

APPENDIX D
DESIGN HYDRAULIC STUDY



COUNTY ROAD 200 AT BRANCH SALT CREEK

Bridge Number 11C0132

GLENN COUNTY, CALIFORNIA



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Design Hydraulic Study
COUNTY ROAD 200 BRIDGE AT BRANCH SALT CREEK

GLENN COUNTY, CALIFORNIA

Bridge Number 11C0132

DECEMBER 4, 2019

PREPARED FOR:

GLENN COUNTY PUBLIC WORKS AGENCY

Prepared by:

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EXECUTIVE SUMMARY

The County Road 200 bridge over a Branch of Salt Creek in Glenn County, California is proposed for replacement by Glenn County Public Works Agency. The proposed bridge will be located in the same location as the existing bridge. It will be a single span cast in place pre-stressed concrete box girder. It will accommodate 2 travel lanes with two 2-foot shoulders, as shown in the General Plan (Appendix A).

The Branch of Salt Creek flows south easterly through the north of Glenn County (County). It drains an approximate 20.6 square miles at the project site. The discharges used for the bridge hydraulic analysis are shown in Table 1.

Table 1. Estimated discharges and water surface elevations for bridge design

	Design	Base	Flood of Record
Frequency (years)	50	100	>50
Discharge (cubic feet per second)	5,420	6,400	4,450
Water Surface (elevation in feet at upstream face of Bridge)	620.1	620.9	619.2

This report follows the California Department of Transportation (Caltrans) Final Hydraulic Report Format and has been prepared in accordance with the Caltrans Local Assistance Program Guidelines (Caltrans 2019) and Memos to Designers 16-1¹.

¹ Caltrans Memo to Designers 16-1 December 2017 (<http://www.dot.ca.gov/hq/esc/techpubs/manual/bridgemanuals/bridge-memo-to-designer/page/Section%201/16-1m.pdf>).

GENERAL

This design hydraulic study has been prepared for the sole purpose of meeting the requirements of the Caltrans “Local Assistance Program Guidelines.” 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 for which Avila and Associates Consulting Engineers, Inc. (Avila and Associates) intended and without their written authorization is not endorsed or encouraged and is at the sole risk of the entity reusing the information.

Avila and Associates was retained to complete the bridge hydrology, hydraulics, and scour analysis for the replacement of the existing County Road 200 Bridge over a Branch Salt Creek in Glenn County. The location of this project is approximately 2,000 feet from Newville, CA and 18 miles west of Orland, CA as shown in Figure 1 and Figure 2. The following scope of work has been completed to develop this report:

1. Obtain backup information and field review.
2. Obtain discharge information.
3. Estimate scour, channel bed degradation and bank protection parameters.
4. Prepare draft report for comment.
5. Prepare final report.

The existing bridge is located along County Road 200 over a Branch Salt Creek as shown in Figure 1. The existing bridge was constructed in 1925; it is a three-span reinforced concrete T-beam bridge. It has a sufficiency rating of 48.9 as of 2012 and is structurally deficient (Glenn County, 2012).



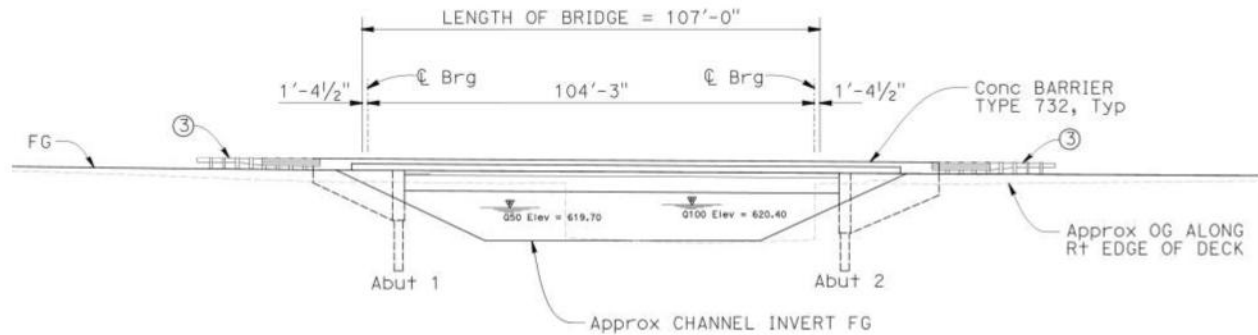
Figure 1. Bridge location map.



Figure 2. Zoomed in map of project location

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The datum elevation used for this study is NAVD-88². The proposed bridge will be located in the same location as the existing bridge. It will be a single span cast in place pre-stressed concrete box girder. It will accommodate 2 travel lanes with two 2-foot shoulders, as shown in Figure 3 and in the attached General Plan (Appendix A). The superstructure will be supported by reinforced concrete abutments on concrete piles.



² As shown in the Survey Control: Vertical Datum statement on the project topographic survey provided by Richard Uhlmann, Willdan, via electronic mail to Cathy Avila, Avila and Associates on October 1, 2018.

BRIDGE HISTORY

Avila and Associates reviewed the pertinent bridge maintenance records for the existing bridge to review the typical impacts to bridges along this reach. Details of the bridge are shown in

Table 2 below.

Table 2. Bridge information from adjacent bridges on the reach

	Road 200 at Salt Creek Bridge
Bridge Number	11C0132
Bridge Length (ft)	62
Span Lengths (ft)	3 @ 20
Bridge Type	3-span continuous reinforced concrete T-Beam (4) on reinforced concrete abutments on unknown foundation
Debris Challenges	1987 ³ , 1996 ⁴
Cross Sections Available for	none available
NBIS Item 113 (scour) code	U
ELI Flag 361 Condition State	2
Pier Type	Reinforced concrete solid wall piers on spread footings
Year Built	1925
Year Widened	N/A
Scour Challenges	1978 ⁵ , 1983 ⁶ , 1991 ⁷ , 1993 ⁸ , 2002 ⁹ , 2004 ¹⁰ , 2006 ¹¹ , 2009 ¹² , 2012 ¹³

³ A large log is wedged against Pier 3.

⁴ Debris has accumulated at the nose of Piers 2 and 3.

⁵ The ends of all pier footings are slightly undermined with upstream side of pier 2 undermined about 4 ft for the full width of the footing.

⁶ The stream channel under the structure has been paved with concrete. It appears that erosion may be occurring under the concrete at Pier 2.

⁷ The downstream end is scoured about 2 feet to sound rock.

⁸ Same as 1991.

⁹ The streambed below the structure is overlaid with concrete as indicated during the 2/83 investigation. The concrete lining is breaking apart with large potholes under spans 2 and 3. There is undermining (0.5m deep) below the concrete lining approximately 1 m downstream from the structure. The footings of Piers 2 and 3 are exposed but there is no indication of undermining at this time.

¹⁰ Same as 2002.

¹¹ Same as 2004.

¹² Same as 2006.

¹³ Same as 2009.

DISCHARGE

Salt Creek at the Road 200 bridge drains an approximate 20.6 square miles as shown in Figure 4. The mean annual precipitation of the project watershed is approximately 30.5 inches/year¹⁴.

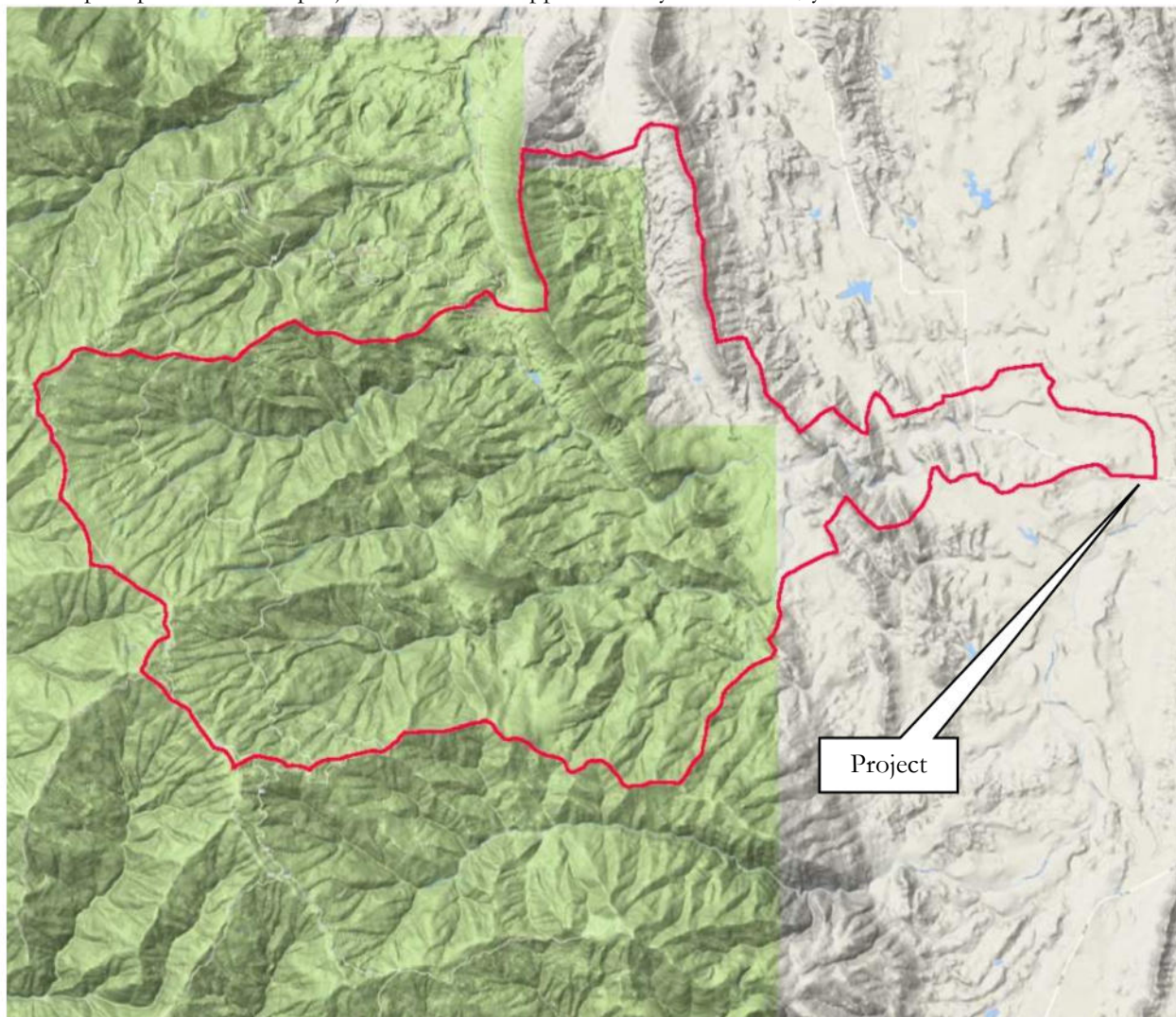


Figure 4. Basin contributing to the bridge discharge (USGS streamstats).

Just downstream from the project, Salt Creek joins North Fork (NF) Stony Creek. NF Stony Creek drains an approximate 15.1 square miles at the confluence as shown in Figure 5. NF Stony Creek is included in this analysis because the combined discharges may impact the water surface elevation downstream from the project due to the proximity of the confluence. The mean annual precipitation of the NF Stony Creek watershed is approximately 25.1 inches/year.

¹⁴ www.streamstatsags.cr.usgs.gov (U.S.G.S.)

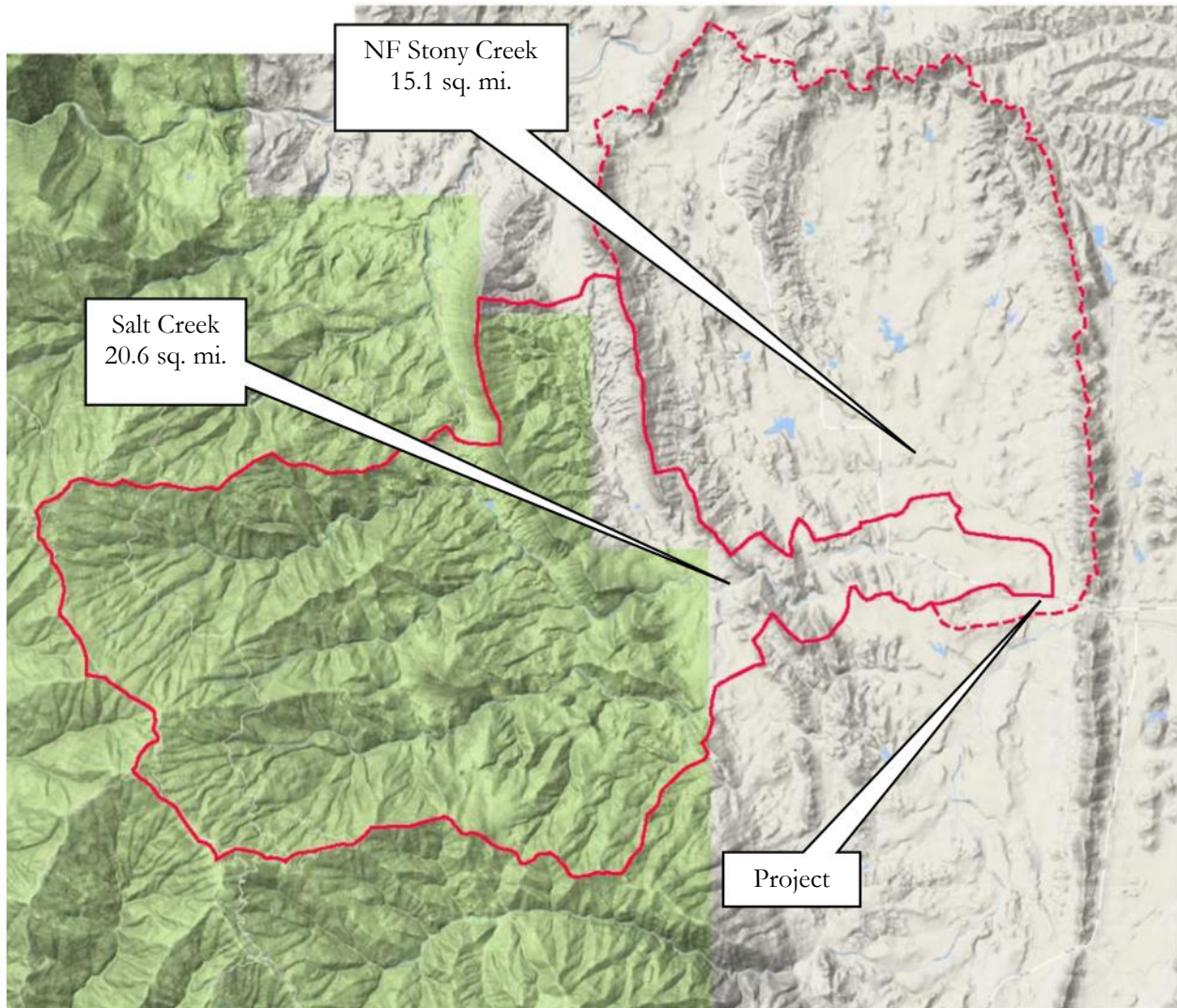


Figure 5. Basin contributing to overall hydrology (USGS streamstats).

Discharge at the bridge reach was calculated using three methods:

- A regression analysis ¹⁵
- A basin transfer of results of a statistical analysis of gage data ¹⁶
- A HEC-HMS analysis

The results of the three methods are shown in Table 3.

¹⁵ *Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006* (USGS SIR 2012-5113)

¹⁶ US. Department of the Interior, Geological Survey. *Guidelines for Determining Flood Flow Frequency, Bulletin #17B of the Hydrology Subcommittee*, Revised September 1981.

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Table 3. Regression, HEC-HMS, and statistical analysis results

Method	Discharge (cfs)			
	Project		Combined	
	50-yr	100-yr	50-yr	100-yr
Regression	3,778	4,455	5,820	6,866
Basin Transfer NF Stony Creek Gage (11387800)	5,420	6,372	8,746	10,258
HEC-HMS	3,135	3,830	4,915	6,007

The results from the Basin Transfer analysis are conservative when compared to the regression and gage analyses and were used for the hydraulic analysis as shown in Table 4.

Table 4. Discharges used for analysis (cfs)

	Project		Combined	
	Design	Base	Design	Base
Frequency (years)	50	100	50	100
Discharge (cubic feet per second)	5,420	6,400	8,770	10,300

See Appendix B for a complete summary of the three methods of analysis.

HEC-RAS ANALYSIS

Existing Condition

The Manning “n” values of 0.035 for the channel bottom and 0.05 for the banks and overbanks were used in the model and are consistent with the field review by Avila and Associates as shown in Figure 6.



Figure 6. Looking upstream from the bridge. The channel bottom is sparsely vegetated and the banks and overbanks are heavily vegetated.

The existing bridge was input into the model as a 3-span span bridge with a minimum soffit elevation of 622.5 feet as illustrated in Figure 7.

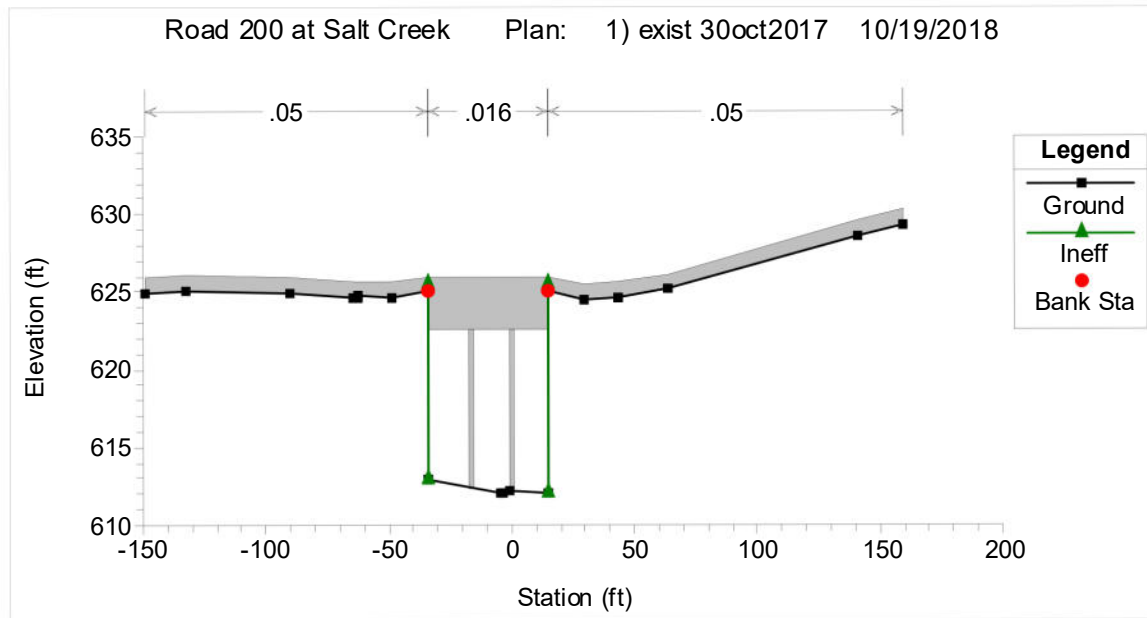


Figure 7. HEC-RAS upstream cross section of the existing bridge.

