Design Hydraulic Study

Hackstaff Road over Long Valley Creek, Bridge 7C-081

Lassen County

Prepared for:

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May 5, 2015

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Executive Summary

Purpose:	Replace deficient bridge	
Funding Program:	HBR	
Design Flood:	Standard Design Flood (Q50, 6410-cfs)	
Clearance for Drift:	2.0-feet	
Design Exception:	No design exception is anticipated for hydraulic conditions	
Recommendations:	Min. Soffit Elevation –	4174.70-feet (to meet recommendations of Caltrans and FHWA)
	Pier Scour Elevation –	4158.6-feet
	Abutment Scour Elevation –	4160.0-feet
	Abutment Protection –	Recommended to reduce the potential for damage to abutments from bank erosion and bank migration.

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 –	4175.54-feet (1.99-ft above Q100, 2.84-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

INTRODUCTION

Background: This bridge hydraulic analysis has been prepared for the sole purpose of meeting the requirements of 23 CFR §650.115 and §650.117 dealing with bridges, structures, and hydraulics. 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: Six span simple timber stringer

Length – 120.75-ft
Skew – 0-degrees
Clear Width – 21.3-ft
Total Width – 22.0-ft
Lanes – 1
Speed Limit – 55-mph
Load Limit – None (legal loads)
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 at a location immediately downstream of the existing bridge and at a skew of approximately 12.5-degrees from perpendicular to the direction of flood flow.

Length – 160.52-ft nominal, 154.0-ft effective hydraulic
Hydraulic Skew – 12.5-degrees
Clear Width – 32-ft
Total Width – 35.33-ft
Lanes – 2
Speed Limit – 55-mph
Load Limit – None
Structure – Two span RC slab on precast girders supported by RC pier and 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 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 looking downstream and upstream from the existing bridge.

Stream Banks: The banks of Long Valley Creek are very steep to near vertical and consist of cohesive silty loam with a light grass cover on shallower slopes.

Existing Bridge: The existing bridge is aligned near perpendicular to the direction of flood flow in Long Valley Creek.

Site Topography: Figure 4, page 17

HYDROLOGIC ANALYSIS

Hydrologic Environment: The Long Valley Creek channel is one of two channels potentially conveying water during infrequent flood events in Long Valley Creek. Long Valley Creek is presently the main channel and Long Valley Creek Overflow is presently the alternate or overflow channel. 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. Under current conditions assuming no bulking (high sediment and debris load), the full peak flow of Long Valley Creek can be contained within the Long Valley Creek channel without overflow to the Long Valley Creek Overflow channel. Therefore the flood hydrologic analysis for Bridge 7C-81 has been conducted assuming the full flow of Long Valley Creek during infrequent flood events.

Hydrologic Stability: Infrequent floods in Long Valley Creek 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 of 0.030 and 0.045 were used to represent the Long Valley Creek channel and overbank areas respectively.

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 channel, was used as the starting slope. Two surveyed, one derived (cut from detailed topography), and one interpolated cross-section 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[1] (feet)	Avg. Channel Velocity[2] (fps)
Standard Design	6410	50	4175.53	10.1
Base	8320	100	4177.03	11.0
Base, no drift	8320	100	4176.20	11.0
Flood of Record	n/a	n/a	n/a	n/a
Overtopping Flood	9600	200±	4177.52	12±

Notes: 1) At cross-section 1580 located approximately 20-feet upstream of the existing bridge.
2) Highest average channel velocity under bridge.

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 – Effective pier width assumed 5-feet (1.5-foot actual)
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 (with drift except as noted)

Flood	Flow (cfs)	Recurrence (years)	W.S. Elevation[1] (feet)	Avg. Channel Velocity[2] (fps)
Standard Design	6410	50	4172.70	8.4
Base	8320	100	4173.50	9.1
Base (no drift)	8320	100	4173.55	9.1
Flood of Record	n/a	n/a	n/a	n/a
Overtopping Flood	>12000	>200	4177.52	10±

Notes: See notes at the end of 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.

Contraction Local: The preferred bridge does not represent a contraction of the flood channel. Application of the live bed contraction scour equation presented in FHWA HEC-18 (Reference 4) indicates limited risk of contraction scour.

Abutment Local: The abutments of the preferred bridge do not encroach significantly within the Long Valley Creek flood channel. FHWA HEC-18 now recommends the NCHRP method for estimating potential abutment scour. The NCHRP method consists of estimating potential contraction scour and applying an amplification factor. Since contraction scour is not anticipated at the preferred bridge, the potential for abutment scour is likewise not anticipated. The abutments should be designed considering or protected against a minimum of 6-feet of potential scour.

Pier Local: Potential pier scour has been estimated to be 8.4-feet using the CSU equation presented in FHWA HEC-18 (reference 4). An effective pier width of 3.5-feet was used for this calculation.

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	Local Scour	Total Scour	Scour Elev.
Abutments	4166.	0.	0.	6.	6.	4160.
Pier	4166.	0.	0.	8.4	8.4	4158.6

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. 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 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, the critical condition establishing the minimum soffit elevation is the most probable 50-year flood with appropriate 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 necessary for bridge hydraulic conditions.
Recommendations:	Minimum Soffit Elevation – The minimum soffit elevation of a bridge meeting the recommendations of Caltrans and FHWA is 4174.70-feet. This represents the elevation of the Standard Design Flood (50-year flood) plus 2.0-feet of clearance for drift.
	Pier Scour Elevation – Piers should be designed considering total potential scour to an elevation of 4158.6-feet.
	Abutment Scour Elevation – Abutments should be designed considering or protected against total potential scour to an elevation of 4160.0-feet.
	Abutment Protection – Recommended to reduce the long term potential for damage to abutments from bank erosion and bank migration.

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 –	4175.54-feet (1.99-ft above Q100, 2.84-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-81,
Hackstaff Road over Long Valley Creek

Photo 2: Looking upstream (south) at Bridge 7C-81,
Hackstaff Road over Long Valley Creek

Photo 3: Looking east at Bridge 7C-81,
Hackstaff Road over Long Valley Creek

Photo 4: Looking west at Bridge 7C-81,
Hackstaff Road over Long Valley Creek

Photo 5: Looking downstream (north) at Long Valley Creek
from Bridge 7C-81, Hackstaff Road

Photo 6: Looking upstream (south) at Long Valley Creek
from Bridge 7C-81, Hackstaff Road

