

Design Hydraulic Study

Hackstaff Road over Long Valley Creek Overflow, Bridge 7C-012

Lassen County

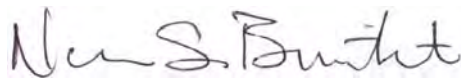
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## Executive Summary

Purpose:	Replace deficient bridge
Funding Program:	HBR
Design Flood:	Base Flood (Q100, 8320-cfs)
Clearance for Drift:	2.0-feet
Design Exception:	No design exception is anticipated for hydraulic conditions
Recommendations:	Min. Soffit Elevation – 4172.33-feet (to meet recommendations of Caltrans and FHWA)
	Abutment Scour Elevation – Abutments should be designed considering or protected against total potential scour to an elevation of 4150.-feet.
	Abutment Protection – Recommended for reducing the potential for damage to abutments from bank erosion and bank migration. Abutment protection should be designed in a manner that avoids a reduction in the flow area under the bridge. Abutment protection should also be placed to the top of bank (approximate elevation 98 upstream of bridge) to reduce the risk of damage associated with transient aggradation events.

Note regarding estimates of potential scour: Potential scour has been estimated using empirical equations presented in FHWA HEC-18. These equations do not consider geotechnical conditions and therefore assume all substrate is erodible. The potential scour estimates identified in this report may be inappropriate if a geotechnical investigation identifies material resistant to erosion at higher elevations.

### Preferred Bridge Characteristics:

Soffit Elevation –	4177.31-feet (4.98-ft above Q100, 7.86-ft above Q50)
Overtopping Flood –	>12000-cfs, >Q200
Impact on Flood Risk –	None
Impact on Channel –	Construction of the preferred bridge is not expected to aggravate channel instability.

Design Hydraulic Study  
Hackstaff Road over Long Valley Creek Overflow

INTRODUCTION

**Background:** This bridge hydraulic analysis has been prepared for the sole purpose of meeting the requirements of 23 CFR §650 dealing with bridge location and design in floodplains. Although potentially useful for other purposes, this analysis has not been prepared for any other purpose. Reuse of information contained in this report for purposes other than those for which this analysis and report are intended is not endorsed or encouraged by the author and is at the sole risk of the entity reusing information herein contained. Estimates of peak flows for frequent flood peaks (5-year or more frequent), if shown in this report, should not be considered accurate unless an overtopping flood of 5-year or more frequent recurrence is identified.

Analyses to meet the requirements of FEMA, the State of California Reclamation Board, low flow environmental or construction concerns and for other purposes may be provided as additional services.

**Design Standards:** Hydraulic design of the preferred bridge is based on standards recommended by Caltrans (Local Programs Manual - reference 1). Exceptions to these design standards are recommended only if meeting the standard is found to be impractical or unreasonably costly for the proposed project and the exception does not result in an increased risk of damage during floods. Local design standards that have been provided in writing prior to the preparation of the hydraulic analysis have also been considered.

**Funding:** HBR

**Existing Bridge:** Three span

Length – 62-ft

Skew – 0-degrees

Clear Width – 21.3-ft

Total Width – 22.0-ft

Lanes – 2

Speed Limit – 55-mph

Load Limit – 16-tons per vehicle, 25-tons per semi-trailer combination,  
31-tons per truck and full trailer

Posting – 12-tons per vehicle, 25-tons per semi-trailer combination,  
31-tons per truck and full trailer

Structure – Timber stringer on timber pile bents and abutments

Sketch – Figure 1, page 14

Significance: Vital Route – Yes  
 Bus Route – No  
 Road Classification – Local road  
 Present ADT – 138  
 Estimated future ADT – 414  
 Trucks or Commercial Vehicles – 15%  
 Description of Service – Residences, ranches, access to undeveloped land  
 Length of Detour – 11-miles  
 Description of Road – Substantially flat, mildly winding

Photos 1-4, pages 11, 12

Preferred Bridge: The preferred bridge crosses Long Valley Creek Overflow at a location immediately downstream of the existing bridge and at a skew of approximately 30-degrees from perpendicular to the direction of flood flow.

Length – 85.17-ft nominal, 68.86-ft effective hydraulic  
 Hydraulic Skew – 30-degrees  
 Clear Width – 32.0-ft  
 Total Width – 35.33-ft  
 Lanes – 2  
 Speed Limit – 55-mph  
 Load Limit – None  
 Structure – Single span RC slab on precast girders on RC abutments.  
 Traffic During Construction – Maintained on existing bridge  
 General Plan – Figure 2, page 15

## DESCRIPTION OF BASIN

Geographic Location: Above the proposed bridge, Long Valley Creek drains a modest area of high mountain desert east of the Sierra Nevada range and north of Lake Tahoe.

Receiving Waters: Honey Lake, a closed basin

Characteristics: Area of basin – 403 sq-mi  
 Shape – “F”, outlet at top left corner  
 Highest elevation – 8750-ft on Babbitt Peak near south end of basin  
 Lowest elevation – 4230-feet near bridge site  
 Elevation index – 5.2  
 Average annual precipitation (basin wide) – 10-in  
 Aspect – NNW

Land use: Forest activities, rural residential

Vegetation: High desert grasses and shrubs.

Geologic:	Topographic and geologic features indicate substantial potential for significant landslides and bank erosion capable of causing channel instability and risk to bridge integrity.
Basin:	Figure 3, page 16

## DESCRIPTION OF STREAM AND SITE

Stream Channel:	In the vicinity of the preferred bridge, Long Valley Creek Overflow has a well defined, flat bottom, slightly incised flood channel with an average slope of approximately 0.3-percent (decimal 0.003) and bed materials consisting of fine sand and silt. Photos 5 and 6 (page 13) show Long Valley Creek Overflow looking downstream and upstream from the existing bridge.
Stream Banks:	The banks of Long Valley Creek Overflow are steep and consist of cohesive silty loam with a light grass and shrub cover at shallower slopes.
Existing Bridge:	The existing bridge is aligned near perpendicular to the direction of flood flow in Long Valley Creek Overflow.
Site Topography:	Figure 4, page 17

## HYDROLOGIC ANALYSIS

Hydrologic Environment:	Long Valley Creek Overflow is one of two channels conveying water during infrequent flood events in Long Valley Creek. The Long Valley Creek Overflow channel presently forms from the combination of numerous small overflow channels located approximately ½-mile upstream of Hackstaff Road. Based on channel geometry and geologic conditions at the bridge and within the basin, it is likely that the Long Valley Creek Overflow channel was at one time the single main channel conveying all flow in Long Valley Creek. Given a high risk of transient aggradation events and associated potential for full channel avulsion resulting in the Long Valley Creek Overflow channel once again conveying the full flow of Long Valley Creek, the flood hydrologic analysis for Bridge 7C-12 has been conducted assuming the full flow of Long Valley Creek during infrequent flood events.
Hydrologic Stability:	Infrequent floods in Long Valley Creek Overflow are substantially natural and not significantly influenced by land use activities within the drainage basin.
Flood History:	Bridge abutments may have been damaged during the flood of 1964. There is no record of flood water overtopping Hackstaff Road.

Number of Methods: Four methods were investigated for estimating potential infrequent flood peak flows in Long Valley Creek and Long Valley Creek Overflow. These include adjustment (translation) of known flood frequency curves, direct application of the USGS Nevada Region 6 Equation.

Translation Analysis: Approach – Translation analysis consists of estimating the infrequent flood peak flows by comparison with gaged stream or river basins. After identification of representative gaged basins, flood frequency relationships for the gaged basins are determined by plotting annual flood peaks and computing the normal probability Log-Pearson Type III curve fit (reference 7). If the Log-Pearson type III curve fit reasonably represents the plotted data for the less frequent floods, it is considered representative of the gaged basin and used as a basis of comparison. If not, a line of best visual fit may be used as a basis of comparison.

After identifying representative flood-frequency relationships for the gaged basins, candidate flood frequency relationships representing the stream or river at the proposed project site are estimated by adjusting the gaged basin flood frequency relationship to account for differences in characteristics between the gaged basin and the basin above the proposed project. The adjustments are made using the area, elevation and precipitation exponents of the appropriate USGS region equation (reference 8).

Basin Characteristics – Characteristics of gaged basins found to be potentially representative of the basin above the proposed project and having records of adequate length to reasonably identify the infrequent flood peak flows are identified in Table 1.

TABLE 1  
Stream and Gaged Basin Characteristics

Basin Description	USGS Gage Number	Area (sq mi)	Average Annual Precip (in)	Elevation Index	Years of Record
Long Valley Ck at Hackstaff Rd	n/a	403	10	5.2	n/a
Steamboat Ck nr Steamboat NV	10349300	123	n/a	6.0	30

Gaged basin flood frequency curves – Plotted flood frequency data and curves for the gaged basins used in this analysis are shown in Appendix A.

Regional Equations: Approach – The USGS has published regional equations for estimating infrequent flood peak flows in ungaged natural streams and rivers not affected by lakes, reservoirs, substantial development or substantial reclamation projects (reference 8). These equations are useful for planning level and rough preliminary estimates of infrequent flood peak flows and corroboration of flood frequency estimates using more detailed procedures. Flood peak flows estimated by these equations should only be relied upon for design if confidence in other methodologies is low and if verified by other methodologies. The empirical equations estimate flood

peak flows from basin characteristics including area, elevation index and precipitation as appropriate. Use of the area, elevation index and precipitation factor exponents of the regional equation for adjustment of flood characteristics from representative long term gaged basins (described in Translation Analysis above) is generally considered to provide a more reliable estimate of infrequent flood peak flows for the ungaged basin.

**Flood Peak Flows:** Candidate flood frequency relationship – All candidate flood frequency curves derived from translation analysis for the proposed project site are plotted and shown in Appendix A. Estimated 50- and 100-year flood peak flows from all methods investigated are summarized in Tables 2 and 3.

TABLE 2  
Long Valley Creek Estimated 50- and 100-year Flood Peak Flows

Estimated from	50-Year (cfs)	100-Year (cfs)
Steamboat Creek “Same Stream” approach	4530	6670
Steamboat Creek, Nevada Region 6 Exponents	6410	8320
Direct Application of USGS Nevada Region 6 Equation	7700	9600
Steamboat Creek, California Northeast Exponents	4070	5260

Selected flood frequency relationship – The flood frequency relationships estimated from Steamboat Creek adjusted using Nevada Region 6 Exponents has been selected as most appropriate for design of the replacement bridge. This estimate has been selected because of the long length of peak flow records at the Steamboat Creek streamgage and because of corroboration by other methods. Other estimates were not selected because of the more regional nature of the methods. The selected flood frequency relationship is shown in Figure 5 (page 18).

**Flood of Record:** There are no adequate records of streamflow in Long Valley Creek from which to identify a flood of record.

## HYDRAULIC ANALYSIS

**Backwater Model:** Backwater program – The Corps of Engineers’ HEC-RAS version 4.1.0 backwater program (reference 3) has been selected for modeling hydraulic characteristics representing existing conditions, preliminary bridge configurations and the proposed bridge. This program has been selected because of its long history of use (derived from HEC-2), wide acceptance, and great flexibility for evaluating bridge configurations.

Cross-section data – Stream cross-sections and Manning’s roughness coefficients upstream and downstream of the proposed project have been assumed constant for all models. Cross-sections used in the backwater models were from a recent ground survey. Locations of cross-sections used in the backwater model are shown on Figure 6 (page 19). Cross-



sections have been adjusted for skew as appropriate. Interpolated cross-sections were inserted as appropriate to improve model reliability. Interpolated cross-sections were checked to avoid interpolation error.

Elevation Datum – NAVD88

Manning's Roughness Coefficients – Manning's Roughness Coefficients were estimated by observation and comparison with similar channels identified in Roughness Coefficients of Natural Channels (reference 6). Manning's roughness coefficients ranging from 0.030 to 0.035 was used to represent the Long Valley Creek Overflow channel. A Mannings roughness coefficient of 0.045 was used to represent overbank areas.

Contraction and Expansion Coefficients – Contraction and expansion coefficients of 0.1 and 0.3 respectively were used to represent the natural channels. These were raised to 0.3 and 0.5 respectively in the vicinity of the bridges.

Downstream starting water surface elevation assumption – The normal depth method in HEC-RAS was selected for estimating the downstream water surface elevation. A slope of 0.003, estimated from the average slope of the Long Valley Creek Overflow channel, was used as the starting slope. Two surveyed, one derived (cut from detailed topography), and one interpolated cross-sections were used to isolate the effects of downstream starting water surface elevation assumption from water surface elevations at the bridge.

Existing Bridge: Purpose – The existing condition backwater model has been prepared to identify and document existing hydraulic conditions and to serve as a basis of comparison with which to evaluate preliminary and proposed bridge configurations.

Channel roughness coefficient at bridge – 0.030

Overbank roughness coefficient at bridge – 0.045

Contraction coefficient – 0.3 (at bridge)

Expansion coefficient – 0.5 (at bridge)

Bridge modeling method – Energy

Drift assumption – Effective pier width assumed 3-feet (1-foot actual)

Figure 7 (page 20) shows how the existing bridge is represented in model

Model results – Existing hydraulic conditions are summarized in Table 3. Existing condition flood profiles and a stage discharge curve at cross-section 1580 are shown in Figures 9 and 10 (pages 22, 23). Summary output tables from the existing condition HEC-RAS backwater model are included in Appendix B.

TABLE 3  
Existing Hydraulic Conditions (with drift except as noted)

Flood	Flow (cfs)	Recurrence (years)	W.S. Elevation <sup>1</sup> (feet)	Avg. Channel Velocity <sup>2</sup> (fps)
Standard Design	6410	50	4173.07	12.8
Base	8320	100	4175.50	14.1
Base, no drift	8320	100	4174.45	14.1
Flood of Record	n/a	n/a	n/a	n/a
Overtopping Flood	7800	80±	4174.82	>15

Notes: 1) At cross-section 1580 located approximately 20-feet upstream of the existing bridge.  
2) Highest average channel velocity under bridge. The very high velocities are due to the assumption that all flow of Long Valley Creek is being conveyed in the overflow channel and indicate that during such an event, substantial erosion can be expected.

