APPENDIX D

DESIGN HYDRAULIC STUDY

COUNTY ROAD 303 AT SOUTH FORK WILLOW CREEK

Bridge Number 11C0163

GLENN COUNTY, CALIFORNIA

AVILA & ASSOCIATES

DRAFT

Design Hydraulic Study

COUNTY ROAD 303 BRIDGE AT SOUTH FORK WILLOW CREEK

GLENN COUNTY, CALIFORNIA

Bridge Number 11C0163

DECEMBER 4, 2019

PREPARED FOR:

GLENN COUNTY PUBLIC WORKS AGENCY

Prepared by:

AVILA AND ASSOCIATES
CONSULTING ENGINEERS, INC.

Catherine M.C. Avila, P.E

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EXECUTIVE SUMMARY

The County Road 303 bridge at South Fork Willow Creek in Glenn County, California is proposed for replacement by Glenn County Public Works Agency. The proposed bridge will be a 105-foot long 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 attached General Plan (Appendix A). The superstructure will be supported by abutments with CIDH piles.

The south fork of Willow Creek flows northerly through central part of Glenn County and is approximately 14 miles west of Willows, CA. The discharges used for the bridge hydraulic analysis are shown below:

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)	4,000	4,500	2,390
Water surface elevation (ft)	563.0	563.6	560.4

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 16-1 [1].

[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 303 Bridge over South Fork Willow Creek in Glenn County. The location of this project is shown in Figure 1. 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 303 at South Fork Willow Creek as shown in Figure 1. The existing bridge was constructed in 1920; it is a Single span haunched reinforced concrete T-Beam (4) on reinforced concrete abutments on unknown foundations. It has a sufficiency rating of 44.1 as of 2012 and is structurally deficient (Glenn County, 2012).

Figure 1. Bridge location map

The datum elevation used for this study is NAVD-88[2]. The proposed bridge will be a 110-foot long 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 attached General Plan (Appendix A). The superstructure will be supported by abutments with CIDH piles.

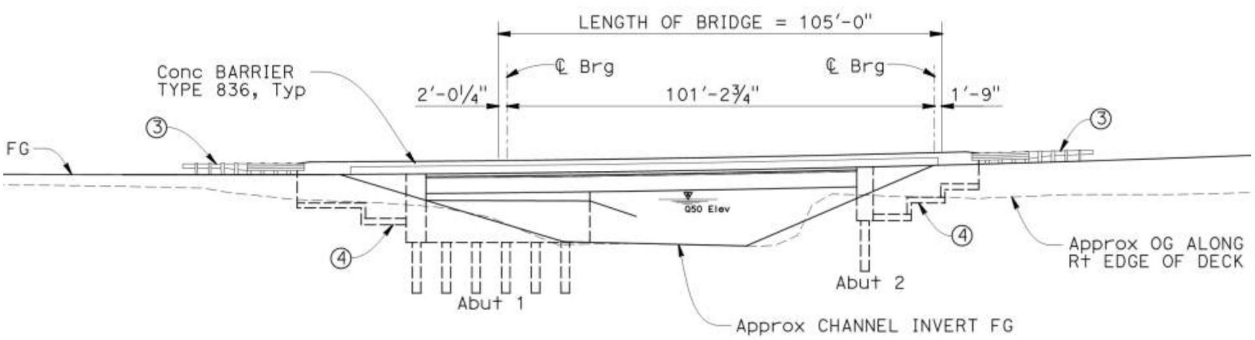

Figure 2. Proposed bridge profile view

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.

[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.

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Table 2. Bridge information from adjacent bridges on the reach

	Road 303 at SF Willow Creek
Bridge Number	11C0163
Bridge Length (ft)	31
Span Lengths (ft)	1 @ 29
Bridge Type	Single span haunched reinforced concrete T-Beam (4) on reinforced concrete abutments on unknown foundations
Debris Challenges	1978[3]
Cross Sections Available for	N/A
NBIS Item 113 (scour) code	U
ELI Flag 361 Condition State	N/A
Pier Type	N/A
Year Built	1920
Year Widened	N/A
Scour Challenges	1995[4], 2004[5], 2006[6], 2009[7], 2012[8]

[3] There is a large limb of an oak tree in the stream about 20 feet downstream from the bridge.
[4] Abutment 1 and 2 have undermining and scouring at the right wingwall along the footing.
[5] There is a scour forming at Abutment 2, right side.
[6] There was a scour hole 1.5 m diameter, 1 m deep of standing water at the right of Abutment 2. There is a drainage pipe at the roadway, behind the right side of Abutment 2 which is exacerbating and possibly causing the scour hole at the right side of Abutment 2.
[7] Same as 2006. Additionally, there is a void behind the right side of Abutment 2 which is 2 ft x 1 ft x 3.5 ft deep. This is the first time this condition has been noted.
[8] Same as 2009.

BASIN AND DISCHARGE

South Fork (SF) Willow Creek at the Road 303 bridge drains an approximate 24.4 square miles as shown on the Hydrology Map. The mean annual precipitation of the project watershed is approximately 21.8 inches/year.[9].

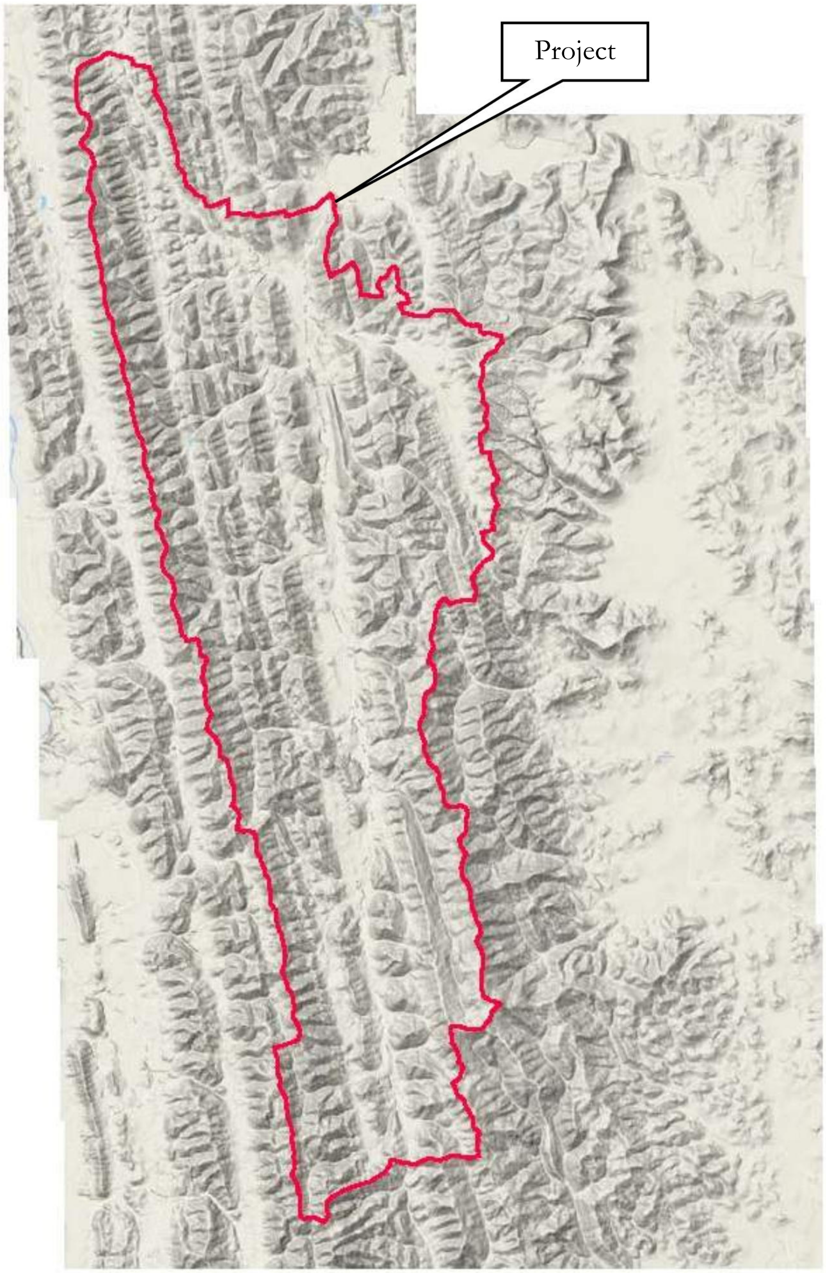

Figure 3. Basin contributing to the bridge discharge (USGS streamstats).

[9] **www.streamstatsags.cr.usgs.gov** (U.S.G.S.)

Discharge at the bridge reach was calculated using three methods:

- A regression analysis [10]
- A basin transfer of results of a statistical analysis of gage data [11]
- A HEC-HMS analysis

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

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

Method	**Discharge** (cfs)	
	50-yr	100-yr
Regression	3,592	4,280
Basin Transfer Stone Corral Creek Gage (11390672)	3,963	4,450
HEC-HMS	2,135	2,590

The results from the basin transfer analysis are conservative when compared to the regression and HEC-HMS analyses and were used for the hydraulic analysis as shown in Table 4.