Starting Water Surface Elevation

Various downstream starting water surface elevation boundary conditions were analyzed as follows: 1) critical depth and 2) normal depth. As can be seen in Figure 8, all of the WSE profiles converged approximately 700 feet downstream from the existing bridge. Normal depth was used as the downstream boundary condition for establishing the starting WSE.

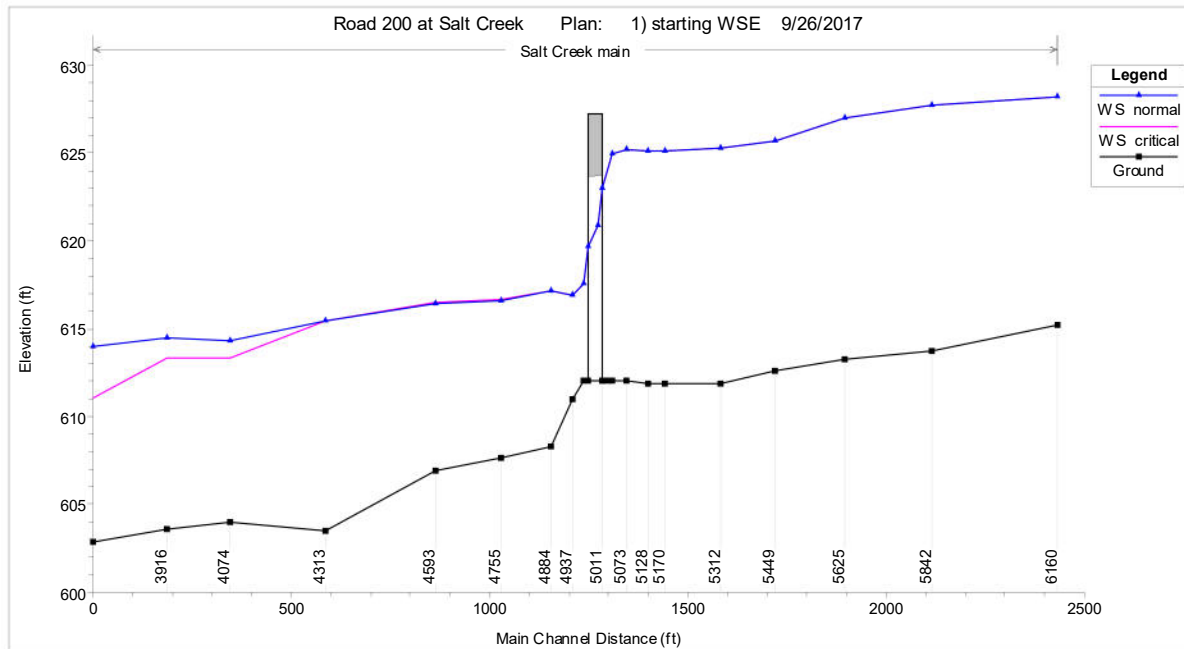


Figure 8. WSE profile comparison using various downstream boundary conditions

Proposed Condition Model

The HEC-RAS model was re-run for the proposed condition by replacing the existing bridge with the proposed bridge. The proposed bridge along the same alignment as the existing bridge and was modeled as a single-span bridge with minimum soffit elevation of 623 feet as shown in Figure 9.

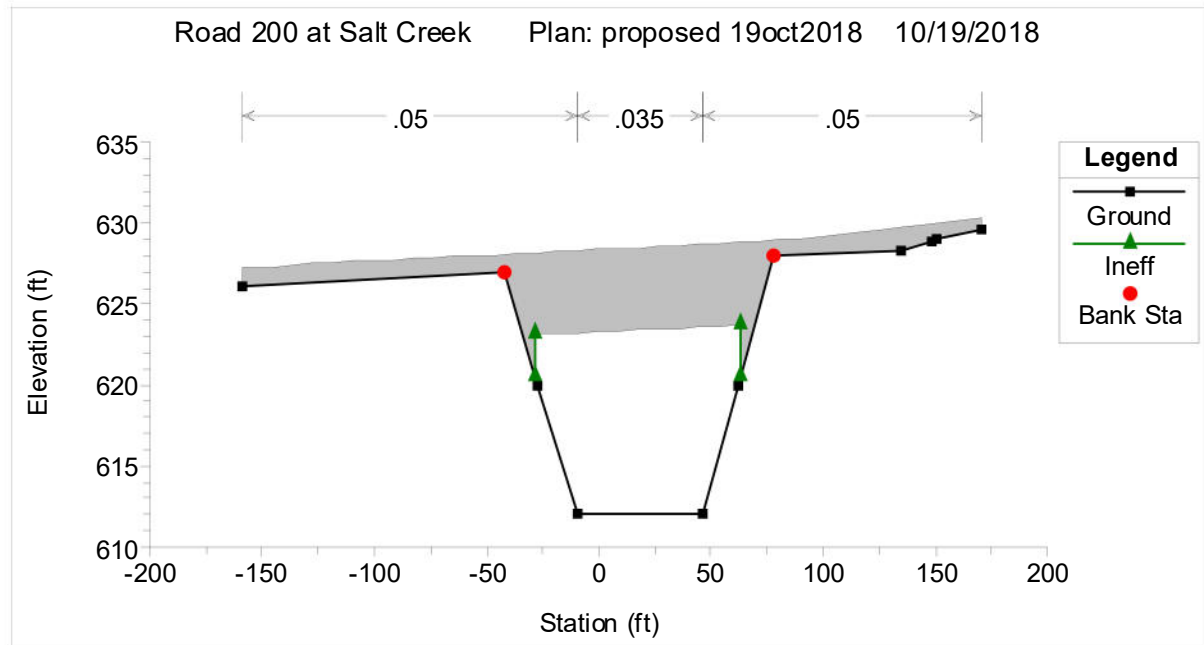


Figure 9. HEC-RAS upstream cross section of the proposed bridge.

Figure 10, Figure 11, and Table 5 show a comparison of the existing to the proposed water surface elevation (WSE) profiles for the 50-yr and 100-yr discharges. The RSP will have only a very small change to the Manning's "n" value which will not have a discernable effect on the water surface elevation. As can be seen, the WSE is significantly lowered upstream of the bridge, slightly higher just downstream of the bridge, and unchanged farther downstream with the proposed bridge.

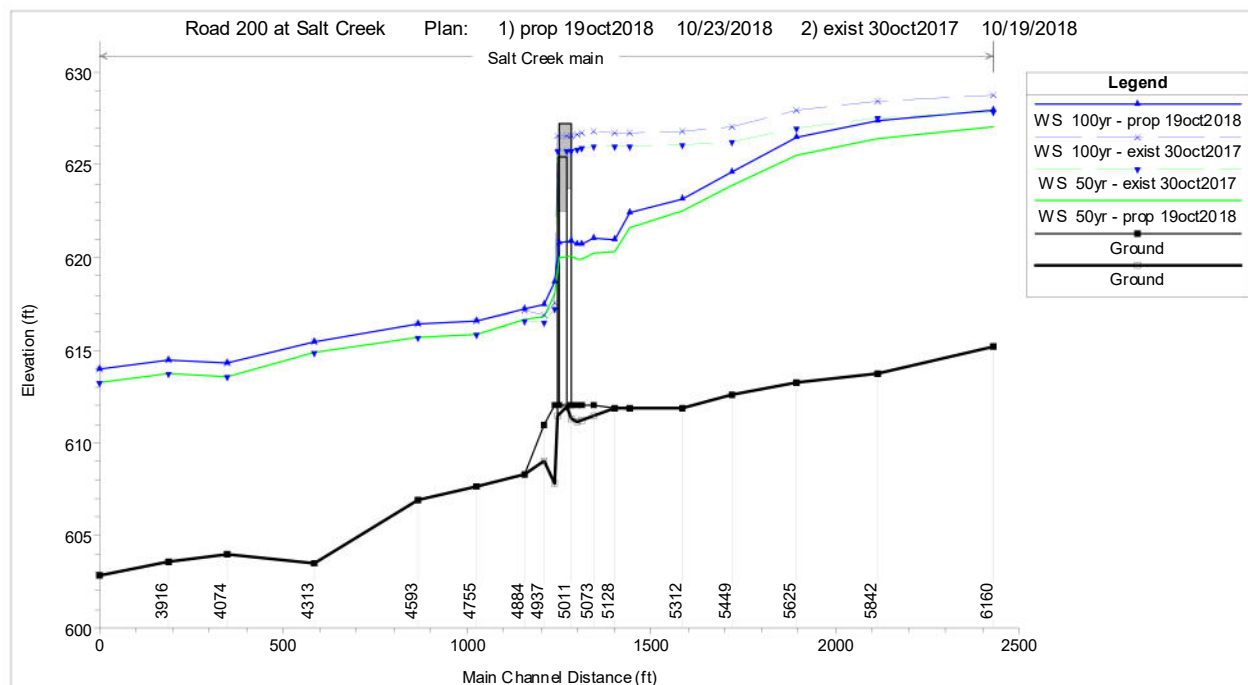


Figure 10. Water surface elevation profile comparison of existing to proposed for the 50 and 100-year discharges.

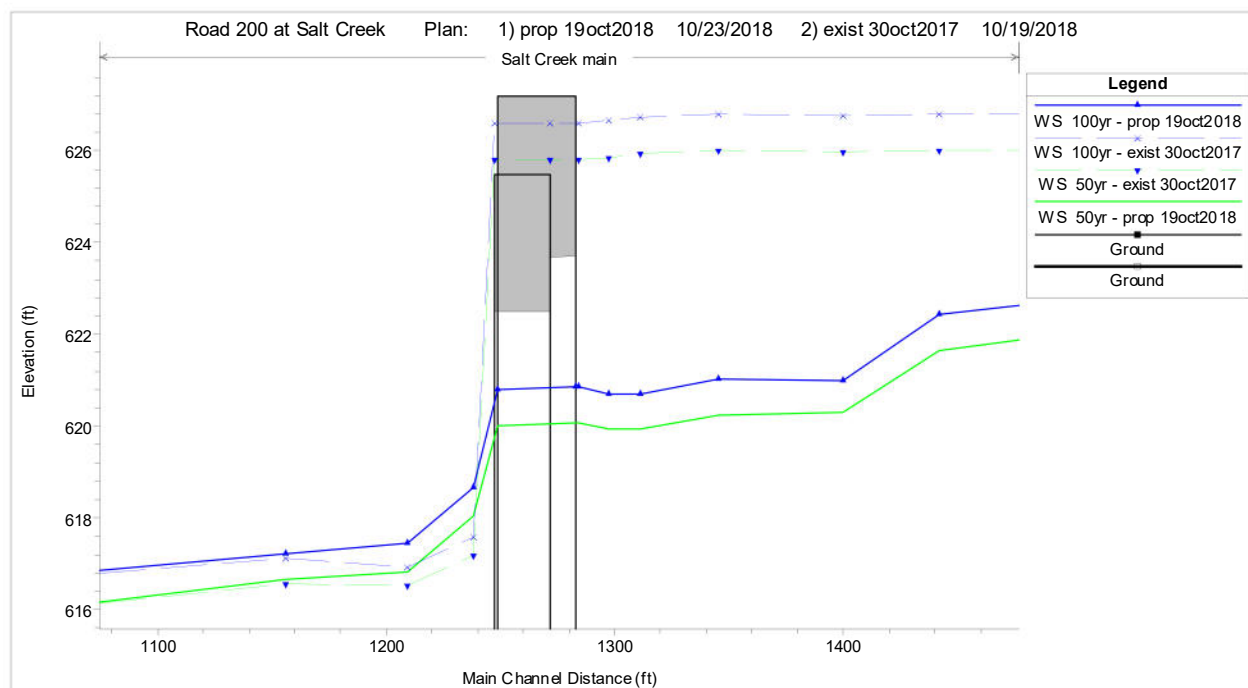


Figure 11. Zoomed in view of Figure 10.

Table 5. Water Surface Elevation (WSE) comparison of existing to proposed condition for the 50-yr and 100-yr discharges.

River Station	50-yr			100-yr		
	existing	proposed	difference	existing	proposed	difference
6160	627.9	627.04	-0.86	628.81	627.94	-0.87
5842	627.52	626.45	-1.07	628.43	627.37	-1.06
5625	627.02	625.5	-1.52	627.94	626.48	-1.46
5449	626.27	623.89	-2.38	627.04	624.65	-2.39
5312	626.06	622.52	-3.54	626.83	623.21	-3.62
5170	626.01	621.66	-4.35	626.79	622.43	-4.36
5128	625.98	620.29	-5.69	626.76	621	-5.76
5073	626.02	620.24	-5.78	626.8	621.04	-5.76
5039	625.94	619.94	-6.00	626.72	620.69	-6.03
5025.5	625.84	619.94	-5.90	626.65	620.7	-5.95
5012	625.8	620.08	-5.72	626.59	620.87	-5.72
Bridge						
4966	617.2	618.03	0.83	617.58	618.68	1.10
4937	616.52	616.83	0.31	616.91	617.44	0.53
4884	616.56	616.64	0.08	617.12	617.22	0.10
4755	615.87	615.87	0.00	616.62	616.62	0.00
4593	615.73	615.73	0.00	616.46	616.46	0.00
4313	614.86	614.86	0.00	615.43	615.43	0.00
4074	613.56	613.56	0.00	614.34	614.34	0.00
3916	613.71	613.71	0.00	614.43	614.43	0.00
3728	613.29	613.29	0.00	614.02	614.02	0.00

HYDRAULIC CRITERIA

Chapter 800 of the Caltrans Highway Design Manual (HDM) delineates the hydraulic design criteria for bridges (Caltrans, 2001). The basic HDM rule for hydraulic design is that bridges should be designed to pass the Q_{50} with sufficient freeboard and convey the Q_{100} without freeboard. Exceptions may be granted if the bridge designer can provide sufficient evidence that less freeboard is needed. The HDM notes that 2 feet of freeboard is often assumed to be appropriate for preliminary bridge designs, but leaves the recommendation for freeboard to the judgment of the hydraulic engineer based primarily upon the debris anticipated at the bridge.

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Since the minimum soffit elevation under proposed conditions is 623 feet, 2.9 feet of freeboard will be provided above the 50-year WSE (620.1 feet) and 2.1 feet of freeboard will be provided above the 100-year water surface elevation (620.9), which meets the HDM criteria.

DRIFT

Avila and Associates researched the available Bridge Maintenance Reports for the existing bridge to determine if floating debris catches on the bridge. There were two noted instances of debris caught on the pier of the existing bridge and obstructing flow. At the time of the field inspection, there were large quantities of drift and debris on the piers as shown in Figure 12.



Figure 12. Drift and debris on both piers of the existing bridge

The proposed bridge will improve the hydraulics due to the removal of the two piers. This will reduce obstructions to flow lowering the upstream water surface elevation, and the ability of the structure to capture debris.

STREAMBED/BANK PROTECTION

Riprap size was calculated using the FHWA Hydraulic Engineering Circular No. 23 (HEC 23) guidelines for RSP (FHWA, 2009). The riprap revetment design guidelines outlined in HEC 23 are based on flume

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studies performed by Stephen Maynard in 1989 and 1990 and were published in the U.S. Army Corp of Engineers (USACE) Engineering Manual (EM) 1601 in 1991.

Calculations were performed using the recommended minimum safety factor of 1.1 and the parameters provided in Table 6.

Table 6. Rock riprap calculation parameters and results through the proposed bridge reach.

Cross-Section	5039	5012	5011 BR U	5011 BR D	4966
Width (ft)	84.3	93.6	92.0	92.0	84.7
Average Velocity (ft/s)	11.3	9.7	9.7	8.9	13.4
Hydraulic Depth (ft)	6.7	7.2	7.2	7.8	5.6
Calculated D50 (in)	27.1	19.0	18.9	20.6	60.7
Calculated Weight (lbs)	1627.0	560.8	552.2	708.1	18196.6
Class (based on size)	VIII	VI	VI	VI	>XI
D50 (in)	30	21	21	21	>46
Weight (lbs)	2000	750	750	750	>8000
Thickness (in)	48	42	42	42	>67

The RSP is quite large and consideration could be given to designing to a lower recurrence interval. RSP should extend up to the 100-year WSE of 620.9 ft and be underlain with Type 8 filter fabric. The RSP should be keyed down to the scour depth, or a mounded toe approach should be used as shown in Figure 13.

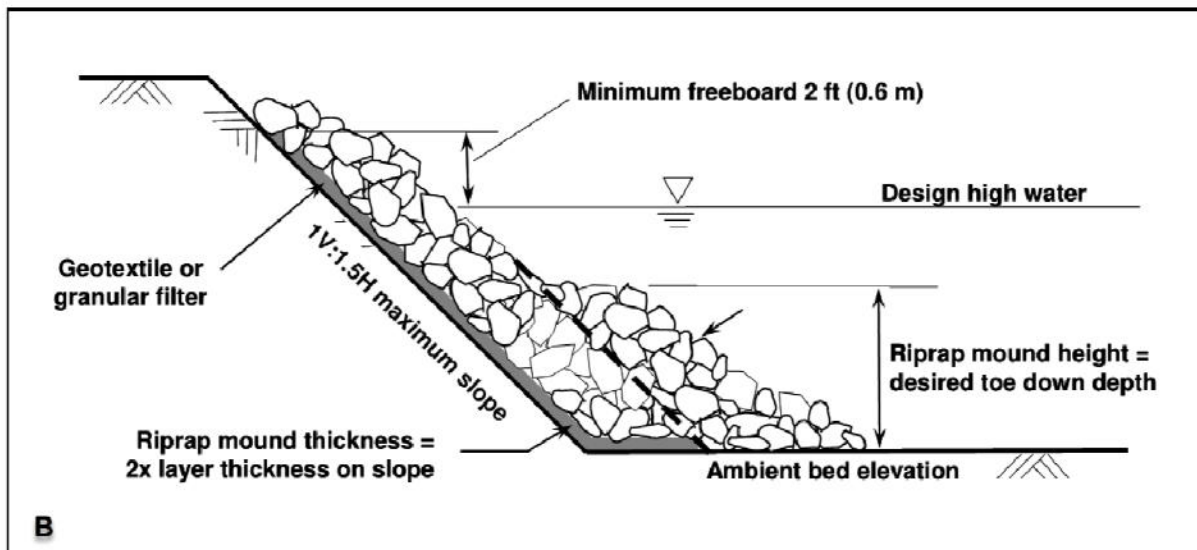
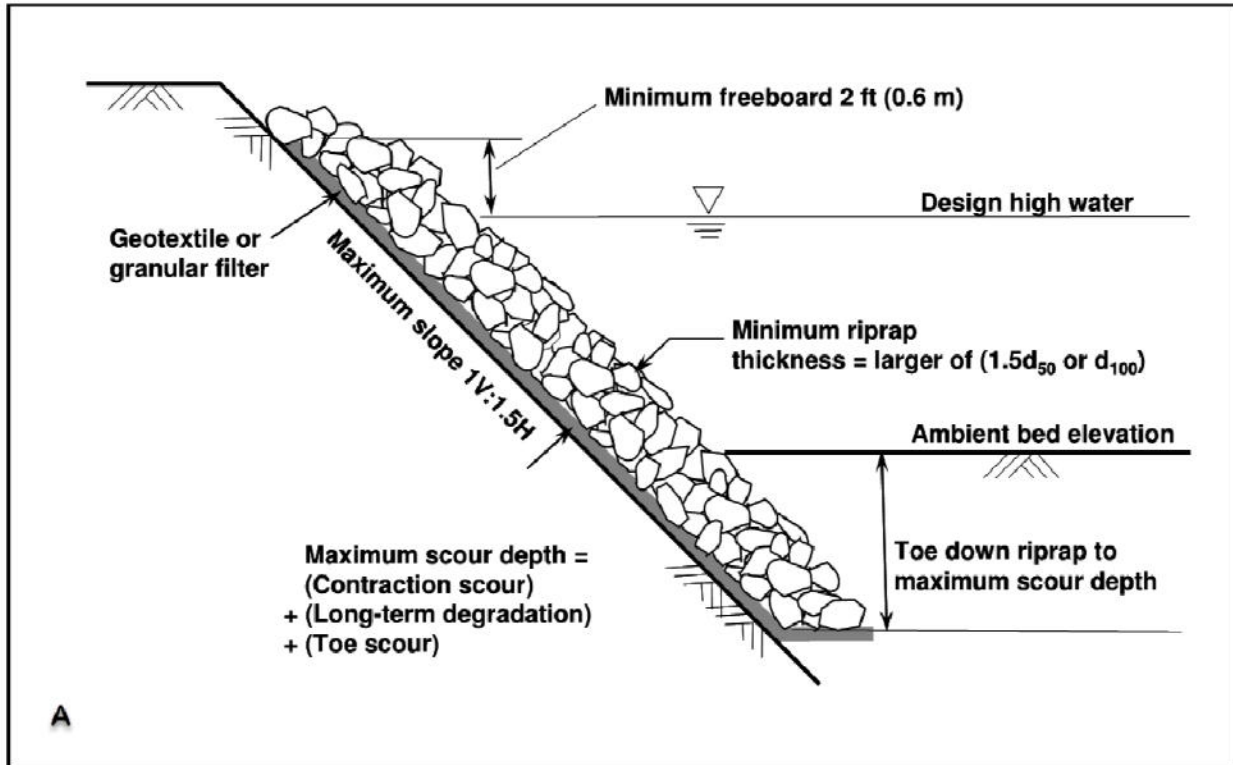


Figure 13. Bank RSP termination options: A) key down to the scour depth and B) Mounded Toe (Lagasse et al. 2009)

See Appendix E for bank protection calculations.