**Preliminary Bridges:** Backwater models were prepared to represent a variety of candidate bridge configurations. Results from these models were provided to the bridge design engineer in the form of memoranda and e-mail. Using information provided in the memoranda and e-mail and considering additional factors not related to hydraulic conditions, the preferred bridge configuration was selected for final design.

**Preferred Bridge:** The preferred bridge backwater model has been prepared to identify preferred bridge hydraulic requirements and impacts. The preferred bridge backwater model assumes the existing bridge and approaches are removed.

Channel roughness coefficient at bridge – 0.030  
Overbank roughness coefficient at bridge – 0.045  
Contraction coefficient – 0.3 (at bridge)  
Expansion coefficient – 0.5 (at bridge)  
Bridge modeling method – Energy  
Drift assumption – None (no piers)

Figure 8 (page 21) shows how preferred bridge is represented in model

Model results – Preferred bridge hydraulic conditions are summarized in Table 4. Preferred bridge flood profiles and a stage discharge curve at cross-section 1580 are shown in Figures 9 and 10 (page 22, 23). Summary output tables from the preferred bridge HEC-RAS backwater model are included in Appendix B.

TABLE 4  
Preferred Bridge Hydraulic Conditions (no drift)

Flood	Flow (cfs)	Recurrence (years)	W.S. Elevation <sup>1</sup> (feet)	Avg. Channel Velocity <sup>2</sup> (fps)
Standard Design	6410	50	4169.45	8.8
Base	8320	100	4172.33	10.2
Flood of Record	n/a	n/a	n/a	n/a
Overtopping Flood	>12000	>200	4176.7	>12

Notes: See notes for Table 3

### SCOUR AND EROSION

**Channel Stability:** In the vicinity of Hackstaff Road is it not likely that the Long Valley Creek channel has been greatly influenced by land use activities in the basin. The combination of topography and geology in the basin, however, are conducive of channels having low stability with the potential for rapid and substantial changes during infrequent flood events. Rapid channel changes are normal and expected during infrequent flood events in Long Valley Creek and Long Valley Creek Overflow whether or not the existing bridge is replaced with the preferred bridge.

Long Valley Creek is likely to experience transient aggradation events associated with upstream landslides and bank erosion during flood events (Reference 11).

Construction of the preferred bridge is not expected to significantly impact energy slope or sediment transport during floods up to the most probable 100-year flood and therefore is not expected to aggravate instability in Long Valley Creek Overflow.

**Contraction Local:** The preferred bridge represents a minor contraction of the flood channel. Potential contraction scour has been estimated to be approximately 4.1-feet using the live bed contraction scour equation presented in FHWA HEC-18 (Reference 4).

**Abutment Local:** Including potential contraction scour, the total potential scour at abutments was estimated to be 10.0-feet using the NCHRP live bed equation presented in FHWA HEC-18.

**Total Scour:** Total potential scour and scour elevations at abutments are summarized in Table 5. Scour computations and data are included in Appendix C.

TABLE 5  
Total Potential Scour (feet)

Location	Ground Elev.	Degradation	Contraction Scour <sup>1</sup>	Local Scour	Total Scour	Scour Elev.
Abutments	4160.	0.	0	10	10	4150.

Notes: 1) Contraction scour included in NCHRP local abutment scour estimate

#### OTHER CONSIDERATIONS:

- Drift:** There is a moderate potential for significant volumes of small to medium size drift (branches to small tree trunks) in Long Valley Creek Overflow. Drift has been considered in the design of the preferred bridge by selecting a clear span structure that provides more than the recommended clearance for drift.
- Geologic Risk:** Transient aggradation due to upstream landslides or excessive bank erosion is likely to occur in Long Valley Creek and Long Valley Creek Overflow during infrequent flood events. During such events, considerable bedload causes water surface elevations to be much higher than estimated by a fixed geometry backwater model. Although the probability of occurrence of such events cannot be quantified, risk of damage to the project may be minimized by considering the possibility of higher water surface elevations. At the Long Valley Creek Overflow site, the risk of damage to the replacement bridge can be reduced by providing bank protection to the top of bank. Potential damage to the bridge structure can be further reduced by designing the abutment foundations such that the bridge will remain stable in the event bank materials are eroded from behind the abutments. A paper documenting potential transient aggradation risks to the bridge is included in Appendix E.
- Flood Risk:** Replacement of the existing bridge with the preferred bridge is expected to reduce the water surface elevations of infrequent floods in Long Valley Creek.
- FEMA:** The preferred bridge is not located within an area having flood risk mapped by FEMA using detailed study methods. As such, projects may encroach into the floodplain to the extent they result in a 1.0-foot increase in the water surface elevation during the most probable 100-year flood provided the increase does not result in an increased risk of damage to structures or other negative impacts. Replacement of the existing bridge with the preferred bridge is not expected to produce an increase in the water surface elevations during the most probable 100-year flood.

## CONCLUSIONS AND RECOMMENDATIONS

- Design Flood:** Caltrans and FHWA recommend that the lowest soffit elevation of new and replacement bridges pass the most probable 100-year flood (Base Flood) under the bridge soffit without a clearance for drift and that it pass the most probable 50-year flood (Standard Design Flood) with appropriate clearance for drift, whichever is higher. At Long Valley Creek Overflow, the critical condition establishing the minimum soffit elevation is the most probable 100-year flood with no clearance for drift.
- Clearance for Drift:** The minimum clearance for drift recommended by Caltrans and FHWA for bridges over small streams of 2.0-feet is appropriate at this site.
- Design Exception:** No design exception is anticipated for bridge hydraulic conditions.
- Recommendations:**
- Minimum Soffit Elevation –** The minimum soffit elevation of a bridge meeting the recommendations of Caltrans and FHWA is 4172.33-feet. This represents the elevation of the Base Design Flood (100-year flood) with no clearance for drift.
  - Abutment Scour Elevation –** Abutments should be designed considering or protected against total potential scour to an elevation of 4150.-feet.
  - Abutment Protection –** Recommended for reducing the potential for damage to abutments from bank erosion and bank migration. Abutment protection should be designed in a manner that avoids a reduction in the flow area under the bridge. Abutment protection should also be placed to the top of bank (approximate elevation 4175-feet) to reduce the risk of damage associated with transient aggradation events.

Note regarding estimates of potential scour: Potential scour has been estimated using empirical equations presented in FHWA HEC-18. These equations do not consider geotechnical conditions and therefore assume all substrate is erodible. The potential scour estimates identified in this report may be inappropriate if a geotechnical investigation identifies material resistant to erosion at higher elevations.

### Preferred Bridge Characteristics:

- Soffit Elevation –** 4177.31-feet (4.98-ft above Q100, 7.86-ft above Q50)
- Overtopping Flood –** >12000-cfs, >Q200
- Impact on Flood Risk –** None
- Impact on Channel –** Construction of the preferred bridge is not expected to aggravate channel instability.



Photo 1: Looking downstream (north) at Bridge 7C-12,  
Hackstaff Road over Long Valley Creek Overflow



Photo 2: Looking upstream (south) at Bridge 7C-12,  
Hackstaff Road over Long Valley Creek Overflow





Photo 3: Looking east at Bridge 7C-12,  
Hackstaff Road over Long Valley Creek Overflow



Photo 4: Looking west at Bridge 7C-12,  
Hackstaff Road over Long Valley Creek Overflow





Photo 5: Looking downstream (north) at Long Valley Creek Overflow  
from Bridge 7C-12, Hackstaff Road



Photo 6: Looking upstream (south) at Long Valley Creek Overflow  
from Bridge 7C-12, Hackstaff Road