Table 4. Discharges used for analysis (cfs)

	Discharge (cfs)	
	Design	Base
Frequency (years)	50	100
Discharge (cubic feet per second)	4,000	4,500

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

[10] *Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006* (USGS SIR 2012-5113)
[11] US. Department of the Interior, Geological Survey. *Guidelines for Determining Flood Flow Frequency, Bulletin #17B of the Hydrology Subcommittee*, Revised September 1981.

HEC-RAS ANALYSIS

Existing Condition

The Manning "n" values of 0.035 for the channel bottom and 0.045 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 4.

Figure 4. Looking towards the bridge. The channel bottom is sparsely vegetated and the banks and overbanks are vegetated.

The existing bridge was input into the model as a single span bridge with a minimum soffit elevation of 562 feet as illustrated in Figure 5 and a downstream lateral structure.

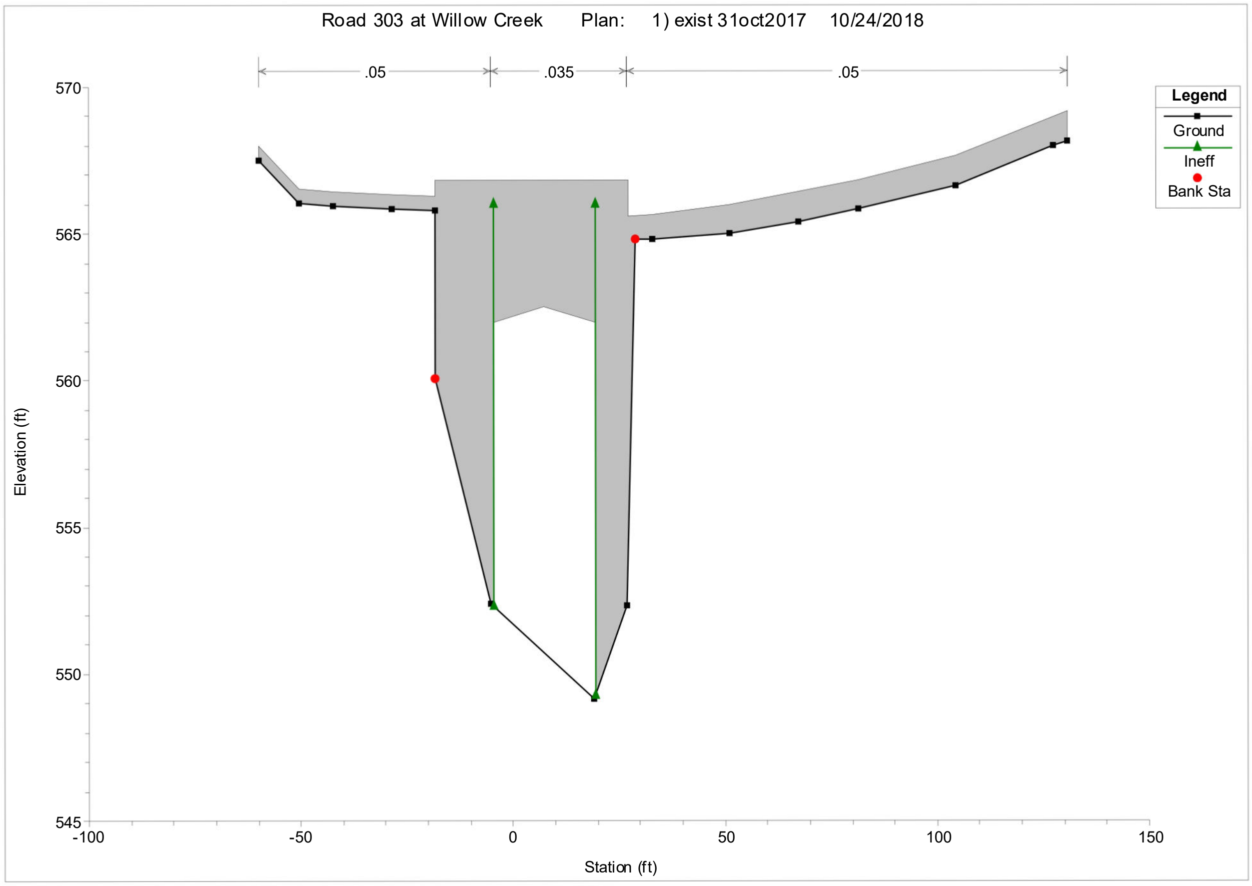

Figure 5. HEC-RAS upstream cross section of the existing bridge.

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 6, all of the WSE profiles converged approximately 900 feet downstream from the existing bridge. Normal depth was used as the downstream boundary condition for establishing the starting WSE.

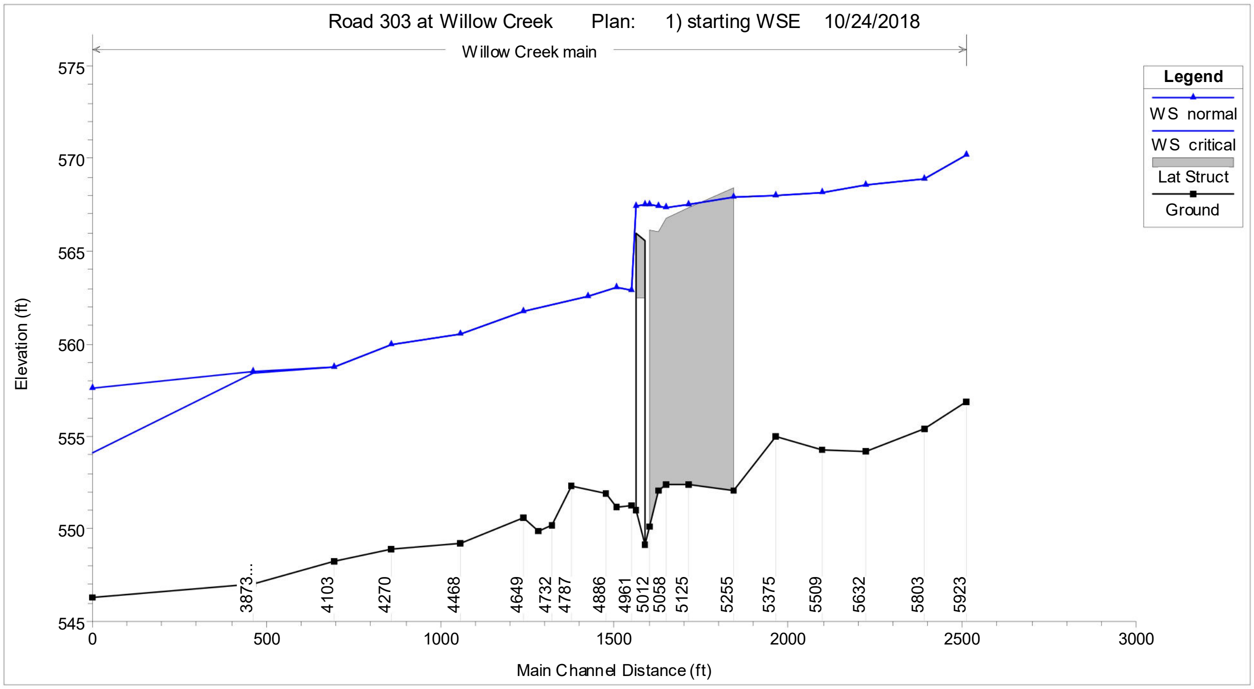

Figure 6. WSE profile comparison using various downstream boundary conditions

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 will be approximately 130 feet downstream of the existing bridge and was modeled as a single-span bridge with minimum soffit elevation of 565.4 feet as shown in Figure 7.