SCOUR

The County Road 200 Bridge was determined to have significant scour problems by Caltrans with scour issues first noted in the report dated 1978 resulting in significant undermining of the concrete lining of the channel under the structure as shown in Figure 14.



Figure 14. Looking toward the bridge, the concrete lining of the channel is undermined.

Degradation

There were no available cross sections for this bridge, so channel degradation could not be assessed from historical cross sections. However, Figure 14 clearly shows undermining of the concrete lining indicating the channel is degrading. Furthermore, as shown in the thalweg profile in Figure 15, there are two potential head cuts downstream of the bridge, indicating potential channel lowering ranging from 4 to 9 feet.

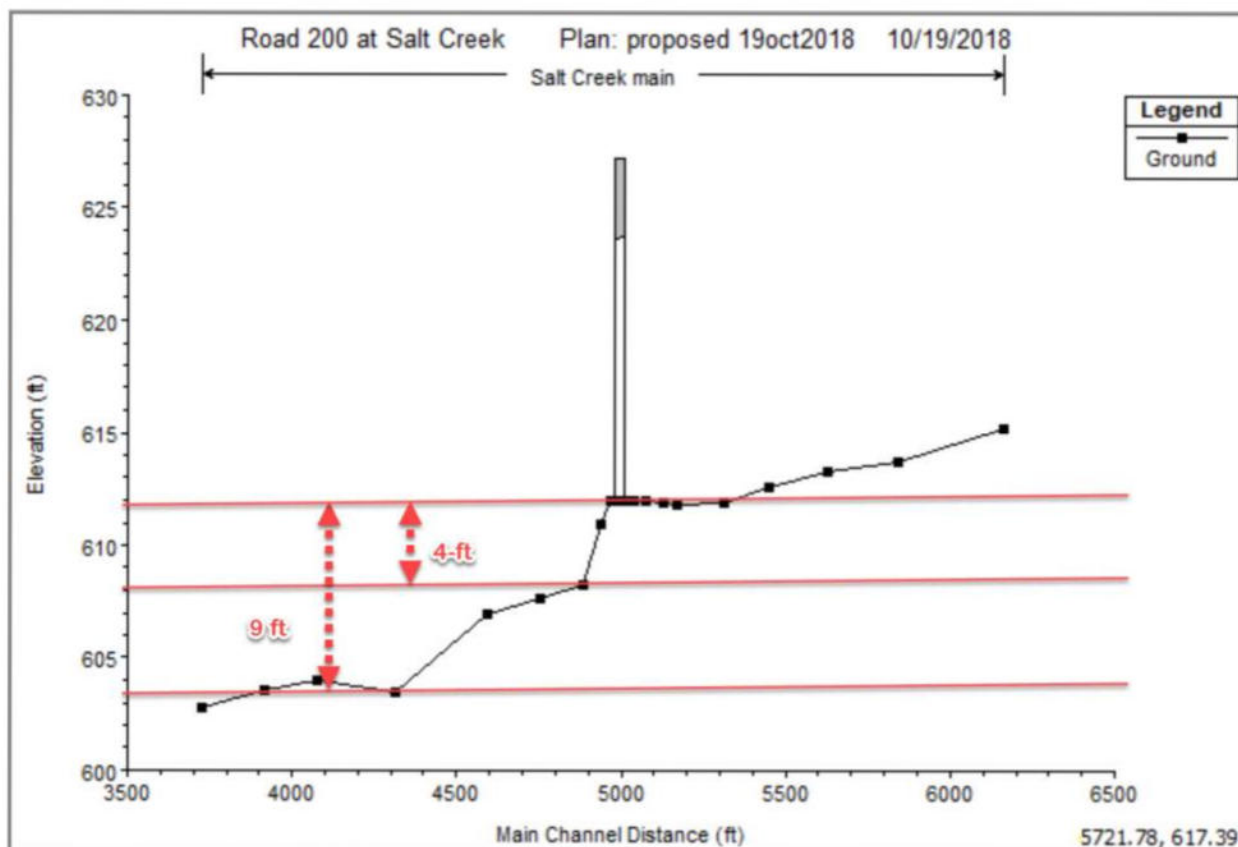


Figure 15. Channel thalweg profile and head cuts indicate potential degradation.

As shown in Figure 14, the channel lining below the existing bridge acts as a check dam which may have prevented the channel from headcutting upstream the initial 4 feet shown in Figure 15. Once the bridge and channel lining are removed, it is likely the channel profile will drop to create an equilibrium between the upstream and downstream ground elevation. Therefore, the channel thalweg without the concrete lining should be considered to be at elevation 608 ft, instead of the current thalweg of 612ft, to account for the removal of the concrete. The anticipated additional degradation after headcutting is 5 feet to elevation 603 ft.

Contraction Scour

The proposed bridge does not significantly constrict the channel and there is no calculated contraction scour.

Abutment Scour

Abutment scour was calculated using two methods outlined in the NCHRP 24-20 report: Scour Condition A and Scour Condition C. Scour condition A assumed the channel can migrate laterally to the abutments, and the equations are inclusive of contraction scour. The resulting abutment scour from Condition A is 2 feet (elevation 606).

Scour Condition C assumes the bridge will be designed to stand if the abutment fills are washed out, leaving the abutment as piers in the channel, during the 100-year event. The abutment scour is calculated as the local pier scour. For abutment piles of 2 feet, the resulting pier scour under Condition C is 5 feet. Unless

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it is determined the channel cannot migrate laterally, thalweg migration to the abutment could occur. Therefore, the abutment scour elevation should be determined from the channel thalweg of 608 ft to elevation 603.

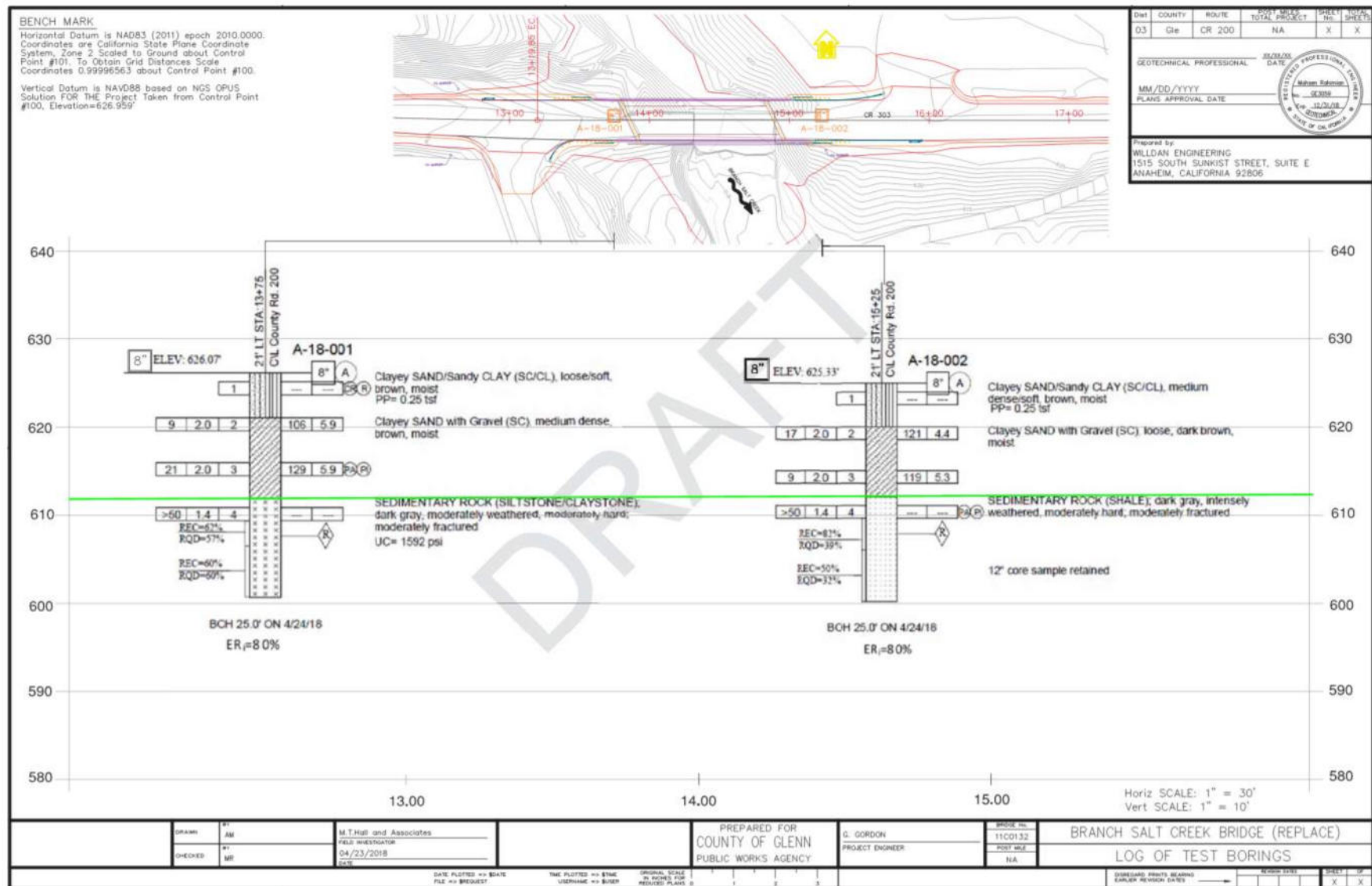
Total Scour

The theoretical Scour Summary Table is provided in Table 7. All scour depths are estimated based on the assumption of alluvial material. The Log of Test Borings for the project (Figure 16) show a layer of sedimentary rock at each abutment near elevation 612 ft (Willdan, 2018). If this formation is confirmed to be erosion resistant by the geotechnical engineer, this would limit the theoretical scour.

Table 7. Scour Summary Table

Long Term & Short-Term Scour Depths			
Support No.	Degradation Scour Depth (ft)	Contraction Scour Depth (ft)	Short Term (Local) Scour Depth (ft)
A1	5	0	2
A2	5	0	2

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SUMMARY TABLES

The following Hydrologic Summary Table is provided for your use for placement on the Foundation Plan:

Drainage Area: 20.6 Square miles

	Design	Base	Flood of Record
Frequency (Years)	50	100	>50
Discharge (Cubic feet per second)	5,420	6,400	4,450
Water Surface (Elevation at u/s face of Bridge)	620.1	620.9	619.2

The following Scour Data Table is provided for placement on the Foundation Plan assuming a thalweg elevation of 612 ft:

Support No.	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)
A1	603*	2
A2	603*	2

*Assuming migration of the thalweg (elevation 608ft) to the abutments

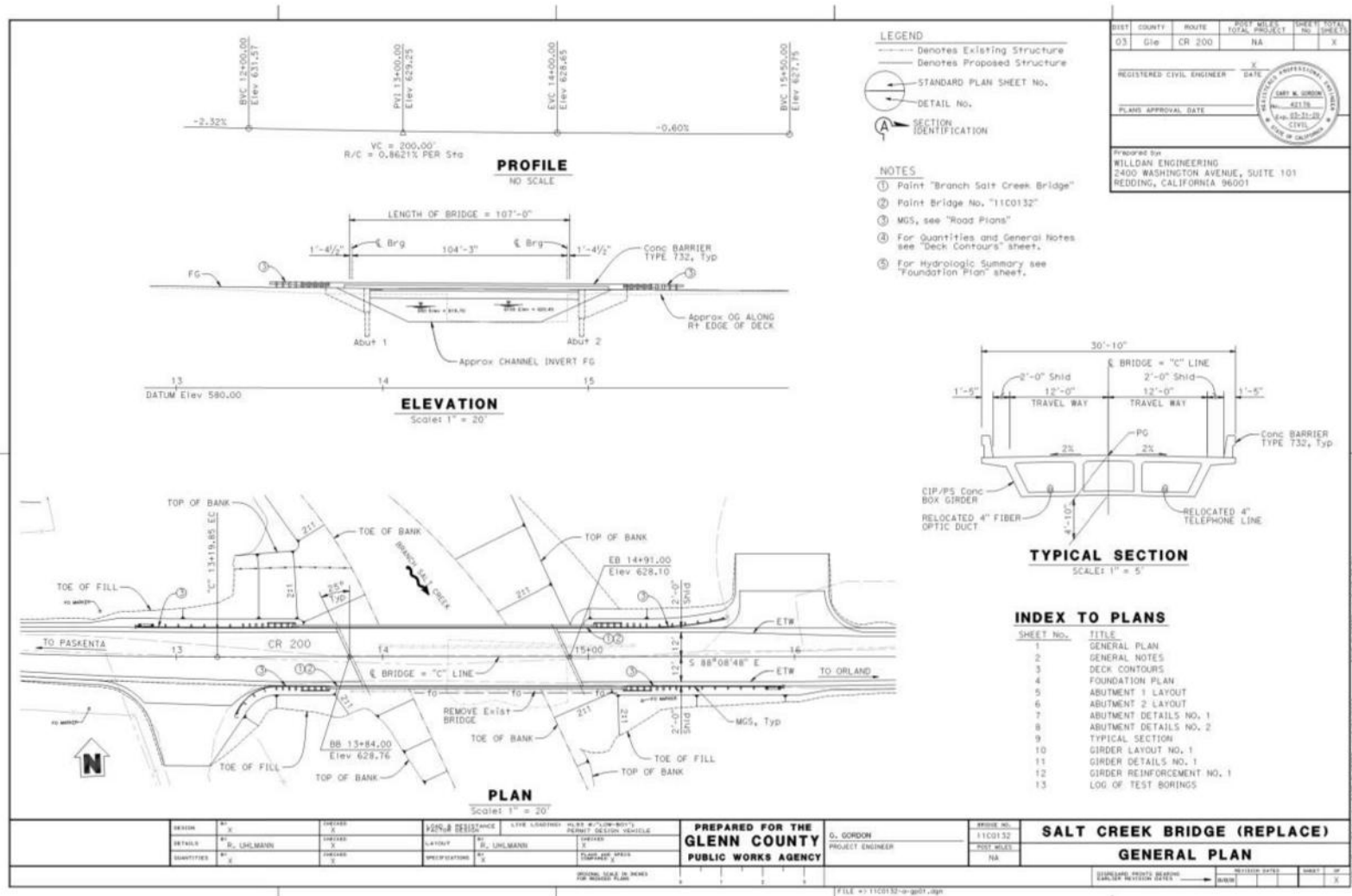
Flood plain data are based upon information available when the plans were prepared and are shown to meet Federal requirements. The accuracy of said information is not warranted by the County and interested or affected parties should make their own investigation.

The Floodplain Evaluation Report as outlined in 23 CFR 650 Subpart A, Section 650.111(b)(c)(d) is included in Appendices G and H.

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APPENDICES



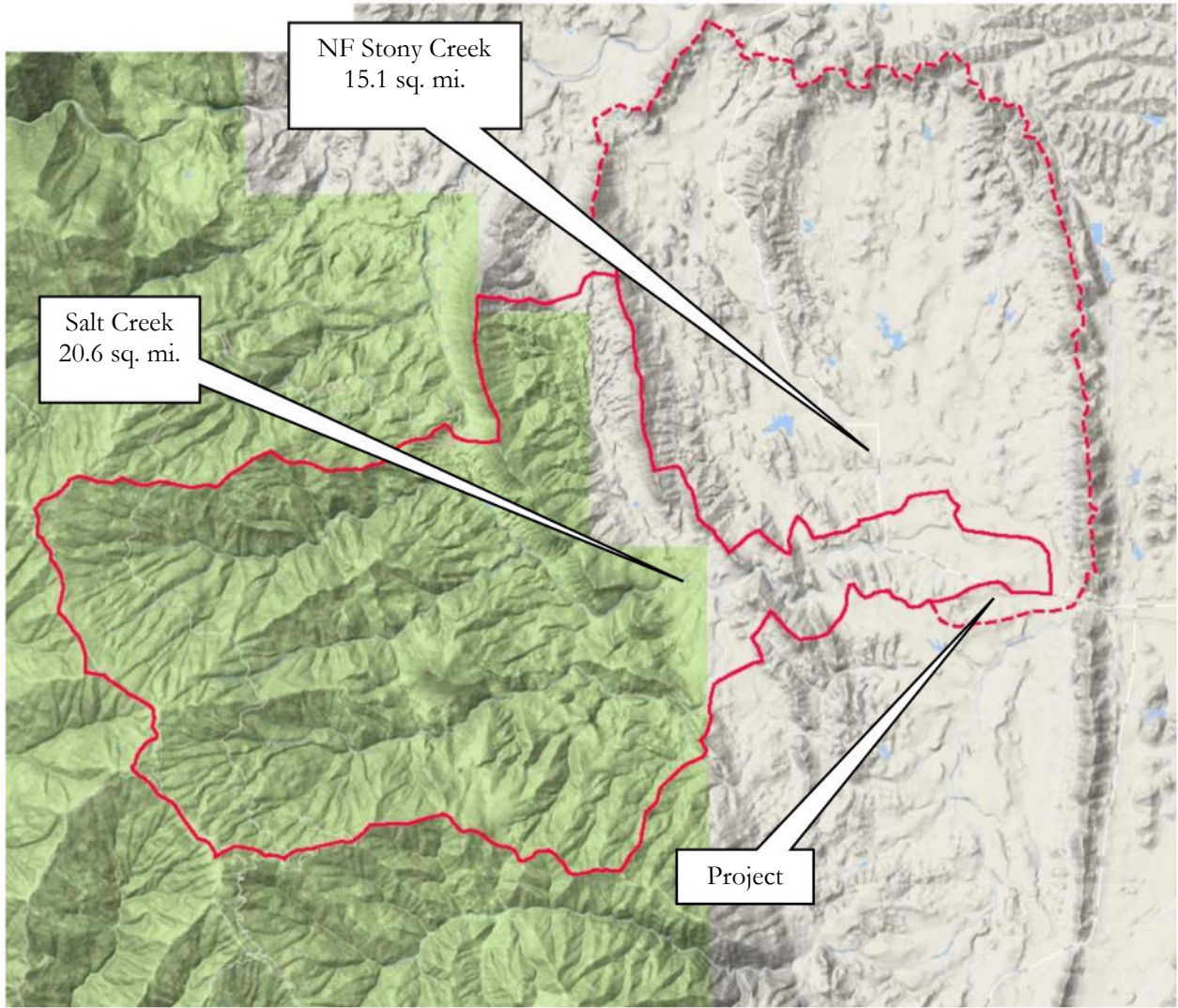
APPENDIX B – REGRESSION, STATISTICAL ANALYSIS, AND HEC-HMS DISCHARGES

Salt Creek at the Road 200 bridge drains an approximate 20.6 square miles as shown on the Project Hydrology Map. The mean annual precipitation of the project watershed is approximately 30.5 inches/year (streamstats).