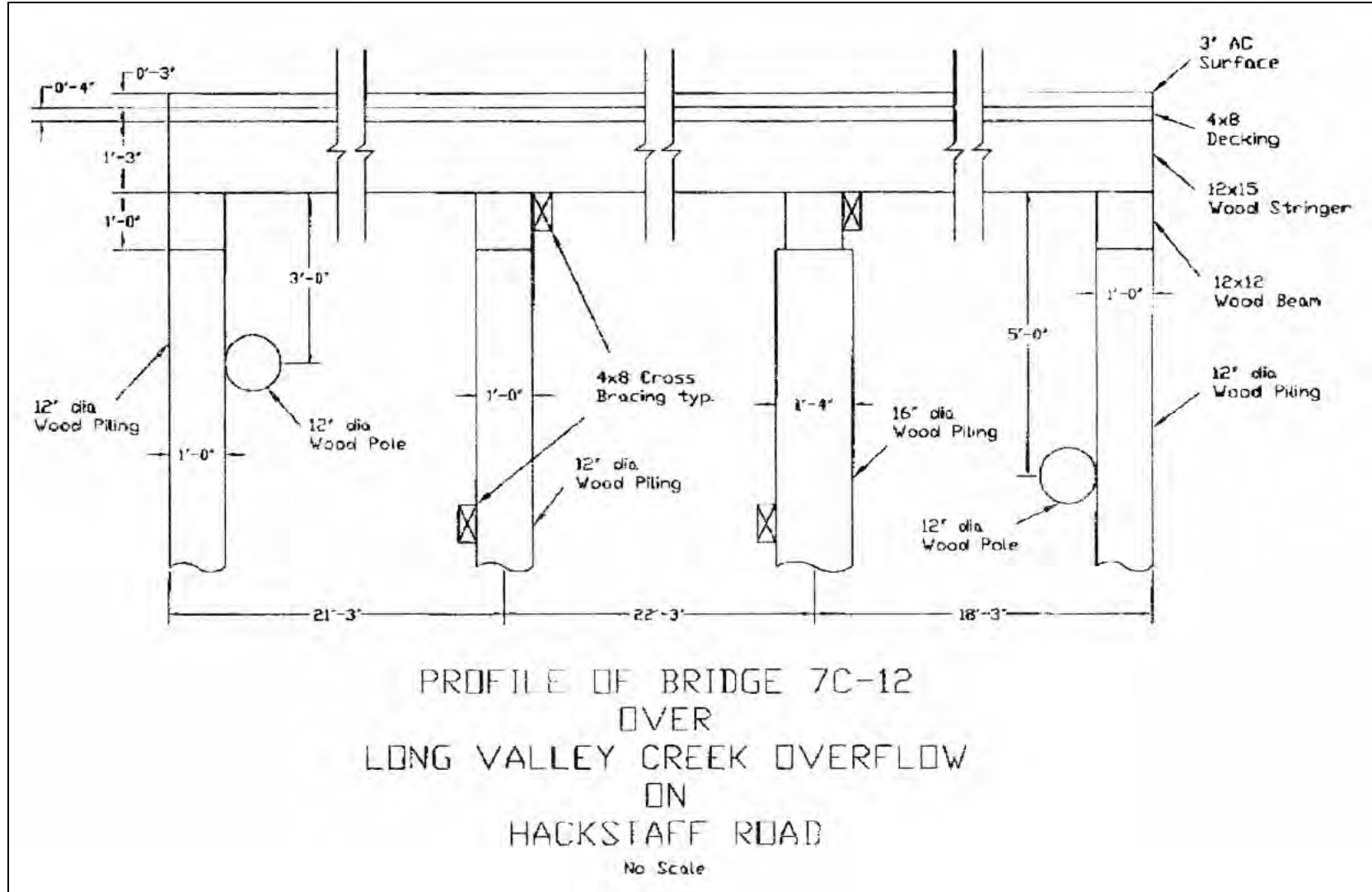


Figure 1: Sketch of Existing Bridge

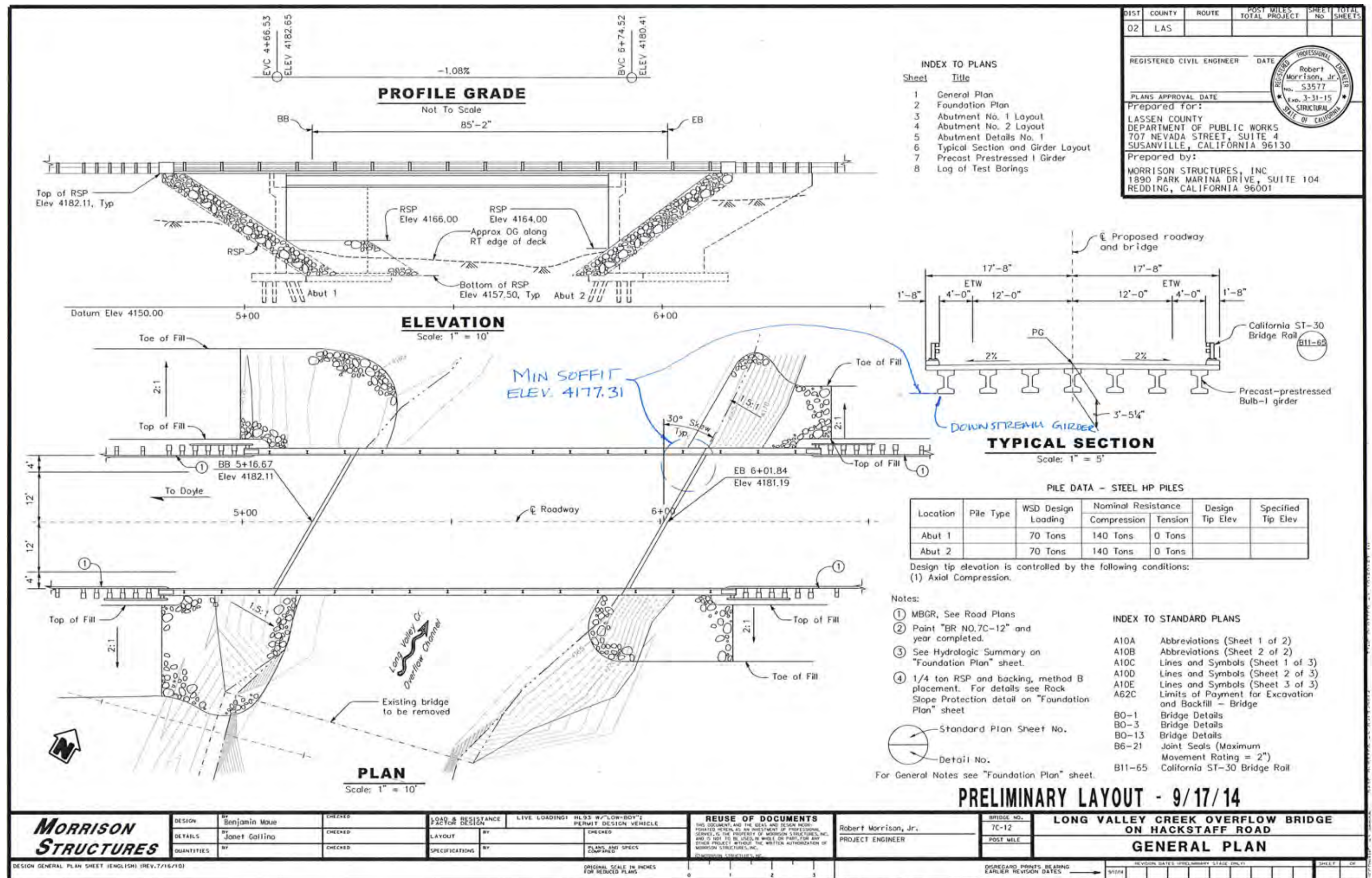


Figure 2: Preferred Bridge

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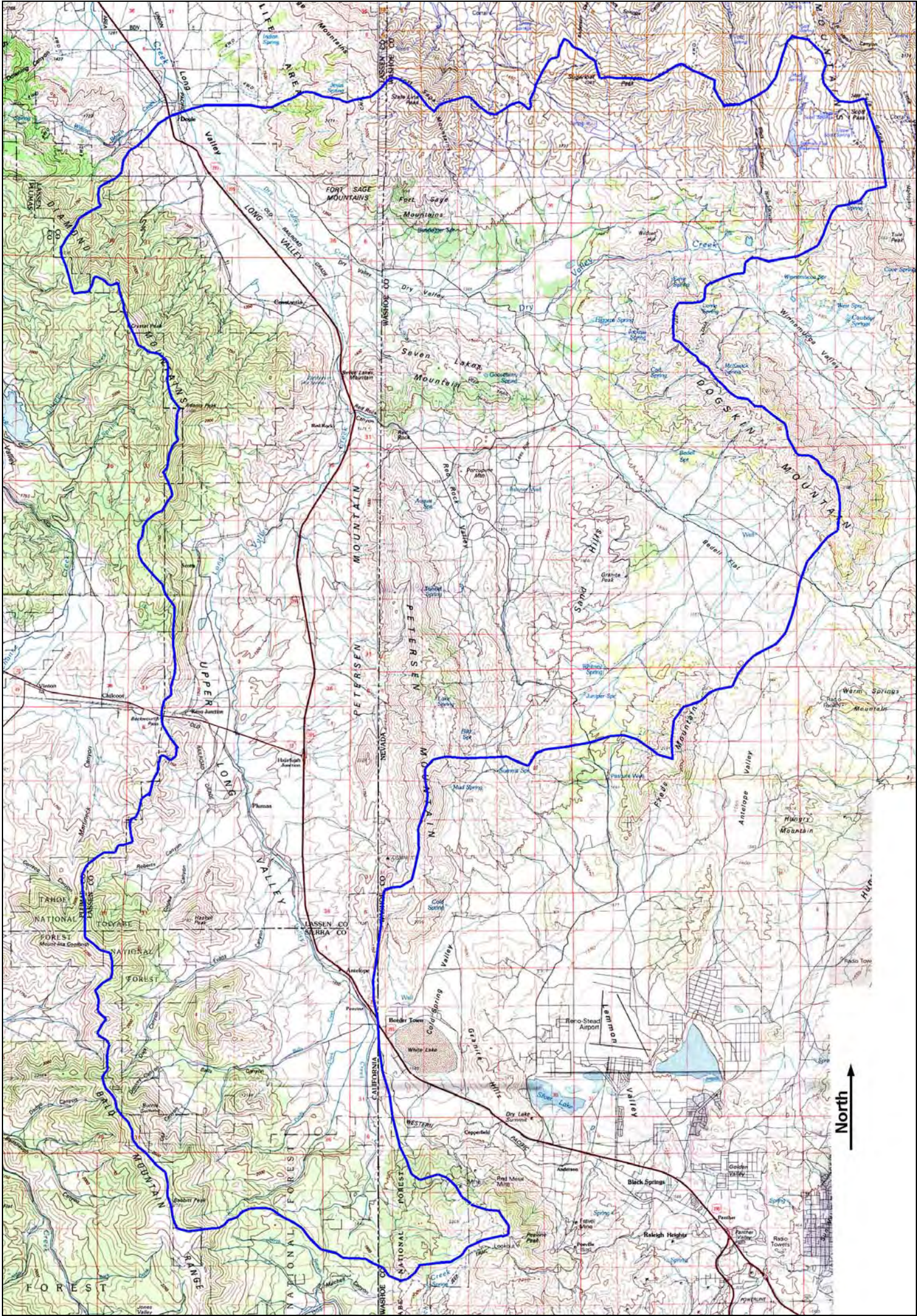


