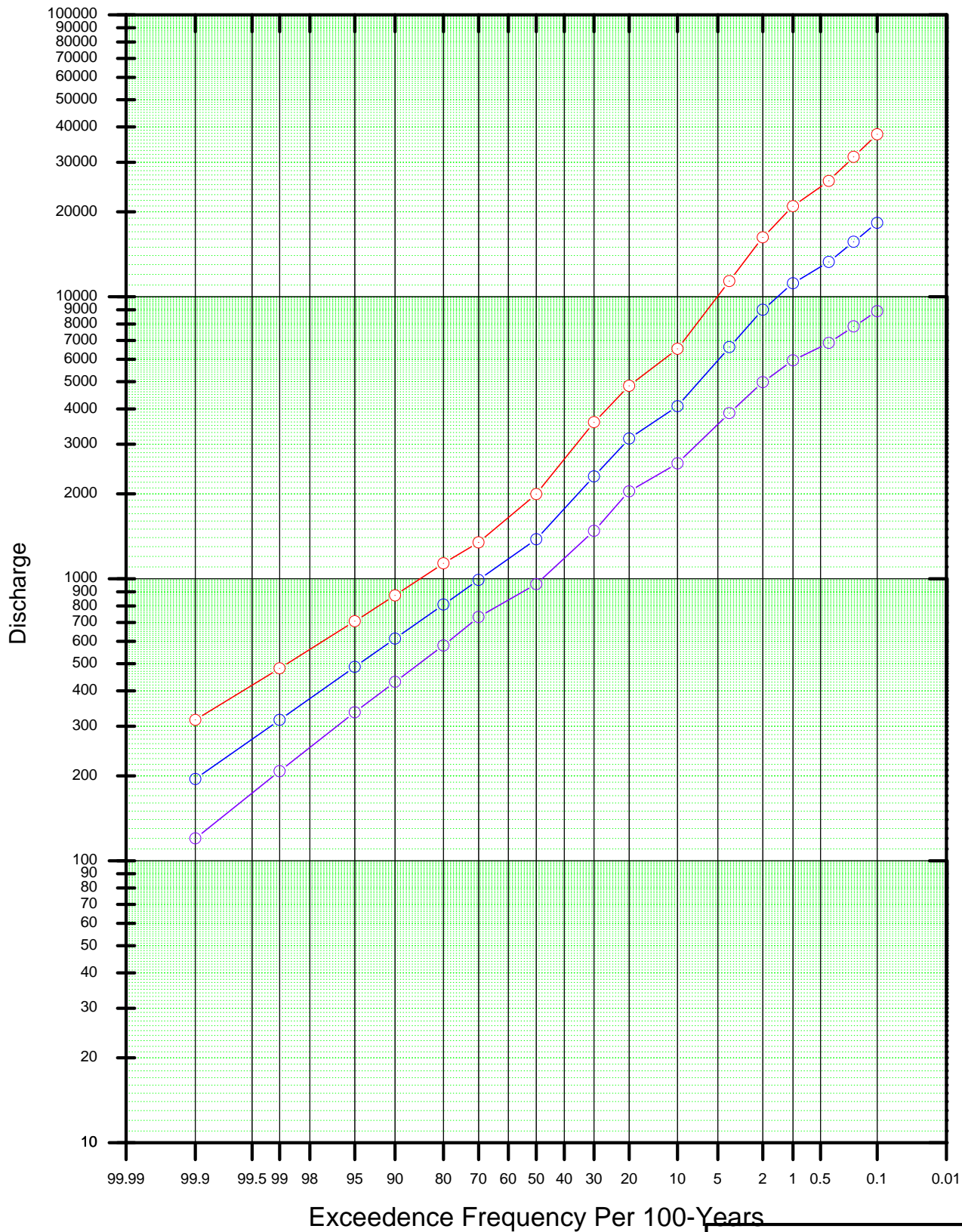
The spreadsheet calculates the specific and concurrent hyetographs for a given sub-basin by specifying a particular “order number” or “rank” in Column BI, Row 24.