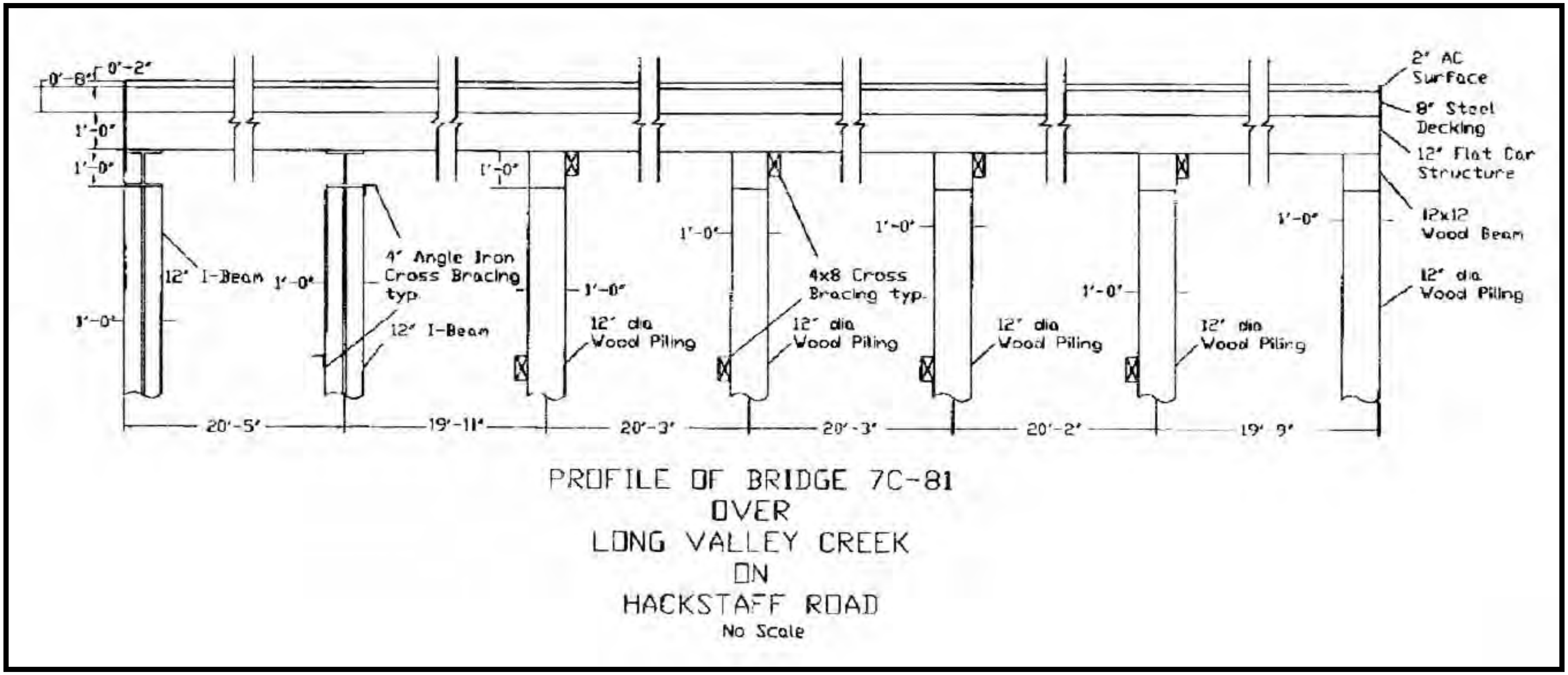


Figure 1: Sketch of Existing Bridge

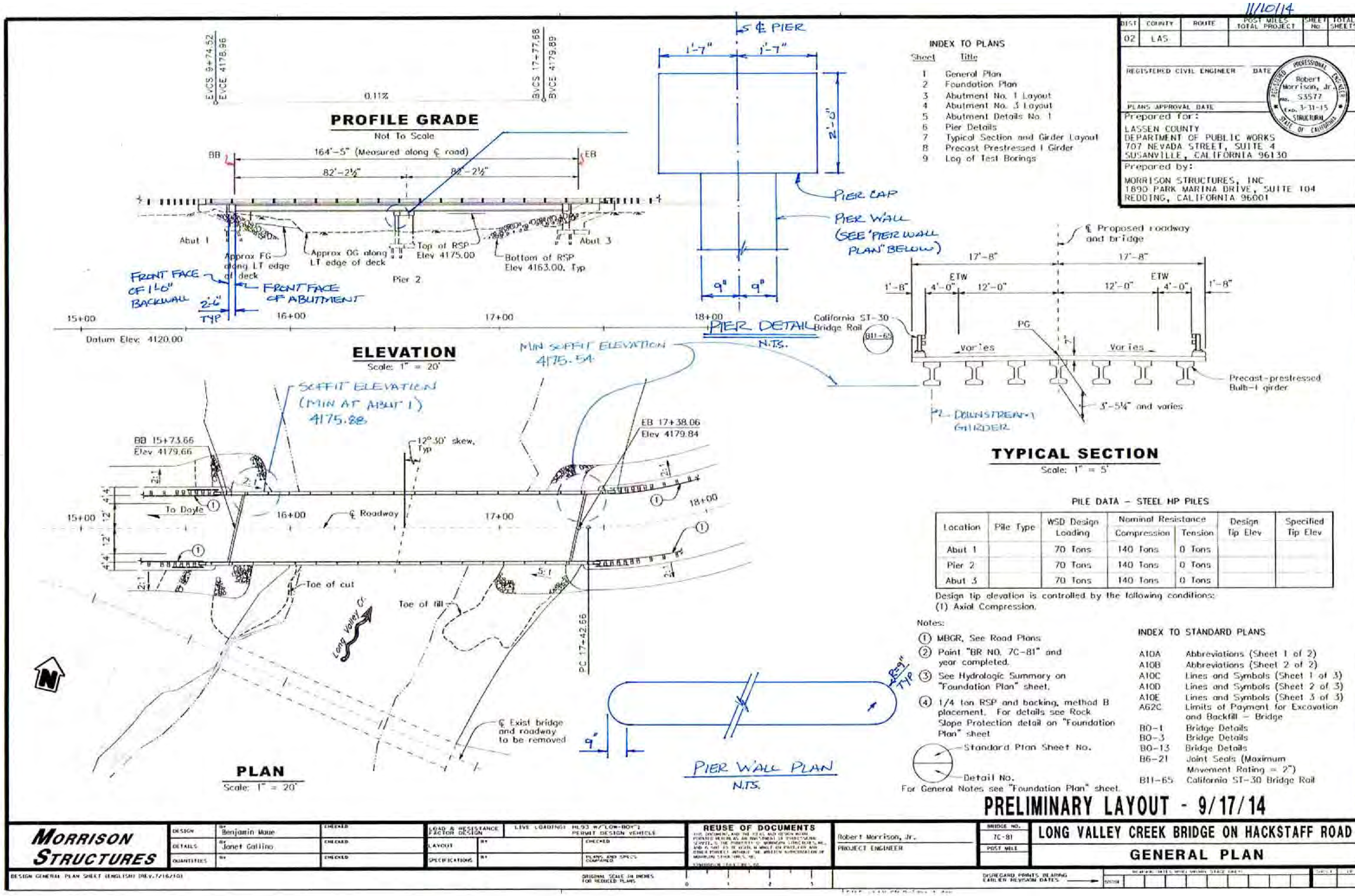


Figure 2: Preferred Bridge

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

Figure 4: Site Topography

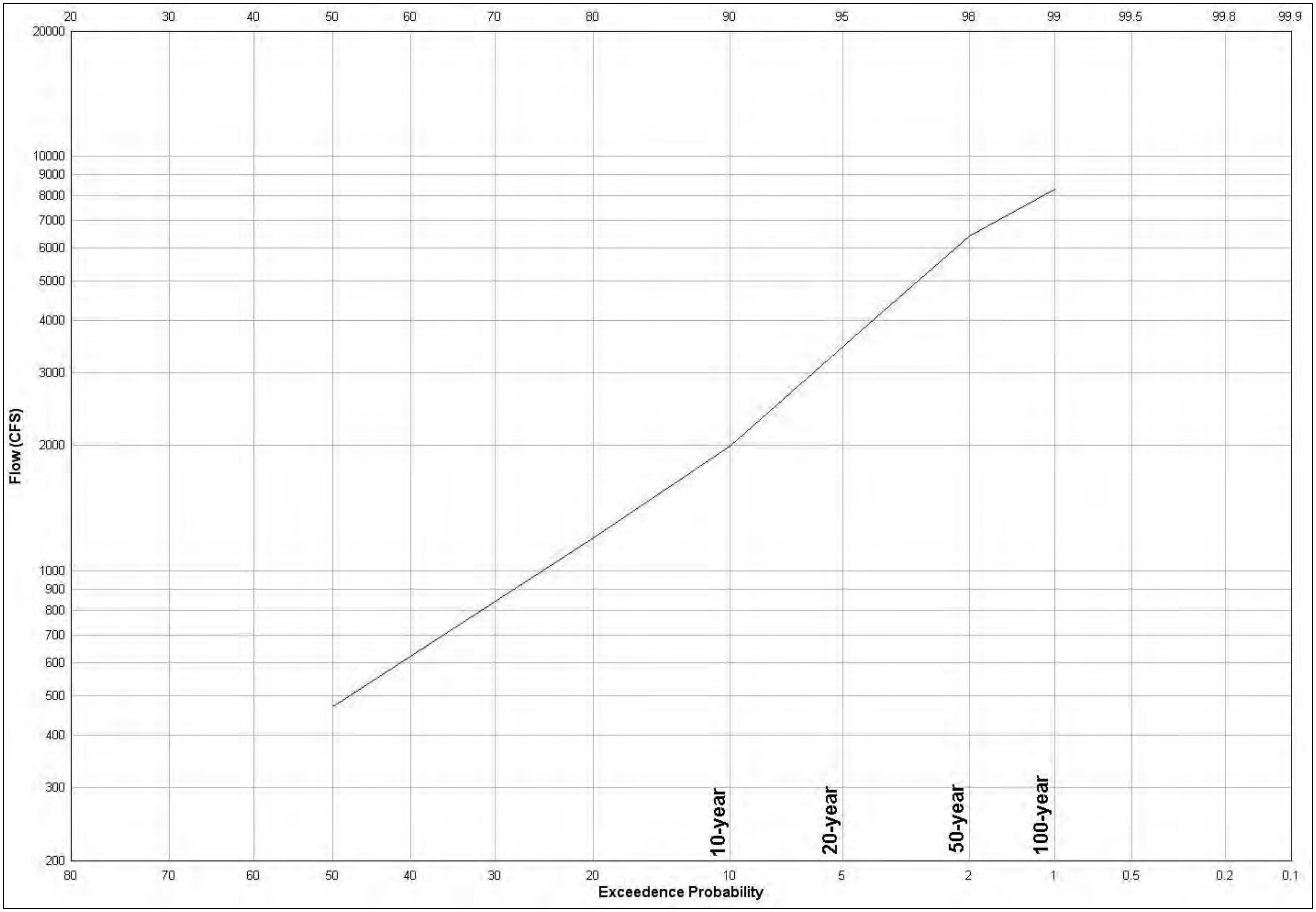


Figure 5: Flood Frequency Curve, Long Valley Creek at Hackstaff Road

Figure 6: Approximate Location of Cross-sections

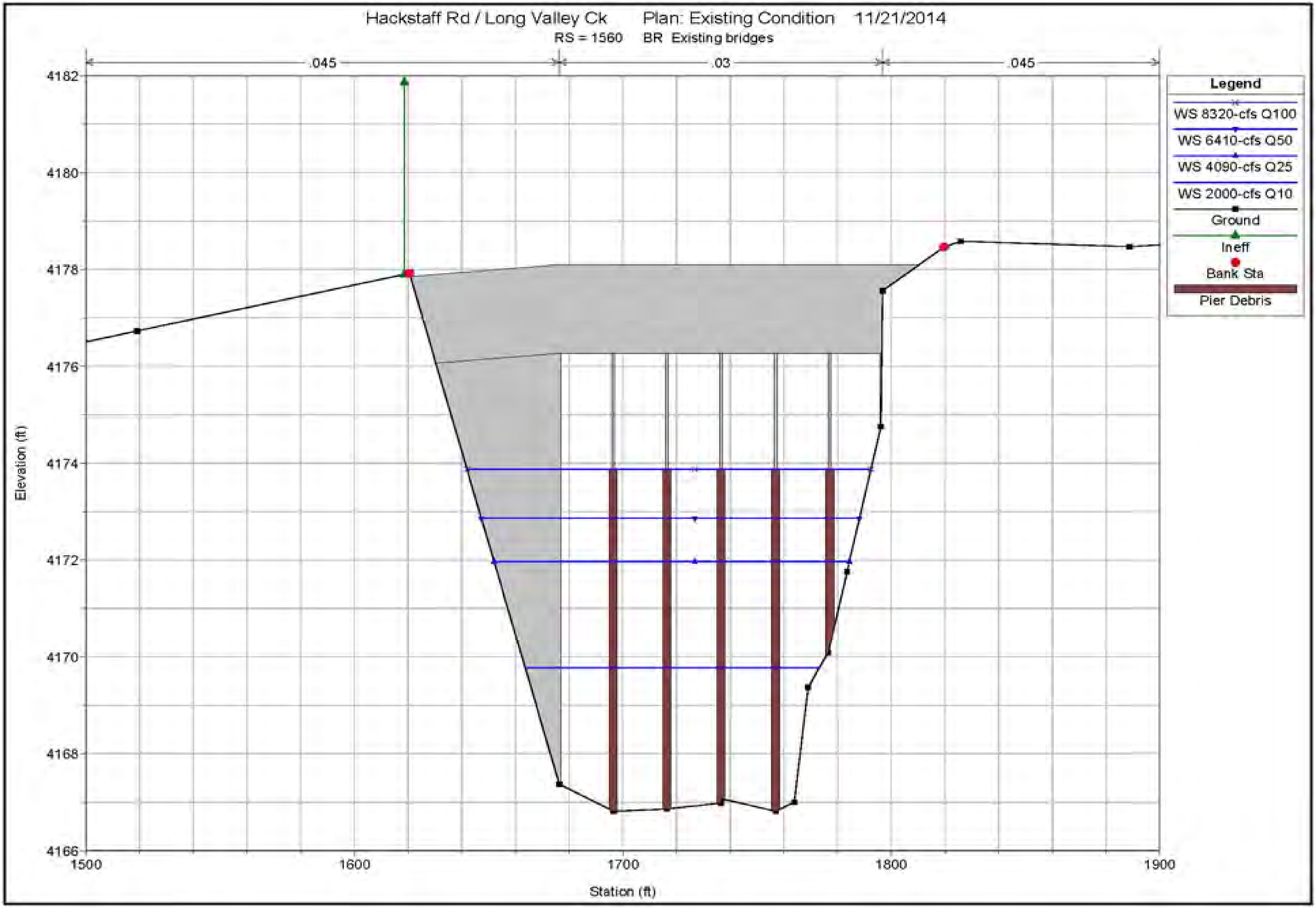


Figure 7: Existing Bridge as Represented in Backwater Model