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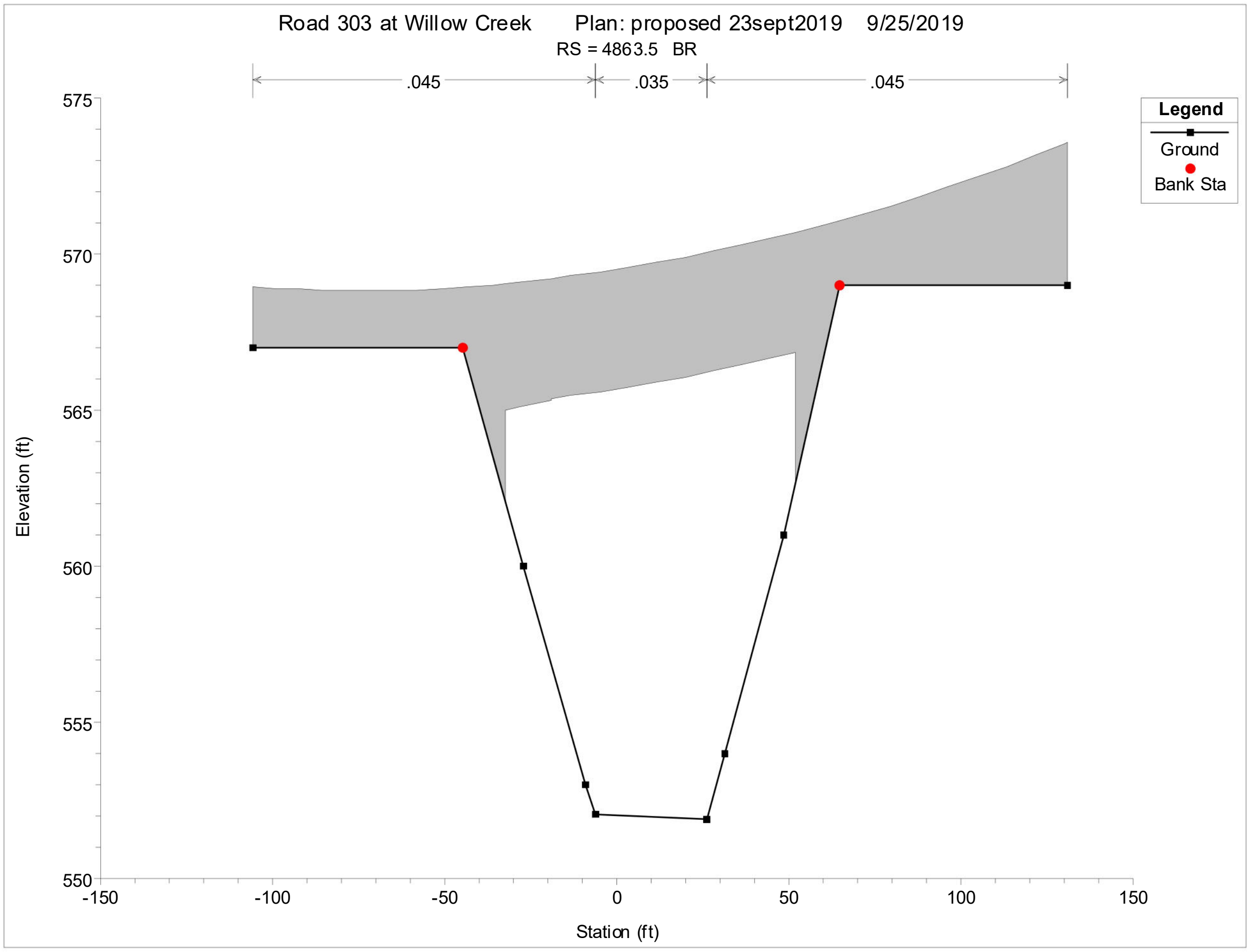


Figure 7. HEC-RAS upstream cross section of the proposed bridge.

Figure 8, Figure 9, 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 and downstream of the proposed and existing bridge locations, and is slightly higher between the existing and proposed bridges.

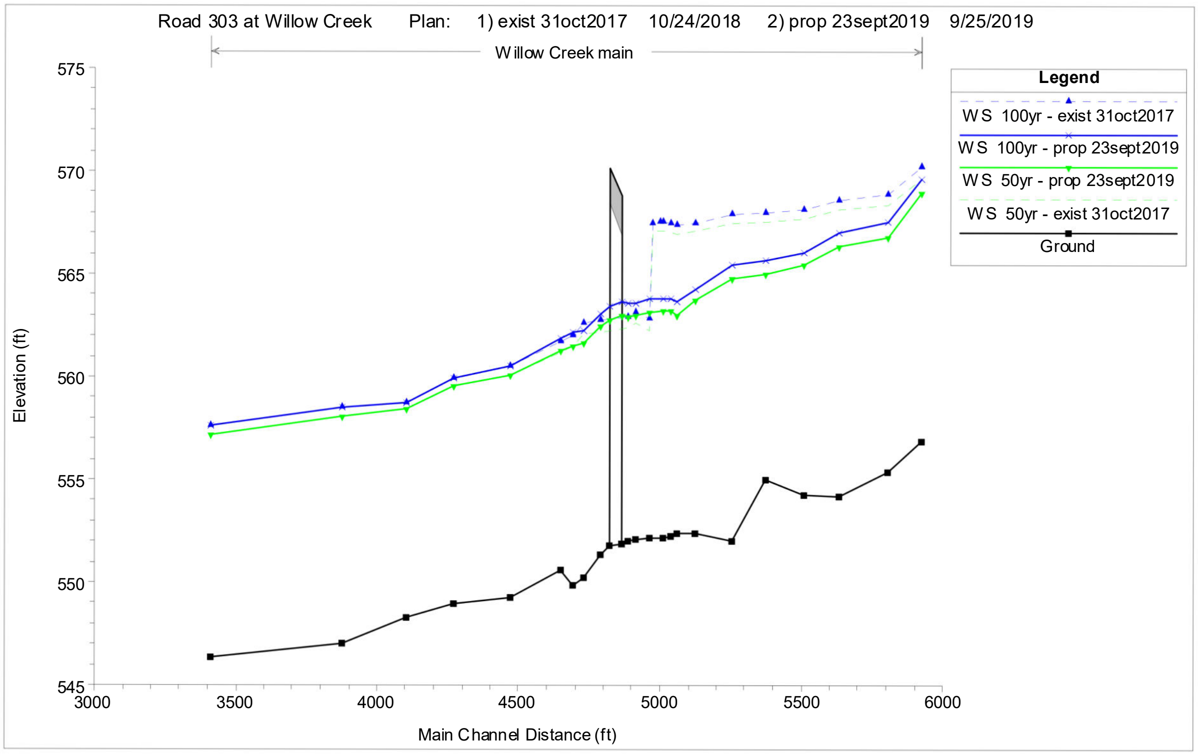


Figure 8. Water surface elevation profile comparison of existing to proposed for the 50 and 100-year discharges.

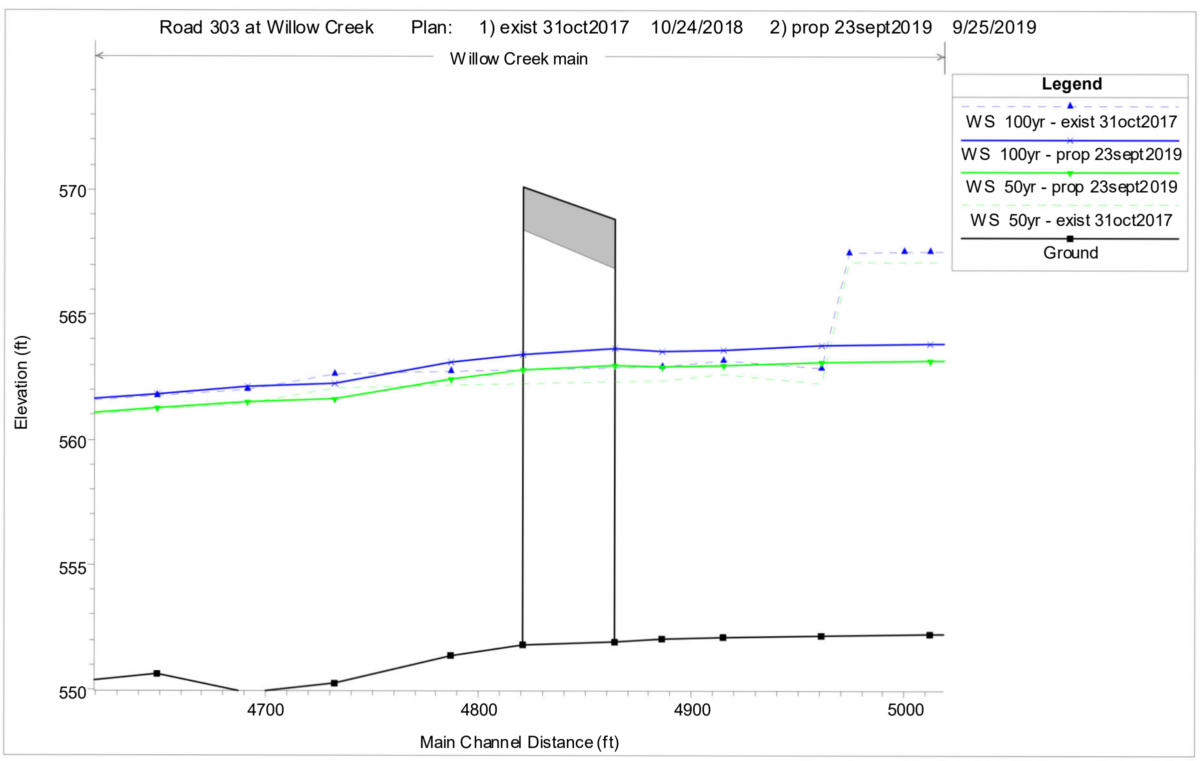


Figure 9. Zoomed in view of Figure 8.