Concurrent centering is selected by placing the number “2” in Column BI, Row 26 (“1” is for specific centering only).

The rainfall pattern (Pattern A, B, C, D, or E) is selected by specifying 1, 2, 3, 4, or 5 (respectively) in Column BI, Row 27.

Once a desired hyetograph has been created, the spreadsheet is able to conduct various graphical representations and comparisons of the specific and concurrent precipitation patterns.

Appendix E

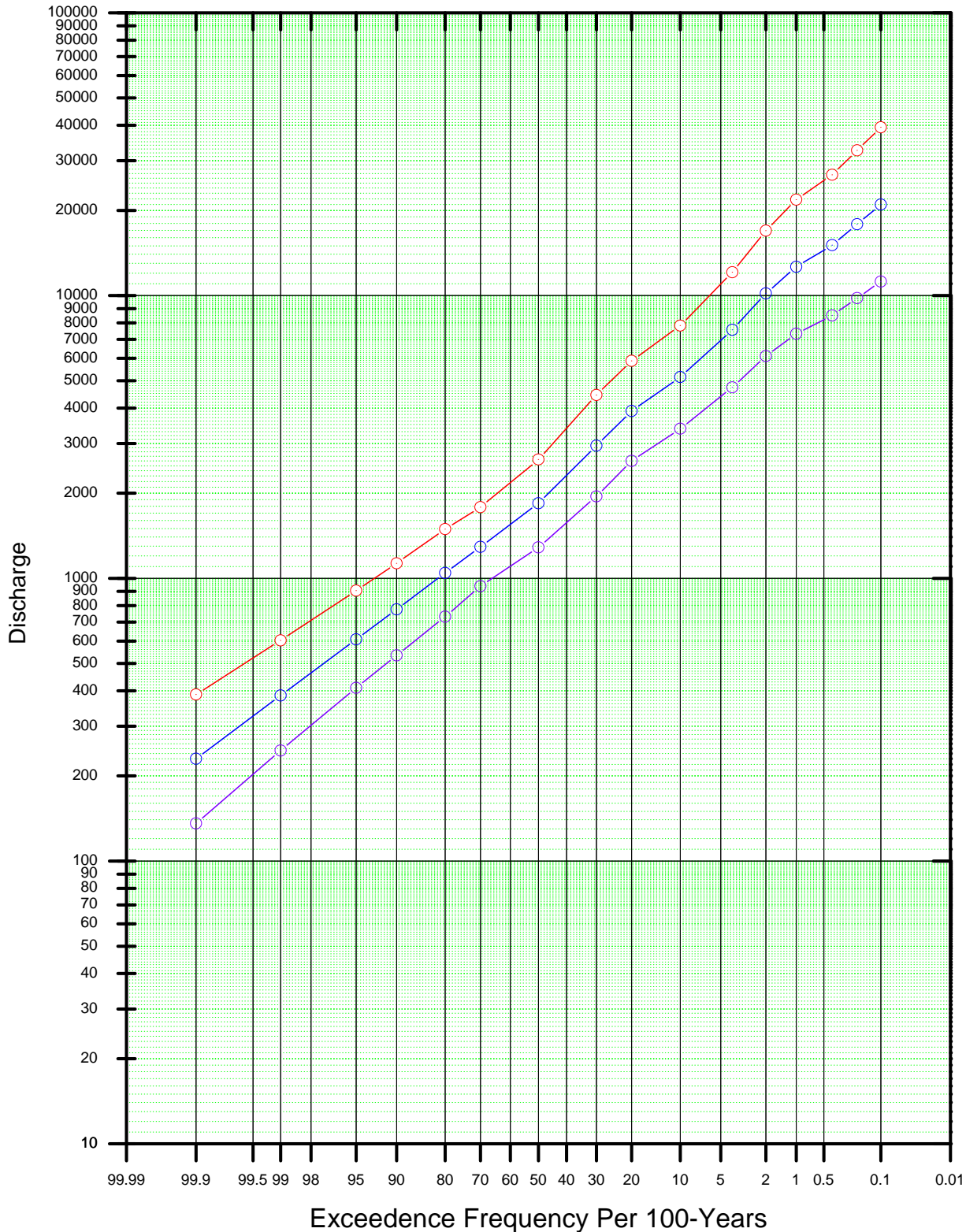


Legend:

- Guadalupe Rv us Ross Ck (flow)
- Guadalupe Rv us Ross Ck 0.95 Confidence Limits
- Guadalupe Rv us Ross Ck 0.05 Confidence Limits

November 2009

Upper Guadalupe Hydrologic Study, Santa Clara County, CA
Flow Frequency Curve Guadalupe River Upstream Ross Ck (J Guad & Alamito Cr Confl)
US Army Corps of Engineers San Francisco District

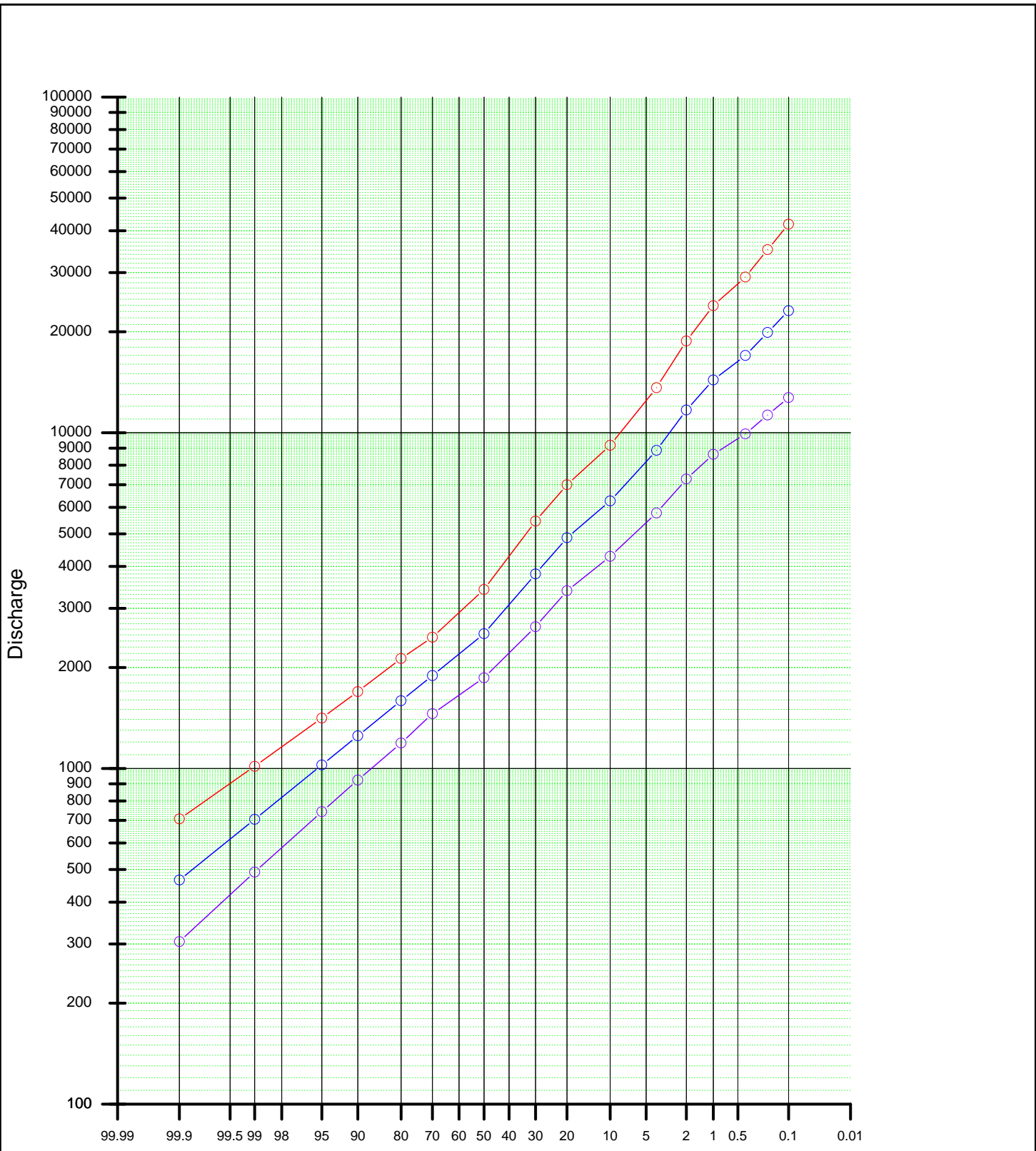


Legend:

- Guadalupe Rv ds Ross Ck (flow)
- Guadalupe Rv ds Ross Ck 0.95 Confidence