Figure 3: Long Valley Creek Basin  
Scale 1:200,000



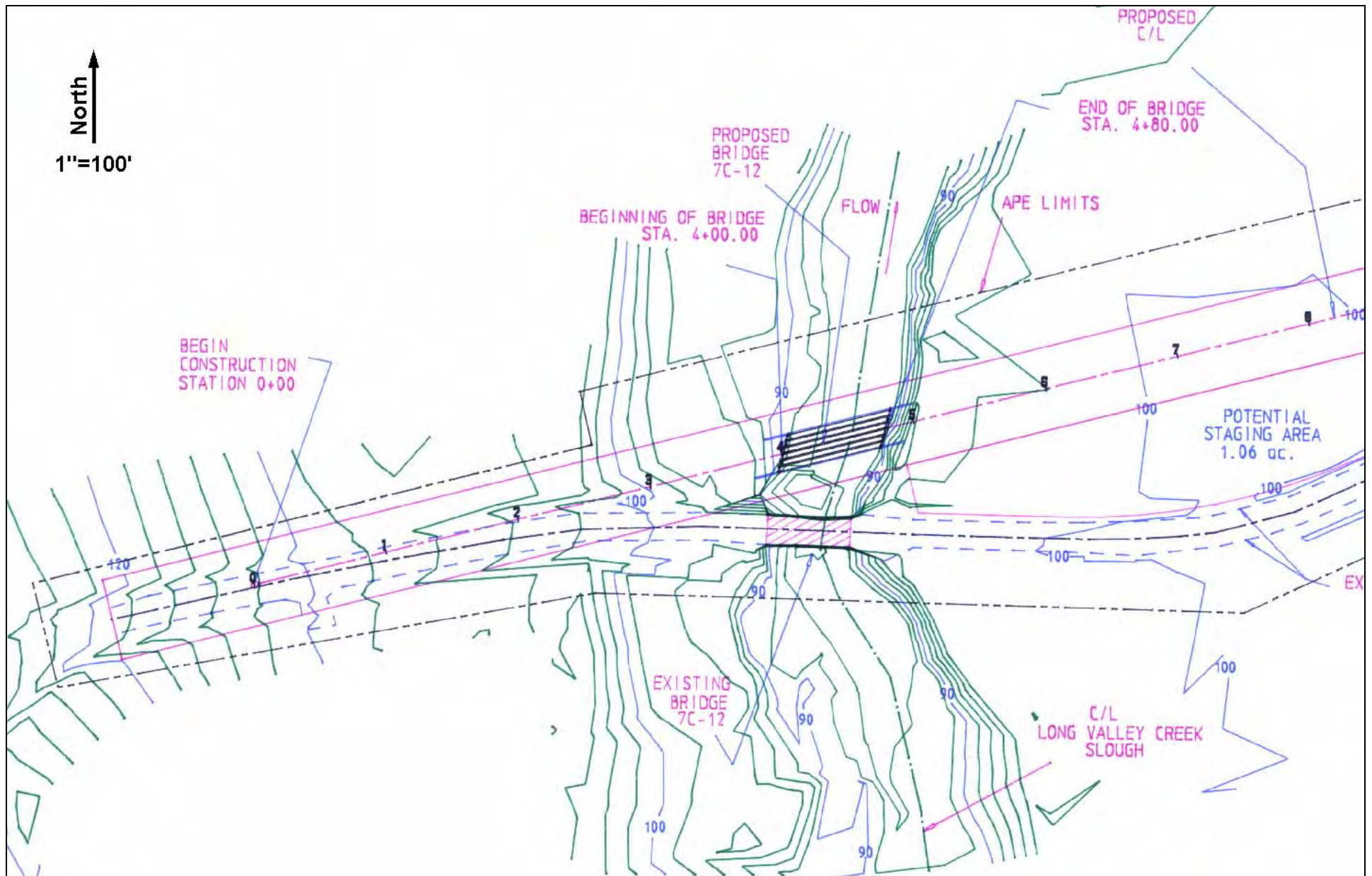


Figure 4: Site Topography

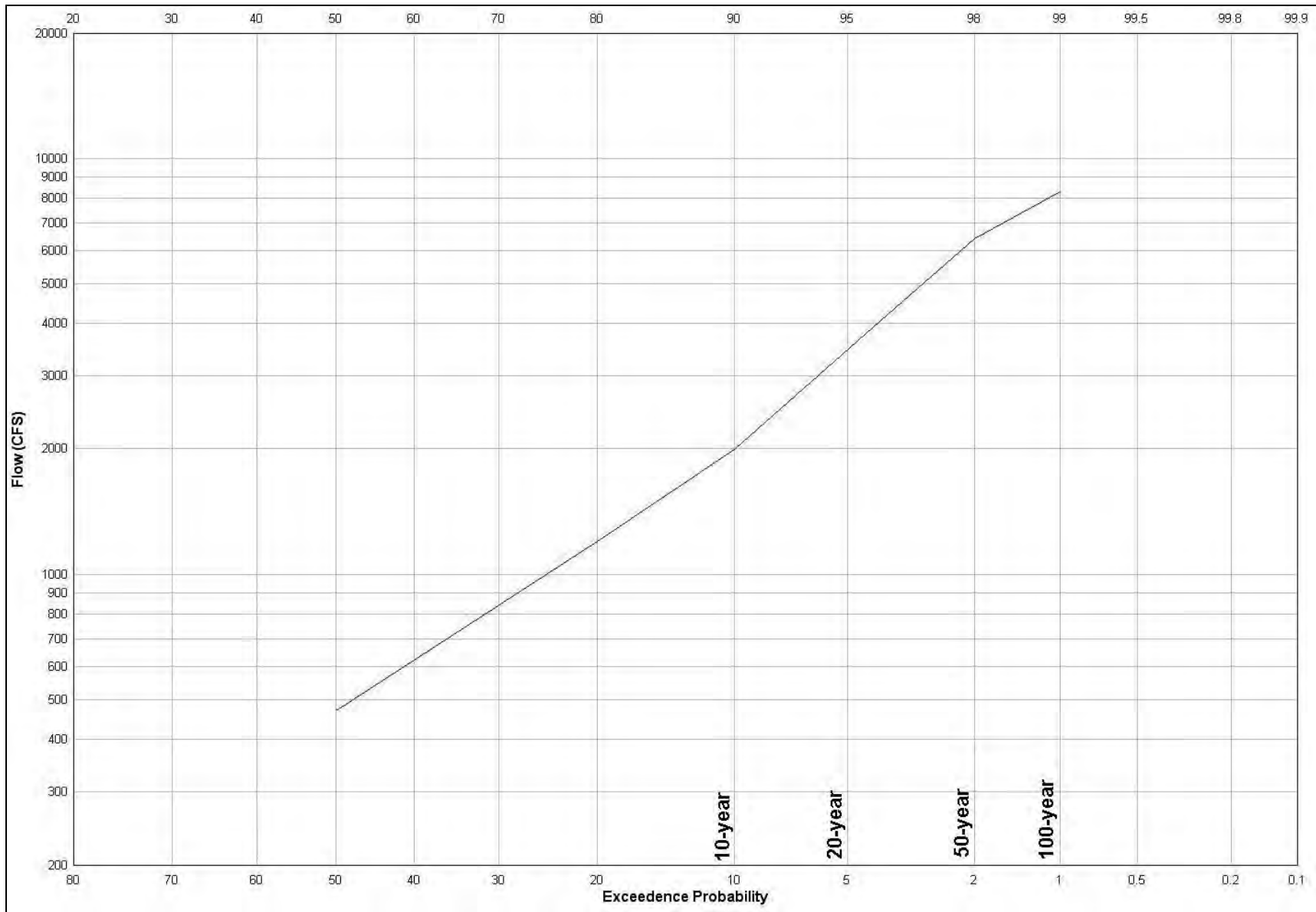


Figure 5: Flood Frequency Curve, Long Valley Creek at Hackstaff Road





Figure 6: Approximate Location of Cross-sections

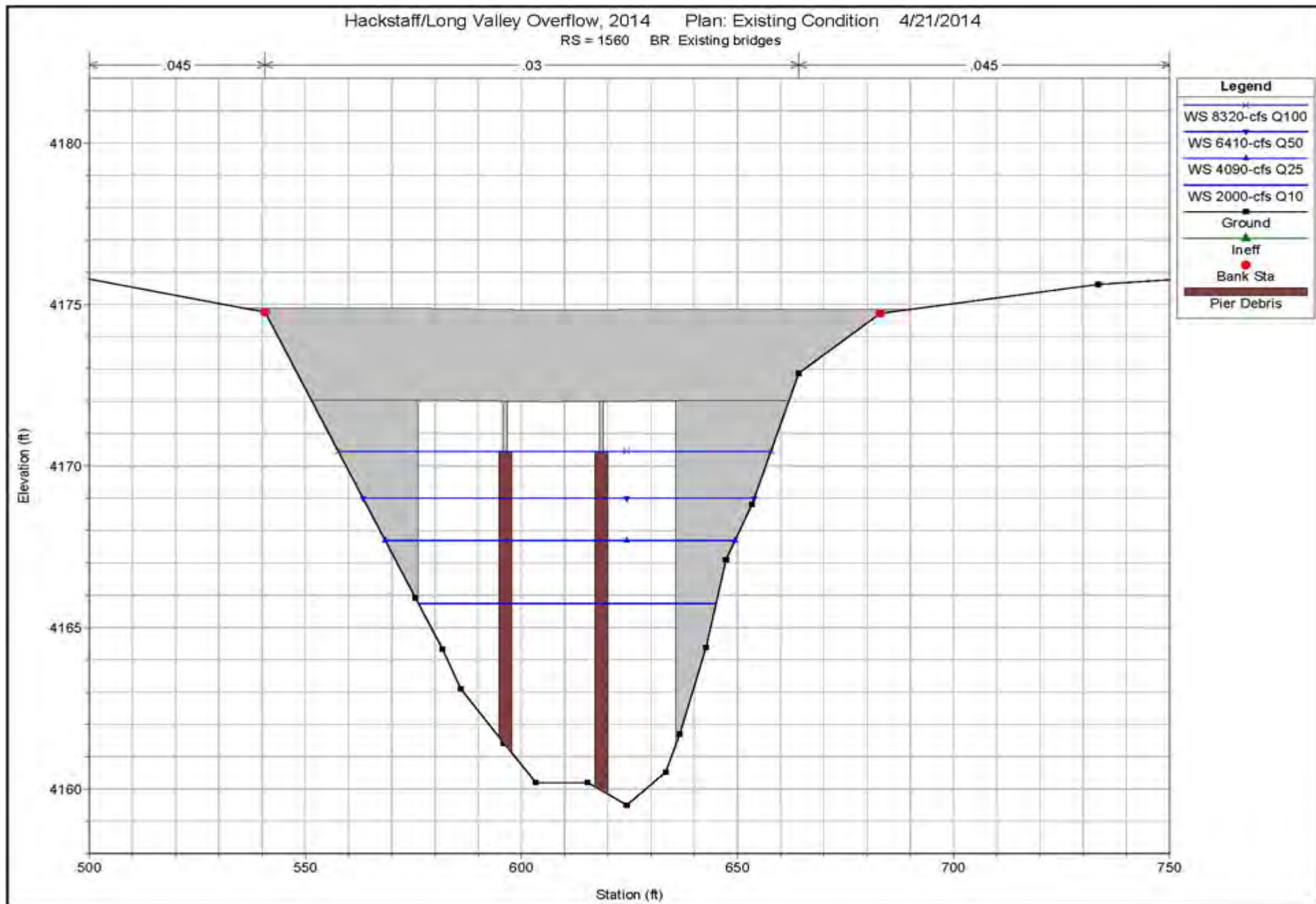


Figure 7: Existing Bridge as Represented in Backwater Model



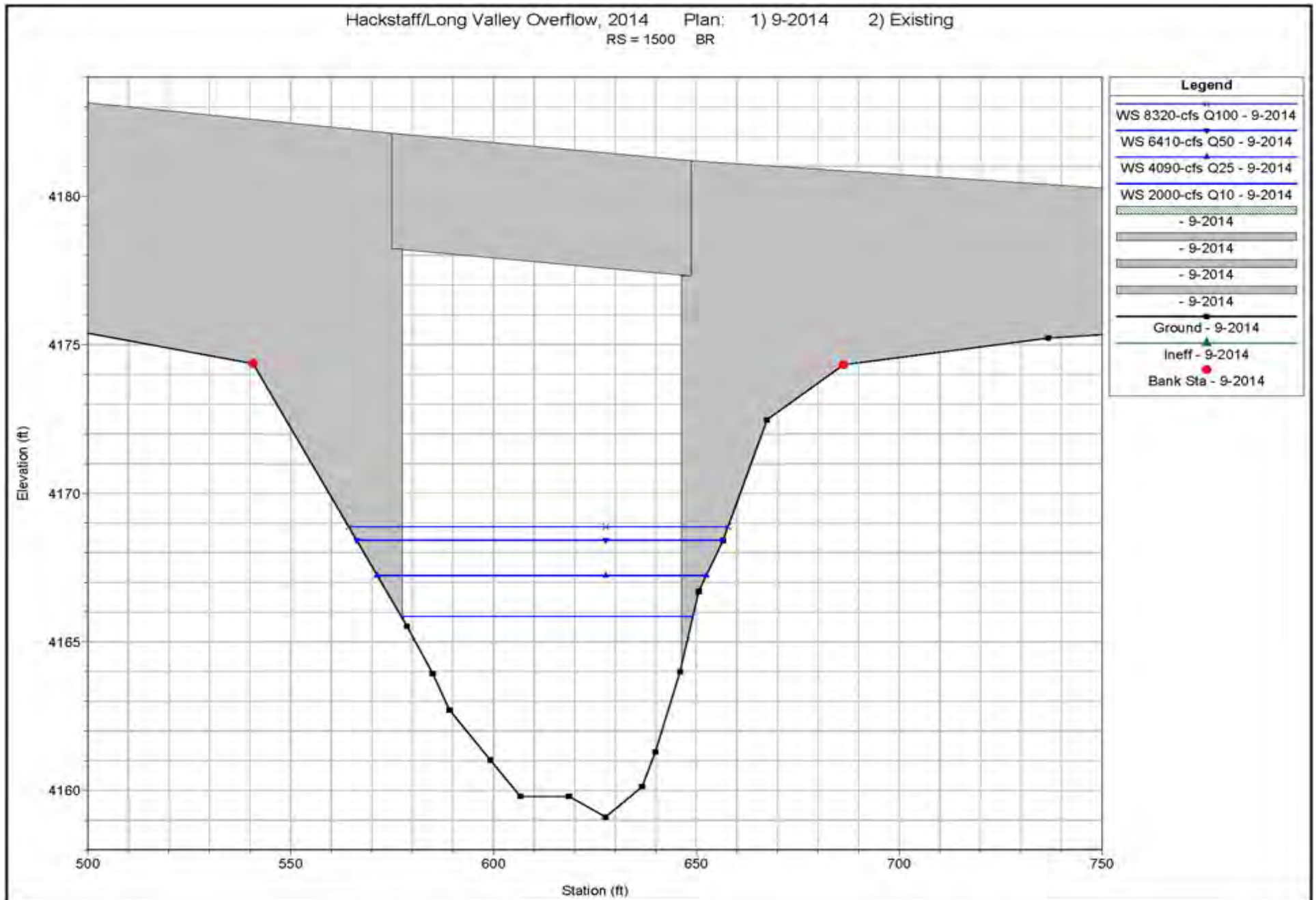


Figure 8: Preferred Bridge as Represented in Backwater Model



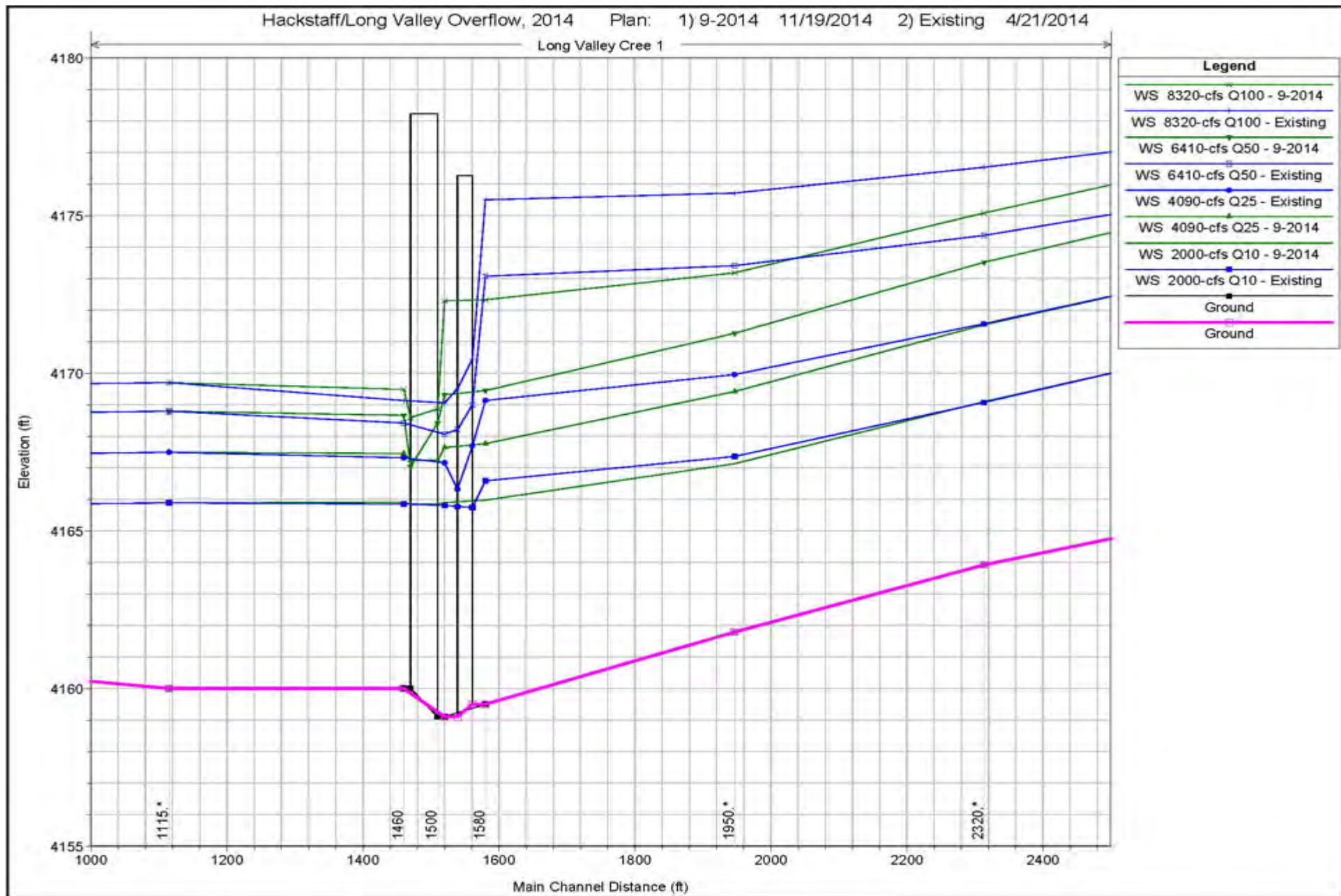


Figure 9: Existing and Preferred (9-2014) Bridge Flood Profiles

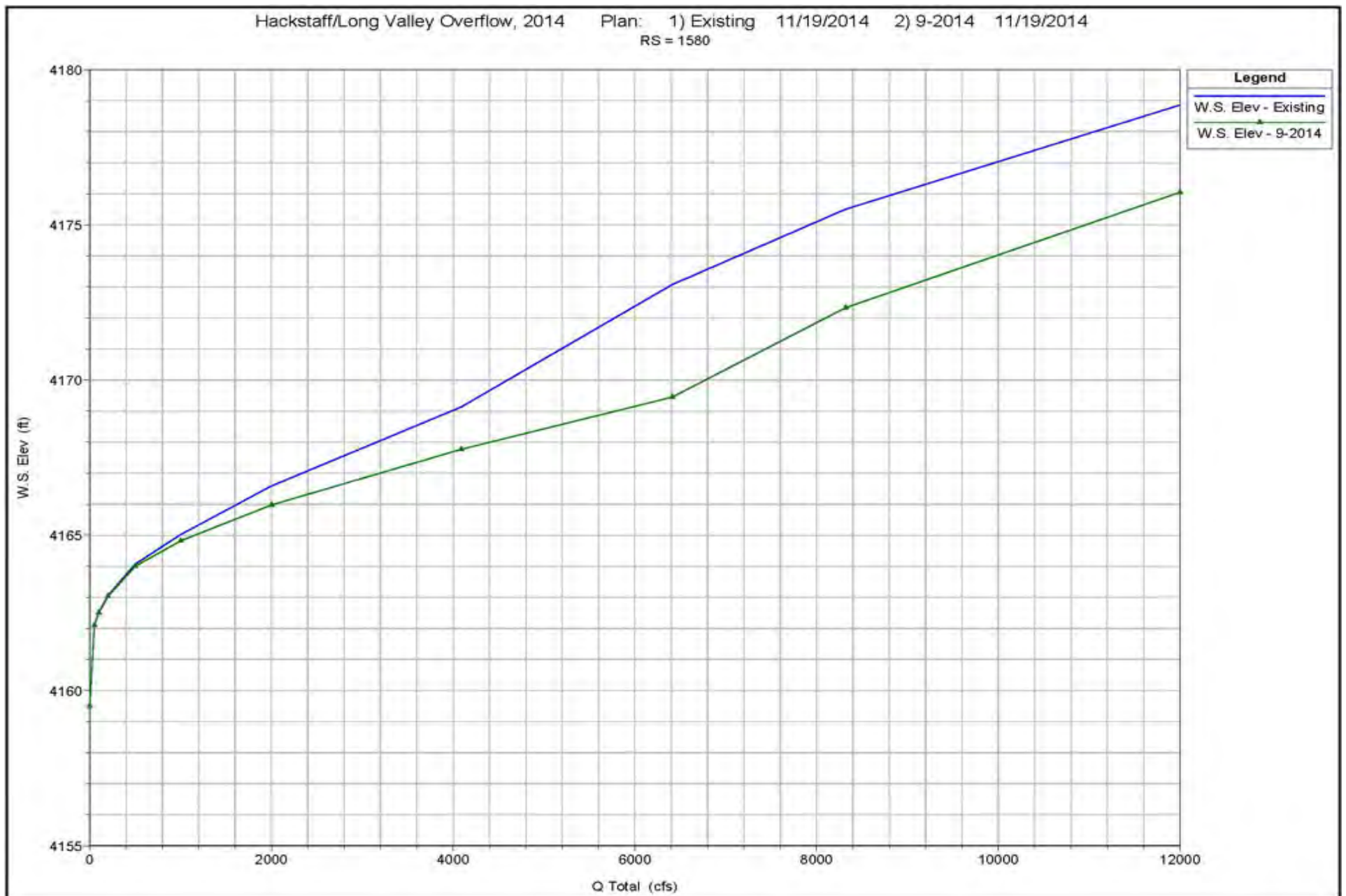


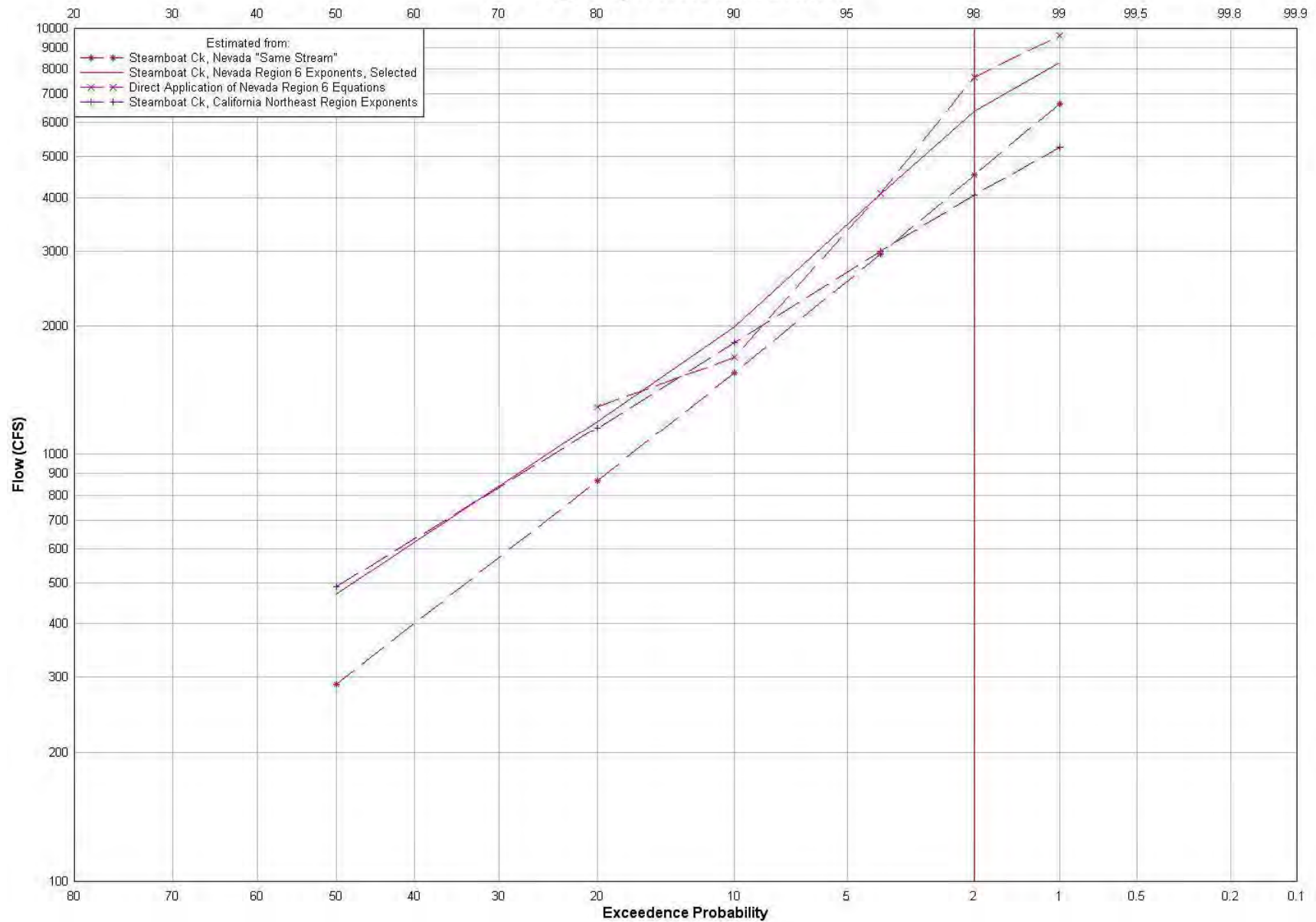
Figure 10: Existing and Preferred (9-2014) Bridge Stage-Discharge Curve at Cross-section 1580

# APPENDIX A

## Additional Hydrologic Data

# Candidate Flood Frequency Curves

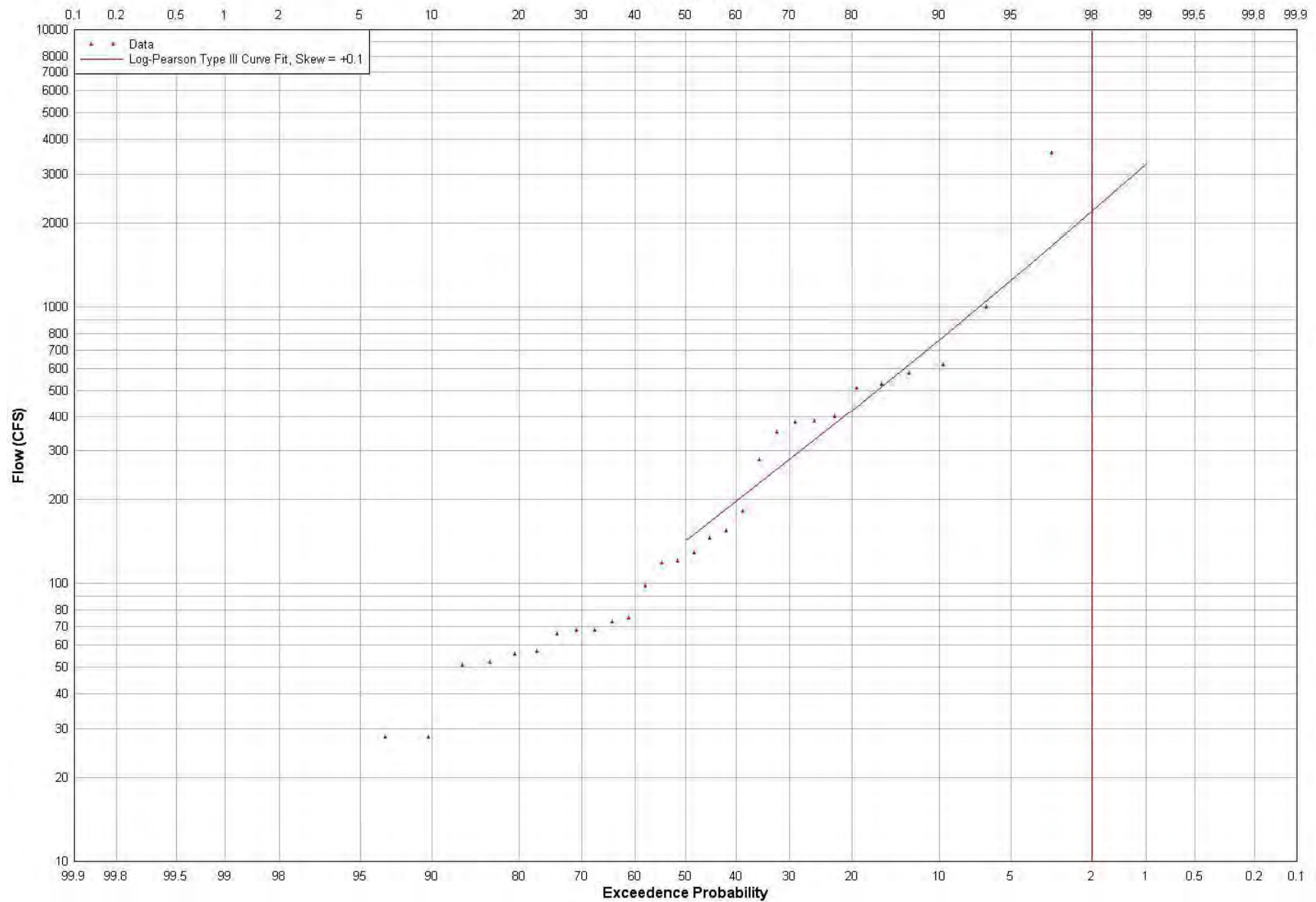
Long Valley Creek at Hackstaff Road



# Steamboat Creek at Steamboat, Nevada

USGS Streamgage 10349300: 30-years, 123-sq mi, 6000-ft avg

Discontinued



# APPENDIX B

## Additional Hydraulic Data



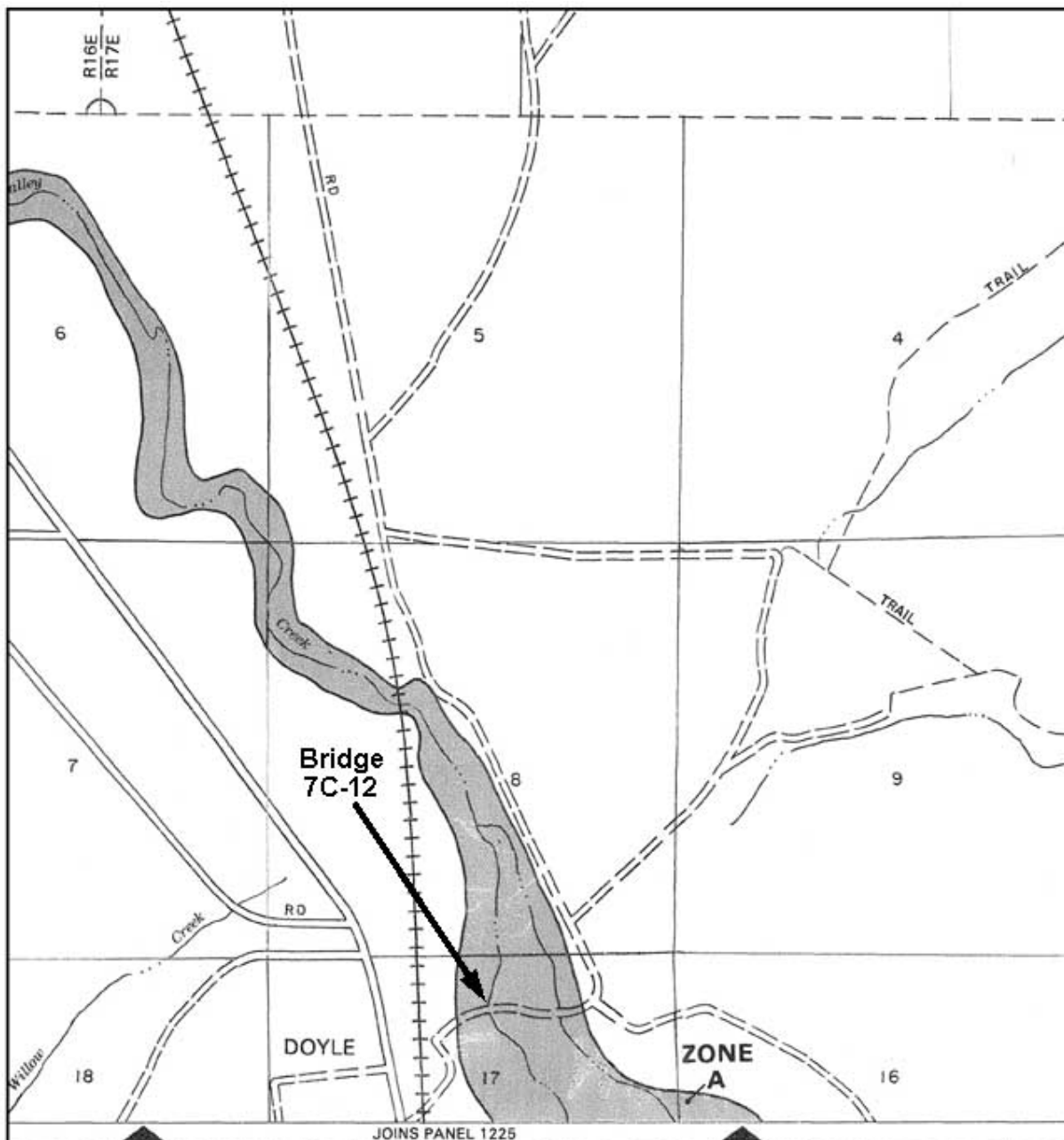
Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	3660	2000-cfs Q10	2000	4166.46	4173.92	4170.84	4174.26	0.001442	4.86	471	239.51	0.358
1	3660	4090-cfs Q25	4090	4166.46	4176.50	4173.26	4176.95	0.001553	5.84	876	333.85	0.373
1	3660	6410-cfs Q50	6410	4166.46	4178.46	4174.88	4178.97	0.001609	6.47	1283	382.02	0.381
1	3660	8320-cfs Q100	8320	4166.46	4179.80	4175.97	4180.33	0.001601	6.77	1610	414.85	0.381
1	3336.66*	2000-cfs Q10	2000	4166.43	4173.24	4170.88	4173.69	0.002083	5.45	387	225.10	0.436
1	3336.66*	4090-cfs Q25	4090	4166.43	4175.68	4173.08	4176.35	0.002163	6.80	693	301.48	0.465
1	3336.66*	6410-cfs Q50	6410	4166.43	4177.62	4174.80	4178.37	0.002103	7.50	1060	361.51	0.470
1	3336.66*	8320-cfs Q100	8320	4166.43	4178.96	4176.02	4179.74	0.002074	7.80	1388	450.89	0.472
1	3013.33*	2000-cfs Q10	2000	4166.28	4172.31	4170.62	4172.87	0.003065	6.03	334	205.03	0.519
1	3013.33*	4090-cfs Q25	4090	4166.28	4174.67	4172.60	4175.51	0.003060	7.44	582	282.70	0.545
1	3013.33*	6410-cfs Q50	6410	4166.28	4176.67	4174.23	4177.59	0.002749	8.03	925	383.70	0.533
1	3013.33*	8320-cfs Q100	8320	4166.28	4178.18	4175.72	4179.04	0.002324	7.96	1249	466.59	0.498
1	2690	2000-cfs Q10	2000	4165.55	4170.89	4169.75	4171.64	0.004756	6.95	288	207.29	0.633
1	2690	4090-cfs Q25	4090	4165.55	4173.29	4171.65	4174.33	0.004278	8.21	511	289.94	0.633
1	2690	6410-cfs Q50	6410	4165.55	4175.50	4173.47	4176.59	0.003443	8.52	801	354.69	0.588
1	2690	8320-cfs Q100	8320	4165.55	4177.14	4174.54	4178.20	0.002876	8.49	1066	426.09	0.549
1	2320.*	2000-cfs Q10	2000	4163.92	4169.08	4167.88	4169.89	0.004754	7.19	278	90.99	0.639