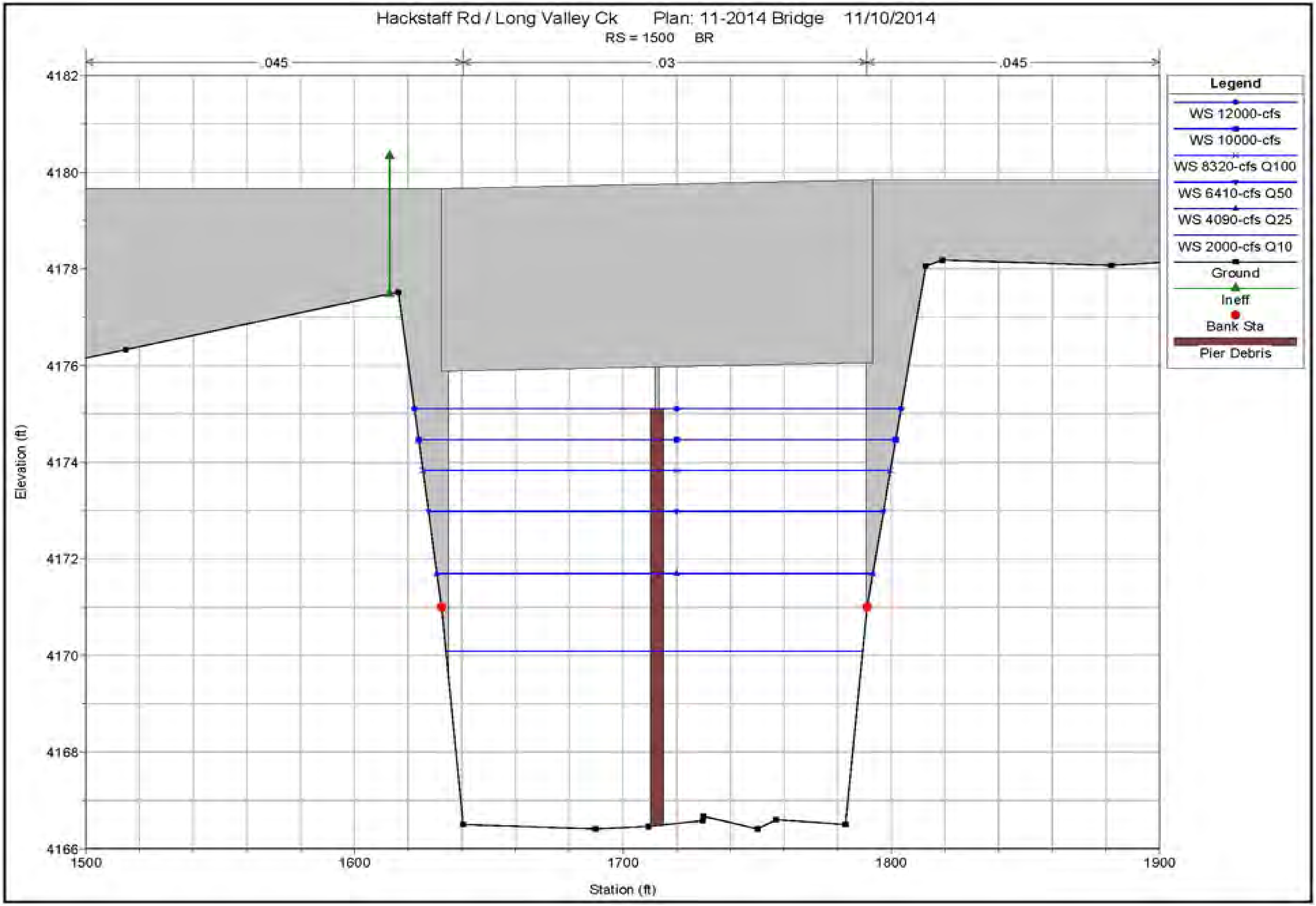


Figure 8: Preferred Bridge as Represented in Backwater Model

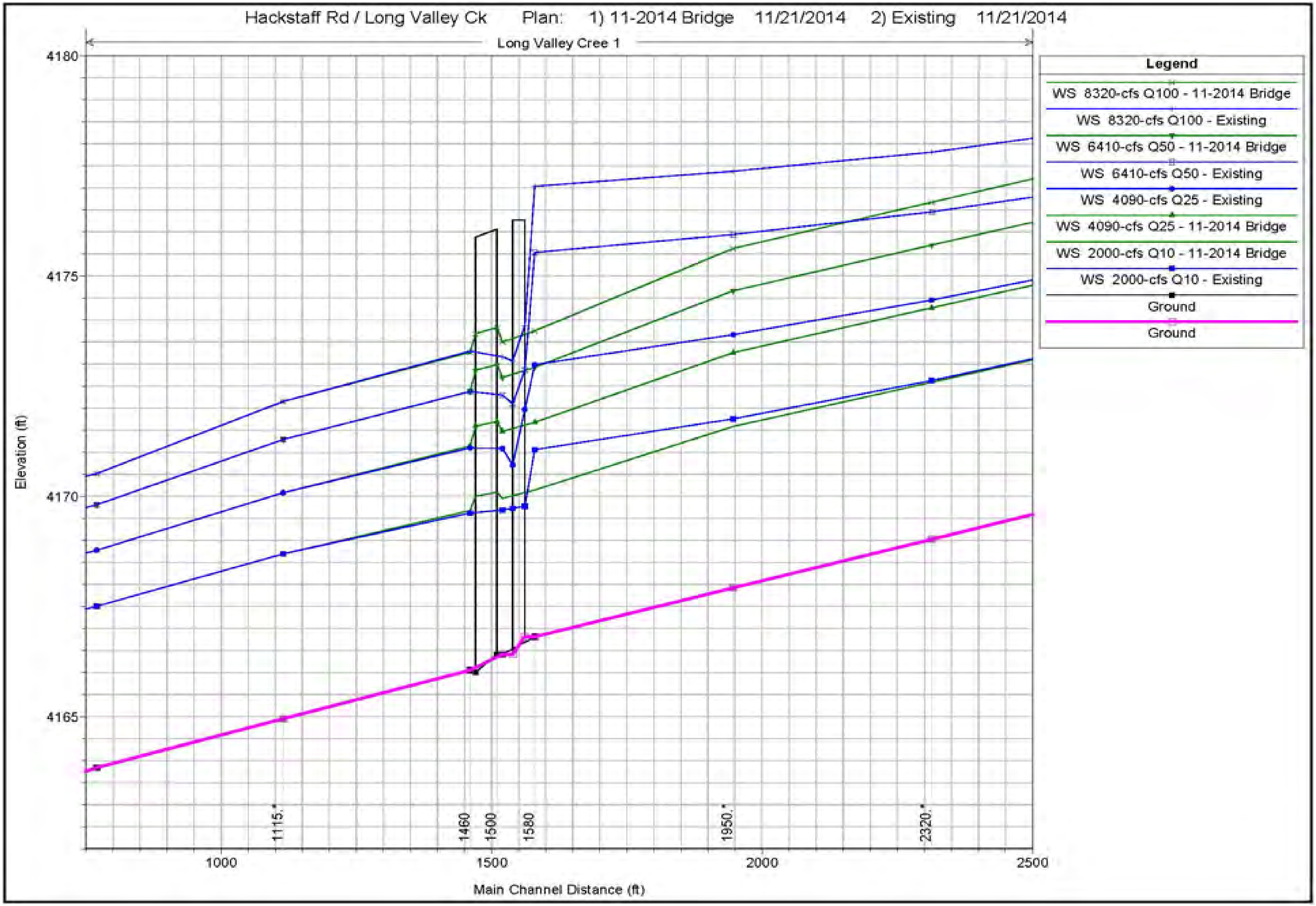


Figure 9: Existing and Preferred (9-2014) Bridge Flood Profiles

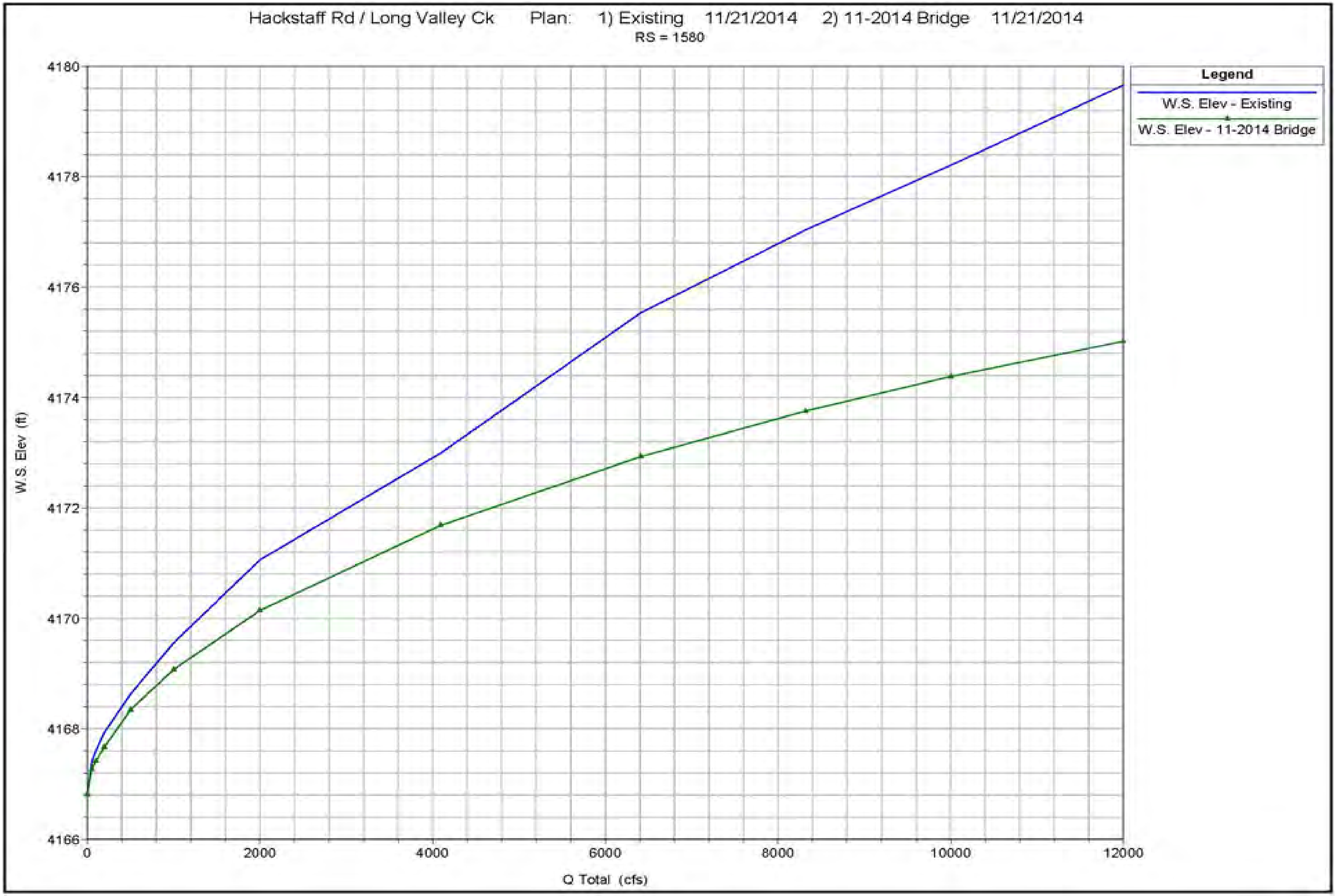
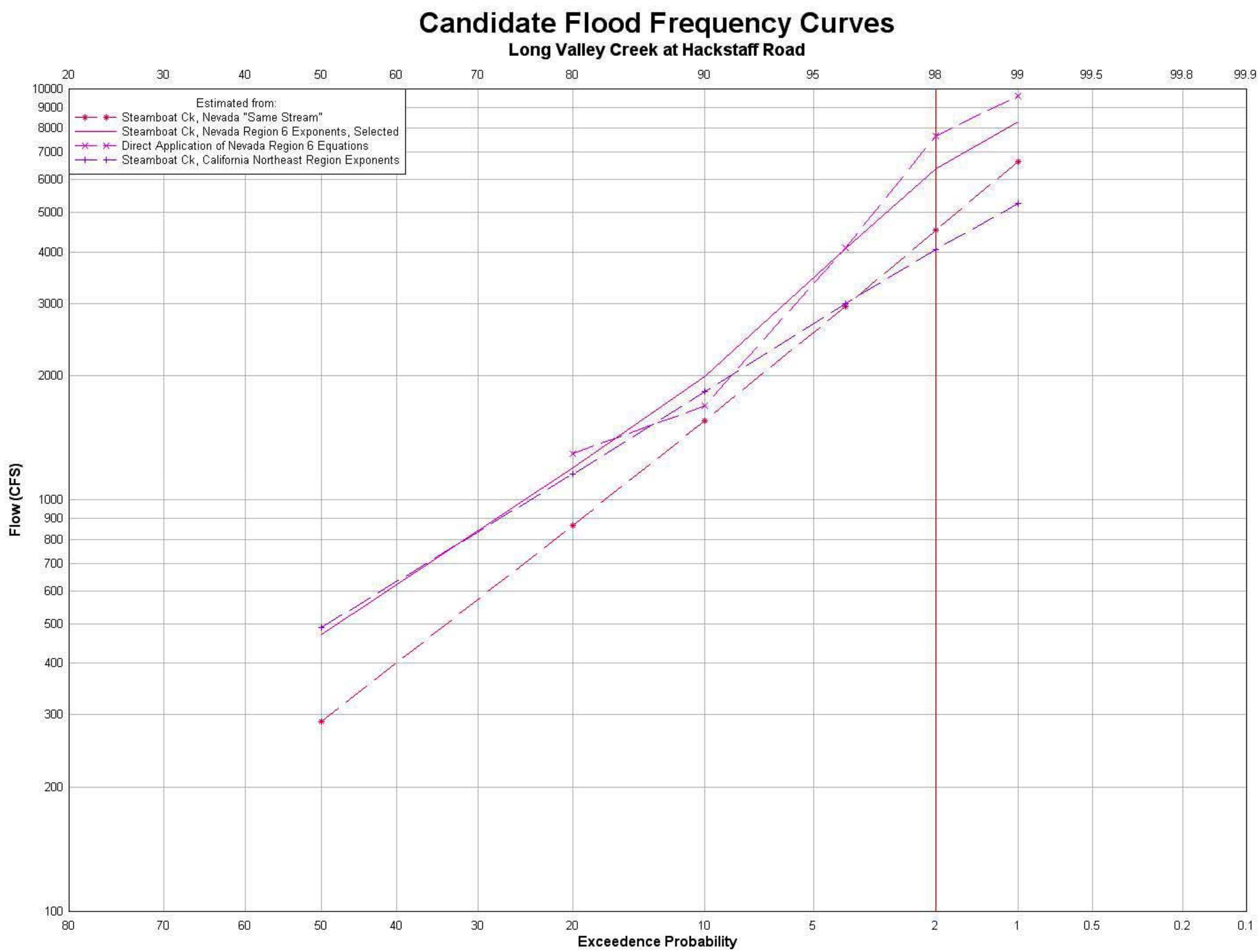


Figure 10: Existing and Preferred (9-2014) Bridge Stage-Discharge Curve at Cross-section 1580

APPENDIX A

Additional Hydrologic Data

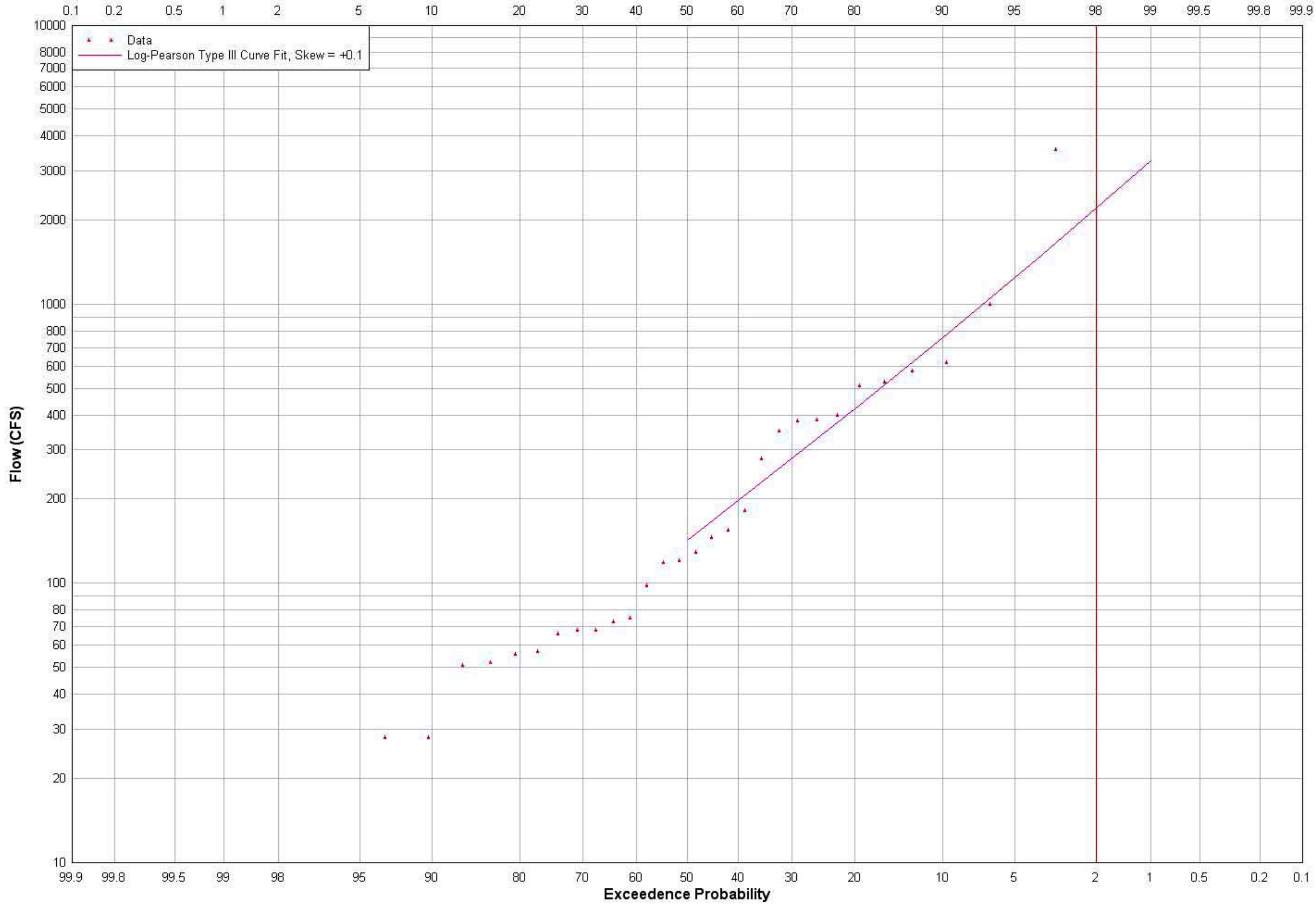


Candidate Flood Frequency Curves
Long Valley Creek at Hackstaff Road
Estimated from:
Steamboat Ck, Nevada "Same Stream"
Steamboat Ck, Nevada Region 6 Exponents, Selected
Direct Application of Nevada Region 6 Equations
Steamboat Ck, California Northeast Region Exponents
Flow (CFS)
Exceedence Probability
20 30 40 50 60 70 80 90 95 98 99 99.5 99.8 99.9
80 70 60 50 40 30 20 10 5 2 1 0.5 0.2 0.1
10000 9000 8000 7000 6000 5000 4000 3000 2000 1000 900 800 700 600 500 400 300 200 100