Project Hydrology Map (Google Maps Terrain)

Just downstream from the project, Salt Creek joins North Fork (NF) Stony Creek. NF Stony Creek drains an approximate 15.1 square miles at the confluence as shown on the Overall Hydrology Map. NF Stony Creek is included in this analysis because the combined discharges may impact the water surface elevation downstream from the project due to the proximity of the confluence. The mean annual precipitation of the NF Stony Creek watershed is approximately 25.1 inches/year (streamstats).



Overall Hydrology Map (Google Maps Terrain)

Three methods of analysis were performed to estimate the design discharges for the hydraulic analysis:

1. Regional Regression
2. Statistical Analysis of gage data.
3. HEC-HMS analysis.

METHOD 1: Regional Regression

U.S. Geological Survey website application Streamstats (water.usgs.gov/osw/streamstats/) was used to obtain the basin characteristics and flow statistics for the project watershed. Flow characteristics are based on *Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006 (USGS SIR 2012-5113)*.

Project (Salt Creek)

North Coast Region 1

Area = 20.6 sq. mi.

MAP = 30.5 in/yr

C	DRNAREA	y	DRNAREA ^y	PRECIP	z	PRECIP ^z	Q (cfs)	Recurrence
1.82	20.6	0.904	28.0419	30.5	0.983	28.7784	807	2
8.11	20.6	0.887	118.6919	30.5	0.772	13.9920	1661	5

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14.8	20.6	0.88	212.0630	30.5	0.696	10.7913	2288	10
26	20.6	0.874	365.8418	30.5	0.628	8.5534	3129	25
36.3	20.6	0.87	504.6277	30.5	0.589	7.4860	3778	50
48.5	20.6	0.866	666.1174	30.5	0.556	6.6876	4455	100
61	20.6	0.863	830.2278	30.5	0.531	6.1399	5098	200
79.3	20.6	0.86	1069.5448	30.5	0.503	5.5796	5968	500

Combined (Salt Creek and NF Stony Creek)

North Coast Region 1

Area = 35.7 sq. mi.

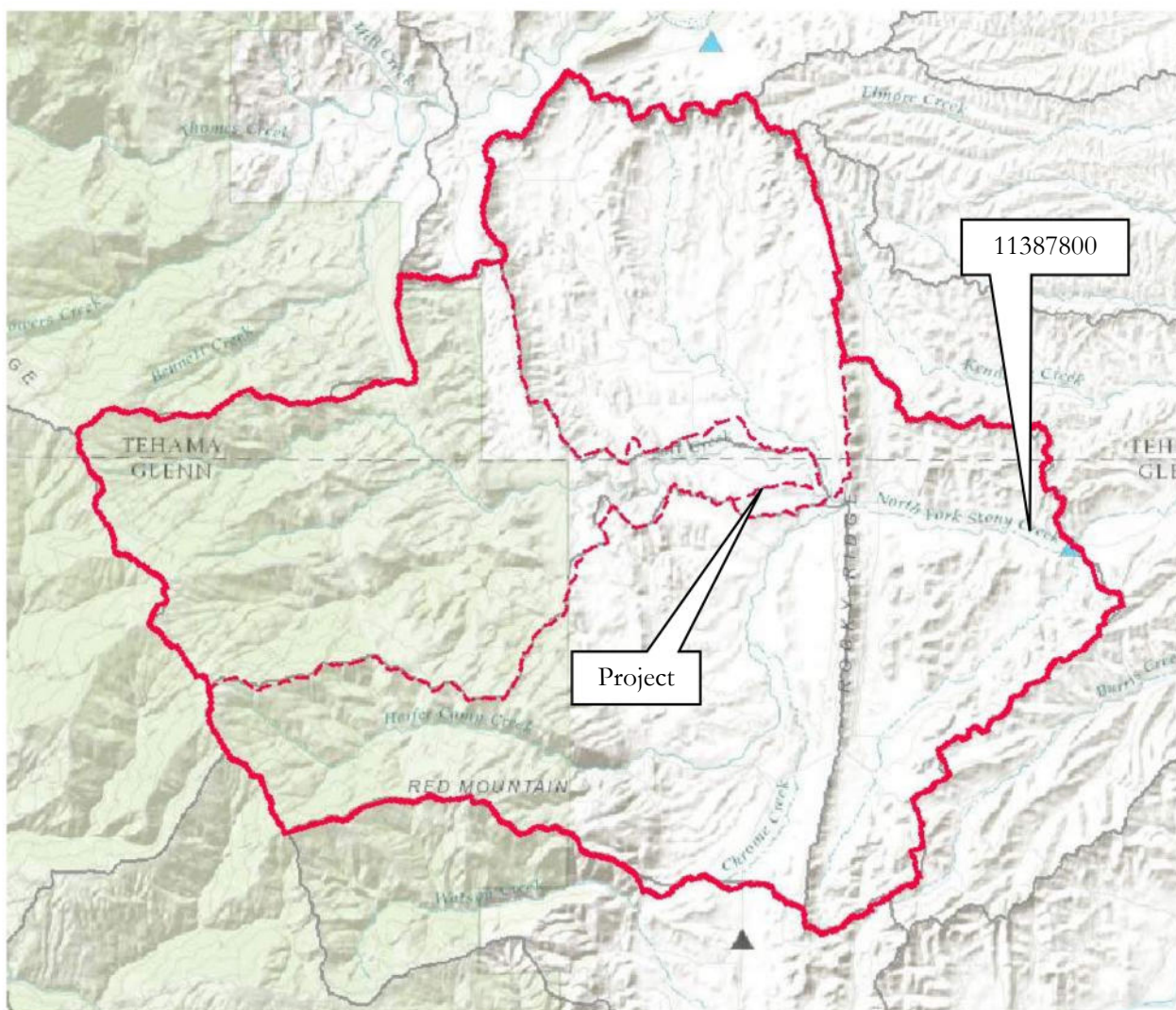
MAP = 28.2 in/yr (weighted average)

C	DRNAREA	y	DRNAREA^y	PRECIP	z	PRECIP^z	Q (cfs)	Recurrence
1.82	35.7	0.904	46.0981	28.2	0.983	26.6437	1228	2
8.11	35.7	0.887	193.3025	28.2	0.772	13.1702	2546	5
14.8	35.7	0.88	344.0406	28.2	0.696	10.2182	3515	10
26	35.7	0.874	591.5689	28.2	0.628	8.1425	4817	25
36.3	35.7	0.87	814.1941	28.2	0.589	7.1482	5820	50
48.5	35.7	0.866	1072.3892	28.2	0.556	6.4023	6866	100
61	35.7	0.863	1334.3892	28.2	0.531	5.8896	7859	200
79.3	35.7	0.86	1716.1998	28.2	0.503	5.3638	9205	500

METHOD 2: Statistical Analysis of Gage Data

There is an existing stream gage on NF Stony Creek approximately 3.1 miles downstream from the confluence (USGS gage #11387800). The area of the watershed at the gage is approximately 67.9 square miles and has 16 peak stream flow records taken between 1963 and 1978. During this period, the highest peak flow recorded was 12,500 cfs in January 1965. Other significant flows recorded are 6,050 cfs in January 1970 and 5,860 cfs in January 1978.

The location of the gage, the project, and their corresponding watersheds are shown on the Gage Location Map.



Gage Location Map

Results of the Gage 11387800 statistical analysis were obtained from USGS Scientific Investigations Report (SIR) 2012-5113, *Methods for determining magnitude and frequency of floods in California based on data through water year 2006*. Results of the statistical analysis from SIR 2012-5113 and a basin transfer to the project and to the confluence are shown in table below.

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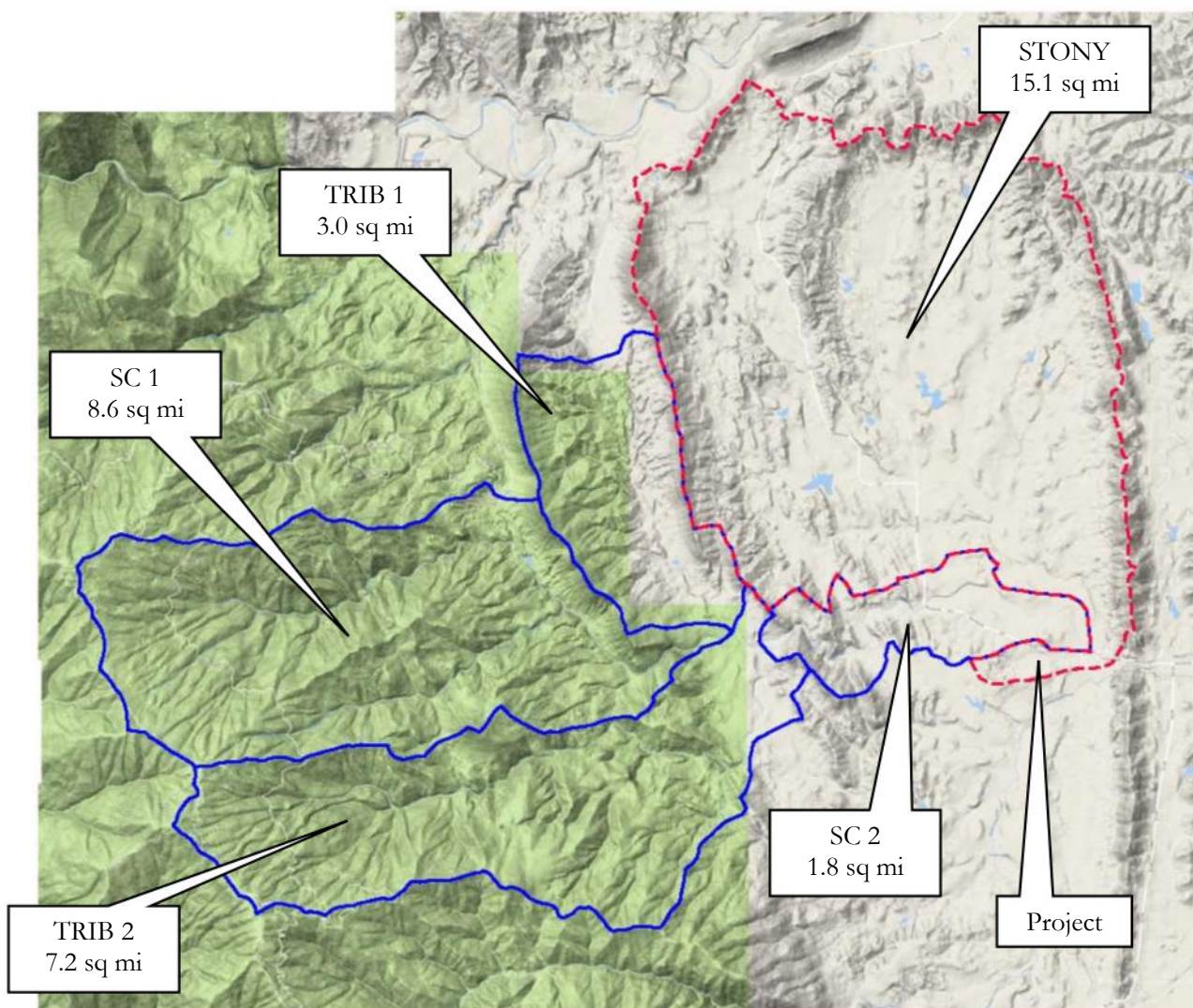
Recurrence interval (yr)	Discharge (cfs)		
	Statistical analysis (SIR 2012-5113)	Basin Transfer	
		to Project	to Confluence
2	3,060	1,041	1,711
5	6,490	2,253	3,669
10	9,120	3,193	5,180
25	12,600	4,443	7,184
50	15,300	5,420	8,746
100	17,900	6,372	10,258
500	23,800	8,533	13,692

METHOD 3: HEC-HMS analysis

The following methods and parameters were used for the HEC-HMS analysis:

- SCS Curve Number loss method
 - Initial Abstraction = 0.2
- SCS Unit Hydrograph transform method
 - Type 1A storm distribution
 - Lag time = 0.6 x time of concentration

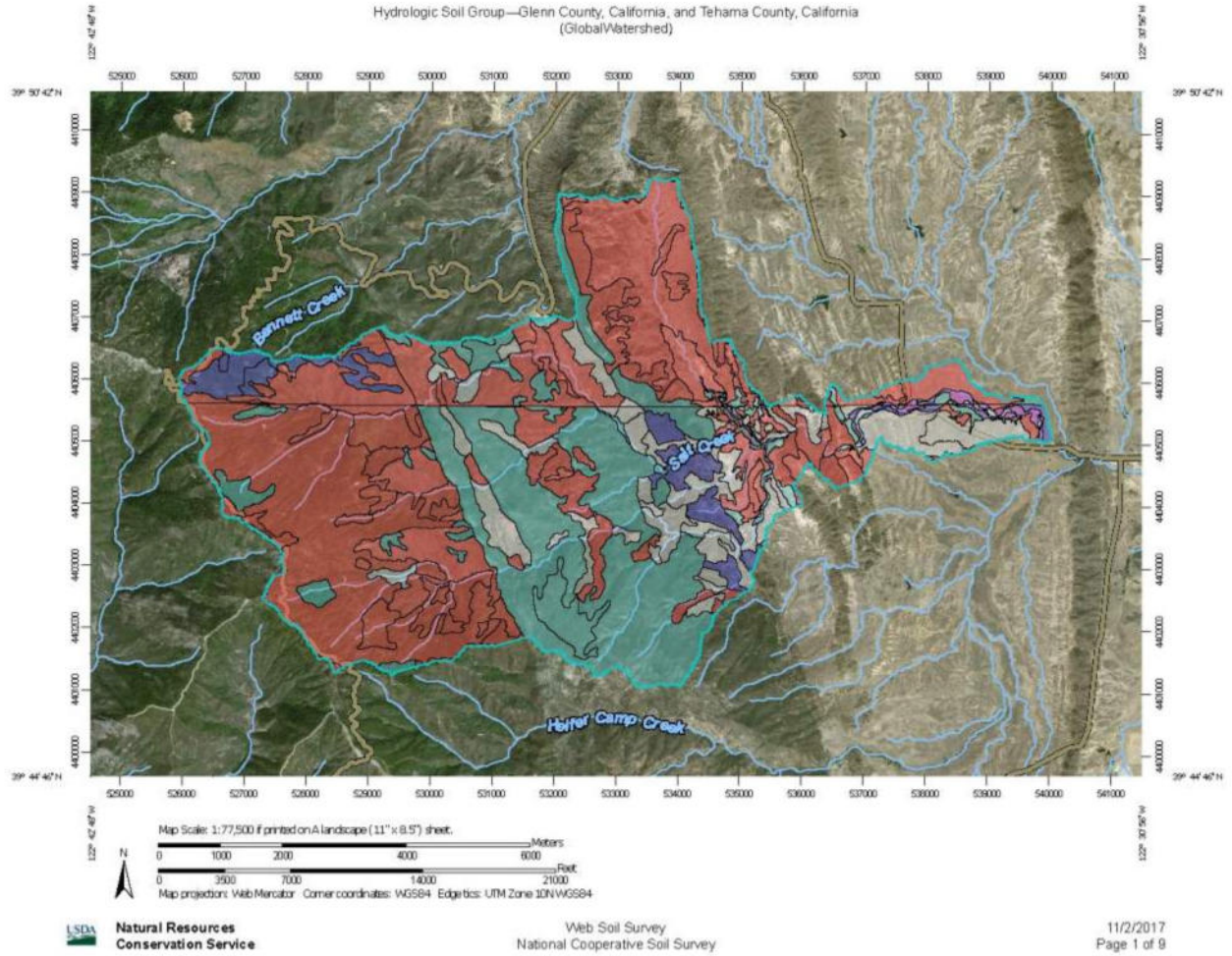
The project watershed was divided into four sub-basins and NF Stony Creek was analyzed as a single watershed as shown below.



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HEC-HMS Sub-basins

The project watershed is composed of soils from four hydrologic soils groups as shown on the Project Watershed Soils Map.