1	2320.*	4090-cfs Q25	4090	4163.92	4171.54	4169.96	4172.71	0.004526	8.66	472	199.54	0.655
1	2320.*	6410-cfs Q50	6410	4163.92	4173.96	4171.68	4175.25	0.003755	9.14	701	276.50	0.618
1	2320.*	8320-cfs Q100	8320	4163.92	4175.89	4172.87	4177.11	0.002995	8.95	979	367.69	0.562
1	1950.*	2000-cfs Q10	2000	4161.79	4167.24	4166.14	4168.10	0.004966	7.43	269	67.27	0.654
1	1950.*	4090-cfs Q25	4090	4161.79	4169.72	4168.21	4170.98	0.004822	9.01	454	183.88	0.677
1	1950.*	6410-cfs Q50	6410	4161.79	4172.74	4169.99	4173.97	0.003160	8.89	721	221.94	0.564
1	1950.*	8320-cfs Q100	8320	4161.79	4174.79	4171.15	4176.04	0.002840	8.98	940	270.05	0.539
1	1580	2000-cfs Q10	2000	4159.50	4166.32	4164.34	4166.89	0.002071	6.08	329	72.12	0.502
1	1580	4090-cfs Q25	4090	4159.50	4168.69	4166.42	4169.65	0.002511	7.90	518	186.45	0.575
1	1580	6410-cfs Q50	6410	4159.50	4172.26	4168.25	4173.09	0.001473	7.32	876	247.16	0.461
1	1580	8320-cfs Q100	8320	4159.50	4174.45	4169.48	4175.27	0.001175	7.25	1147	294.38	0.444
1	1560		Bridge									
1	1520	2000-cfs Q10	2000	4159.10	4165.81	4163.95	4166.41	0.002220	6.23	321	71.62	0.518
1	1520	4090-cfs Q25	4090	4159.10	4167.16	4166.04	4168.61	0.004333	9.66	423	170.24	0.743
1	1520	6410-cfs Q50	6410	4159.10	4168.07	4167.85	4170.62	0.006852	12.83	500	183.87	0.946
1	1520	8320-cfs Q100	8320	4159.10	4169.06	4169.06	4172.14	0.007375	14.10	590	198.89	0.996
1	1460	2000-cfs Q10	2000	4160.00	4165.85	4163.27	4166.18	0.001199	4.63	432	95.09	0.383
1	1460	4090-cfs Q25	4090	4160.00	4167.32	4165.08	4168.09	0.002153	7.07	579	206.42	0.529
1	1460	6410-cfs Q50	6410	4160.00	4168.42	4166.59	4169.73	0.003093	9.19	697	248.36	0.646
1	1460	8320-cfs Q100	8320	4160.00	4169.14	4167.67	4170.91	0.003811	10.69	778	257.12	0.725
1	1115.*	2000-cfs Q10	2000	4160.00	4165.89	4162.09	4165.96	0.000216	2.09	955	258.43	0.166
1	1115.*	4090-cfs Q25	4090	4160.00	4167.49	4163.12	4167.65	0.000368	3.22	1270	342.96	0.225
1	1115.*	6410-cfs Q50	6410	4160.00	4168.79	4164.06	4169.07	0.000503	4.18	1534	354.60	0.270
1	1115.*	8320-cfs Q100	8320	4160.00	4169.70	4164.72	4170.06	0.000596	4.83	1723	363.91	0.298
1	770	2000-cfs Q10	2000	4160.68	4165.81	4163.25	4165.87	0.000289	1.94	1030	373.35	0.182
1	770	4090-cfs Q25	4090	4160.68	4167.40	4164.04	4167.52	0.000356	2.73	1499	451.17	0.214
1	770	6410-cfs Q50	6410	4160.68	4168.70	4164.71	4168.88	0.000414	3.40	1888	457.43	0.239
1	770	8320-cfs Q100	8320	4160.68	4169.62	4165.21	4169.85	0.000450	3.85	2164	461.75	0.254
1	0	2000-cfs Q10	2000	4160.52	4164.97	4164.07	4165.32	0.003000	4.75	421	348.56	0.545
1	0	4090-cfs Q25	4090	4160.52	4166.32	4165.16	4166.87	0.003001	5.95	688	459.13	0.578
1	0	6410-cfs Q50	6410	4160.52	4167.38	4166.12	4168.14	0.003004	7.00	915	470.31	0.602
1	0	8320-cfs Q100	8320	4160.52	4168.14	4166.71	4169.06	0.003003	7.69	1082	478.22	0.616