Steamboat Creek at Steamboat, Nevada

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

Discontinued



APPENDIX B

Additional Hydraulic Data

HEC-RAS Plan: Existing River: Long Valley Cree Reach: 1 Backwater Model Summary Output, Existing Condition Without Drift

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	3660	2000-cfs Q10	2000	4173.03	4176.26	4175.27	4176.63	0.002569	4.89	409	327.72	0.517
1	3660	4090-cfs Q25	4090	4173.03	4177.96	4176.38	4178.55	0.002241	6.16	664	369.65	0.518
1	3660	6410-cfs Q50	6410	4173.03	4179.57	4177.41	4180.34	0.002011	7.05	909	409.17	0.512
1	3660	8320-cfs Q100	8320	4173.03	4180.73	4178.14	4181.64	0.001903	7.63	1090	587.34	0.509
1	3336.66*	2000-cfs Q10	2000	4172.07	4175.41	4174.36	4175.80	0.002507	5.04	397	292.57	0.518
1	3336.66*	4090-cfs Q25	4090	4172.07	4177.19	4175.54	4177.82	0.002233	6.37	642	342.70	0.523
1	3336.66*	6410-cfs Q50	6410	4172.07	4178.85	4176.63	4179.68	0.002034	7.32	876	447.01	0.521
1	3336.66*	8320-cfs Q100	8320	4172.07	4180.02	4177.42	4181.00	0.001964	7.97	1044	626.02	0.523
1	3013.33*	2000-cfs Q10	2000	4171.10	4174.56	4173.46	4174.99	0.002516	5.23	383	277.29	0.524
1	3013.33*	4090-cfs Q25	4090	4171.10	4176.38	4174.72	4177.08	0.002338	6.71	610	373.38	0.539
1	3013.33*	6410-cfs Q50	6410	4171.10	4178.04	4175.88	4178.98	0.002211	7.79	823	461.58	0.545
1	3013.33*	8320-cfs Q100	8320	4171.10	4179.18	4176.70	4180.32	0.002202	8.56	972	565.16	0.555
1	2690	2000-cfs Q10	2000	4170.13	4173.59	4172.58	4174.10	0.002939	5.73	349	297.48	0.566
1	2690	4090-cfs Q25	4090	4170.13	4175.31	4173.94	4176.20	0.003020	7.57	540	349.01	0.609
1	2690	6410-cfs Q50	6410	4170.13	4176.91	4175.17	4178.13	0.002963	8.88	722	420.15	0.625
1	2690	8320-cfs Q100	8320	4170.13	4178.07	4176.10	4179.48	0.002876	9.53	873	551.58	0.708
1	2320.*	2000-cfs Q10	2000	4169.02	4172.59	4171.52	4173.05	0.002723	5.47	366	233.92	0.547
1	2320.*	4090-cfs Q25	4090	4169.02	4174.36	4172.81	4175.13	0.002662	7.06	579	306.95	0.576
1	2320.*	6410-cfs Q50	6410	4169.02	4176.09	4174.07	4177.09	0.002403	8.01	800	378.44	0.570
1	2320.*	8320-cfs Q100	8320	4169.02	4177.30	4174.92	4178.47	0.002302	8.66	960	681.91	0.570
1	1950.*	2000-cfs Q10	2000	4167.92	4171.61	4170.44	4172.06	0.002679	5.39	371	210.55	0.542
1	1950.*	4090-cfs Q25	4090	4167.92	4173.46	4171.87	4174.17	0.002475	6.73	607	229.97	0.555
1	1950.*	6410-cfs Q50	6410	4167.92	4175.40	4173.06	4176.23	0.002017	7.31	877	292.63	0.524
1	1950.*	8320-cfs Q100	8320	4167.92	4176.70	4173.90	4177.64	0.001857	7.78	1070	659.17	0.514
1	1580	2000-cfs Q10	2000	4166.81	4170.64	4169.45	4171.09	0.002606	5.38	371	221.06	0.539
1	1580	4090-cfs Q25	4090	4166.81	4172.60	4170.89	4173.27	0.002366	6.54	625	252.73	0.542
1	1580	6410-cfs Q50	6410	4166.81	4174.80	4172.08	4175.50	0.001758	6.73	952	307.39	0.485
1	1580	8320-cfs Q100	8320	4166.81	4176.20	4172.97	4176.97	0.001565	7.05	1180	577.39	0.467
1	1560		Bridge									
1	1520	2000-cfs Q10	2000	4166.41	4169.69	4169.03	4170.35	0.004618	6.53	306	213.06	0.701
1	1520	4090-cfs Q25	4090	4166.41	4171.08	4170.49	4172.24	0.005249	8.63	474	235.68	0.786
1	1520	6410-cfs Q50	6410	4166.41	4172.29	4171.72	4173.88	0.005519	10.10	634	255.31	0.830
1	1520	8320-cfs Q100	8320	4166.41	4173.16	4172.53	4175.04	0.005583	10.98	758	274.62	0.849
1	1460	2000-cfs Q10	2000	4166.05	4169.62	4168.62	4170.01	0.002682	5.01	399	262.30	0.532
1	1460	4090-cfs Q25	4090	4166.05	4171.10	4169.80	4171.77	0.002838	6.60	620	303.29	0.579
1	1460	6410-cfs Q50	6410	4166.05	4172.38	4170.81	4173.32	0.002932	7.80	822	352.92	0.609
1	1460	8320-cfs Q100	8320	4166.05	4173.29	4171.59	4174.43	0.002970	8.56	972	400.74	0.626
1	1115.*	2000-cfs Q10	2000	4164.95	4168.69	4167.75	4169.08	0.002701	4.97	402	353.68	0.532
1	1115.*	4090-cfs Q25	4090	4164.95	4170.08	4168.86	4170.77	0.002954	6.67	613	368.10	0.591
1	1115.*	6410-cfs Q50	6410	4164.95	4171.29	4169.88	4172.28	0.003097	7.98	803	383.31	0.627
1	1115.*	8320-cfs Q100	8320	4164.95	4172.15	4170.62	4173.36	0.003170	8.83	942	394.20	0.647
1	770	2000-cfs Q10	2000	4163.84	4167.50	4166.87	4167.96	0.003865	5.44	368	451.67	0.622
1	770	4090-cfs Q25	4090	4163.84	4168.77	4167.93	4169.59	0.003964	7.23	566	457.77	0.671
1	770	6410-cfs Q50	6410	4163.84	4169.81	4168.90	4171.01	0.004290	8.80	729	462.65	0.724
1	770	8320-cfs Q100	8320	4163.84	4170.51	4169.63	4172.03	0.004550	9.89	841	467.07	0.760
1	0	2000-cfs Q10	2000	4160.52	4164.97	4164.09	4165.32	0.003000	4.75	421	348.56	0.545
1	0	4090-cfs Q25	4090	4160.52	4166.33	4165.17	4166.87	0.003001	5.90	693	459.23	0.577
1	0	6410-cfs Q50	6410	4160.52	4167.38	4166.11	4168.13	0.003003	6.96	921	470.28	0.601
1	0	8320-cfs Q100	8320	4160.52	4168.13	4166.73	4169.03	0.003004	7.64	1089	478.10	0.615