Magenta = Class A, Blue = Class B, Aqua = Class C, Salmon = Class D, Tan = Not rated or N/A
Project Watershed Soils Map (USDA NRCS Web Soil Survey)

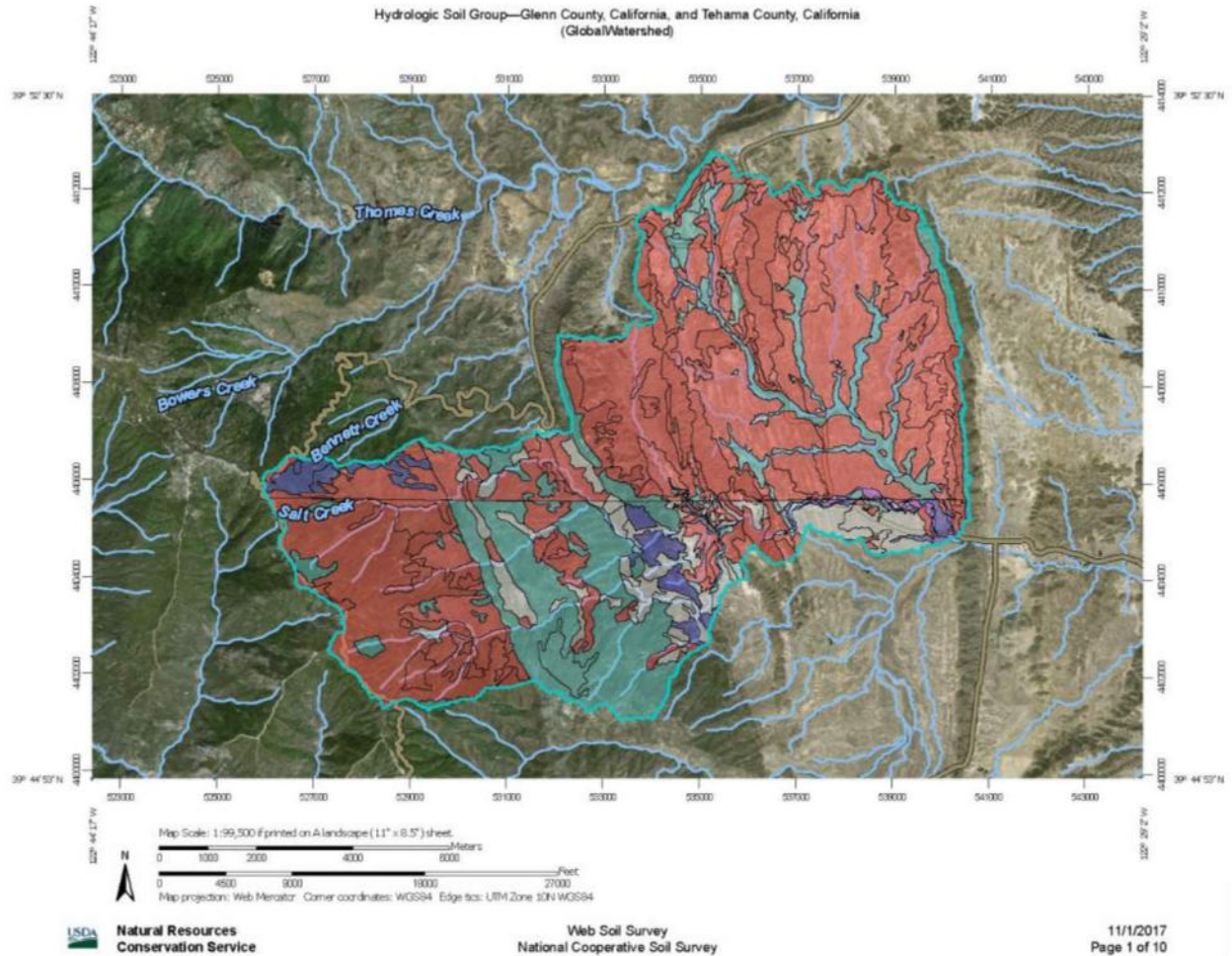
A breakdown of the soils types for the project watershed is shown in the table below along with the composite CN value.

Soil Class	Area (acre)	CN (see note)	Area x CN
A	71.7	30	2151.0
B	711.5	55	39132.5
C	3419.7	70	239379.0
D	8975.3	77	691098.1
Total	13178.2		971760.6
Project		Composite	74

Note: Cover type is primarily woods with hydrologic condition good.

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The NF Stony Creek watershed is composed of soils from four hydrologic soils groups as shown on the Overall Watershed Soils Map.



Magenta = Class A, Blue = Class B, Aqua = Class C, Salmon = Class D, Tan = Not rated or N/A
Overall Watershed Soils Map (USDA NRCS Web Soil Survey)

A breakdown of the soils types for the NF Stony Creek watershed is shown in the table below along with the composite CN value. The areas of each soils class were determined by subtracting the project areas from the overall areas.

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Soil Class	Area (acre)	CN (see note)	Area x CN
A	12	30	360.0
B	77.6	48	3724.8
C	1404.1	65	91266.5
D	8172.1	73	596563.3
Water	40.8	99	4039.2
Total	9706.6		695953.8
NF Stony Creek		Composite	72

Note: Cover type is primarily brush with hydrologic condition good.

Precipitation data was obtained from the NOAA's National Weather Service Hydrometeorological Design Studies Center Precipitation Frequency Data Server by manually entering the latitude and longitude of the centroid of the project watershed area. http://hdsc.nws.noaa.gov/hdsc/pfds/pfds_map_cont.html?bkmrk=ca (NOAA Atlas 14)

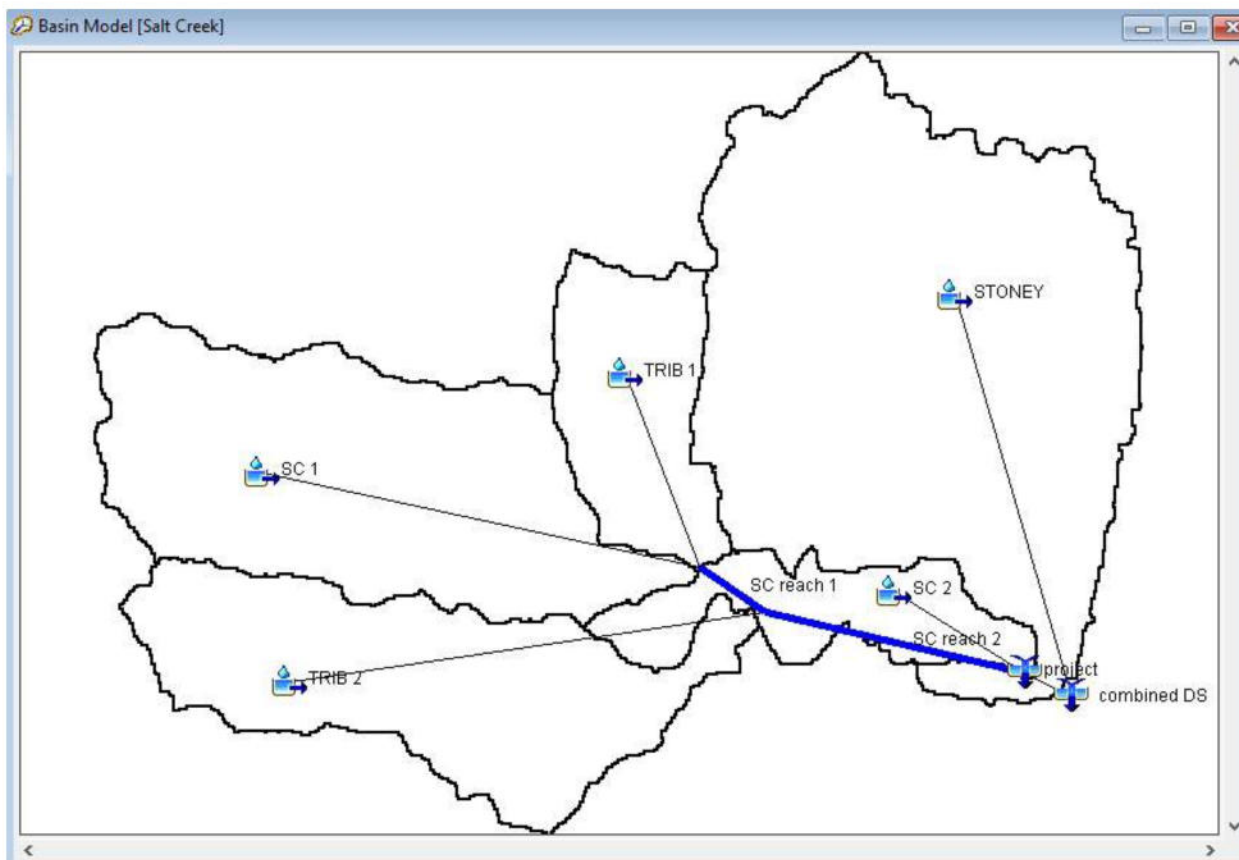
For this analysis, the 12-hour storm was analyzed and the precipitation depth for the 50-yr and 100-yr storm for the project watershed is 3.72 and 4.18 inches respectively.

Times of concentration for all sub-basins were determined by adding the travel times for overland, shallow, and channel flows within each sub-basin. The times of concentration were used to calculate the lag times. Lag times for reaches were determined by calculating the travel time in the channel between points of concentration. The time of concentration and lag time calculations are shown in the table below.

Sub-basin	Overland Flow					Shallow Flow				Channel Flow			TOC (min)	Lag Time (min)
	L (ft)	s (ft/ft)	n	T (hr)	T (min)	L (ft)	s (ft/ft)	v (ft/s)	T (min)	L (ft)	v (ft/s)	T (min)		
SC 1	300	0.3	0.4	0.31	18.6	6073	0.29	8.5	11.9	30205	13.7	36.7	67.2	40
TRIB 1	300	0.53	0.4	0.246	14.8	4313	0.16	6.5	11.1	14661	6.7	36.5	62.3	37
TRIB 2	300	0.02	0.4	0.86	51.6	3639	0.29	8.5	7.1	32431	14.4	37.5	96.3	58
SC 2	300	0.52	0.13	0.101	6.1	835	0.24	8	1.7	20936	7.3	86.6	94.3	57
STONY	300	0.13	0.13	0.129	7.7	1105	0.14	6	3.1	35737	7.5	79.9	90.7	54
Reach 1										1529	7.4	3.4		4
Reach 2										21740	7.3	49.4		49

A schematic of the HEC-HMS model is shown below.

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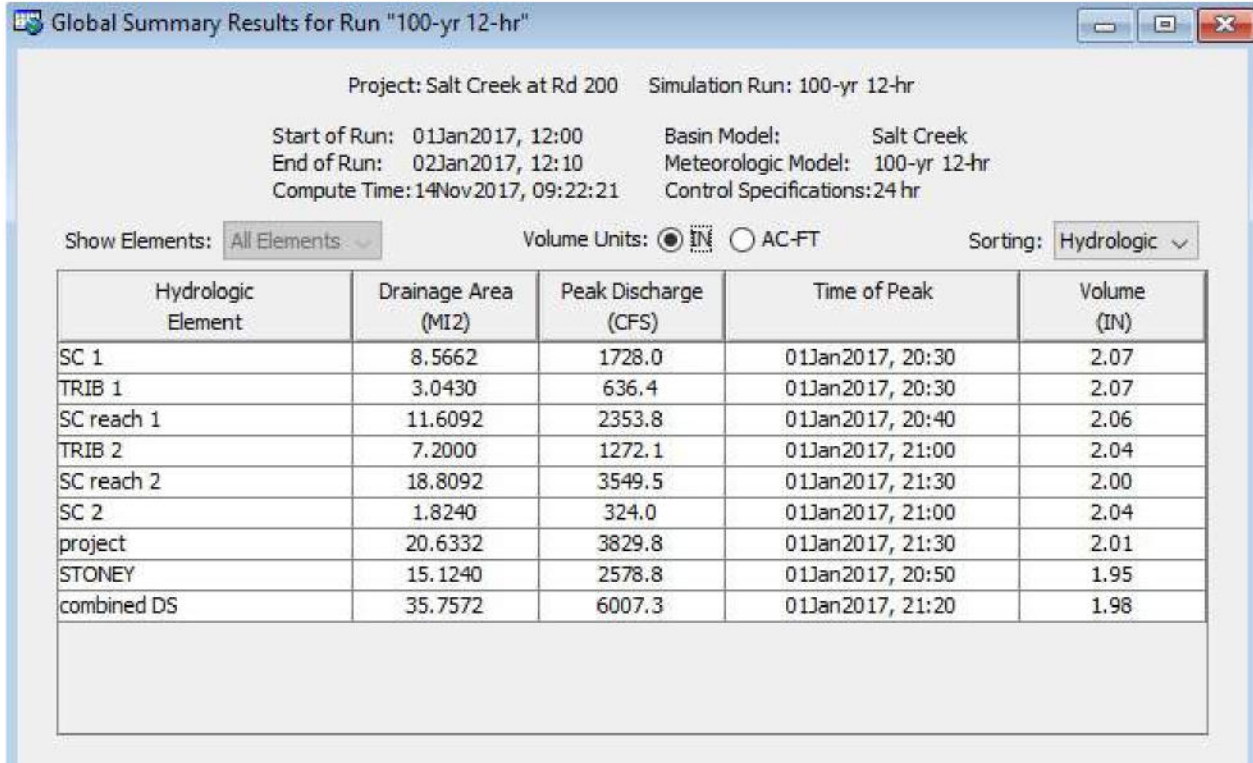


HEC-HMS Model Schematic

50-yr and 100-yr peak discharges from the HEC-HMS analysis are shown below.

Global Summary Results for Run "50-yr 12-hr"				
Project: Salt Creek at Rd 200 Simulation Run: 50-yr 12-hr				
Start of Run: 01Jan2017, 12:00		Basin Model: Salt Creek		
End of Run: 02Jan2017, 12:10		Meteorologic Model: 50-yr 12-hr		
Compute Time: 14Nov2017, 09:22:22		Control Specifications: 24 hr		
Show Elements: All Elements		Volume Units: ☒ IN ☐ AC-FT		Sorting: Hydrologic
Hydrologic Element	Drainage Area (MI ²)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
SC 1	8.5662	1416.1	01Jan2017, 20:40	1.72
TRIB 1	3.0430	520.8	01Jan2017, 20:30	1.73
SC reach 1	11.6092	1927.9	01Jan2017, 20:40	1.72
TRIB 2	7.2000	1042.4	01Jan2017, 21:00	1.70
SC reach 2	18.8092	2904.1	01Jan2017, 21:30	1.67
SC 2	1.8240	265.6	01Jan2017, 21:00	1.70
project	20.6332	3135.2	01Jan2017, 21:30	1.67
STONEY	15.1240	2103.6	01Jan2017, 20:50	1.62
combined DS	35.7572	4915.2	01Jan2017, 21:20	1.65

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A summary of the results of the three methods of analysis are shown in the table below.

Summary of Results from three methods of analysis.

Method	Discharge (cfs)			
	Project		Combined	
	50-yr	100-yr	50-yr	100-yr
Regression	3,778	4,455	5,820	6,866
Basin Transfer NF Stony Creek Gage (11387800)	5,420	6,372	8,746	10,258
HEC-HMS	3,135	3,830	4,915	6,007

The results from the gage analysis are conservative when compared to the regression and HEC-HMS analyses and will be used for the hydraulic analysis as shown in the table below.

Estimated discharges used for design.

	Project		Combined	
	Design	Base	Design	Base
Frequency (years)	50	100	50	100
Discharge (cubic feet per second)	5,420	6,400	8,770	10,300

APPENDIX C- HEC-RAS RESULTS

HEC-RAS Plan: exist 30oct2017 River: Salt Creek Reach: main

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
main	6160	50yr	5420.00	615.21	627.90		628.63	0.002139	7.09	846.67	117.75	0.41
main	6160	100yr	6400.00	615.21	628.81		629.61	0.002188	7.46	955.13	121.17	0.41
main	5842	50yr	5420.00	613.74	627.52		627.98	0.001519	5.64	1065.48	141.64	0.32
main	5842	100yr	6400.00	613.74	628.43		628.94	0.001548	5.98	1197.37	148.74	0.32
main	5625	50yr	5420.00	613.28	627.02		627.59	0.001995	6.61	986.98	145.61	0.37
main	5625	100yr	6400.00	613.28	627.94		628.54	0.002016	6.89	1124.75	155.49	0.37
main	5449	50yr	5420.00	612.60	626.27		627.16	0.002557	7.86	755.70	89.49	0.42
main	5449	100yr	6400.00	612.60	627.04		628.08	0.002824	8.53	825.80	92.00	0.45
main	5312	50yr	5420.00	611.88	626.06		626.81	0.001983	7.97	871.04	105.92	0.39
main	5312	100yr	6400.00	611.88	626.83		627.69	0.002143	8.61	953.01	107.85	0.41
main	5170	50yr	5420.00	611.83	626.01		626.51	0.001266	6.33	1062.08	131.21	0.32
main	5170	100yr	6400.00	611.83	626.79		627.37	0.001378	6.88	1165.48	136.69	0.33
main	5128	50yr	5420.00	611.88	625.98		626.45	0.001166	6.10	1112.33	143.29	0.31
main	5128	100yr	6400.00	611.88	626.76		627.30	0.001267	6.62	1225.46	149.70	0.32
main	5073	50yr	5420.00	611.48	626.02		626.36	0.000796	5.27	1363.10	177.46	0.26
main	5073	100yr	6400.00	611.48	626.80		627.20	0.000883	5.78	1512.57	202.47	0.28
main	5039	50yr	5420.00	611.21	625.94	619.93	626.32	0.001040	5.33	1235.15	207.39	0.28
main	5039	100yr	6400.00	611.21	626.72	620.65	627.16	0.001101	5.73	1401.75	216.70	0.29
main	5025.5	50yr	5420.00	611.12	625.84	619.66	626.30	0.001096	5.99	1243.30	221.85	0.30
main	5025.5	100yr	6400.00	611.12	626.65	620.32	627.14	0.001119	6.31	1429.49	235.76	0.30
main	5012	50yr	5420.00	611.28	625.80	619.67	626.29	0.000805	6.01	1260.88	232.57	0.29
main	5012	100yr	6400.00	611.28	626.59	620.53	627.12	0.000851	6.42	1456.00	255.90	0.30
main	5000		Bridge									
main	4966	50yr	5420.00	607.84	617.20	615.14	619.10	0.004363	11.08	489.38	81.33	0.66
main	4966	100yr	6400.00	607.84	617.58	615.90	620.02	0.005269	12.53	510.95	82.59	0.73
main	4937	50yr	5420.00	609.05	616.52	616.04	618.81	0.009396	12.47	468.45	86.62	0.88
main	4937	100yr	6400.00	609.05	616.91	616.76	619.70	0.010766	13.79	502.77	89.07	0.95
main	4884	50yr	5420.00	608.29	616.56		618.17	0.007329	10.37	565.74	119.24	0.73
main	4884	100yr	6400.00	608.29	617.12	616.26	618.94	0.007493	11.10	633.95	126.48	0.75
main	4755	50yr	5490.00	607.64	615.87		617.22	0.006192	11.74	703.22	150.52	0.77
main	4755	100yr	6455.00	607.64	616.62		617.97	0.005544	11.85	817.00	154.08	0.74
main	4593	50yr	5490.00	606.95	615.73		616.43	0.002435	8.15	941.25	160.30	0.52
main	4593	100yr	6455.00	606.95	616.46		617.23	0.002374	8.55	1060.40	164.10	0.52
main	4313	50yr	5490.00	603.48	614.86		615.76	0.002226	7.86	774.95	105.52	0.45
main	4313	100yr	6455.00	603.48	615.43		616.52	0.002486	8.64	836.40	107.62	0.48
main	4074	50yr	5490.00	604.02	613.56		614.96	0.004837	10.25	695.68	167.90	0.63
main	4074	100yr	6455.00	604.02	614.34		615.71	0.004415	10.40	827.38	172.34	0.61
main	3916	50yr	8770.00	603.55	613.71		614.30	0.001946	6.38	1516.86	235.83	0.40
main	3916	100yr	10300.00	603.55	614.43		615.09	0.001970	6.76	1694.07	258.84	0.40
main	3728	50yr	8770.00	602.82	613.29	610.61	613.92	0.002000	7.27	1568.10	252.06	0.42
main	3728	100yr	10300.00	602.82	614.02	611.09	614.71	0.002002	7.65	1753.63	257.36	0.43