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

River Station	50-year			100-year		
	existing	proposed	difference	existing	proposed	difference
5923	569.61	568.98	-0.63	570.24	569.66	-0.58
5803	568.35	566.82	-1.53	568.91	567.53	-1.38
5632	568.10	566.36	-1.74	568.62	567.04	-1.58
5509	567.69	565.42	-2.27	568.14	566.06	-2.08
5375	567.55	564.99	-2.56	567.99	565.63	-2.36
5255	567.49	564.76	-2.73	567.92	565.42	-2.50
5125	567.10	563.71	-3.39	567.50	564.27	-3.23
5058	566.97	562.98	-3.99	567.38	563.63	-3.75
5038	567.03	563.18	-3.85	567.44	563.82	-3.62
5012	567.12	563.19	-3.93	567.55	563.84	-3.71
Existing bridge						
4961	562.26	563.12	0.86	562.87	563.78	0.91
4915	562.60	562.96	0.36	563.15	563.60	0.45
4886	562.39	562.90	0.51	562.93	563.56	0.63
Proposed bridge						
4787	562.17	562.43	0.26	562.73	563.09	0.36
4732	562.06	561.62	-0.44	562.62	562.22	-0.40
4691	561.47	561.52	0.05	562.02	562.13	0.11
4649	561.22	561.26	0.04	561.75	561.84	0.09
4468	560.10	560.07	-0.03	560.55	560.50	-0.05
4270	559.52	559.52	0.00	559.94	559.94	0.00
4103	558.46	558.46	0.00	558.74	558.74	0.00
3873	558.06	558.06	0.00	558.51	558.51	0.00
3410	557.16	557.16	0.00	557.60	557.60	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.

Since the minimum soffit elevation under proposed conditions is 565.4 feet, 2.4 feet of freeboard will be provided above the 50-year WSE (563.0 feet) and 1.8 feet of freeboard will be provided above the 100-year water surface elevation (563.6), 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 was only one instance in the records of debris on the bridge from 1978, which recorded drift in the channel, but not caught on the bridge structure.

The proposed bridge will improve the hydraulics by increasing the channel capacity, or opening, through the bridge from approximately 24 ft to 84 feet (accounting for approximately 25 degrees of hydraulic skew).

STREAMBED/BANK PROTECTION

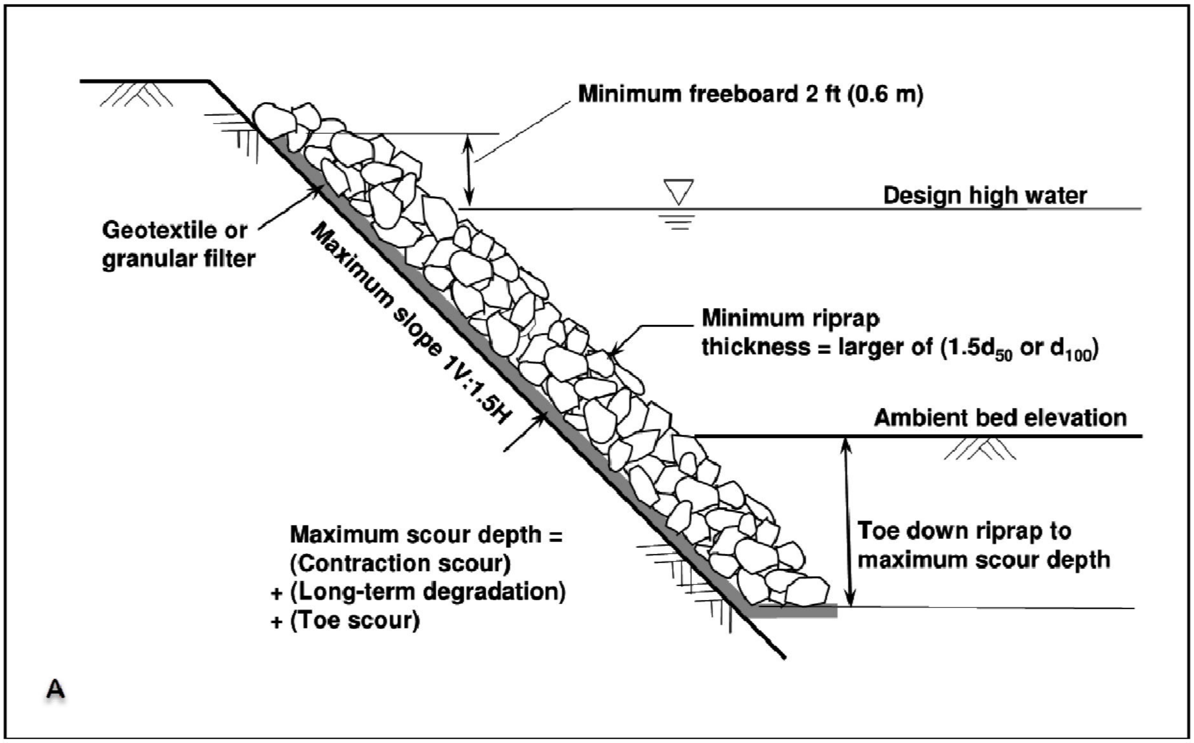
Riprap size was calculated using the FHWA Hydraulic Engineering Circular No. 23 (HEC 23) guidelines for RSP (FHWA, 2009). The riprap revetment design guidelines outlined in HEC 23 are based on flume studies performed by Stephen Maynord in 1989 and 1990 and were published in the U.S. Army Corp of Engineers (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	**4915**	**4886**	**4863.5 BR U**	**4863.5 BR D**	**4787**
Width (ft)	73.4	80.8	84.3	77.1	81.1
Average Velocity (ft/s)	7.8	7.4	6.3	6.7	7.6
Hydraulic Depth (ft)	7.9	7.5	8.5	8.7	7.3
Calculated D50 (in)	12.3	11.6	7.5	8.4	12.2
Calculated Weight (lbs)	150.1	128.4	34.7	48.5	148.0
Class (based on size)	IV	III	II	II	IV
D50 (in)	15	12	9	9	15
Weight (lbs)	300	150	60	60	150
Thickness (in)	30	24	18	18	30

Class IV (15-inch, 150 lb.) rock slope protection should be used to protect the banks of the proposed bridge. Rock slope protection should extend up to elevation 562.8, the approximate design flood elevation, and be lined with Type B filter fabric. The RSP should be keyed down to the scour depth, or a mounded toe approach should be used as shown in Figure 10.



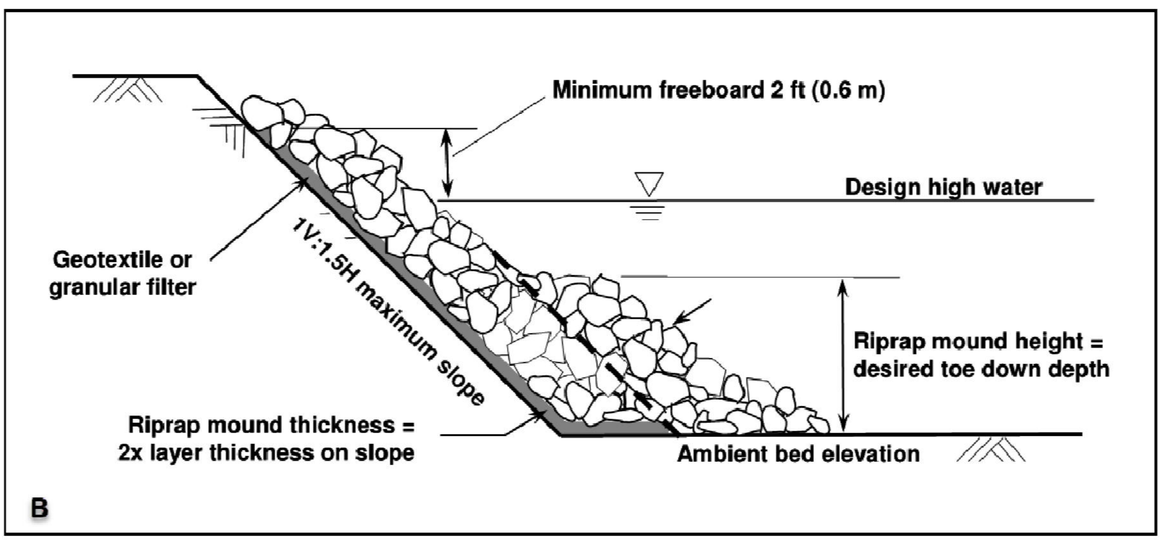


Figure 10. Bank RSP termination options: A) key down to the scour depth and B) Mounded Toe (Lagasse et al. 2009)

See Appendix E for bank protection calculations.

SCOUR

The County Road 303 Bridge was determined to have significant scour problems by Caltrans with scour issues first noted in the report dated 1995 resulting in significant undermining of the concrete lining of the channel under the structure as shown in Figure 11. The item 113 (scour) code for the bridge is a U, meaning the bridge has an unknown foundation and could not be evaluated for scour.

Figure 11. Undermining and scour at Abutment 2 of the existing bridge.

Degradation

There were no available cross sections for this bridge, so channel degradation could not be assessed. However, Figure 11 clearly shows undermining of the existing structure indicating the channel is degrading. Therefore, a minimum 3 feet of degradation is assumed for the proposed bridge.

Contraction Scour

The proposed replacement structure does not constrict the channel. Thus, there is no anticipated 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 550).

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. In this case, contraction scour is negligible since the channel would be opened. The abutment scour is then calculated as the local pier scour. For abutment piles of 2 feet, the resulting pier scour under Condition C is 5 feet. Unless 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 552 ft to elevation 547 ft.

Total Scour

The total scour values are presented in the Scour Summary table, Table 7.