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

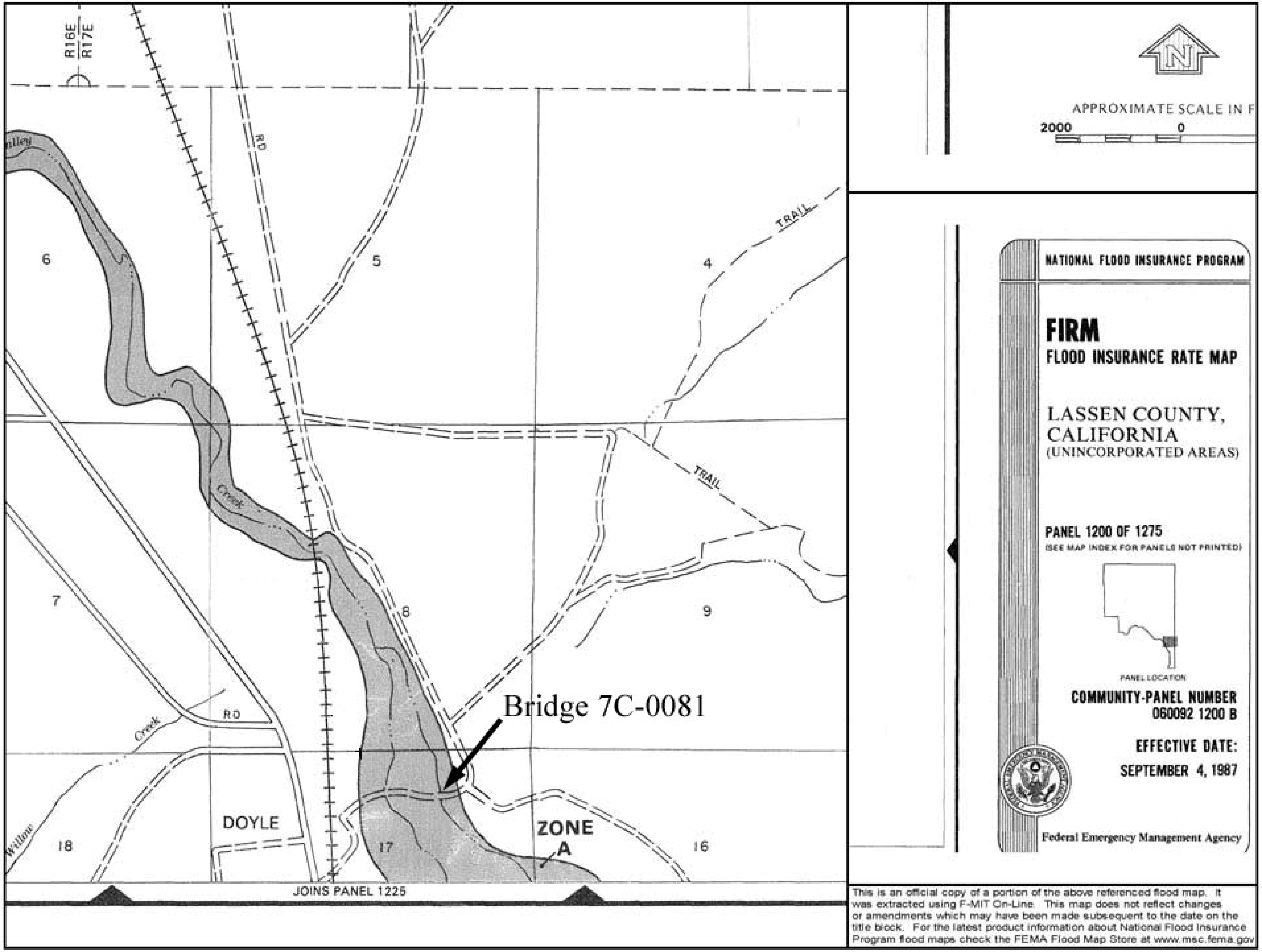
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	3660	2000-cfs Q10	2000	4173.03	4176.26	4175.27	4176.63	0.002569	4.89	409	327.72	0.517
1	3660	4090-cfs Q25	4090	4173.03	4177.96	4176.38	4178.55	0.002236	6.16	664	369.72	0.517
1	3660	6410-cfs Q50	6410	4173.03	4179.60	4177.41	4180.36	0.001975	7.01	914	409.99	0.508
1	3660	8320-cfs Q100	8320	4173.03	4180.79	4178.14	4181.68	0.001856	7.57	1099	599.34	0.504
1	3336.66*	2000-cfs Q10	2000	4172.07	4175.41	4174.36	4175.80	0.002507	5.04	397	292.57	0.518
1	3336.66*	4090-cfs Q25	4090	4172.07	4177.20	4175.54	4177.83	0.002223	6.36	643	342.89	0.522
1	3336.66*	6410-cfs Q50	6410	4172.07	4178.90	4176.63	4179.72	0.001978	7.25	884	448.84	0.514
1	3336.66*	8320-cfs Q100	8320	4172.07	4180.10	4177.42	4181.07	0.001894	7.88	1056	688.20	0.514
1	3013.33*	2000-cfs Q10	2000	4171.10	4174.57	4173.46	4174.99	0.002512	5.22	383	277.37	0.523
1	3013.33*	4090-cfs Q25	4090	4171.10	4176.39	4174.72	4177.09	0.002317	6.69	612	373.85	0.537
1	3013.33*	6410-cfs Q50	6410	4171.10	4178.14	4175.88	4179.05	0.002108	7.67	835	465.10	0.533
1	3013.33*	8320-cfs Q100	8320	4171.10	4179.31	4176.70	4180.41	0.002084	8.41	989	713.74	0.541
1	2690	2000-cfs Q10	2000	4170.13	4173.60	4172.58	4174.11	0.002915	5.71	350	297.68	0.564
1	2690	4090-cfs Q25	4090	4170.13	4175.35	4173.94	4176.23	0.002945	7.51	545	350.14	0.602
1	2690	6410-cfs Q50	6410	4170.13	4177.12	4175.17	4178.26	0.002670	8.59	746	425.48	0.596
1	2690	8320-cfs Q100	8320	4170.13	4178.42	4176.10	4179.67	0.002459	8.94	931	895.51	0.677
1	2320.*	2000-cfs Q10	2000	4169.02	4172.63	4171.52	4173.08	0.002616	5.40	370	234.64	0.537
1	2320.*	4090-cfs Q25	4090	4169.02	4174.45	4172.81	4175.19	0.002500	6.92	591	309.86	0.560
1	2320.*	6410-cfs Q50	6410	4169.02	4176.45	4174.07	4177.34	0.002011	7.56	848	408.26	0.525
1	2320.*	8320-cfs Q100	8320	4169.02	4177.81	4174.92	4178.82	0.001855	8.06	1032	845.77	0.545
1	1950.*	2000-cfs Q10	2000	4167.92	4171.75	4170.44	4172.16	0.002318	5.14	389	212.24	0.507
1	1950.*	4090-cfs Q25	4090	4167.92	4173.67	4171.87	4174.31	0.002168	6.44	635	236.27	0.522
1	1950.*	6410-cfs Q50	6410	4167.92	4175.94	4173.06	4176.64	0.001552	6.71	956	444.84	0.464
1	1950.*	8320-cfs Q100	8320	4167.92	4177.38	4173.90	4178.16	0.001410	7.10	1172	987.22	0.452
1	1580	2000-cfs Q10	2000	4166.81	4171.06	4169.45	4171.41	0.001774	4.73	423	227.86	0.451
1	1580	4090-cfs Q25	4090	4166.81	4172.99	4170.89	4173.55	0.001867	6.02	679	259.76	0.485
1	1580	6410-cfs Q50	6410	4166.81	4175.53	4172.08	4176.09	0.001240	5.99	1070	381.59	0.412
1	1580	8320-cfs Q100	8320	4166.81	4177.03	4172.97	4177.65	0.001121	6.30	1322	1073.58	0.400
1	1560		Bridge									
1	1520	2000-cfs Q10	2000	4166.41	4169.69	4169.03	4170.35	0.004618	6.53	306	213.06	0.701
1	1520	4090-cfs Q25	4090	4166.41	4171.08	4170.49	4172.24	0.005249	8.63	474	235.68	0.786
1	1520	6410-cfs Q50	6410	4166.41	4172.29	4171.72	4173.88	0.005519	10.10	634	255.31	0.830
1	1520	8320-cfs Q100	8320	4166.41	4173.16	4172.53	4175.04	0.005583	10.98	758	274.62	0.849
1	1460	2000-cfs Q10	2000	4166.05	4169.62	4168.62	4170.01	0.002682	5.01	399	262.30	0.532
1	1460	4090-cfs Q25	4090	4166.05	4171.10	4169.80	4171.77	0.002838	6.60	620	303.29	0.579
1	1460	6410-cfs Q50	6410	4166.05	4172.38	4170.81	4173.32	0.002932	7.80	822	352.92	0.609
1	1460	8320-cfs Q100	8320	4166.05	4173.29	4171.59	4174.43	0.002970	8.56	972	400.74	0.626
1	1115.*	2000-cfs Q10	2000	4164.95	4168.69	4167.75	4169.08	0.002701	4.97	402	353.68	0.532
1	1115.*	4090-cfs Q25	4090	4164.95	4170.08	4168.86	4170.77	0.002954	6.67	613	368.10	0.591
1	1115.*	6410-cfs Q50	6410	4164.95	4171.29	4169.88	4172.28	0.003097	7.98	803	383.31	0.627
1	1115.*	8320-cfs Q100	8320	4164.95	4172.15	4170.62	4173.36	0.003170	8.83	942	394.20	0.647
1	770	2000-cfs Q10	2000	4163.84	4167.50	4166.87	4167.96	0.003865	5.44	368	451.67	0.622
1	770	4090-cfs Q25	4090	4163.84	4168.77	4167.93	4169.59	0.003964	7.23	566	457.77	0.671
1	770	6410-cfs Q50	6410	4163.84	4169.81	4168.90	4171.01	0.004290	8.80	729	462.65	0.724
1	770	8320-cfs Q100	8320	4163.84	4170.51	4169.63	4172.03	0.004550	9.89	841	467.07	0.760
1	0	2000-cfs Q10	2000	4160.52	4164.97	4164.09	4165.32	0.003000	4.75	421	348.56	0.545
1	0	4090-cfs Q25	4090	4160.52	4166.33	4165.17	4166.87	0.003001	5.90	693	459.23	0.577
1	0	6410-cfs Q50	6410	4160.52	4167.38	4166.11	4168.13	0.003003	6.96	921	470.28	0.601
1	0	8320-cfs Q100	8320	4160.52	4168.13	4166.73	4169.03	0.003004	7.64	1089	478.10	0.615

HEC-RAS Plan: 11-2014 Bridge River: Long Valley Cree Reach: 1 Backwater Model Summary Output, Preferred Bridge Without Drift