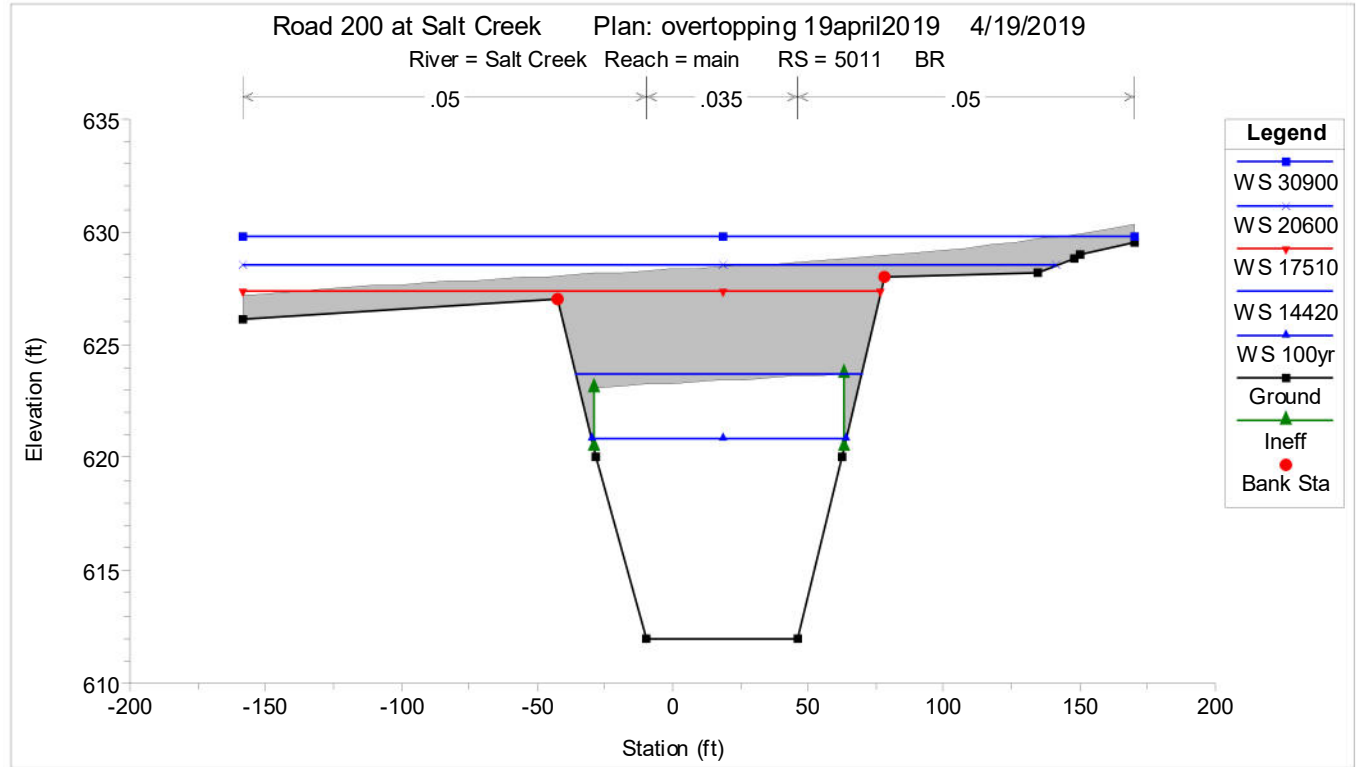
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HEC-RAS Plan: prop 19oct2018 River: Salt Creek Reach: main

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
main	6160	50yr	5420.00	615.21	627.04		627.98	0.002934	7.96	746.70	114.51	0.47
main	6160	100yr	6400.00	615.21	627.94		628.95	0.002941	8.33	851.29	117.90	0.48
main	5842	50yr	5420.00	613.74	626.45		627.06	0.002274	6.46	918.23	133.26	0.38
main	5842	100yr	6400.00	613.74	627.37		628.03	0.002239	6.78	1043.87	140.44	0.38
main	5625	50yr	5420.00	613.28	625.50		626.42	0.003577	8.27	777.55	129.14	0.49
main	5625	100yr	6400.00	613.28	626.48		627.41	0.003408	8.44	908.70	139.68	0.48
main	5449	50yr	5420.00	612.60	623.89		625.55	0.005782	10.55	551.90	81.89	0.63
main	5449	100yr	6400.00	612.60	624.65		626.53	0.006090	11.27	615.46	84.23	0.65
main	5312	50yr	5420.00	611.88	622.52		624.59	0.007901	12.82	524.48	86.74	0.74
main	5312	100yr	6400.00	611.88	623.21		625.52	0.008160	13.67	585.67	91.25	0.76
main	5170	50yr	5420.00	611.83	621.66	620.74	623.41	0.007312	11.39	557.16	100.02	0.71
main	5170	100yr	6400.00	611.83	622.43	621.34	624.31	0.007059	11.90	636.72	106.18	0.71
main	5128	50yr	5420.00	611.88	620.29	620.29	622.92	0.012888	13.56	444.89	88.07	0.92
main	5128	100yr	6400.00	611.88	621.00	621.00	623.83	0.012282	14.15	510.88	96.68	0.91
main	5073	50yr	5420.00	612.00	620.24		622.03	0.009679	10.75	504.51	86.94	0.77
main	5073	100yr	6400.00	612.00	621.04		622.97	0.009238	11.16	578.35	98.45	0.77
main	5039	50yr	5420.00	612.00	619.94		621.73	0.008500	10.72	505.51	81.17	0.76
main	5039	100yr	6400.00	612.00	620.69		622.67	0.008616	11.27	567.98	84.31	0.77
main	5025.5	50yr	5420.00	612.00	619.94		621.57	0.007460	10.24	529.11	83.57	0.72
main	5025.5	100yr	6400.00	612.00	620.70		622.51	0.007560	10.77	594.04	86.61	0.73
main	5012	50yr	5420.00	612.00	620.08	618.12	621.39	0.005618	9.17	591.20	90.37	0.63
main	5012	100yr	6400.00	612.00	620.87	618.76	622.31	0.005523	9.65	663.10	93.55	0.63
main	5011		Bridge									
main	4966	50yr	5420.00	612.00	618.03	618.03	620.59	0.014128	12.83	422.56	82.22	1.00
main	4966	100yr	6400.00	612.00	618.68	618.68	621.48	0.013951	13.44	476.24	84.67	1.00
main	4937	50yr	5420.00	611.00	616.83	616.83	619.16	0.017229	12.26	442.24	94.85	1.00
main	4937	100yr	6400.00	611.00	617.44	617.44	619.97	0.016998	12.76	501.72	99.23	1.00
main	4884	50yr	5420.00	608.29	616.64		618.10	0.006958	9.95	592.90	124.84	0.71
main	4884	100yr	6400.00	608.29	617.22		618.86	0.007016	10.60	666.89	132.54	0.72
main	4755	50yr	5490.00	607.64	615.87		617.22	0.006192	11.74	703.22	150.52	0.77
main	4755	100yr	6455.00	607.64	616.62		617.97	0.005544	11.85	817.00	154.08	0.74
main	4593	50yr	5490.00	606.95	615.73		616.43	0.002435	8.15	941.25	160.30	0.52
main	4593	100yr	6455.00	606.95	616.46		617.23	0.002374	8.55	1060.40	164.10	0.52
main	4313	50yr	5490.00	603.48	614.86		615.76	0.002226	7.86	774.95	105.52	0.45
main	4313	100yr	6455.00	603.48	615.43		616.52	0.002486	8.64	836.40	107.62	0.48
main	4074	50yr	5490.00	604.02	613.56		614.96	0.004837	10.25	695.68	167.90	0.63
main	4074	100yr	6455.00	604.02	614.34		615.71	0.004415	10.40	827.38	172.34	0.61
main	3916	50yr	8770.00	603.55	613.71		614.30	0.001946	6.38	1516.86	235.83	0.40
main	3916	100yr	10300.00	603.55	614.43		615.09	0.001970	6.76	1694.07	258.84	0.40
main	3728	50yr	8770.00	602.82	613.29	610.61	613.92	0.002000	7.27	1568.10	252.06	0.42
main	3728	100yr	10300.00	602.82	614.02	611.09	614.71	0.002002	7.65	1753.63	257.36	0.43

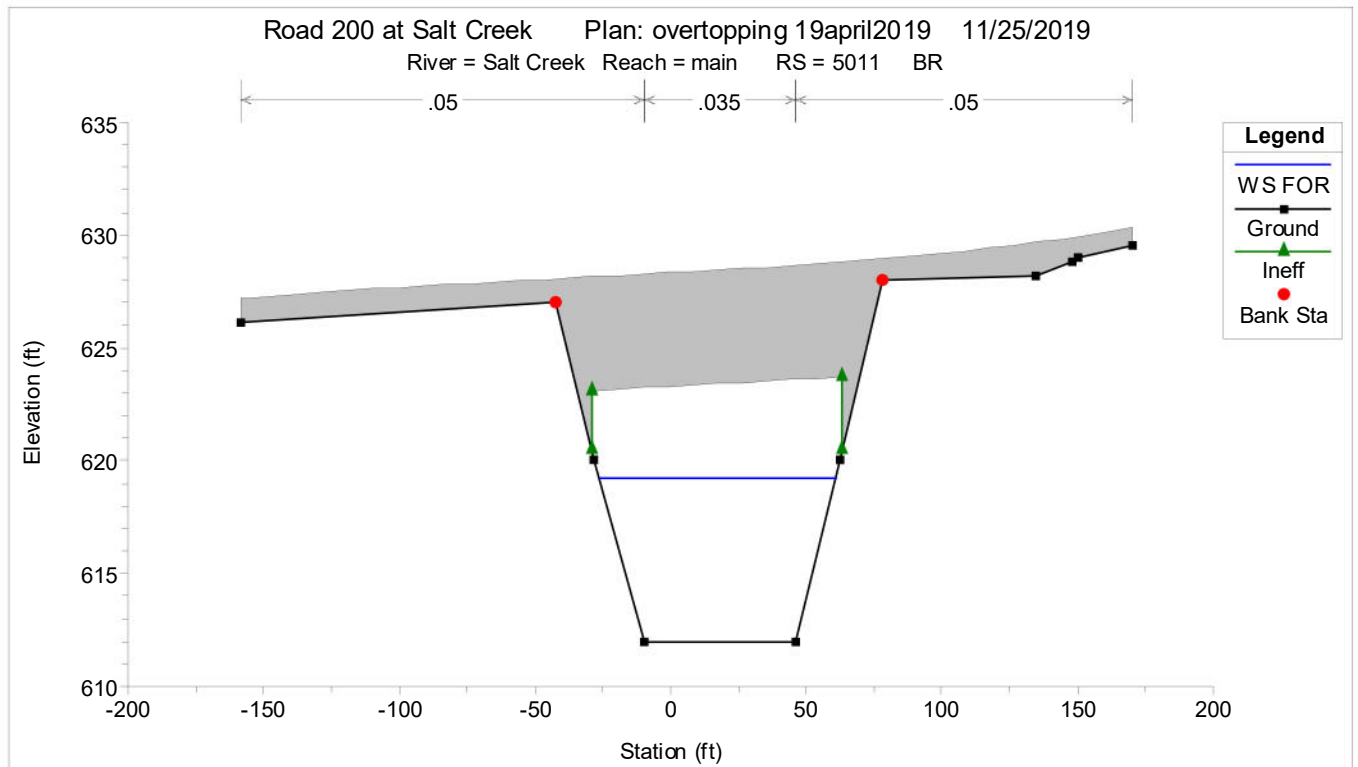
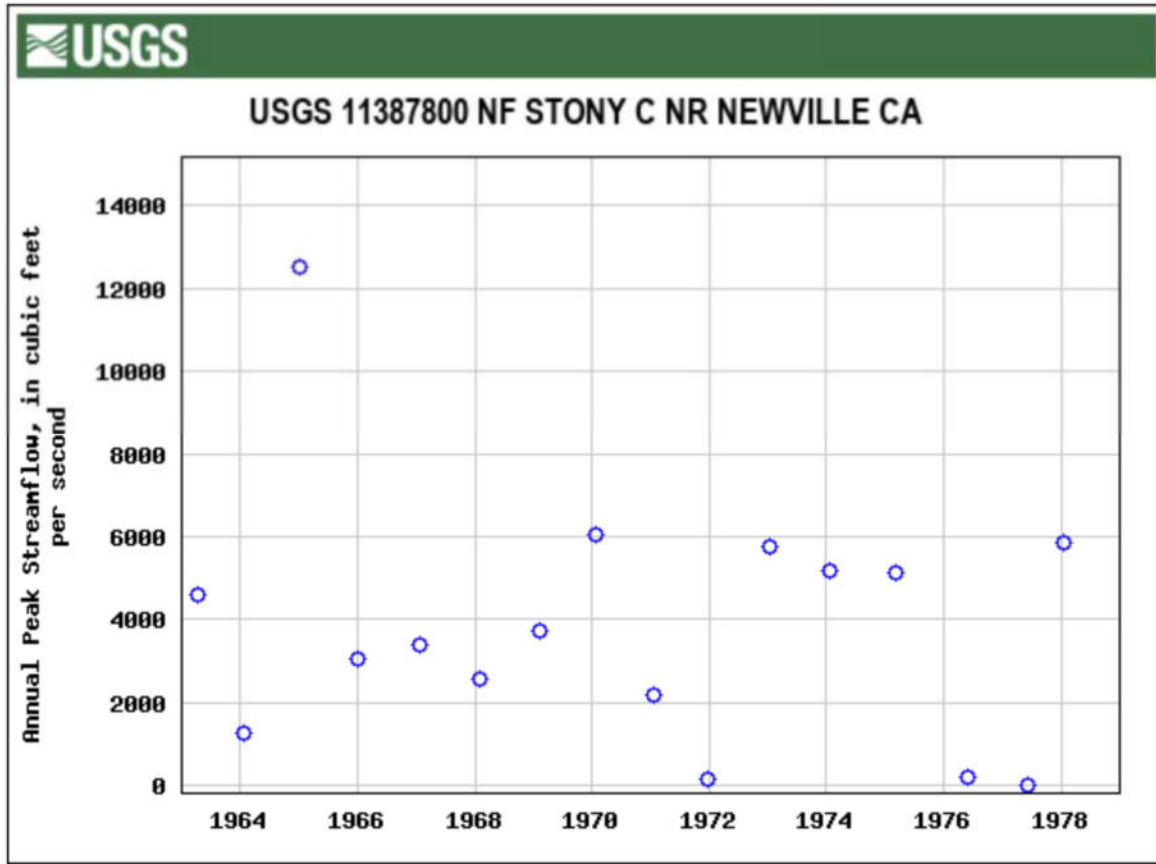
APPENDIX D- OVERTOPPING AND FLOOD OF RECORD

Water first overtops the roadway of Road 200 when the combined discharge in Salt Creek is 17,510 cfs. This corresponds to a discharge of 10,880 cfs at the bridge and is greater than the 200-year event. This results in a WSE of 627.3 ft at the upstream face of the proposed bridge.



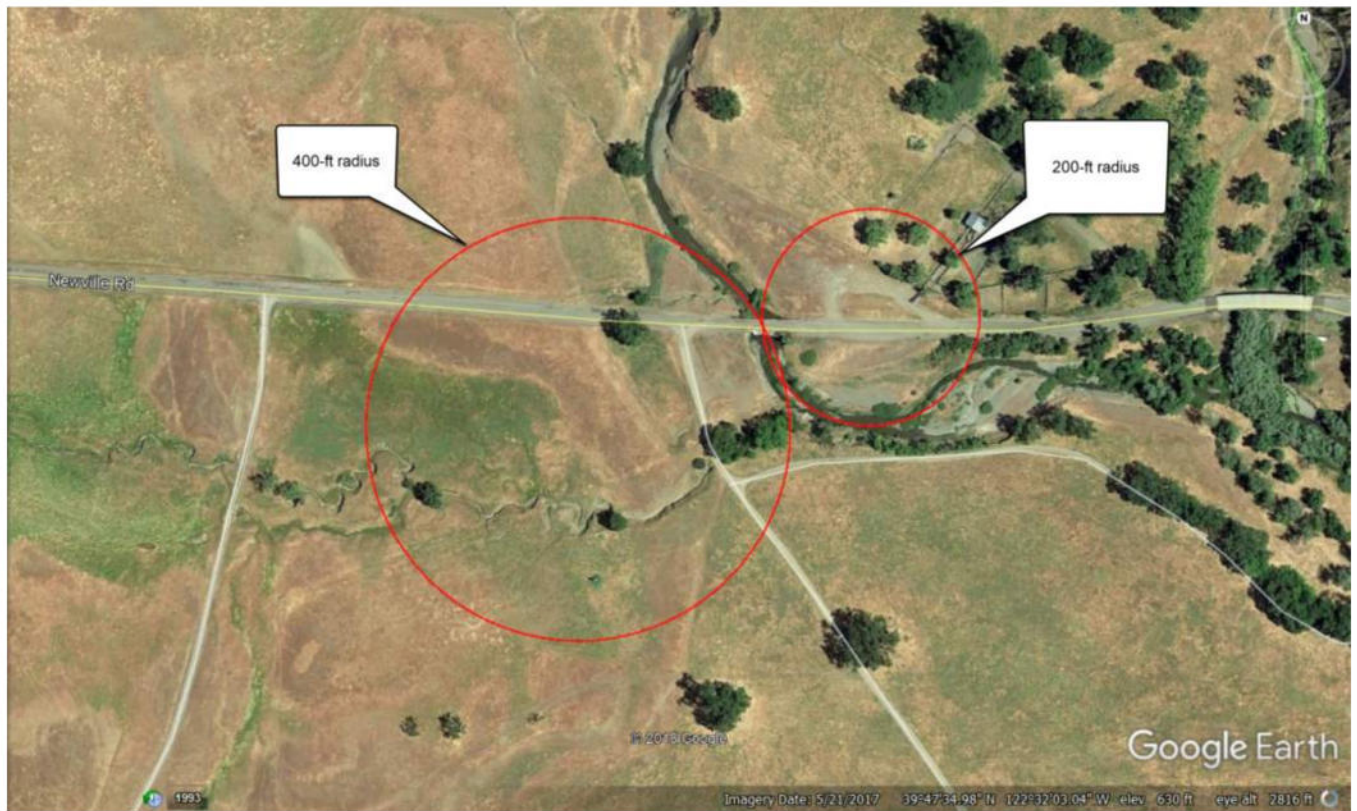
The Flood of Record recorded at the Stony Creek stream gage (USGS gage No. 11387800) occurred on January 5, 1965. This discharge was 12,500 cfs. A basin transfer was completed for the project site which resulted in a discharge of 4,450 cfs at the project site and 7,163 cfs at the confluence with Stony Creek. The HEC-RAS model was re-run with this discharge which resulted in a WSE of 619.2 feet at the project bridge.