Table 7. Scour summary table assuming Abutment Scour Condition A.

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

Detailed scour calculations are provided in Appendix F.

SUMMARY TABLES

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

Drainage Area: 24.4 Square miles

	Design	**Base**	**Flood of Record**
Frequency (Years)	50	100	<50
Discharge (Cubic feet per second)	4,000	4,500	2,390
Water Surface (Elevation at u/s face of Bridge)	563.0	563.6	560.4

The following Scour Data Table is provided for placement on the Foundation Plan assuming a thalweg elevation of 552 ft:

Support No.	Long Term (Degradation and Contraction) Scour Elevation (ft)	Short Term (Local) Scour Depth (ft)
A1	552	2
A2	552	2

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.

REFERENCES

California Department of Transportation (Caltrans). 1998. “Local Assistance Procedures Manual, Processing Procedures for Implementing Federal and/or State Funded local Public Transportation Projects.” February.

California Department of Transportation (Caltrans). 2001. “Highway Design Manual Chapter 800.” May.

Federal Highway Administration (FHWA). 2009. “Bridge Scour and Stream Instability Countermeasures,” Federal Highway Administration Report No. FHWA NHI HEC-23. September.

Federal Highway Administration (FHWA). 1995. “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges”. Federal Highway Administration Report No. FHWA-RD-96-0001. December.

H.H. Barnes, Jr. 1967 United States Geological Survey Water Supply Paper 1849.

Glenn County. 2012. Maintenance Records and As-Built Plans for the County Road 303 (Br # 11C0163)

Maynord, S.T., Ruff, J.F., and Abt, S.R. 1989. "Riprap Design," ASCE Journal of Hydraulic Engineering, Vol. 115, No. 7, pp 937-939.

Maynord, S.T. 1990. "Riprap Stability Results from Large Test Channel," Hydraulic Engineering, Proceedings of the 1990 ASCE National Conference, Volume 1, Chang, H.H. and Hill, J.C. (eds), San Diego, CA.

Maynord, S.T., Hebler, M.T. and Knight, S.F. 1998. “User's Manual for CHANLPRO, PC Program for Channel Protection Design” (No. WES/TR/CHL-98-20). Army Engineer Waterways Experiment Station Vicksburg, MS Coastal Hydraulics Lab.

Racin, J.A., Hoover, T.P., and C.M. Crossett Avila. 1997. “California Bank and Shore Rock Slope Protection Design.” FHWA-CA-TL-10, California Department of Transportation, Office of New Technology and Research. Sacramento, CA.

Willdan Engineering Geotechnical Group. 2019. “Foundation Report Replacement of County Road 303 Bridge over Willow Creek Caltrans Bridge No. 11C-0163 Glenn County, California.” September.

U.S. Army Corps of Engineers. 1991. "Hydraulic Design of Flood Control Channels," EM 1110-2-1601, Department of the Army, Washington, D.C.

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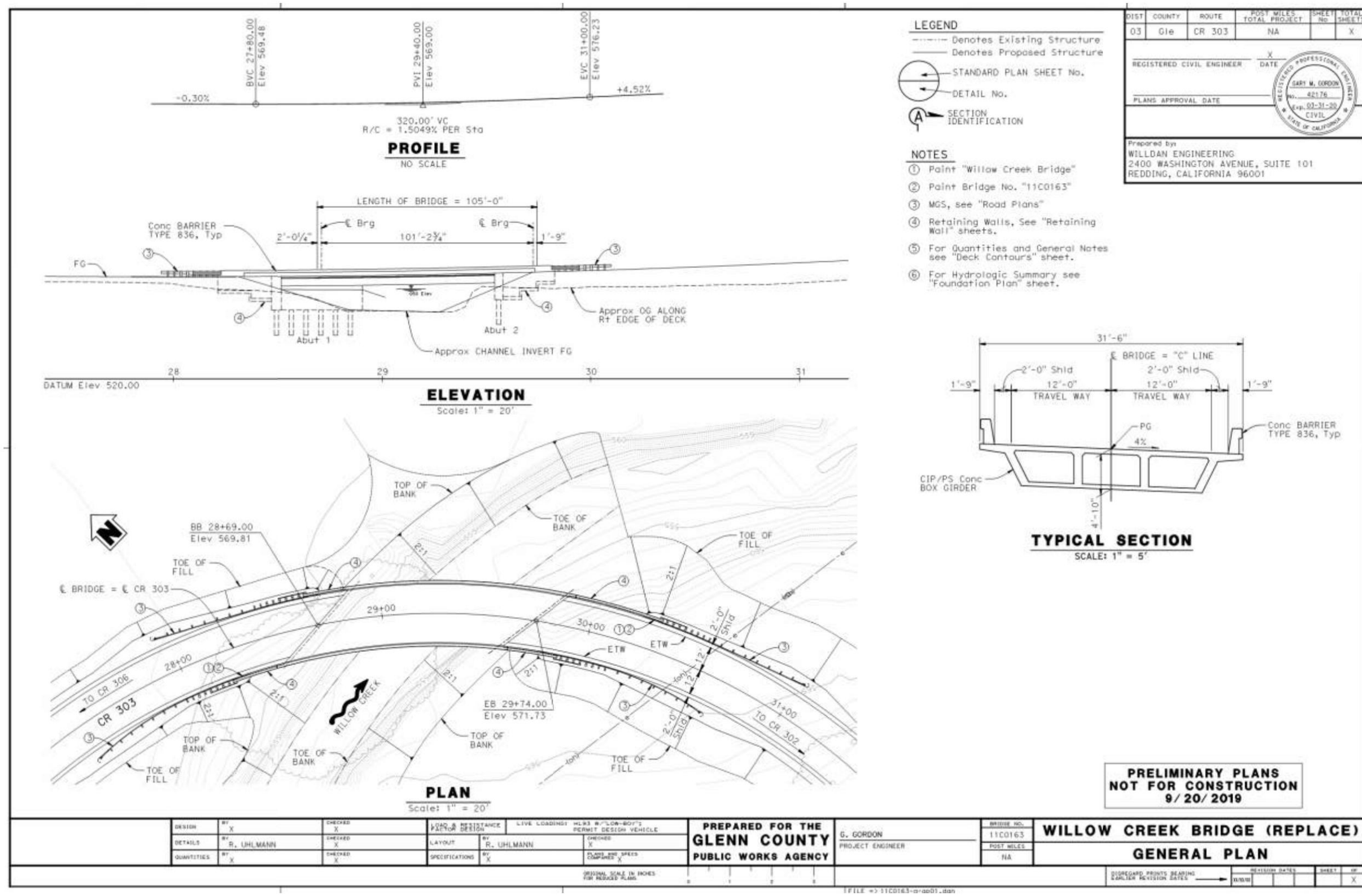
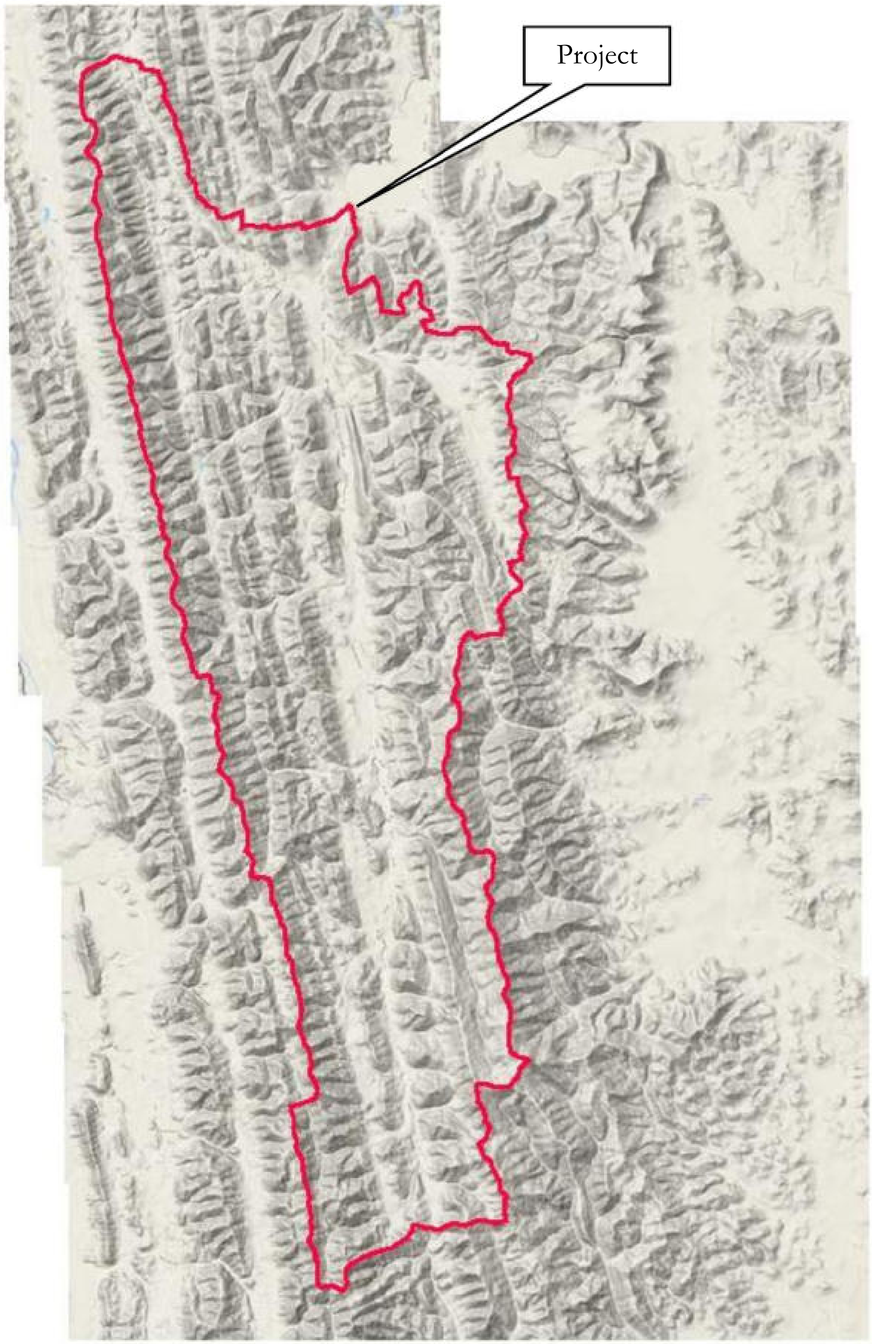