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	3660	2000-cfs Q10	2000	4173.03	4176.26	4175.27	4176.63	0.002567	4.89	409	327.73	0.517
1	3660	4090-cfs Q25	4090	4173.03	4177.96	4176.38	4178.55	0.002244	6.16	664	369.60	0.518
1	3660	6410-cfs Q50	6410	4173.03	4179.54	4177.41	4180.32	0.002038	7.08	905	408.56	0.515
1	3660	8320-cfs Q100	8320	4173.03	4180.69	4178.14	4181.61	0.001937	7.68	1084	569.91	0.514
1	3336.66*	2000-cfs Q10	2000	4172.07	4175.41	4174.36	4175.80	0.002508	5.04	397	292.55	0.518
1	3336.66*	4090-cfs Q25	4090	4172.07	4177.19	4175.54	4177.82	0.002239	6.37	642	342.60	0.524
1	3336.66*	6410-cfs Q50	6410	4172.07	4178.80	4176.63	4179.65	0.002080	7.37	870	445.57	0.526
1	3336.66*	8320-cfs Q100	8320	4172.07	4179.96	4177.42	4180.96	0.002016	8.04	1035	530.87	0.529
1	3013.33*	2000-cfs Q10	2000	4171.10	4174.56	4173.46	4174.99	0.002517	5.23	383	277.27	0.524
1	3013.33*	4090-cfs Q25	4090	4171.10	4176.37	4174.72	4177.07	0.002352	6.72	609	373.05	0.540
1	3013.33*	6410-cfs Q50	6410	4171.10	4177.97	4175.88	4178.93	0.002294	7.88	813	457.07	0.554
1	3013.33*	8320-cfs Q100	8320	4171.10	4179.08	4176.70	4180.25	0.002303	8.68	958	520.22	0.567
1	2690	2000-cfs Q10	2000	4170.13	4173.59	4172.58	4174.10	0.002940	5.73	349	297.47	0.566
1	2690	4090-cfs Q25	4090	4170.13	4175.28	4173.94	4176.18	0.003079	7.61	537	348.14	0.614
1	2690	6410-cfs Q50	6410	4170.13	4176.72	4175.17	4178.02	0.003261	9.15	700	415.39	0.653
1	2690	8320-cfs Q100	8320	4170.13	4177.71	4176.10	4179.31	0.003388	10.15	820	462.03	0.733
1	2320.*	2000-cfs Q10	2000	4169.02	4172.58	4171.52	4173.05	0.002731	5.48	365	233.87	0.548
1	2320.*	4090-cfs Q25	4090	4169.02	4174.27	4172.81	4175.08	0.002811	7.18	569	304.48	0.590
1	2320.*	6410-cfs Q50	6410	4169.02	4175.70	4174.07	4176.84	0.002940	8.55	749	360.25	0.626
1	2320.*	8320-cfs Q100	8320	4169.02	4176.67	4174.92	4178.07	0.003052	9.49	877	415.66	0.650
1	1950.*	2000-cfs Q10	2000	4167.92	4171.58	4170.44	4172.04	0.002740	5.43	368	210.26	0.548
1	1950.*	4090-cfs Q25	4090	4167.92	4173.26	4171.87	4174.03	0.002840	7.04	581	227.13	0.591
1	1950.*	6410-cfs Q50	6410	4167.92	4174.66	4173.06	4175.74	0.002955	8.31	772	266.94	0.625
1	1950.*	8320-cfs Q100	8320	4167.92	4175.62	4173.90	4176.92	0.003046	9.15	909	336.54	0.647
1	1580	2000-cfs Q10	2000	4166.81	4170.15	4169.45	4170.78	0.004320	6.37	314	213.16	0.680
1	1580	4090-cfs Q25	4090	4166.81	4171.69	4170.89	4172.72	0.004441	8.12	503	238.10	0.727
1	1580	6410-cfs Q50	6410	4166.81	4172.96	4172.08	4174.36	0.004672	9.50	675	259.04	0.767
1	1580	8320-cfs Q100	8320	4166.81	4173.80	4172.97	4175.49	0.004893	10.44	797	278.87	0.797
1	1520	2000-cfs Q10	2000	4166.41	4169.96	4169.03	4170.50	0.003456	5.93	337	217.43	0.614
1	1520	4090-cfs Q25	4090	4166.41	4171.49	4170.49	4172.43	0.003858	7.77	527	242.30	0.682
1	1520	6410-cfs Q50	6410	4166.41	4172.73	4171.72	4174.05	0.004238	9.22	695	264.32	0.734
1	1520	8320-cfs Q100	8320	4166.41	4173.55	4172.53	4175.17	0.004543	10.21	815	283.76	0.771
1	1500		Bridge									
1	1460	2000-cfs Q10	2000	4166.05	4169.68	4168.83	4170.16	0.003503	5.61	356	252.13	0.607
1	1460	4090-cfs Q25	4090	4166.05	4171.14	4170.14	4171.95	0.003722	7.21	568	306.15	0.660
1	1460	6410-cfs Q50	6410	4166.05	4172.38	4171.25	4173.46	0.003621	8.38	778	375.40	0.679
1	1460	8320-cfs Q100	8320	4166.05	4173.27	4172.02	4174.53	0.003504	9.06	951	437.33	0.683
1	1115.*	2000-cfs Q10	2000	4164.95	4168.69	4167.75	4169.08	0.002701	4.97	402	353.68	0.532
1	1115.*	4090-cfs Q25	4090	4164.95	4170.08	4168.86	4170.77	0.002954	6.67	613	368.10	0.591
1	1115.*	6410-cfs Q50	6410	4164.95	4171.29	4169.88	4172.28	0.003097	7.98	803	383.31	0.627
1	1115.*	8320-cfs Q100	8320	4164.95	4172.15	4170.62	4173.36	0.003170	8.83	942	394.20	0.647
1	770	2000-cfs Q10	2000	4163.84	4167.50	4166.87	4167.96	0.003865	5.44	368	451.67	0.622
1	770	4090-cfs Q25	4090	4163.84	4168.77	4167.93	4169.59	0.003964	7.23	566	457.77	0.671
1	770	6410-cfs Q50	6410	4163.84	4169.81	4168.90	4171.01	0.004290	8.80	729	462.65	0.724
1	770	8320-cfs Q100	8320	4163.84	4170.51	4169.63	4172.03	0.004550	9.89	841	467.07	0.760
1	0	2000-cfs Q10	2000	4160.52	4164.97	4164.09	4165.32	0.003000	4.75	421	348.56	0.545
1	0	4090-cfs Q25	4090	4160.52	4166.33	4165.17	4166.87	0.003001	5.90	693	459.23	0.577
1	0	6410-cfs Q50	6410	4160.52	4167.38	4166.11	4168.13	0.003003	6.96	921	470.28	0.601
1	0	8320-cfs Q100	8320	4160.52	4168.13	4166.73	4169.03	0.003004	7.64	1089	478.10	0.615

HEC-RAS Plan: 11-2014 Bridge River: Long Valley Cree Reach: 1 Backwater Model Summary Output, Preferred Bridge With Drift

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
1	3660	2000-cfs Q10	2000	4173.03	4176.26	4175.27	4176.63	0.002567	4.89	409	327.73	0.517
1	3660	4090-cfs Q25	4090	4173.03	4177.96	4176.38	4178.55	0.002244	6.16	664	369.60	0.518
1	3660	6410-cfs Q50	6410	4173.03	4179.54	4177.41	4180.32	0.002038	7.08	905	408.56	0.515
1	3660	8320-cfs Q100	8320	4173.03	4180.69	4178.14	4181.61	0.001937	7.68	1084	569.91	0.514
1	3336.66*	2000-cfs Q10	2000	4172.07	4175.41	4174.36	4175.80	0.002508	5.04	397	292.55	0.518
1	3336.66*	4090-cfs Q25	4090	4172.07	4177.19	4175.54	4177.82	0.002239	6.37	642	342.60	0.524
1	3336.66*	6410-cfs Q50	6410	4172.07	4178.80	4176.63	4179.65	0.002080	7.37	870	445.57	0.526
1	3336.66*	8320-cfs Q100	8320	4172.07	4179.96	4177.42	4180.96	0.002016	8.04	1035	530.87	0.529
1	3013.33*	2000-cfs Q10	2000	4171.10	4174.56	4173.46	4174.99	0.002517	5.23	383	277.27	0.524
1	3013.33*	4090-cfs Q25	4090	4171.10	4176.37	4174.72	4177.07	0.002352	6.72	609	373.05	0.540
1	3013.33*	6410-cfs Q50	6410	4171.10	4177.97	4175.88	4178.93	0.002294	7.88	813	457.07	0.554
1	3013.33*	8320-cfs Q100	8320	4171.10	4179.08	4176.70	4180.25	0.002303	8.68	958	520.22	0.567
1	2690	2000-cfs Q10	2000	4170.13	4173.59	4172.58	4174.10	0.002940	5.73	349	297.47	0.566
1	2690	4090-cfs Q25	4090	4170.13	4175.28	4173.94	4176.18	0.003079	7.61	537	348.14	0.614
1	2690	6410-cfs Q50	6410	4170.13	4176.72	4175.17	4178.02	0.003261	9.15	700	415.39	0.653
1	2690	8320-cfs Q100	8320	4170.13	4177.71	4176.10	4179.31	0.003388	10.15	820	462.03	0.733
1	2320.*	2000-cfs Q10	2000	4169.02	4172.58	4171.52	4173.05	0.002731	5.48	365	233.87	0.548
1	2320.*	4090-cfs Q25	4090	4169.02	4174.27	4172.81	4175.08	0.002811	7.18	569	304.48	0.590
1	2320.*	6410-cfs Q50	6410	4169.02	4175.70	4174.07	4176.84	0.002939	8.55	749	360.26	0.626
1	2320.*	8320-cfs Q100	8320	4169.02	4176.67	4174.92	4178.07	0.003051	9.49	877	415.68	0.650
1	1950.*	2000-cfs Q10	2000	4167.92	4171.58	4170.44	4172.04	0.002740	5.43	368	210.26	0.548
1	1950.*	4090-cfs Q25	4090	4167.92	4173.26	4171.87	4174.03	0.002840	7.04	581	227.13	0.591
1	1950.*	6410-cfs Q50	6410	4167.92	4174.66	4173.06	4175.74	0.002954	8.31	772	266.96	0.625
1	1950.*	8320-cfs Q100	8320	4167.92	4175.62	4173.90	4176.92	0.003043	9.15	909	336.70	0.646
1	1580	2000-cfs Q10	2000	4166.81	4170.14	4169.45	4170.77	0.004346	6.38	313	213.08	0.682
1	1580	4090-cfs Q25	4090	4166.81	4171.68	4170.89	4172.71	0.004492	8.16	502	237.85	0.731
1	1580	6410-cfs Q50	6410	4166.81	4172.93	4172.08	4174.35	0.004755	9.56	671	258.34	0.774
1	1580	8320-cfs Q100	8320	4166.81	4173.75	4172.97	4175.47	0.005005	10.52	791	277.86	0.806
1	1520	2000-cfs Q10	2000	4166.41	4169.95	4169.03	4170.50	0.003479	5.94	336	217.32	0.616
1	1520	4090-cfs Q25	4090	4166.41	4171.47	4170.49	4172.42	0.003909	7.80	524	242.00	0.686
1	1520	6410-cfs Q50	6410	4166.41	4172.70	4171.72	4174.03	0.004326	9.28	690	263.49	0.741
1	1520	8320-cfs Q100	8320	4166.41	4173.50	4172.53	4175.15	0.004666	10.31	807	282.55	0.781
1	1500		Bridge									
1	1460	2000-cfs Q10	2000	4166.05	4169.68	4168.83	4170.16	0.003503	5.61	356	252.13	0.607
1	1460	4090-cfs Q25	4090	4166.05	4171.14	4170.14	4171.95	0.003722	7.21	568	306.15	0.660
1	1460	6410-cfs Q50	6410	4166.05	4172.38	4171.25	4173.46	0.003621	8.38	778	375.40	0.679
1	1460	8320-cfs Q100	8320	4166.05	4173.27	4172.02	4174.53	0.003504	9.06	951	437.33	0.683
1	1115.*	2000-cfs Q10	2000	4164.95	4168.69	4167.75	4169.08	0.002701	4.97	402	353.68	0.532
1	1115.*	4090-cfs Q25	4090	4164.95	4170.08	4168.86	4170.77	0.002954	6.67	613	368.10	0.591
1	1115.*	6410-cfs Q50	6410	4164.95	4171.29	4169.88	4172.28	0.003097	7.98	803	383.31	0.627
1	1115.*	8320-cfs Q100	8320	4164.95	4172.15	4170.62	4173.36	0.003170	8.83	942	394.20	0.647
1	770	2000-cfs Q10	2000	4163.84	4167.50	4166.87	4167.96	0.003865	5.44	368	451.67	0.622
1	770	4090-cfs Q25	4090	4163.84	4168.77	4167.93	4169.59	0.003964	7.23	566	457.77	0.671
1	770	6410-cfs Q50	6410	4163.84	4169.81	4168.90	4171.01	0.004290	8.80	729	462.65	0.724
1	770	8320-cfs Q100	8320	4163.84	4170.51	4169.63	4172.03	0.004550	9.89	841	467.07	0.760
1	0	2000-cfs Q10	2000	4160.52	4164.97	4164.09	4165.32	0.003000	4.75	421	348.56	0.545
1	0	4090-cfs Q25	4090	4160.52	4166.33	4165.17	4166.87	0.003001	5.90	693	459.23	0.577
1	0	6410-cfs Q50	6410	4160.52	4167.38	4166.11	4168.13	0.003003	6.96	921	470.28	0.601
1	0	8320-cfs Q100	8320	4160.52	4168.13	4166.73	4169.03	0.003004	7.64	1089	478.10	0.615