- Guadalupe Rv ds Ross Ck 0.05 Confidence

November 2009

Upper Guadalupe Hydrologic Study, Santa Clara County, CA
Flow Frequency Curve Guadalupe River Downstream Ross Ck (J Guad 23B d/s Ross Cr)
US Army Corps of Engineers San Francisco District



Legend:

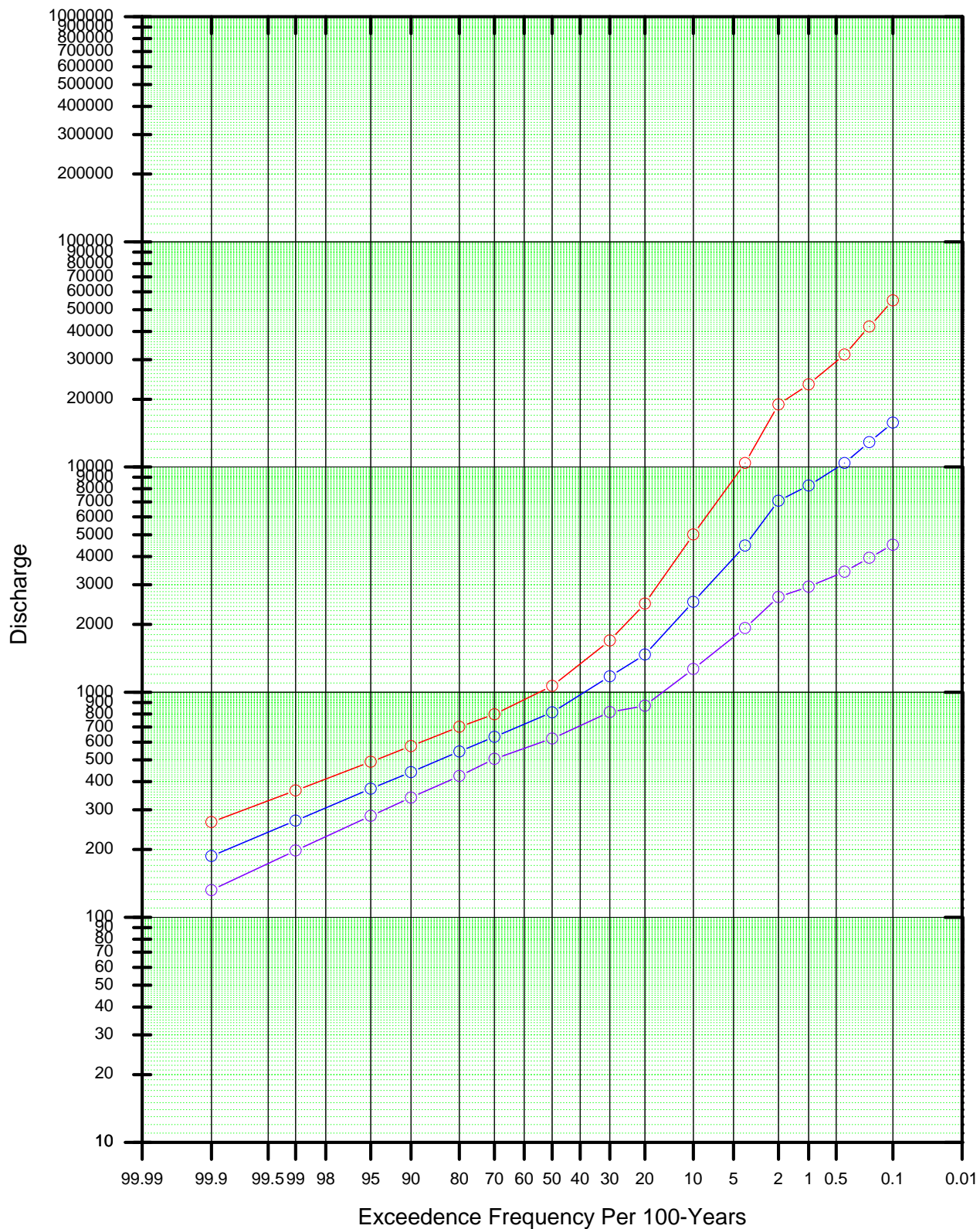
- Guadalupe ds Canoas Ck (flow)
- Guadalupe ds of Canoas .95 Confidence Limits (.95)
- Guadalupe ds of Canoas .05 Confidence Limits (.05)