LEGEND
Denotes Existing Structure
Denotes Proposed Structure
STANDARD PLAN SHEET No.
DETAIL No.
SECTION IDENTIFICATION
NOTES
① Paint "Willow Creek Bridge"
② Paint Bridge No. "11C0163"
③ MGS, see "Road Plans"
④ Retaining Walls, See "Retaining Wall" sheets.
⑤ For Quantities and General Notes see "Deck Contours" sheet.
⑥ For Hydrologic Summary see "Foundation Plan" sheet.
DIST 03 COUNTY Gle ROUTE CR 303 POST MILES TOTAL PROJECT NA
REGISTERED CIVIL ENGINEER
PLANS APPROVAL DATE
Prepared by:
WILLDAN ENGINEERING
2400 WASHINGTON AVENUE, SUITE 101
REDDING, CALIFORNIA 96001
BVC 27+80.00 Elev 569.48
PVI 29+40.00 Elev 569.00
EVC 31+00.00 Elev 576.23
-0.30%
+4.52%
320.00' VC
R/C = 1.5049% PER Sta
PROFILE
NO SCALE
LENGTH OF BRIDGE = 105'-0"
Conc BARRIER TYPE 836, Typ
101'-2¾"
Abut 1
Abut 2
Approx OG ALONG Rt EDGE OF DECK
Approx CHANNEL INVERT FG
DATUM Elev 520.00
ELEVATION
Scale: 1" = 20'
31'-6"
℄ BRIDGE = "C" LINE
2'-0" Shld
12'-0" TRAVEL WAY
PG
4%
CIP/PS Conc BOX GIRDER
TYPICAL SECTION
SCALE: 1" = 5'
BB 28+69.00 Elev 569.81
℄ BRIDGE = ℄ CR 303
TOP OF BANK
TOE OF FILL
TOE OF BANK
EB 29+74.00 Elev 571.73
TO CR 306
CR 303
TO CR 302
WILLOW CREEK
PLAN
Scale: 1" = 20'
PRELIMINARY PLANS
NOT FOR CONSTRUCTION
9/20/2019
R. UHLMANN
PREPARED FOR THE
GLENN COUNTY
PUBLIC WORKS AGENCY
G. GORDON
PROJECT ENGINEER
11C0163
WILLOW CREEK BRIDGE (REPLACE)
GENERAL PLAN

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

South Fork (SF) Willow Creek at the Road 303 bridge drains an approximate 24.4 square miles as shown on the Hydrology Map. The mean annual precipitation of the project watershed is approximately 21.8 inches/year (streamstats).



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).*

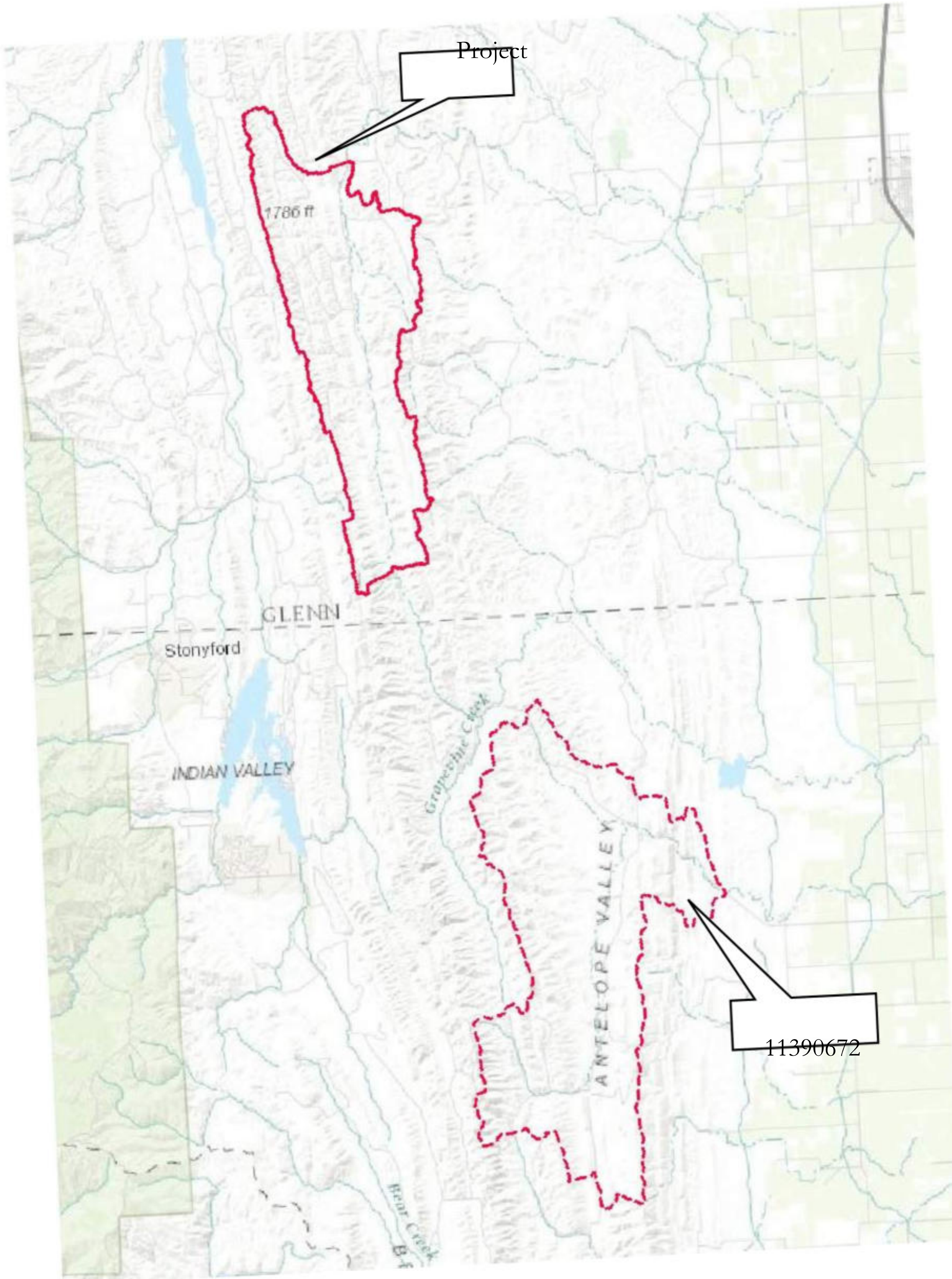
North Coast Region 1
Area = 24.4 sq. mi.
MAP = 21.8 in/yr

C	DRNAREA	y	DRNAREA^y	PRECIP	z	PRECIP^z	**Q**	**Recurrence**
1.82	24.4	0.904	32.6792	21.8	0.983	20.6872	**676**	**2**
8.11	24.4	0.887	137.9227	21.8	0.772	10.7966	**1489**	**5**
14.8	24.4	0.88	246.1301	21.8	0.696	8.5421	**2102**	**10**
26	24.4	0.874	424.1818	21.8	0.628	6.9271	**2938**	**25**
36.3	24.4	0.87	584.7036	21.8	0.589	6.1426	**3592**	**50**
48.5	24.4	0.866	771.2964	21.8	0.556	5.5486	**4280**	**100**
61	24.4	0.863	960.8315	21.8	0.531	5.1371	**4936**	**200**
79.3	24.4	0.86	1237.1673	21.8	0.503	4.7124	**5830**	**500**

METHOD 2: Statistical Analysis of Gage Data

There is an existing stream gage on Stone Corral Creek approximately 19 miles southeast from the project (USGS gage #11390672). The area of the watershed at the gage is approximately 38.4 square miles and has 28 peak stream flow records taken between 1958 and 1985. During this period, the highest peak flow recorded was 5,700 cfs in January 1983. Other significant flows recorded are 5,430 cfs in February 1973 and 4,460 cfs in February 1980.