HEC-RAS Plan: Existing River: Long Valley Cree Reach: 1 Backwater Model Summary Output, Existing Condition With Drift

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	3660	2000-cfs Q10	2000	4166.46	4173.92	4170.84	4174.26	0.001442	4.86	471	239.51	0.358
1	3660	4090-cfs Q25	4090	4166.46	4176.50	4173.26	4176.95	0.001553	5.84	876	333.86	0.373
1	3660	6410-cfs Q50	6410	4166.46	4178.49	4174.88	4178.99	0.001593	6.44	1289	382.62	0.379
1	3660	8320-cfs Q100	8320	4166.46	4179.88	4175.97	4180.40	0.001554	6.69	1631	416.79	0.375
1	3336.66*	2000-cfs Q10	2000	4166.43	4173.24	4170.88	4173.69	0.002084	5.45	387	225.09	0.436
1	3336.66*	4090-cfs Q25	4090	4166.43	4175.69	4173.08	4176.35	0.002162	6.80	693	301.49	0.465
1	3336.66*	6410-cfs Q50	6410	4166.43	4177.66	4174.80	4178.40	0.002068	7.44	1068	362.90	0.466
1	3336.66*	8320-cfs Q100	8320	4166.43	4179.10	4176.02	4179.84	0.001951	7.60	1424	455.50	0.458
1	3013.33*	2000-cfs Q10	2000	4166.28	4172.31	4170.62	4172.87	0.003066	6.03	334	205.03	0.519
1	3013.33*	4090-cfs Q25	4090	4166.28	4174.67	4172.60	4175.51	0.003058	7.44	582	282.75	0.545
1	3013.33*	6410-cfs Q50	6410	4166.28	4176.74	4174.23	4177.64	0.002649	7.92	940	386.43	0.523
1	3013.33*	8320-cfs Q100	8320	4166.28	4178.39	4175.72	4179.19	0.002134	7.67	1300	474.43	0.478
1	2690	2000-cfs Q10	2000	4165.55	4170.89	4169.75	4171.64	0.004758	6.95	288	207.28	0.633
1	2690	4090-cfs Q25	4090	4165.55	4173.29	4171.65	4174.33	0.004270	8.21	512	290.02	0.633
1	2690	6410-cfs Q50	6410	4165.55	4175.67	4173.47	4176.70	0.003192	8.28	826	359.65	0.568
1	2690	8320-cfs Q100	8320	4165.55	4177.48	4174.54	4178.44	0.002508	8.06	1127	445.98	0.515
1	2320.*	2000-cfs Q10	2000	4163.92	4169.07	4167.88	4169.88	0.004783	7.20	278	84.23	0.640
1	2320.*	4090-cfs Q25	4090	4163.92	4171.57	4169.96	4172.72	0.004465	8.62	474	199.93	0.651
1	2320.*	6410-cfs Q50	6410	4163.92	4174.37	4171.68	4175.52	0.003184	8.60	755	307.43	0.571
1	2320.*	8320-cfs Q100	8320	4163.92	4176.53	4172.87	4177.54	0.002365	8.17	1091	410.89	0.502
1	1950.*	2000-cfs Q10	2000	4161.79	4167.36	4166.14	4168.17	0.004559	7.21	277	67.86	0.629
1	1950.*	4090-cfs Q25	4090	4161.79	4169.96	4168.21	4171.12	0.004288	8.64	474	187.48	0.641
1	1950.*	6410-cfs Q50	6410	4161.79	4173.41	4169.99	4174.45	0.002521	8.18	784	229.24	0.506
1	1950.*	8320-cfs Q100	8320	4161.79	4175.71	4171.15	4176.71	0.002167	8.02	1080	348.09	0.472
1	1580	2000-cfs Q10	2000	4159.50	4166.59	4164.34	4167.10	0.001758	5.74	348	73.64	0.465
1	1580	4090-cfs Q25	4090	4159.50	4169.14	4166.42	4169.97	0.002046	7.33	558	192.88	0.523
1	1580	6410-cfs Q50	6410	4159.50	4173.07	4168.25	4173.75	0.001112	6.61	969	261.80	0.408
1	1580	8320-cfs Q100	8320	4159.50	4175.50	4169.48	4176.14	0.000822	6.41	1323	378.42	0.375
1	1560		Bridge									
1	1520	2000-cfs Q10	2000	4159.10	4165.81	4163.95	4166.41	0.002220	6.23	321	71.62	0.518
1	1520	4090-cfs Q25	4090	4159.10	4167.16	4166.04	4168.61	0.004333	9.66	423	170.24	0.743
1	1520	6410-cfs Q50	6410	4159.10	4168.07	4167.85	4170.62	0.006852	12.83	500	183.87	0.946
1	1520	8320-cfs Q100	8320	4159.10	4169.06	4169.06	4172.14	0.007375	14.10	590	198.89	0.996
1	1460	2000-cfs Q10	2000	4160.00	4165.85	4163.27	4166.18	0.001199	4.63	432	95.09	0.383
1	1460	4090-cfs Q25	4090	4160.00	4167.32	4165.08	4168.09	0.002153	7.07	579	206.42	0.529
1	1460	6410-cfs Q50	6410	4160.00	4168.42	4166.59	4169.73	0.003093	9.19	697	248.36	0.646
1	1460	8320-cfs Q100	8320	4160.00	4169.14	4167.67	4170.91	0.003811	10.69	778	257.12	0.725
1	1115.*	2000-cfs Q10	2000	4160.00	4165.89	4162.09	4165.96	0.000216	2.09	955	258.43	0.166
1	1115.*	4090-cfs Q25	4090	4160.00	4167.49	4163.12	4167.65	0.000368	3.22	1270	342.96	0.225
1	1115.*	6410-cfs Q50	6410	4160.00	4168.79	4164.06	4169.07	0.000503	4.18	1534	354.60	0.270
1	1115.*	8320-cfs Q100	8320	4160.00	4169.70	4164.72	4170.06	0.000596	4.83	1723	363.91	0.298
1	770	2000-cfs Q10	2000	4160.68	4165.81	4163.25	4165.87	0.000289	1.94	1030	373.35	0.182
1	770	4090-cfs Q25	4090	4160.68	4167.40	4164.04	4167.52	0.000356	2.73	1499	451.17	0.214
1	770	6410-cfs Q50	6410	4160.68	4168.70	4164.71	4168.88	0.000414	3.40	1888	457.43	0.239
1	770	8320-cfs Q100	8320	4160.68	4169.62	4165.21	4169.85	0.000450	3.85	2164	461.75	0.254
1	0	2000-cfs Q10	2000	4160.52	4164.97	4164.07	4165.32	0.003000	4.75	421	348.56	0.545
1	0	4090-cfs Q25	4090	4160.52	4166.32	4165.16	4166.87	0.003001	5.95	688	459.13	0.578
1	0	6410-cfs Q50	6410	4160.52	4167.38	4166.12	4168.14	0.003004	7.00	915	470.31	0.602
1	0	8320-cfs Q100	8320	4160.52	4168.14	4166.71	4169.06	0.003003	7.69	1082	478.22	0.616



Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # Chl
1	3660	2000-cfs Q10	2000	4166.46	4173.92	4170.84	4174.26	0.001442	4.86	471	239.51	0.358
1	3660	4090-cfs Q25	4090	4166.46	4176.50	4173.26	4176.95	0.001553	5.84	876	333.85	0.373
1	3660	6410-cfs Q50	6410	4166.46	4178.45	4174.88	4178.96	0.001620	6.49	1279	381.61	0.382
1	3660	8320-cfs Q100	8320	4166.46	4179.74	4175.97	4180.29	0.001635	6.83	1596	413.50	0.385
1	3336.66*	2000-cfs Q10	2000	4166.43	4173.24	4170.88	4173.69	0.002084	5.45	387	225.09	0.436
1	3336.66*	4090-cfs Q25	4090	4166.43	4175.68	4173.08	4176.35	0.002163	6.80	693	301.48	0.465
1	3336.66*	6410-cfs Q50	6410	4166.43	4177.59	4174.80	4178.35	0.002127	7.53	1054	359.32	0.472
1	3336.66*	8320-cfs Q100	8320	4166.43	4178.86	4176.02	4179.68	0.002170	7.95	1361	447.52	0.482
1	3013.33*	2000-cfs Q10	2000	4166.28	4172.31	4170.62	4172.87	0.003066	6.03	334	205.03	0.519
1	3013.33*	4090-cfs Q25	4090	4166.28	4174.67	4172.60	4175.51	0.003060	7.44	582	282.70	0.545
1	3013.33*	6410-cfs Q50	6410	4166.28	4176.61	4174.23	4177.56	0.002825	8.12	914	381.68	0.540
1	3013.33*	8320-cfs Q100	8320	4166.28	4178.01	4175.72	4178.93	0.002486	8.20	1211	460.43	0.515
1	2690	2000-cfs Q10	2000	4165.55	4170.88	4169.75	4171.63	0.004765	6.95	288	207.23	0.634
1	2690	4090-cfs Q25	4090	4165.55	4173.29	4171.65	4174.33	0.004280	8.21	511	289.91	0.633
1	2690	6410-cfs Q50	6410	4165.55	4175.37	4173.47	4176.51	0.003670	8.72	780	350.56	0.606
1	2690	8320-cfs Q100	8320	4165.55	4176.84	4174.54	4178.00	0.003255	8.90	1014	418.40	0.582
1	2320.*	2000-cfs Q10	2000	4163.92	4169.10	4167.88	4169.89	0.004710	7.17	279	111.83	0.636
1	2320.*	4090-cfs Q25	4090	4163.92	4171.53	4169.96	4172.70	0.004544	8.67	471	199.42	0.656
1	2320.*	6410-cfs Q50	6410	4163.92	4173.52	4171.68	4175.00	0.004453	9.76	657	252.52	0.670
1	2320.*	8320-cfs Q100	8320	4163.92	4175.08	4172.87	4176.64	0.004062	10.05	853	329.30	0.650
1	1950.*	2000-cfs Q10	2000	4161.79	4167.13	4166.14	4168.04	0.005389	7.64	262	66.73	0.679
1	1950.*	4090-cfs Q25	4090	4161.79	4169.42	4168.21	4170.83	0.005621	9.53	429	179.60	0.728
1	1950.*	6410-cfs Q50	6410	4161.79	4171.27	4169.99	4173.12	0.005684	10.93	586	206.30	0.748
1	1950.*	8320-cfs Q100	8320	4161.79	4173.19	4171.15	4175.04	0.004545	10.92	762	226.19	0.679
1	1580	2000-cfs Q10	2000	4159.50	4165.97	4164.34	4166.65	0.002580	6.57	304	70.19	0.556
1	1580	4090-cfs Q25	4090	4159.50	4167.77	4166.42	4169.11	0.003884	9.30	440	172.79	0.706
1	1580	6410-cfs Q50	6410	4159.50	4169.45	4168.25	4171.30	0.004376	10.92	587	197.93	0.768
1	1580	8320-cfs Q100	8320	4159.50	4172.33	4169.48	4173.71	0.002420	9.41	884	248.33	0.592
1	1520	2000-cfs Q10	2000	4159.10	4165.90	4163.95	4166.48	0.002101	6.11	327	72.15	0.505
1	1520	4090-cfs Q25	4090	4159.10	4167.64	4166.04	4168.85	0.003417	8.83	463	177.46	0.664
1	1520	6410-cfs Q50	6410	4159.10	4169.31	4167.85	4171.00	0.003921	10.43	615	204.62	0.729
1	1520	8320-cfs Q100	8320	4159.10	4172.29	4169.06	4173.53	0.002131	8.91	933	255.30	0.557
1	1500	Bridge										
1	1460	2000-cfs Q10	2000	4160.00	4165.91	4163.27	4166.23	0.001158	4.57	437	95.53	0.377
1	1460	4090-cfs Q25	4090	4160.00	4167.45	4165.08	4168.19	0.002007	6.90	593	213.37	0.512
1	1460	6410-cfs Q50	6410	4160.00	4168.66	4166.59	4169.88	0.002772	8.84	725	251.35	0.614
1	1460	8320-cfs Q100	8320	4160.00	4169.49	4167.67	4171.09	0.003295	10.15	819	261.44	0.678
1	1115.*	2000-cfs Q10	2000	4160.00	4165.89	4162.09	4165.96	0.000216	2.09	955	258.43	0.166
1	1115.*	4090-cfs Q25	4090	4160.00	4167.49	4163.12	4167.65	0.000368	3.22	1270	342.96	0.225
1	1115.*	6410-cfs Q50	6410	4160.00	4168.79	4164.06	4169.07	0.000503	4.18	1534	354.60	0.270
1	1115.*	8320-cfs Q100	8320	4160.00	4169.70	4164.72	4170.06	0.000596	4.83	1723	363.91	0.298
1	770	2000-cfs Q10	2000	4160.68	4165.81	4163.25	4165.87	0.000289	1.94	1030	373.35	0.182
1	770	4090-cfs Q25	4090	4160.68	4167.40	4164.04	4167.52	0.000356	2.73	1499	451.17	0.214
1	770	6410-cfs Q50	6410	4160.68	4168.70	4164.71	4168.88	0.000414	3.40	1888	457.43	0.239
1	770	8320-cfs Q100	8320	4160.68	4169.62	4165.21	4169.85	0.000450	3.85	2164	461.75	0.254
1	0	2000-cfs Q10	2000	4160.52	4164.97	4164.07	4165.32	0.003000	4.75	421	348.56	0.545
1	0	4090-cfs Q25	4090	4160.52	4166.32	4165.16	4166.87	0.003001	5.95	688	459.13	0.578
1	0	6410-cfs Q50	6410	4160.52	4167.38	4166.12	4168.14	0.003004	7.00	915	470.31	0.602
1	0	8320-cfs Q100	8320	4160.52	4168.14	4166.71	4169.06	0.003003	7.69	1082	478.22	0.616



APPROXIMATE SCALE IN FEET  
2000 0

NATIONAL FLOOD INSURANCE PROGRAM

**FIRM**  
FLOOD INSURANCE RATE MAP

LASSEN COUNTY,  
CALIFORNIA  
(UNINCORPORATED AREAS)

PANEL 1200 OF 1275  
(SEE MAP INDEX FOR PANELS NOT PRINTED)



PANEL LOCATION

COMMUNITY-PANEL NUMBER  
060092 1200 B

EFFECTIVE DATE:  
SEPTEMBER 4, 1987



Federal Emergency Management Agency

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# APPENDIX C

## Scour Computation

# Contraction Scour

Hackstaff Rd over Long Valley Ck, Lassen Co, 9-2014

## 5.3 LIVE-BED CONTRACTION SCOUR

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section.<sup>(43)</sup> The original equation is given in Appendix C. The modification is to eliminate the ratio of Manning's n (see the following Note #3). The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left( \frac{Q_2}{Q_1} \right)^{6/7} \left( \frac{W_1}{W_2} \right)^{k_1} \quad (5.2)$$

$$y_s = y_2 - y_0 = (\text{average contraction scour depth}) \quad (5.3)$$

where:

$y_1$  = Average depth in the upstream main channel (ft) =

12.8

$y_2$  = Average depth in the contracted section (ft) =

$y_0$  = Existing depth in the contracted section before scour (ft) =

9.5

(DS face of structure)

$Q_1$  = Flow in the upstream channel transporting sediment (cfs) =

8320

$Q_2$  = 100-year flow in the contracted channel (cfs) =

8320

$W_1$  = Bottom width of upstream channel transporting bed material (ft) =

60

$W_2$  = Bottom width of contracted section less pier width (ft) =

54

$k_1$  = Exponent determined below

0.59

$V_* / \omega$	$k_1$	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

$V_* = (\tau_o / \rho)^{1/2} = (g y_1 S_1)^{1/2}$ , shear velocity in the upstream section, m/s (ft/s)

$\omega$  = Fall velocity of bed material based on the  $D_{50}$ , m/s (Figure 5.8)  
For fall velocity in English units (ft/s) multiply  $\omega$  in m/s by 3.28

$g$  = Acceleration of gravity (9.81 m/s<sup>2</sup>) (32.2 ft/s<sup>2</sup>)

$S_1$  = Slope of energy grade line of main channel, m/m (ft/ft)

$y_2 / y_1 =$  1.06

$y_2 =$  13.6

$y_s =$  4.1

# NCHRP Live Bed Abutment Scour

## Contraction Scour Included

### Hackstaff Rd over Long Valley Ck, Lassen Co, 9-2014

$$y_{\max} = \alpha_A y_c \quad (8.3)$$

$$y_s = y_{\max} - y_0 \quad (8.4)$$

Where:

$y_{\max}$  = Maximum flow depth resulting from abutment scour (ft) 22.8

$y_c$  = Flow depth including live-bed contraction scour (ft) 13.9

$\alpha_A$  = Amplification factor for live bed conditions = 1.64

$y_s$  = Depth of abutment scour (ft) = 10.0

$y_0$  = Flow depth prior to scour (ft) = 12.8

$$y_c = y_1 \left( \frac{q_{2c}}{q_1} \right)^{6/7} \quad (8.5)$$

Where:

$y_1$  = Upstream flow depth (ft) = 12.8

$q_1$  = Upstream unit discharge (cfs/ft) = 110

$q_{2c}$  = Unit discharge in constricted opening (cfs/ft) = 121

$$q_{2c}/q_1 = 1.10$$

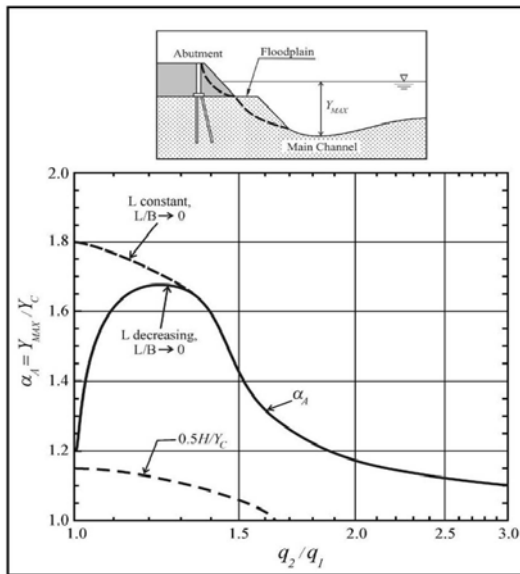


Figure 8.9. Scour amplification factor for spill-through abutments and live-bed conditions (NCHRP 2010b).

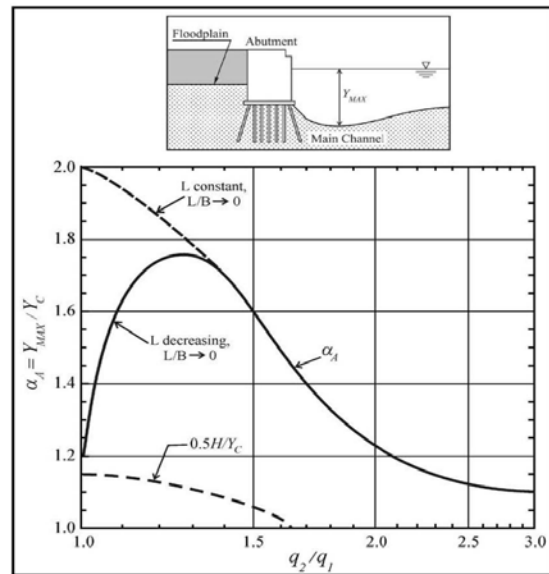


Figure 8.10. Scour amplification factor for wingwall abutments and live-bed conditions (NCHRP 2010b).

# APPENDIX D

## References

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# APPENDIX E

Reference 11



*Norman S. Braithwaite*<sup>1</sup>

## **Bridge Design Considerations - Non Proximate Geologic Hazard**

**Keywords:** Aggradation, Bank Migration, Bridge, Bridge Damage, Bridge Design, Debris Flow, Flood, Geomorphology, Landslide, Risk, Transient Aggradation.

### **Abstract**

*Most bridges are designed considering geologic conditions at the location of the new or replacement bridge. Little thought is generally given to the possibility of upstream landslides and debris flows entering the streams or rivers over which the bridge spans and the potential impact of these landslides on the bridge. Although commonly believed to be an unusual and unpredictable event, a recent series of bridges that were damaged or destroyed as a result of upstream landslides entering the channel begs a closer look. The process by which bridges are damaged during the "transient aggradation" process following a landslide entering a channel is described along with some basic bridge design considerations and measures to minimize the cost of damages should such an event occur. Examples of two recent transient aggradation events that resulted in the destruction of two bridges and significant damages at two additional bridges are discussed.*

### **Introduction**

Most frequently when designing new or replacement bridges, geologic concerns are limited to conditions present at bridge abutments and piers. If located downstream of an area of high landslide potential, however, geologic risks to bridges are not limited to conditions proximate to the bridge. A literature search yielded no published articles addressing the risk of damage to bridges posed by significant upstream landslide potential. This article has been prepared to assist in the identification of risk associated with upstream landslide potential and to suggest some simple design recommendations to minimize damage costs.

Why should a bridge owner or designer be concerned about a landslide if it is not proximate to the bridge? The short answer is that landslides can dramatically affect stream and river channel conditions for many miles downstream of the landslide. Few if any bridge owners would consider designing a bridge with an abutment on a known landslide, so why shouldn't the much greater potential for risks associated with upstream landslides be considered? The risk of damage to bridges presented by upstream landslides has been greatly overlooked because of a lack of knowledge about how upstream landslides affect bridge risk and because of a general belief that landslides are infrequent, unpredictable events. Having recently completed studies for several bridges damaged by landslides entering stream channels

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upstream of bridges, the author believes the risk of damage to bridges from upstream landslides is real and significant.

Not all landslides pose a risk to bridges. Landslides of concern are those that have the ability to deliver a large load of sediment and/or debris to a stream or river channel over a short period of time prior to or during a flood. Soil saturation and toe erosion are primary causes of landslides. Both causes can occur during moderate storms and floods. Therefore, it is reasonable to believe that the greatest risk of a landslide will be during a storm and flood event. In addition to new, unknown and unclassified landslide types that may become active, landslide types classified by the California Department of Mines and Geology as translational/rotational landslide, earthflow, debris landslide, debris flow/torrent track, debris landslide slope, and active landslide are all capable of supplying a significant load of sediment to a stream or river channel during a flood.

### **Stream Reaction to Landslide**

In response to a rapid increase in sediment load (in excess of the streams ability to transport sediment), the stream channel will increase in elevation and develop a wide, shallow, sometimes braided geometry. During the flood and during subsequent floods, the sediment supplied by the landslide will move downstream continuing to impact streambanks and structures as it does so. In some locations, high bank erosion and secondary landslides triggered by the high, wide channel geometry, will further aggravate the process. The length of channel affected and the duration of the process will depend greatly on stream bank conditions (potential for erosion and secondary landslides), the mobility of the added sediments, and the significance and order of subsequent flood events. In some cases, the entire process can occur during one or two moderate floods and leave little clue of having occurred save the presence of unusually high water marks, excessive bank erosion, and damage. The channel response to a rapid increase in sediment load caused by a landslide is succinctly referred to as a transient aggradation process.