November 2009

Upper Guadalupe Hydrologic Study,
Santa Clara County, CA

Flow Frequency Curve
Guadalupe River
downstream Canoas Ck
(J272)

US Army Corps of Engineers
San Francisco District



Legend:

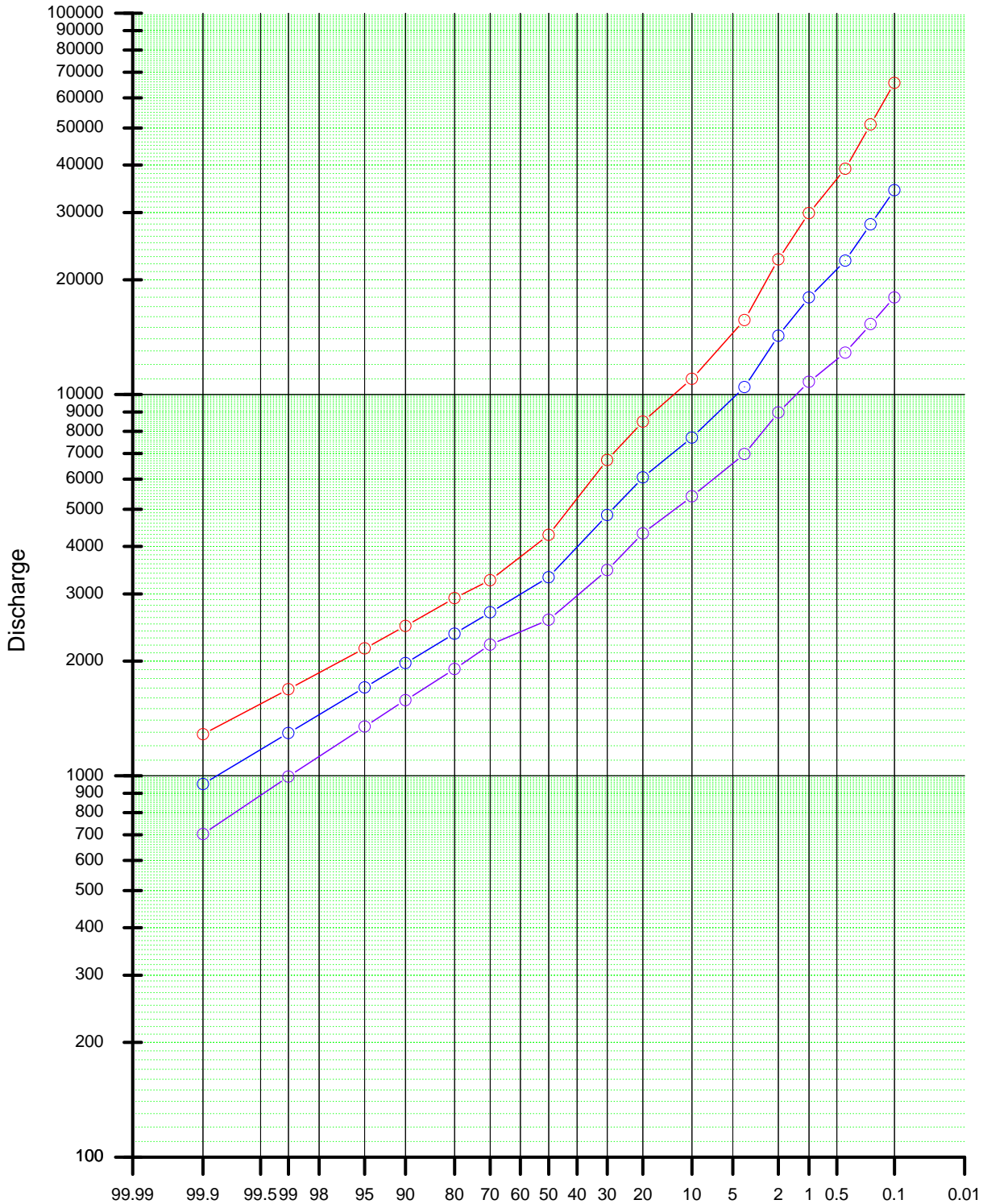
- Los Gatos Ck us Guadalupe Rv (flow)
- Los Gatos Ck us Guadalupe Rv 0.95 Confidence Limits
- Los Gatos Ck us Guadalupe Rv 0.05 Confidence Limits