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 11390672 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 the table below.

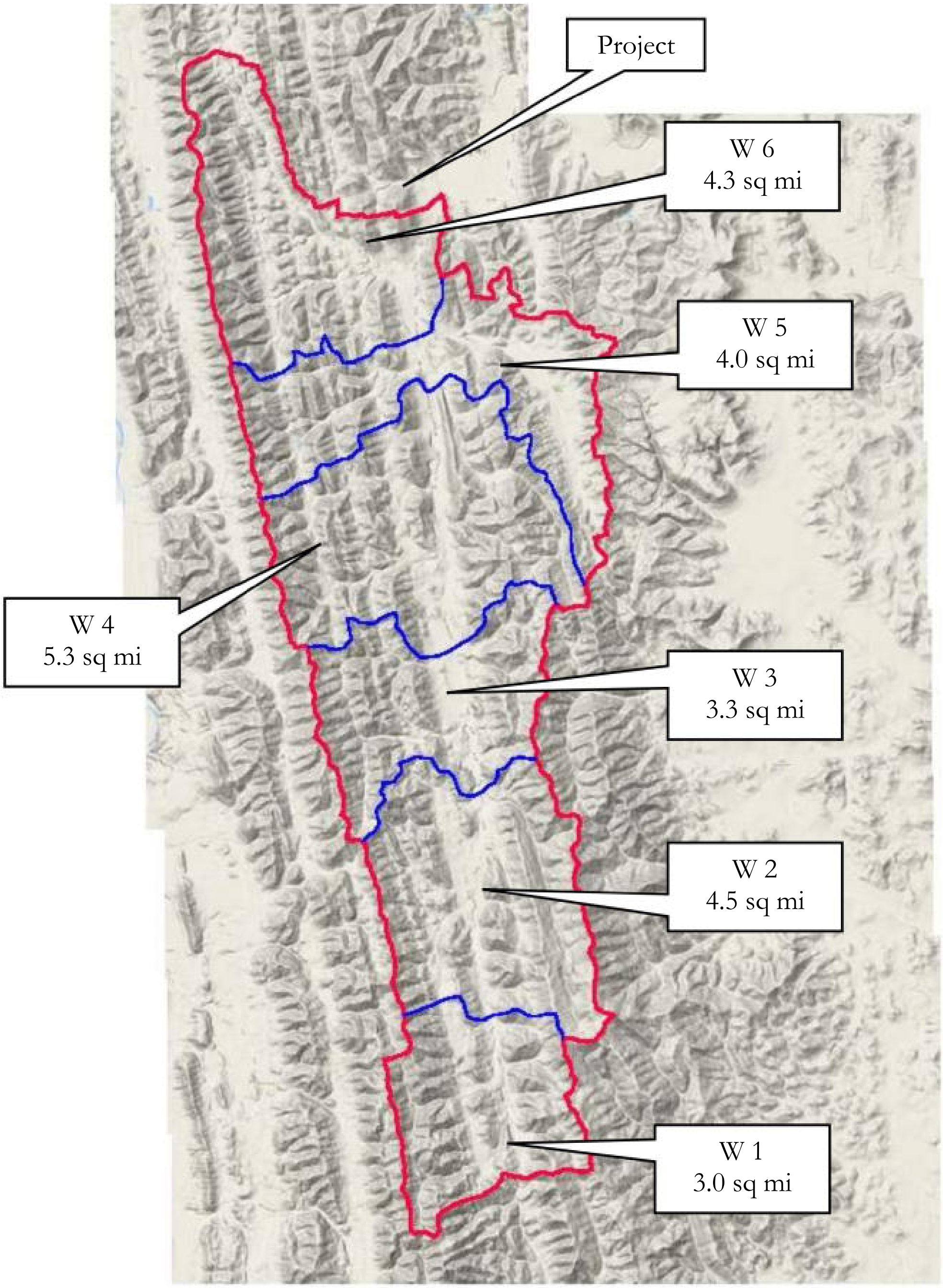
Recurrence interval (yr)	Discharge (cfs)	
	Statistical analysis (SIR 2012-5113)	Basin Transfer to Project
2	1,840	1,221
5	3,160	2,113
10	4,040	2,711
25	5,120	3,445
50	5,880	3,963
100	6,590	4,450
500	8,120	5,498

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 six sub-basins as shown below.



HEC-HMS Sub-basins

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

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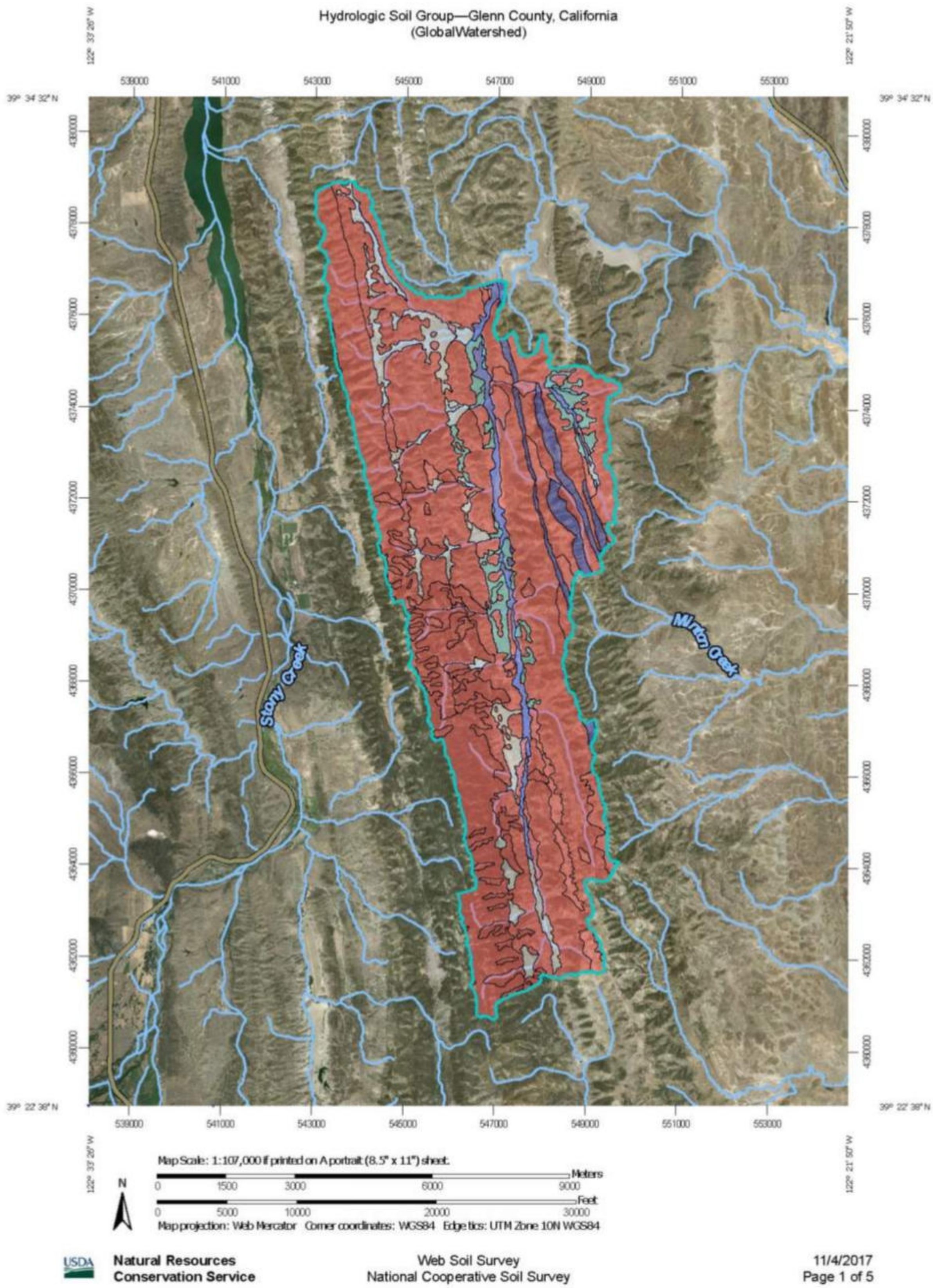

Blue = Class B, Aqua = Class C, Salmon = Class D, Tan = Not rated or N/A

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Watershed Soils Map (USDA NRCS Web Soil Survey)