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APPENDIX E- ROCK RIPRAP SIZING



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Note: FALSE indicates the value is greater than the largest HEC-23 Class

Caltrans HDM Methodology for RSP Size								
USER DATA REQUIRED IN YELLOW								
Road 200 at Branch Salt Creek								
Stone Size (d30)		5128	5039	5012	5011 BR U	5011 BR D	4966	Units
Constants and Coefficients								
Safety Factor (typically 1.1)	Sf	1.10	1.1	1.1	1.1	1.1	1.1	1.1
Stability Coefficient	Cs	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Velocity distribution coefficient	Cv	1.11	1.15	1.16	1.16	1.22	1.21	
Blanket thickness coefficient	CT	1	1	1	1	1	1	1
Specific Gravity of stone (2.5 min)	Sg	2.65	2.65	2.65	2.65	2.65	2.65	2.65
Acceleration due to gravity	g	32.2	32.2	32.2	32.2	32.2	32.2	32.2 ft/s2
Inputs from HEC-RAS								
Width of WS u/s channel bend	W	53.12	84.31	93.55	91.97	91.97	84.67	
Average Velocity	Vavg	14.15	11.27	9.65	9.66	8.88	13.44	ft/s
Hydraulic Depth	y	7.5	6.74	7.21	7.2	7.83	5.64	feet
Other Inputs								
Slope	fraction	2:1	2:1	2:1	2:1	2:1	2:1	
Slope	decimal	0.5	0.5	0.5	0.5	0.5	0.5	
Bank Angle	theta	26.57	26.57	26.57	26.57	26.57	26.57	
Radius of curvature of bend	Rc	400	400	400	400	200	200	feet
Calculations								
$K_1 = \sqrt{1 - \left[\frac{\sin(\theta - 14^\circ)}{\sin 32^\circ} \right]^{1.6}}$	Rc/W	7.53	4.74	4.28	4.35	2.17	2.36	
	K1	0.87	0.87	0.87	0.87	0.87	0.87	
Characteristic velocity	Vdes	18.17	15.65	13.62	13.60	13.89	20.78	ft/s
	d30	2.57	1.89	1.32	1.31	1.43	4.22	feet
	d50	3.09	2.26	1.59	1.58	1.71	5.06	feet
$d_{30} = y(S_f C_f C_p C_T) \left[\frac{V_{des}}{\sqrt{K_1(S_g - 1)gy}} \right]^{2.5}$	d50	37.06	27.15	19.03	18.93	20.57	60.71	inches
Stone Weight (W)								
	Yw	62.4	62.4	62.4	62.4	62.4	62.4	lb/ft3
$W' = 0.85(\gamma_s d^3)$	Sg	2.65	2.65	2.65	2.65	2.65	2.65	
	Ys	165.36	165.36	165.36	165.36	165.36	165.36	lb/ft3
	d	3.1	2.3	1.6	1.6	1.7	5.1	feet
	W	4140.30	1626.97	560.79	552.17	708.06	18196.62	pounds
RSP Class by Size	Class	X	VIII	VI	VI	VI	FALSE	
	Size	42	30	21	21	21	FALSE	inches
	1.5*d50	63.0	45.0	31.5	31.5	31.5	0.0	inches
	d100	61	48	42	42	42	FALSE	inches
	Thickness	63	48	42	42	42	0	inches
	Method	A	A	A or B	A or B	A or B	FALSE	
RSP Class by Weight	Class	X	VIII	VI	VI	VI	FALSE	
	Weight	6000	2000	750	750	750	FALSE	pounds
	W100	18000	9000	6000	6000	6000	FALSE	pounds
	Method	A	A	A or B	A or B	A or B	FALSE	

Table 873.3A
RSP Class by Median Particle Size⁽³⁾

Nominal RSP Class by Median Particle Size ⁽⁵⁾		d ₁₅		d ₅₀		d ₁₀₀	Placement
Class ^{(1), (2)}	Size (in)	Min	Max	Min	Max	Max	Method
I	6	3.7	5.2	5.7	6.9	12.0	B
II	9	5.5	7.8	8.5	10.5	18.0	B
III	12	7.3	10.5	11.5	14.0	24.0	B
IV	15	9.2	13.0	14.5	17.5	30.0	B
V	18	11.0	15.5	17.0	20.5	36.0	B
VI	21	13.0	18.5	20.0	24.0	42.0	A or B
VII	24	14.5	21.0	23.0	27.5	48.0	A or B
VIII	30	18.5	26.0	28.5	34.5	48.0	A or B
IX	36	22.0	31.5	34.0	41.5	52.8	A
X	42	25.5	36.5	40.0	48.5	60.5	A
XI	46	28.0	39.4	43.7	53.1	66.6	A

NOTES:

- (1) Rock grading and quality requirements per Standard Specifications.
- (2) RSP-fabric Type of geotextile and quality requirements per Section 96 Rock Slope Protection Fabric of the Standard Specifications. For RSP Classes I thru VIII, use Class 8 RSP-fabric which has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric. For RSP Classes IX thru XI, use Class 10 RSP-fabric.
- (3) Intermediate, or B dimension (i.e., width) where A dimension is length, and C dimension is thickness.

Table 873.3B
RSP Class by Median Particle Weight⁽³⁾

Nominal RSP Class by Median Particle Weight		W ₁₅		W ₅₀		W ₁₀₀	Placement
Class ^{(1), (2)}	Weight	Min	Max	Min	Max	Max	Method
I	20 lb	4	11	15	27	140	B
II	60 lb	14	39	50	94	470	B
III	150 lb	32	94	120	220	1,100	B
IV	300 lb	63	180	250	440	2,200	B
V	1/4 ton	110	300	400	700	3,800	B
VI	3/8 ton	180	520	650	1,100	6,000	A or B
VII	1/2 ton	250	750	1000	1,700	9,000	A or B
VIII	1 ton	520	1,450	1,900	3,300	9,000	A or B
IX	2 ton	870	2,500	3,200	5,800	12,000	A
X	3 ton	1,350	4,000	5,200	9,300	18,000	A
XI	4 ton	1,800	5,000	6,800	12,200	24,000	A

NOTES:

- (1) Rock grading and quality requirements per Standard Specifications.
- (2) RSP-fabric Type of geotextile and quality requirements per Section 96 Rock Slope Protection Fabric of the Standard Specifications. For RSP Classes I thru VIII, use Class 8 RSP-fabric which has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric. For RSP Classes IX thru XI, use Class 10 RSP-fabric.
- (3) Values shown are based on Table 873.3A dimensions and an assumed specific gravity of 2.65. Weight will vary based on density of rock available for the project.

APPENDIX F- SCOUR ESTIMATES

For a D_{50} of 0.2 mm the fall velocity is 0.06 fps.

2). HEC-18 Figure 6.8 (Alternative):

D_{50} :	2	mm	USER INPUT REQUIRED
Fall Velocity (ω):	0.018	m/s	← Insert estimated Fall Velocity (ω) from Figure 6.8 below in the orange box to convert units.
	0.06	ft/s	
	18.0	cm/s	

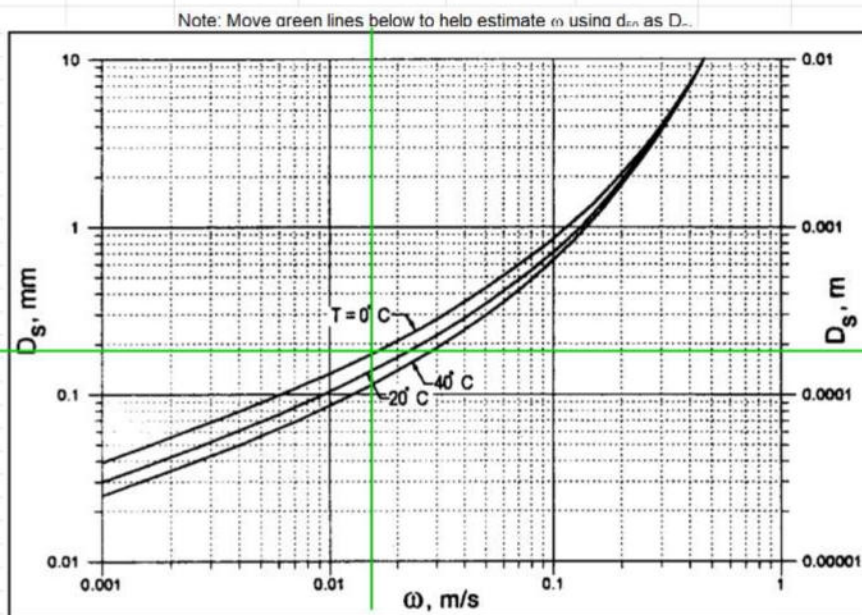


Figure 6.8: Fall velocity of sand-sized particles with specific gravity of 2.65 in metric units.

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The critical velocity is less than the channel velocity therefore the scour condition is Live Bed.



HEC-18 5th Edition - Scour Calculation Spreadsheet (1D)

Critical Velocity Calculation (Clear vs. Live Bed Determination)

Critical Velocity (V_c): *The velocity above which the bed material of size D , D_{50} , etc. and smaller will be transported. Critical velocity is used as an indicator for clear-water or live-bed scour.*

- If the mean velocity (V) of the upstream reach is equal to or less than the critical velocity (V_c) of the median diameter (D_{50}) of the bed material, then contraction and local scour will be clear-water.
- If the mean velocity (V) of the upstream reach is greater than the critical velocity (V_c) of the median diameter (D_{50}) of the bed material, then contraction and local scour will be live-bed.

Parameter	Metric		US	
Median Diameter of Bed Material (D_{50}):	0.20	(mm)	0.2	(mm)
Average Upstream Depth (y):	2.16	(m)	7.09	(ft)
Critical Velocity Parameter (K_u):	6.19	($m^{1/2}/s$)	11.17	($ft^{1/2}/s$)
Average Upstream Velocity (V):	2.941	(m/s)	9.65	(ft/s)

$$V_c = K_u y^{1/6} D^{1/3}$$

*Note: To determine Live Bed Scour vs Clear Scour, D in the equation above is set equal to D_{50}

Critical Velocity (V_c):	0.412	(m/s)	1.4	(ft/s)
--	-------	-------	------------	--------

Upstream $V \leq V_c$: Clear Water Contraction Scour

Upstream $V > V_c$: Live Bed Contraction Scour

Proceed to Live Bed Contraction Scour Tab

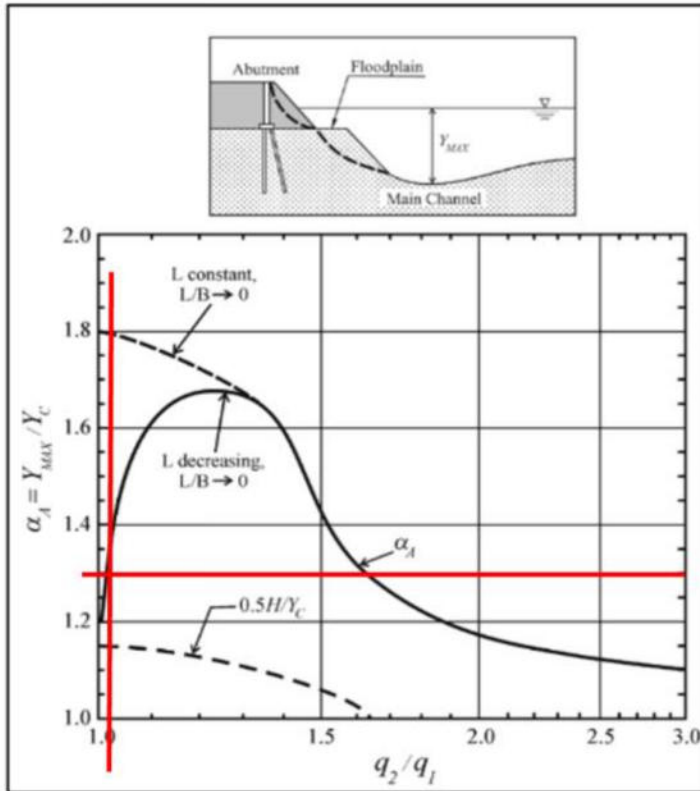
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The contraction scour is 3- feet.

	Live Bed Contraction Scour					
<u>Live Bed Contraction Scour</u> : Scour at a contraction when the bed material in the channel upstream of the bridge is moving at the flow causing bridge scour.						
Modified Laursen's Equation (1):		$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1} \right)^{6/7} \left(\frac{W_1}{W_2} \right)^{k_1}$		Average Contraction Scour Depth:		
		$y_s = y_2 - y_0$				
Parameter	Description	Metric Units		US Units		Notes
y_0	Existing Depth in the Contracted Section Before Scour	2.19	(m)	7.20	(ft)	Flow area of bridge / W_2
y_1	Average Depth in the Upstream Channel	2.16	(m)	7.09	(ft)	Data from Chosen Upstream XS
y_2	Average Depth in the Contraction Section	2.19	(m)	7.17	(ft)	Modified Laursen's Equation
Q_1	Flow in the Upstream Channel Transporting Sediment	181.23	(m ³ /s)	6400.00	(cfs)	Flow in the main channel upstream of the bridge, not including overbank flow.
Q_2	Flow in the Contracted Channel	181.23	(m ³ /s)	6400.00	(cfs)	Flow at the bridge section (through the bridge opening)
W_1	Bottom Width of the Upstream Main Channel that is Transporting Bed Material	28.51	(m)	93.55	(ft)	Can be estimated by Upstream Channel Top Width. Data from Chosen Upstream XS
W_2	Bottom width of the Contracted Section Minus Pier and Debris Width	28.04	(m)	92.00	(ft)	Effective Bridge Width Calculated Given Bridge, Pier, and Debris Width
S_1	Slope of EGL of Upstream Channel	0.01	(m/m)	0.01	(ft/ft)	Data from Chosen Upstream XS
V^*	Shear Velocity in the Upstream Main Channel	0.34	(m/s)	1.12	(ft/s)	Calculated from data from Chosen Upstream XS(s). [$V^* = (gy_1 S_1)^{0.5}$]
ω	Fall Velocity of Bed Material based on D50	0.13	(m/s)	0.44	(ft/s)	See Fall Velocity Tab
V^*/ω	Ratio of Shear Velocity to Fall Velocity	2.537	-	2.537	-	Determines Mode of Bed Transport and k_1
k_1	Modified Laursen's Equation Exponent	0.69	-	0.69	-	See Table 2 to the right.
Average Live Bed Contraction Scour Depth (y_s)				0.0	(ft)	
				0.0	(m)	

Abutment Scour Condition A:

The abutment scour amplification factor is 1.3.



The abutment scour is 2 feet.

2a) Scour occurring when the abutment is in or close to the main channel (Live Bed)

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

$$y_{\max} = \alpha_A y_c$$

$$y_s = y_{\max} - y_0$$

Parameter	Description	Metric Units		US Units		Notes
y_1	Upstream flow depth	2.16	(m)	7.09	(ft)	Flow area of bridge / W_2
y_0	Flow depth prior to scour	2.19	(m)	7.20	(ft)	Data from chosen upstream XS
α_a	Amplification factor for live-bed conditions	1.30	-	1.30	-	For spill through abutments: Use Figure 8.9 For wingwall abutments: Use Figure 8.10
W_1	Width of the upstream channel	28.51	(m)	93.55	(ft)	Width of Flow upstream of the bridge section
Q_1	Flow in the upstream channel	181.23	(m ³ /s)	6400.0	(ft ³ /s)	Flow upstream of the bridge section
q_{2c}	Unit discharge in the constricted opening accounting for non-uniform flow distribution	6.46	(m ² /s)	69.57	(ft ² /s)	Estimated as the total discharge in the bridge opening divided by the width of the bridge opening: Q_2 / W_2
q_1	Upstream unit discharge	6.36	(m ² /s)	68.41	(ft ² /s)	Q_1 / W_1
q_2/q_1	Ratio of unit discharge	1.02	(m)	1.02	(ft)	Value used in Figure 8.9 and Figure 8.10 to determine amplification factor
y_c	Flow depth including live-bed contraction scour	2.19	(m)	7.19	(ft)	Equation Above
$y_{\max}$	Max flow depth resulting from abutment scour	2.85	(m)	9.35	(ft)	Equation Above

Live Bed Abutment Scour Depth (y_s)	2.1	(ft)
	0.7	(m)

Abutment Scour Condition C:

Pier Scour is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties, and the geometry of the pier and footing.

1). HEC-18 5th Edition Pier Scour Equation (based on the CSU Equation)

HEC-18 Equation:

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 \left(\frac{a}{y_1} \right)^{0.65} Fr_1^{0.43}$$

In terms of y_s/a :

$$\frac{y_s}{a} = 2.0 K_1 K_2 K_3 \left(\frac{y_1}{a} \right)^{0.35} Fr_1^{0.43}$$

Parameter	Description	Metric Units		US Units		Notes
y_1	Flow depth directly upstream of the pier	3.38	(m)	11.10	(ft)	Obtained from (BR U) Flow Distribution Table; Bridge Information Macro
θ	Angle of attack of the flow (skew)	0	(deg)	0	(deg)	Bridge Skew
K_1	Correction factor for Pier nose shape	1.0	-	1.0	-	Use Figure 7.3 and Table 7.1 If $\theta > 5$ degrees, $K_1 = 1.0$
K_2	Correction factor for angle of attack of flow	1.0	-	1.0	-	$K_2 = [(\cos(\theta) + \sin(\theta) * L/A)^{0.65}]$ (where $L/A_{max} = 12$)
K_3	Correction factor for bed condition	1.1	-	1.1	-	Use Table 7.3
a	Pier Width (including bottom width)	0.6	(m)	2.00	(ft)	Bottom Pier Width; no floating debris included
L	Length of Pier	0.0	(m)	0.0	(ft)	See Figure 7.3 for Guidance
V_1	Velocity of flow directly upstream of the pier	2.34	(m/s)	7.69	(ft/s)	Obtained from (BR U) Flow Distribution Table; Bridge Information Macro
Fr_1	Froude Number directly upstream of the pier	0.41	-	0.41	-	$Fr_1 = [V_1 / (gy_1)]^{1/2}$

HEC-18 Equation Maximum Pier Scour Depth (y_s)	5.4	(ft)
	1.7	(m)

***Note for Round Nose Piers:** Maximum Scour Depth (y_s) is typically $\leq (2.4 * a)$ for $Fr \leq 0.8 \rightarrow 2.4 * a = 4.80$
Maximum Scour Depth (y_s) is typically $\leq (3.0 * a)$ for $Fr > 0.8 \rightarrow 3.0 * a = 6.00$

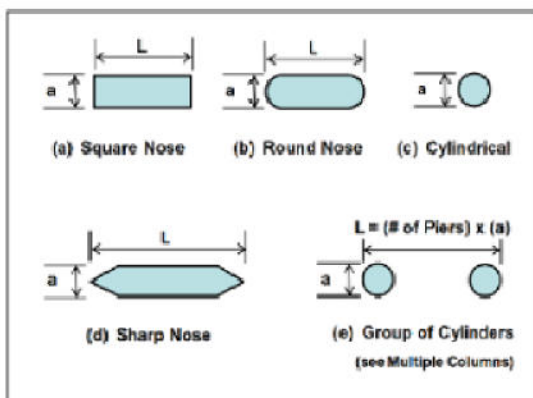


Figure 7.3. Common pier shapes.

Table 7.1. Correction Factor, K_1 , for Pier Nose Shape.	
Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 7.3. Increase in Equilibrium Pier Scour Depths, K_3 , for Bed Condition.		
Bed Condition	Dune Height ft	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$10 > H \geq 2$	1.1
Medium Dunes	$30 > H \geq 10$	1.2 to 1.1
Large Dunes	$H \geq 30$	1.3

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APPENDIX G- LOCATION HYDRAULIC STUDY FORM

LOCATION HYDRAULIC STUDY FORM

Dist. 3 Co. Glenn Rte. County Road 200 Project ID: Bridge #11C0132
Federal-Aid Project Number: BRLO 5911(058)

Floodplain Description:

Salt Creek flows south easterly through the project site through the northern part of Glenn County (County). It drains an approximate 20.6 square miles at the project site. The area surrounding the project site is rural. The channel width (top of bank to top of bank) varies from approximately 100 ft upstream to 200 feet downstream of the project bridge. The channel is sparsely vegetated and the overbanks are heavily vegetated. The area of the proposed bridge is within an existing FEMA Floodplain Zone A, an area subject to flooding by the 100-year event, but for which base flood elevations have not been determined.