R16E
R17E
RD
TRAIL
Valley
Creek
Creek
Willow
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NATIONAL FLOOD INSURANCE PROGRAM
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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
This is an official copy of a portion of the above referenced flood map. It was extracted using F-MIT On-Line. This map does not reflect changes or amendments which may have been made subsequent to the date on the title block. For the latest product information about National Flood Insurance Program flood maps check the FEMA Flood Map Store at www.msc.fema.gov

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.[43] 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.

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

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

where:

y_1 = Average depth in the upstream main channel (ft) =	7
y_2 = Average depth in the contracted section (ft) =	
y_0 = Existing depth in the contracted section before scour (ft) =	7.2
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) =	100
W_2 = Bottom width of contracted section less pier width (ft) =	142
k_1 = Exponent determined below	0.59

V_*/ω	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)
ω = Fall velocity of bed material based on the D_{50}, m/s (Figure 5.8)
For fall velocity in English units (ft/s) multiply ω in m/s by 3.28
g = Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)
S_1 = Slope of energy grade line of main channel, m/m (ft/ft)

y_2/y_1 = 0.81

y_2 = 5.7

y_s = -1.5

Potential Pier Scour Using CSU Equation, 100 - Year flood
Hackstaff Road over Long Valley Creek, Lassen County, 11-24-2014

Ys = Depth of potential scour, Ft.

K1 = 1.0 Nose shape coefficient, round

Theta = 0 Angle between direction of flow and pier, degrees

Length of pier = 30.0 Ft

K2 = 1.00 Angle coefficient

K3 = 1.1 Bed condition coefficient

a = Pier Width, assume 3.5 Ft

Y1 = 7.2 Ft, Maximum depth expected in front of pier, 100 - Year flood.

V1 = 10.5 FPS, Maximum velocity in front of pier, 100 - Year flood.

Fr1 = Froude Number in front of pier

Fr1 = V1/[(g x Y1)^(1/2)] = 0.690

Ys/Y1 = 2.0 x K1 x K2 x K3 x [(a/Y1)^0.65] x Fr1^0.43 = 1.17

Ys = Y1 x (Ys/Y1) = 8.4 Ft.

NOTE: For Fr1 values less than 0.8 the value of Ys/a is not recommended to exceed 2.4.

Ys = 2.4 x a = 8.4 Ft.

APPENDIX D

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APPENDIX E

Reference 11

Journal of Floodplain Management
Floodplain Management Association
June 2002 Vol. 3, No. 1

Bridge Design Considerations - Non Proximate Geologic Hazard

Norman S. Braithwaite [1]

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 unpredictible 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 Pinnicles 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 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 to 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.

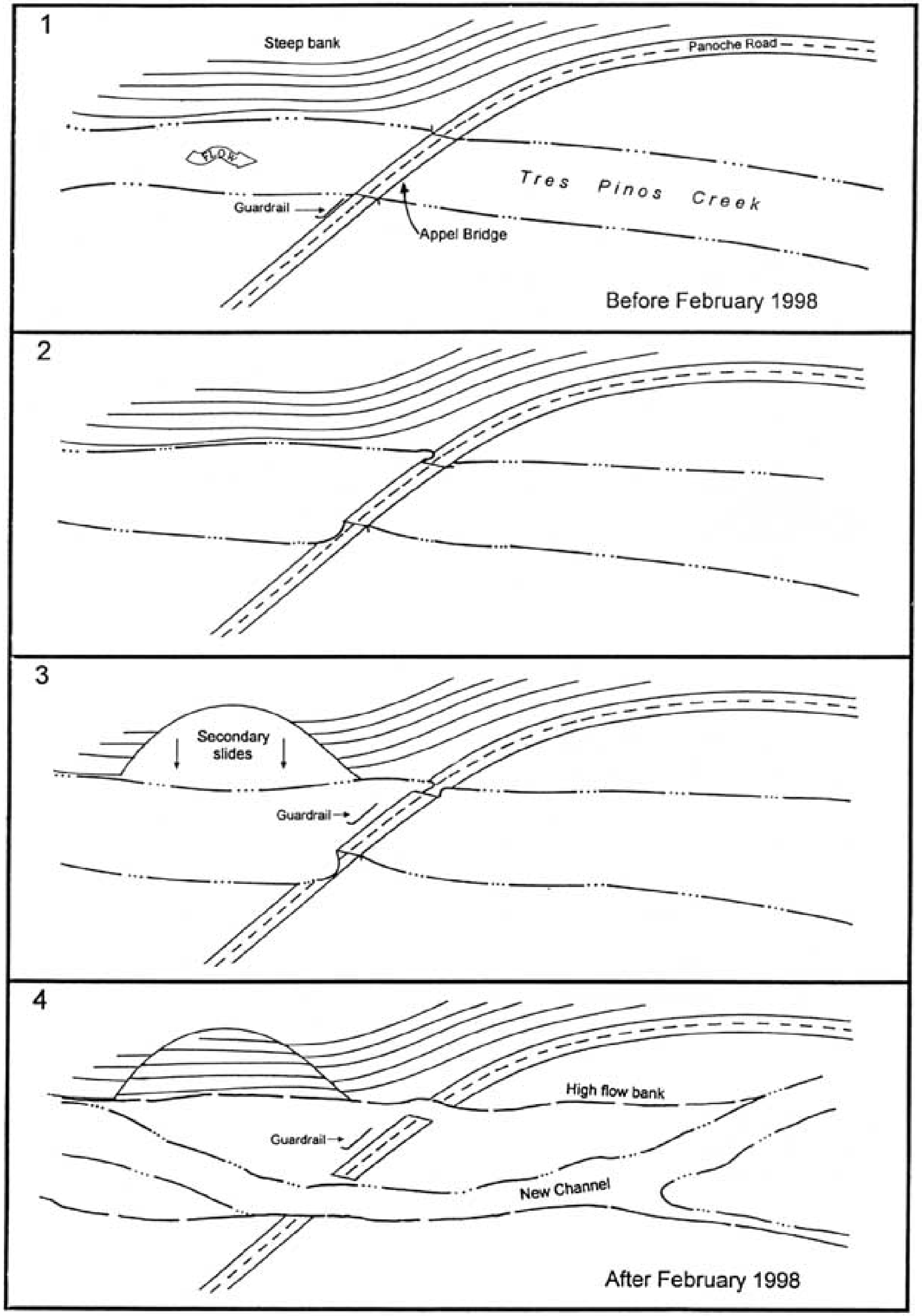

Figure 3. Progression of the Appel Bridge failure.

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.)

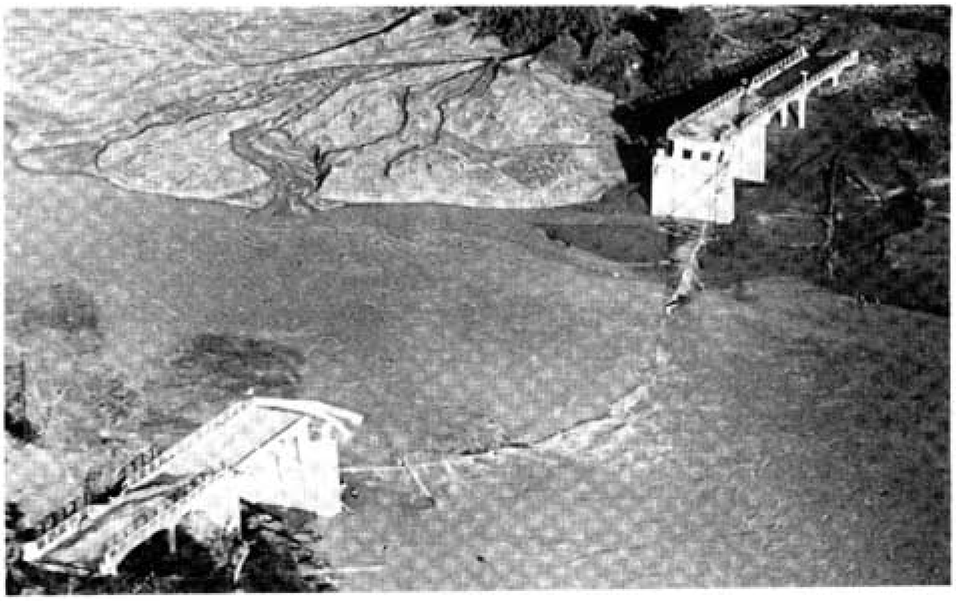

Figure 7. *Orleans Bridge over Klamath River (California Highways and Public Works, January-February 1965)*

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.