November 2009

Upper Guadalupe Hydrologic Study,
Santa Clara County, CA

Flow Frequency Curve
Los Gatos Creek
Upstream Guadalupe River
(J Gage Los Gatos u/s Guad)

US Army Corps of Engineers
San Francisco District



Legend:

Exceedence Frequency Per 100-Years

- Guadalupe ds of Los Gatos Ck (flow)
- Guadalupe ds of Los Gatos Ck .95 Confidence Limits (.95)
- Guadalupe ds of Los Gatos Ck .05 Confidence Limits (.05)

November 2009

Upper Guadalupe Hydrologic Study,
Santa Clara County, CA

Flow Frequency Curve
Guadalupe River
Downstream Los Gatos Creek
(J Gage at San Jose)

US Army Corps of Engineers
San Francisco District

Parcel Count Los Gatos Creek Floodplain Summary

Scenario	Google Impacted Parcels Count				Non-Google Impacted Parcels Count				All Impacted Parcels Count		
	<1' (X Zone)	>1' (A Zone)	Σ	% of Total	<1' (X Zone)	>1' (A Zone)	Σ	% of Total	<1' (X Zone)	>1' (A Zone)	Σ
Existing Conditions	19	58	77	14%	148	318	466	86%	167	376	543
In-Channel Mitigation	19	42	61	22%	82	139	221	78%	101	181	282
In-Channel Mitigation AND W San Fernando St Bridge Reconstruction	15	14	29	30%	19	48	67	70%	34	62	96

Note: Impacted areas within roadways and R.O.W. were not incorporated; does not incorporate Caltrain changes to D2/D3

Parcel Area Los Gatos Creek Floodplain Summary

Scenario	Total Flooded Parcel Area	Google Flooded Parcel Area		Total Flooded Parcel Area Removed	Google Flooded Parcel Area Removed		Other Flooded Parcel Area Removed
	(ac)	(ac)	% of Total	(ac)	(ac)	% of Total	% of Total
Existing Conditions	53.6	15.6	29%	n/a	n/a	n/a	n/a
In-channel mitigation (XS 3871-807) to reduce n-values	45.7	14.5	32%	7.8	1.1	14%	86%
In-channel mitigation (XS 3871-807) to reduce n-values AND W San Fernando St Bridge Reconstruction	21.4	4.9	23%	32.1	10.6	33%	67%

Note: Impacted areas within roadways and R.O.W. were not incorporated; does not incorporate Caltrain changes to D2/D3