### **Bridge Risk**

Risk of damage to bridges during a transient aggradation process is primarily due to high water elevations and bank erosion. During the transient aggradation process high water surface elevations, sometimes much higher than would be expected for floods of much greater magnitude, can result in substantial accumulation of drift (floating debris) on bridge structural elements not designed for such loadings. Steel truss and steel stringer bridges are particularly vulnerable to damage from drift accumulation. Bank erosion at the unusually high water surface elevations can remove material from behind bank protection and abutments leaving the abutments unsupported. As the bulk of excess sediment moves downstream and the stream channel develops a new stable equilibrium (a function of uniform sediment transport, not fixed channel geometry), the reformed stream channel may be located anywhere within the confines of the wide, shallow channel geometry present during the peak of the transient aggradation process. Consequently, after the transient aggradation process has passed, the channel may reform in its previous location, at the location of a former bridge abutment or even off the end of the bridge leaving the bridge high and dry. Damage from pier scour is not likely to be a problem during the transient aggradation process but may become a significant risk as the channel reforms after the process.

## Bridge Risk Reduction Measures

Several reasonable cost measures can be incorporated in the design of a new or replacement bridge to minimize bridge damage. At a minimum the following measures should be considered:

- Using existing geologic hazard maps or by new study, investigate and understand upstream landslide risk prior to bridge design.
- Avoid steel truss and other bridge structures with light decks where upstream landslide potential is high. These structures do not fare well under debris loadings.
- Design abutment foundations so they will not fail if the bridge approach is washed out. Deep large diameter shaft foundations are good where geologic conditions allow.
- Where geometrics and flood risk impacts allow, design bridges with a soffit elevation higher than the minimum recommended by state and federal agencies.
- Design abutment bank protection to elevations above that estimated for the most probable 100-year flood.

If a landslide occurs at a time of low flow when the ability of a stream to transport sediment is limited, it may be possible to mitigate impending impacts by implementing emergency measures to limit transport of excess sediment during a subsequent flood. Prior to taking such action, however, the risks and mitigation measures should be evaluated by qualified hydraulic and geotechnical engineers (appropriate state and federal agencies should also be consulted).

## Recent Examples

### *Case Study 1: Tres Pinos Creek, San Benito County, California, Storm of February 3, 1998.*

During the modest storm of February 3, 1998, a known landslide upstream of several bridges slipped and deposited a substantial volume of rock in Tres Pinos Creek. The weather in 1998 was influenced by a strong El Nino pattern. As such, coastal regions in California received much more precipitation than normal and ground saturation levels were much greater than normal. During a frontal storm event in the first week of February, a modest storm cell tracked from the Big Sur area, through Pinnacles State Park, and through the Tres Pinos Creek basin. Based on regional flood damage the magnitude of the Tres Pinos Creek flood flows are believed to be on the order of a 50- to 75-year flood event. Better estimates of the significance of the Tres Pinos Creek flood were hampered by the loss of streamgages, the lack of precipitation gages, and the high cost of rainfall-runoff modeling using NEX-RAD data. The transient aggradation process resulting from the landslide and flood event damaged the recently constructed Appel Bridge located 1-mile downstream, destroyed the Callens Bridge located 6-miles downstream and left the Graves Bridge, 8-miles downstream, high and dry. Two additional bridges located in the reach suffered minor damage. A bridge damage survey report was prepared to identify and document the nature and causes of damage at the bridges. The "Jones Slide", as it became known, is shown in Figure 1.



*Figure 1. Jones Slide, February 1998. Larger boulders in Tres Pinos Creek are over 20-feet.*

Of particular interest were the damages to the Appel Bridge located closest to the landslide. The Appel bridge was built in 1995 after the previous structure had been seriously damaged by floods earlier that year. Recognizing risks associated with channel instability, deep, large diameter shaft foundations were selected for the replacement bridge piers and abutments. Because the bridge was designed to meet currently recommended minimum bridge design standards, damage at this bridge was unexpected. The damage consisted of the loss of both bridge approaches (Figure 2). The bridge structure itself did not sustain damage. The reformed stream channel was located off the end of the bridge. The former location of the stream channel had been filled with approximately 3 feet of new deposits. High water marks indicated that the bridge and road had not been overtopped.





Figure 2. *Appel Bridge after Jones Slide and flood of February 1998.*

An evaluation of conditions observed at the site confirmed that the stream had developed a wide, shallow geometry and had eroded through Panoche Road by bank erosion at high elevations (without flowing over the road or bridge). The high water marks observed at this site were approximately 2-feet above the water surface elevations estimated for the most probable 100-year flood at the time the bridge was designed. Bank protection that was designed to minimize the risk of damage at the abutments during floods up to the most probable 100-year flood had been overtopped and failed due to the loss of supporting embankment. Progression of the Appel Bridge failure is illustrated in Figure 3. If the bridge abutments had been founded on slab or driven pile foundations relying on bank protection to prevent structural damage, the risk of substantial bridge damage would have been very high.

After evaluating options and recognizing that the bridge hydraulic performance would be less than that for which the bridge was designed, the decision was made to place the bridge back in service by excavating a reasonably sustainable channel under the bridge and replacing the bridge approaches. New bank protection was placed considering floods with higher water surface elevations. Over time, as flood flows continue to transport material downstream, the hydraulic capacity of the bridge is expected to approach the original bridge design hydraulic capacity.

The Callens Bridge was a 95-foot long, 5-span flat slab bridge on substantially unknown foundations. The pier walls were of a design known to capture drift (floating debris). Damage to this bridge consisted of a complete loss of one abutment and three spans (Figure 4). High water marks indicated water surface elevations approximately 10-feet above the water surface elevation estimated for the most probable 100-year flood. Given the very high elevation of water marks left by the February 3, 1998 flood, bridge failure is believed to have resulted from a combination of drift blockage and transient aggradation. Modest deposition of fine soil materials at high elevations upstream of the bridge further supports this mode of failure.

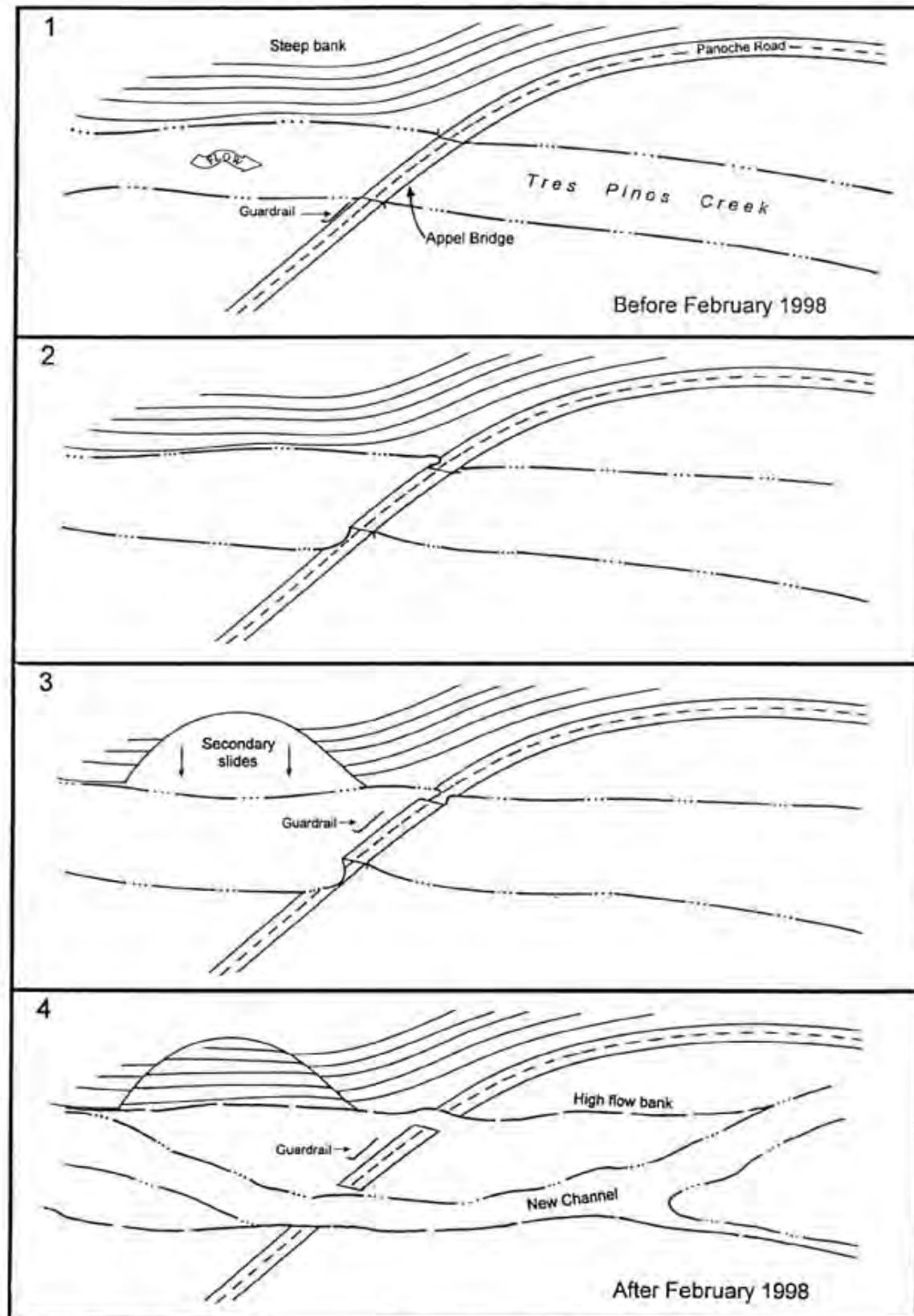


Figure 3. Progression of the Appel Bridge failure.



*Figure 4. Callens Bridge after Jones Slide and flood of February 1998.*

A 110-foot long clear span bridge on vertical wall abutments was selected to replace the failed bridge. Substantial bank protection was placed at and upstream of both abutments to reduce the risk of damage to abutments.

As at the Appel Bridge, Tres Pinos Creek moved out from under the Graves Bridge during the transient aggradation process and reestablished outside of the bridge after the event (Figure 5). The left abutment of the bridge consisted of a short abutment wall supported on two 1-foot diameter cast-in-drilled-hole piles. After the event, more than 10 feet of the piles were exposed and unsupported. Miraculously, these piles were not damaged during the flood and the bridge structure survived. High water marks left by the February 1998 event were approximately 1 foot above the estimated high water for the most probable 100-year flood.

Recognizing that the existing bridge would remain at high risk of failure due to future pier scour, abutment scour, and bank migration, extensive evaluation was conducted before deciding to restore the existing bridge back to service. The scour risks were mitigated by placing buried cable-concrete mats in the channel and considerable rock slope protection along both banks (providing complete protection of the cross-section).





*Figure 5. Graves Bridge after Jones Slide and flood of February 1998. The stream channel had been placed back under the bridge structure but the contractor had difficulty keeping the stream channel in this location. A substantial secondary landslide is visible behind the bridge.*

In spite of all of the damage at these bridges, two intermediate bridges received only minor damage, and some reaches of the stream did not show unusual high water marks or signs of significant flood or sediment load problems. Through these reaches, it is believed the stream channel geometry and hydraulics were such that the stream was capable of transporting all sediment delivered to these reaches.

#### ***Case Study 2: Eagle Creek Loop Road over Trinity River.***

During the January 1, 1997 flood, the Eagle Creek Loop Road bridge over the Trinity River was destroyed. The former bridge was a 75-foot steel truss with short approach spans. Abutments and piers consisted of reinforced concrete walls on flat slab foundations. Based on Trinity River streamgage records, the January 1, 1997 flood peak had a return period of approximately 25 years. Considerable bank erosion was present upstream of the bridge, and considerable deposition at high elevations was present downstream of the bridge. High water marks left by the January 1, 1997 flood indicated substantial blockage at the bridge sometime during the flood. It is believed that this bridge failed due to a transient aggradation process that resulted from a landslide approximately 3 miles upstream. It is not known if the bridge failed due to abutment and pier failure, due to excessive accumulation of debris on the bridge, or due to a combination of the two. Both modes of failure are associated with transient aggradation processes. The high water marks did not help identify the mode of failure because the failed structure would certainly have become a significant obstruction to flood flows. Figure 6 shows the remains of the former bridge.



*Figure 6. Eagle Creek Loop Road over Trinity River after flood of January 1, 1997. One pier wall is submerged. The standing pier and abutment walls are on bedrock.*

### **Historic Examples**

Other clear examples of bridges damaged as a result of transient aggradation processes can be found in sketchy records. From photographic records of flood damage in California Highways and Public Works magazine, several examples of bridges failing as a result of transient aggradation can be identified. A photo of one of these failed bridges, a suspension bridge over the Klamath River at Orleans (Figure 7), clearly shows an immense delta formation immediately upstream of the failed bridge. The formation was most likely the result of a significant landslide into the tributary visible upstream of the bridge. A recent flood frequency analysis of streamflow data at this location indicates the bridge failed during a flood with a peak flow of approximately 50-year recurrence (based on a visual straight line fit of 72-years of flood peak data; based on a computed log-Pearson type III fit the flood peak was of approximately 75-year recurrence). Without a transient aggradation event, this flood should have been conveyed under the bridge soffit. Many more bridges that may have failed from such a process cannot be identified because of the limited extent of photographic and other records.

BELOW: Until December, this was the beautiful State suspension bridge at Orleans, which won an award for design in 1940. Flood topped deck before bridge gave way. (Photo courtesy Six Rivers Forest, U.S. Forest Service.)



Figure 7. Orleans Bridge over Klamath River (California Highways and Public Works, January-February 1965)

## Conclusions

From these examples and untold previous bridge failures, it is apparent that significant bridge damage and failure due to transient aggradation processes resulting from upstream landslides may be much more frequent than commonly believed. Given our present understanding of the transient aggradation process following a landslide, it is not reasonable to design bridges such that no damage will result from a modest transient aggradation event. Even if considering the risks associated with upstream landslides in the design of structures, at many locations damage to bridge approaches, roads, and other infrastructure will be unavoidable consequences of such an event. The measures suggested above, however, will minimize the cost of damage and time to restore service at bridges after a transient aggradation event. To affect a better understanding of bridge risk associated with non-proximate landslides, more thorough bridge failure investigations and documentation should be included in bridge damage assessment reports.