1. Description of Proposal *(include any physical barriers i.e. concrete barriers, sound walls, etc. and design elements to minimize floodplain impacts)*

The County proposes to replace the existing structure (11C0132) on County Road 200 over a Branch of Salt Creek and construct the minimum approach work to accommodate the project. This will consist of approximately 400 feet of roadway reconstruction at both the east and west approaches to the bridge. The horizontal and vertical alignments will basically remain unchanged. The new structure is anticipated to be a 1-span, cast-in-place, post-tensioned, box girder bridge.

2. ADT: Current 150 Projected 180 (2030)

3. Hydraulic Data: Base Flood Q100= 6,400 CFS
WSE100= 620.9 ft
The flood of record, if greater than Q100:
Q= n/a CFS WSE= n/a
Overtopping flood Q= 17,510 CFS WSE= 627.3 ft

Are NFIP maps and studies available? NO YES X

The project is within a FEMA designated Zone A floodplain as shown on Figure 1. A Zone A floodplain is an area subject to flooding by the 100-year event, but where no detailed studies have been completed and thus no water surface elevations have been determined by FEMA.

National Flood Hazard Layer FIRMette

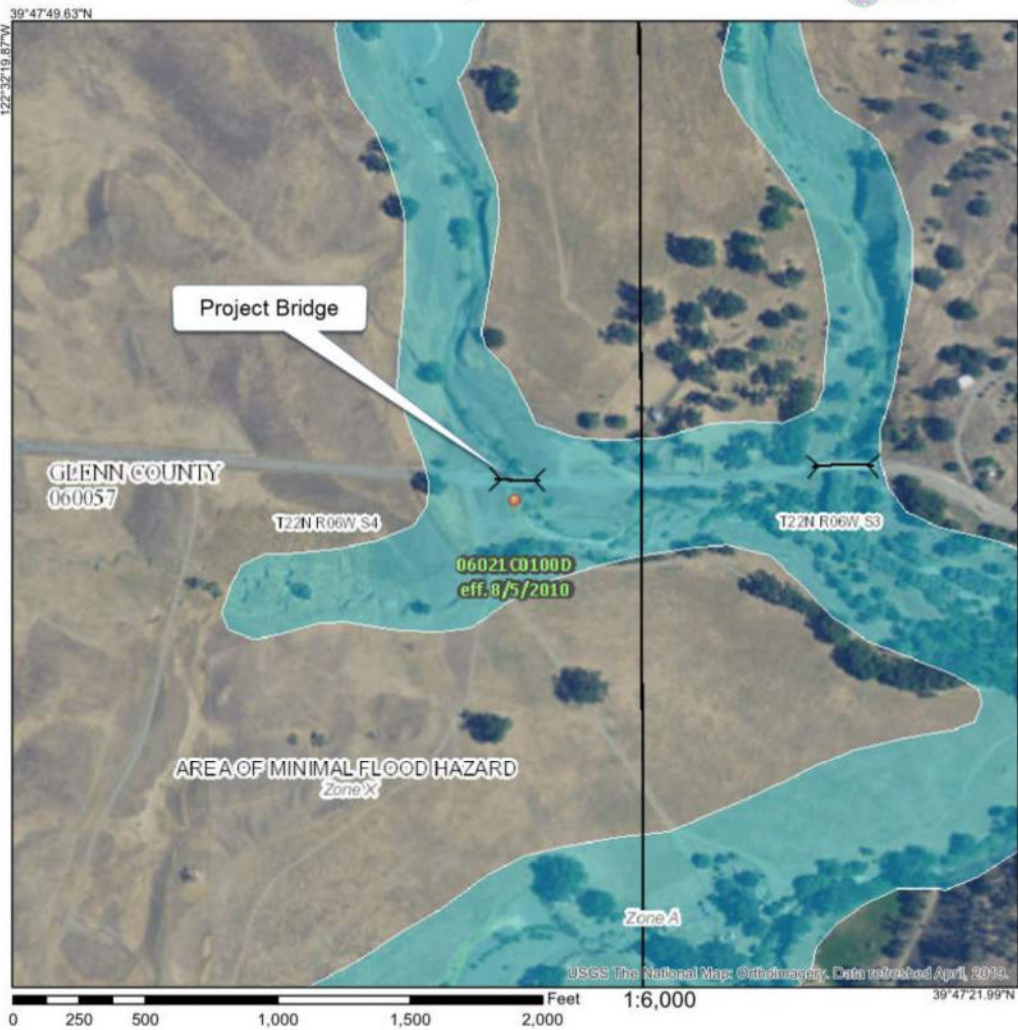
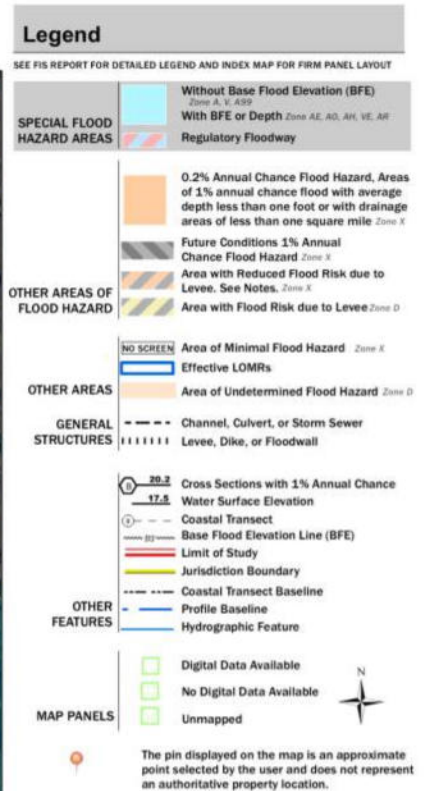


Figure 1. FEMA FIRMette of Map Number 06021C0100D dated August 5, 2010

4. Is the highway location alternative within a regulatory floodway?

NO X YES

5. Attach map with flood limits outlined showing all buildings or other improvements within the base floodplain.

As shown in Figure 2, Figure 3, and Figure 4 the water surface elevation is lowered 6 feet upstream of the proposed bridge, slightly higher just downstream of the bridge, and unchanged farther downstream with the proposed bridge in place. The downstream increase in the water surface elevation is due to the altered drawdown curve through the proposed bridge. The existing bridge greatly constricted the channel, resulting in a higher water surface elevation upstream and lower water surface elevations downstream. With the proposed bridge, the hydraulics are improved and this improvement results in approximately 1 ft higher water surface elevations just downstream of the bridge (approximately 30 feet) which tapers off to no change approximately 200 feet downstream. As shown in Figure 4, this slight increase in the water surface elevation will result in no adverse impacts to any insurable structures or floodplain values.

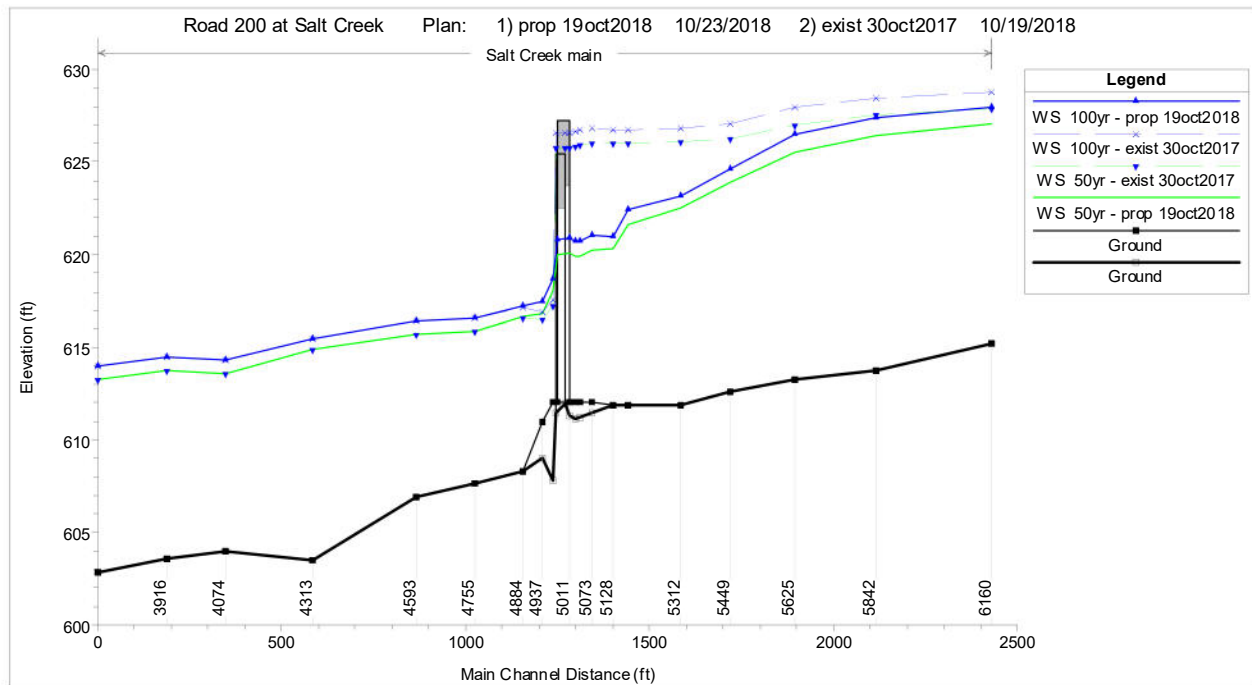


Figure 2. WSE comparison between existing (dashed) and proposed (solid) conditions for the 50-yr and 100-yr discharge.

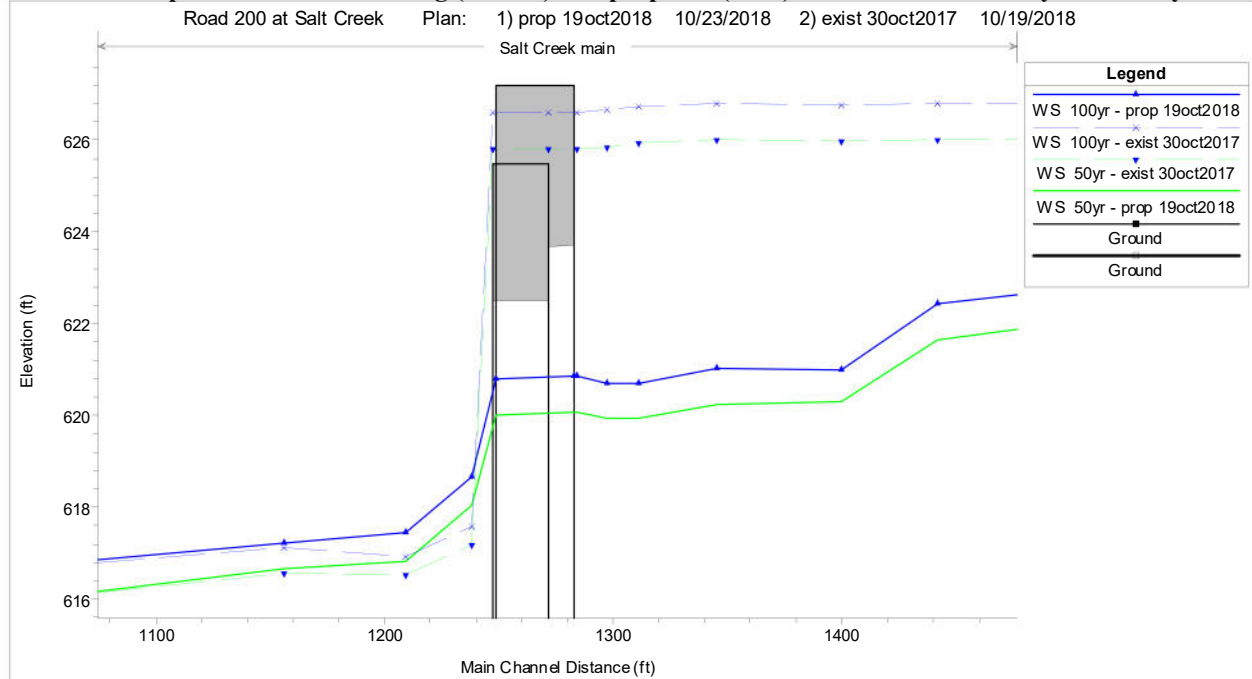


Figure 3. Zoomed in Figure 2.



Figure 4. Water surface extents comparison between existing (blue) and proposed (green) conditions for the 100-yr discharge.

Potential Q100 backwater damages:

A. Residences? NO X YES

There are no residences adjacent to the project that could be impacted by the project.

B. Other Bldgs? NO X YES

As a result of the project, the water surface elevation will be lowered 6 feet upstream and only raised 1 ft downstream tapering to no increase within 200-feet downstream from the bridge. These changes will have no adverse impact to the stable shade structure adjacent to the project.

C. Crops? NO X YES

There are no crops surrounding the project.

D. Natural and beneficial Floodplain values? NO X YES

"Natural and beneficial flood-plain values" shall include but are not limited to fish, wildlife, plants, open space, natural beauty, scientific study, outdoor recreation, agriculture, aquaculture, forestry, natural moderation of floods, water quality maintenance, and groundwater recharge.

The water surface elevation is lowered 6 feet upstream and only increased 1 ft downstream due to the improved hydraulics of the proposed bridge. These changes will not affect the natural and beneficial floodplain values.

6. Type of Traffic:

A. Emergency supply or evacuation route?	NO <u> X </u>	YES <u> </u>
B. Emergency vehicle access?	NO <u> </u>	YES <u> X </u>
C. Practicable detour available?	NO <u> </u>	YES <u> X </u>
D. School bus or mail route?	NO <u> </u>	YES <u> X </u>

7. Estimated duration of traffic interruption for 100-year event hours: 0

8. Estimated value of Q100 flood damages (if any) – moderate risk level.

A.	Roadway	\$ <u> 0 </u>
B	Property	\$ <u> 0 </u>
	Total	\$ <u> 0 </u>

9. Assessment of Level of Risk Low X
 Moderate
 High

For High Risk projects, during design phase, additional Design Study Risk Analysis may be necessary to determine design alternative.

LOCATION HYDRAULIC STUDY FORM cont.

Dist. 3 Co. Glenn Rte. County Road 200 P.M. Federal-Aid Project Number: BRLO 5911(058)
Project ID _____ Bridge No. 11C0132

PREPARED BY:

Signature:

I certify that I have conducted a Location Hydraulic Study consistent with 23 CFR 650 and that the information summarized in items numbers 3, 4, 5, 7, and 9 of this form is accurate.

District Hydraulic Engineer (capital and 'on' system projects) Date _____

Local Agency/Consulting Hydraulic Engineer (local assistance projects) Date _____

Is there any longitudinal encroachment, significant encroachment, or any support of incompatible Floodplain development? NO X YES _____

If yes, provide evaluation and discussion of practicability of alternatives in accordance with 23 CFR 650.113

Information developed to comply with the Federal requirement for the Location Hydraulic Study shall be retained in the project files.

I certify that item numbers 1, 2, 6 and 8 of this Location Hydraulic Study Form are accurate and will ensure that Final PS&E reflects the information and recommendations of said report:

District Project Engineer (capital and 'on' system projects) Date _____

Local Agency Project Engineer (local assistance projects) Date _____

CONCURRED BY:

I have reviewed the quality and adequacy of the floodplain submittal consistent with the attached checklist, and concur that the submittal is adequate to meet the mandates of 23 CFR 650.

District Project Manager (capital and 'on' system projects) Date _____

Local Agency Project Manager (Local Assistance projects) Date _____

District Local Assistance Engineer (or District Hydraulic Branch for very complex projects or when required expertise is unavailable. Note: District Hydraulic Branch review of local assistance projects shall be based on reasonableness and concurrence with the information provided). Date _____

I concur that the natural and beneficial floodplain values are consistent with the results of other studies prepared pursuant to 23 CFR 771, and that the NEPA document or determination includes environmental mitigation consistent with the Floodplain analysis.

District Senior Environmental Planner (or Designee) Date _____

Note: If a significant floodplain encroachment is identified as a result of floodplains studies, FHWA will need to approve the encroachment and concur in the Only Practicable Alternative Finding.

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APPENDIX H- SUMMARY FLOODPLAIN ENCROACHMENT REPORT

SUMMARY FLOODPLAIN ENCROACHMENT REPORT

Dist. 3 Co. Glenn Rte. County Road 200 K.P.

Federal-Aid Project Number: BRLO 5911(058)

Project No.: Bridge No. 11C0132

Limits:

The County proposes to replace the existing structure (11C0132) on County Road 200 over a Branch of Salt Creek and construct the minimum approach work to accommodate the project. This will consist of approximately 400 feet of roadway reconstruction at both the east and west approaches to the bridge. The horizontal and vertical alignments will basically remain unchanged. The new structure is anticipated to be a 1-span, cast-in-place, post-tensioned, box girder bridge.

Floodplain Description:

Salt Creek flows south easterly through the project site through the northern part of Glenn County (County). It drains an approximate 20.6 square miles at the project site. The area surrounding the project site is rural. The channel width (top of bank to top of bank) varies from approximately 100 ft upstream to 200 feet downstream of the project bridge. The channel is sparsely vegetated and the overbanks are heavily vegetated. The area of the proposed bridge is within an existing FEMA Floodplain Zone A, an area subject to flooding by the 100-year event, but for which base flood elevations have not been determined.

- | | No | Yes |
|--|----------|---------|
| 1. Is the proposed action a longitudinal encroachment of the base floodplain?
<i>The proposed bridge is not a longitudinal encroachment.</i> | <u>X</u> | <u></u> |
| 2. Are the risks associated with the implementation of the proposed action significant?
<i>The level of risk to the floodplain of the project site is low because the action is to replace the existing bridge with a bridge that is approximately 45 feet longer and with 2 fewer piers, thus improving the hydraulics through the proposed structure.</i> | <u>X</u> | <u></u> |
| 3. Will the proposed action support probable incompatible floodplain development?
<i>The proposed bridge replacement will be 45 feet longer than the existing bridge and will not support incompatible floodplain development.</i> | <u>X</u> | <u></u> |
| 4. Are there any significant impacts on natural and beneficial floodplain values?
<i>The proposed construction will have only minor impact to the existing riparian habitat in the creek at the bridge site</i> | <u>X</u> | <u></u> |

5. Routine construction procedures are required to minimize impacts on the floodplain. Are there any special mitigation measures necessary to minimize impacts or restore and preserve natural and beneficial floodplain values? If yes, explain. X
Best management practices for erosion control measures should be used for proposed construction to minimize temporary impacts to the floodplain during construction.
6. Does the proposed action constitute a significant floodplain encroachment as defined in 23 CFR, Section 650.105(q). X
The proposed action is not a significant floodplain encroachment.
7. Are Location Hydraulic Studies that document the above answers on file? If not explain. X

PREPARED BY:

_____ *Date* _____
District Project Engineer (*capital and 'on' system projects*)

_____ *Date* _____
Local Agency Project Engineer (*local assistance projects*)

CONCURRED BY:

_____ *Date* _____
District Project Manager (*capital and 'on' system projects*)

_____ *Date* _____
District Local Assistance Engineer (*Local Assistance projects*)

I concur that impacts to natural and beneficial floodplain values are consistent with the results of other studies prepared pursuant to 23 CFR 771, and that the NEPA document or determination includes environmental mitigation consistent with the Floodplain analysis.

_____ *Date* _____
District Senior Environmental Planner (*or Designee*)

Note: If a significant floodplain encroachment is identified as a result of floodplains studies, FHWA will need to approve the encroachment and concur in the Only Practicable Alternative Finding.