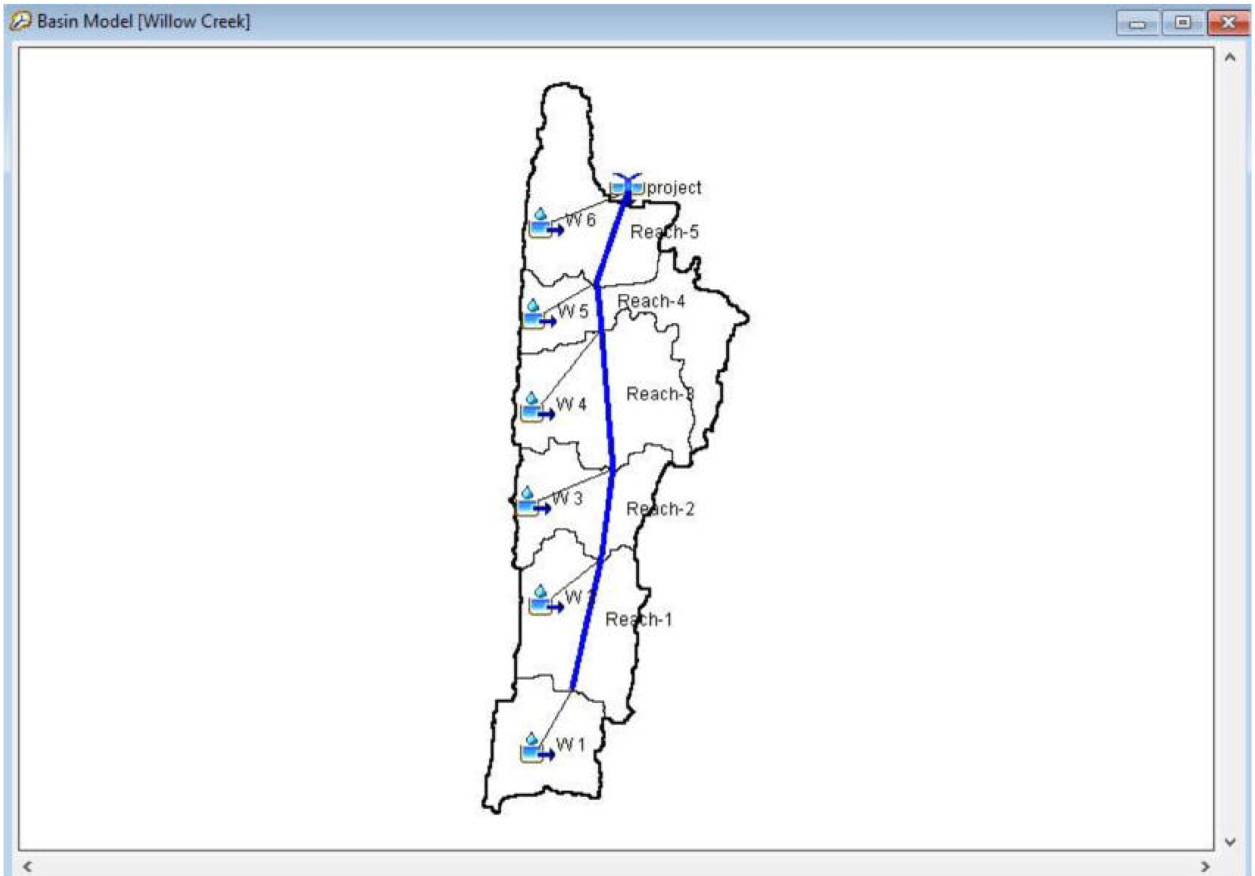
A breakdown of the soils types for the project watershed is shown in the table below along with the composite CN value.

Soil Class	**Area** (acre)	**CN** (see note)	**Area x CN**
B	985.7	48	47313.6
C	801.8	65	52117
D	13842.2	73	1010481
Water	2.0	99	198
Total	15631.7		1110109
		Composite	**71**

Note: Cover type is primarily brush with hydrologic condition good.
Precipitation data was obtained from the NOAA's National Weather Service Hydrometerological 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 3.17 and 3.53 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**	**Lag Time**
	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)	(min)	(min)
W 1	300	0.52	0.4	0.272	16.3	420	0.21	7.5	0.9	14118	8.9	26.3	43.5	**26**
W 2	300	0.28	0.4	0.345	20.7	1534	0.18	7.0	3.7	13958	8.9	26.0	50.4	**30**
W 3	300	0.35	0.4	0.317	19.0	1640	0.19	7.0	3.9	12996	7.6	28.5	51.4	**31**
W 4	300	0.44	0.4	0.291	17.4	690	0.22	7.5	1.5	14407	9.0	26.7	45.6	**27**
W 5	300	0.30	0.4	0.338	20.3	1350	0.21	7.5	3.0	16537	7.2	38.4	61.7	**37**
W 6	300	0.28	0.4	0.345	20.7	2553	0.18	7.0	6.1	19573	6.8	48.3	75.1	**45**
Reach 1										12508	5.6	37.2		**37**
Reach 2										8872	6.4	23.1		**23**
Reach 3										15051	7.2	35.0		**35**
Reach 4										3427	6.2	9.1		**9**
Reach 5										7882	7.6	17.3		**17**

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



HEC-HMS Model Schematic

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

Global Summary Results for Run "50yr 12hr"

Project: SF Willow Creek ar Rd 303 Simulation Run: 50yr 12hr

Start of Run: 01Jan2017, 12:00 Basin Model: Willow Creek
End of Run: 02Jan2017, 12:10 Meteorologic Model: 50-yr 12-hr
Compute Time: 14Nov2017, 15:25:07 Control Specifications: 24 hr

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
W 2	4.4878	574.6	01Jan2017, 20:25	1.23
W 1	2.9759	399.5	01Jan2017, 20:20	1.23
Reach-1	2.9759	398.0	01Jan2017, 20:55	1.21
Reach-2	7.4637	839.3	01Jan2017, 21:10	1.21
W 3	3.2783	415.7	01Jan2017, 20:25	1.23
Reach-3	10.7420	1121.7	01Jan2017, 21:40	1.19
W 4	5.3085	704.9	01Jan2017, 20:20	1.23
Reach-4	16.0505	1482.7	01Jan2017, 21:30	1.20
W 5	4.0493	481.5	01Jan2017, 20:30	1.22
Reach-5	20.0998	1800.5	01Jan2017, 21:40	1.19
W 6	4.3238	477.4	01Jan2017, 20:40	1.22
project	24.4236	2135.1	01Jan2017, 21:35	1.19

Global Summary Results for Run "100yr 12hr"

Project: SF Willow Creek ar Rd 303 Simulation Run: 100yr 12hr

Start of Run: 01Jan2017, 12:00 Basin Model: Willow Creek
End of Run: 02Jan2017, 12:10 Meteorologic Model: 100-yr 12-hr
Compute Time: 14Nov2017, 15:25:07 Control Specifications: 24 hr

Show Elements: All Elements Volume Units: IN AC-FT Sorting: Hydrologic

Hydrologic Element	Drainage Area (MI2)	Peak Discharge (CFS)	Time of Peak	Volume (IN)
W 2	4.4878	699.6	01Jan2017, 20:25	1.47
W 1	2.9759	486.5	01Jan2017, 20:20	1.48
Reach-1	2.9759	484.9	01Jan2017, 20:55	1.45
Reach-2	7.4637	1020.5	01Jan2017, 21:10	1.44
W 3	3.2783	506.3	01Jan2017, 20:25	1.47
Reach-3	10.7420	1362.4	01Jan2017, 21:40	1.42
W 4	5.3085	858.7	01Jan2017, 20:20	1.48
Reach-4	16.0505	1800.9	01Jan2017, 21:30	1.43
W 5	4.0493	586.7	01Jan2017, 20:30	1.46
Reach-5	20.0998	2185.9	01Jan2017, 21:40	1.42
W 6	4.3238	581.6	01Jan2017, 20:40	1.46
project	24.4236	2590.9	01Jan2017, 21:35	1.43

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)	
	50-yr	100-yr
Regression	3,592	4,280
Basin Transfer Stone Corral Creek Gage (11390672)	3,963	4,450
HEC-HMS	2,135	2,590

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.

	Discharge (cfs)	
	Design	Base
Frequency (years)	50	100
Discharge (cubic feet per second)	4,000	4,500

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APPENDIX C- HEC-RAS RESULTS

HEC-RAS Plan: exist 31oct2017 River: Willow Creek Reach: main

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)	
main	5923	50yr	4000.00	556.88	569.61		569.94	0.000886	4.98	889.01	100.17	0.28
main	5923	100yr	4500.00	556.88	570.24		570.61	0.000922	5.22	952.75	102.11	0.28
main	5803	50yr	4000.00	555.37	568.35		569.64	0.005171	9.59	463.10	69.82	0.57
main	5803	100yr	4500.00	555.37	568.91		570.29	0.005305	9.97	502.86	72.89	0.58
main	5632	50yr	4000.00	554.16	568.10		568.91	0.002400	7.31	564.81	64.96	0.42
main	5632	100yr	4500.00	554.16	568.62		569.53	0.002592	7.77	598.68	66.51	0.44
main	5509	50yr	4000.00	554.23	567.69		568.58	0.002885	7.58	527.57	57.55	0.44
main	5509	100yr	4500.00	554.23	568.14		569.17	0.003206	8.12	554.00	58.72	0.47
main	5375	50yr	4000.00	555.01	567.55		568.21	0.001836	6.51	614.19	65.41	0.37
main	5375	100yr	4500.00	555.01	567.99		568.75	0.002050	7.00	643.29	66.46	0.40
main	5255	50yr	4000.00	552.04	567.49	560.89	567.97	0.001361	5.56	720.70	81.35	0.32
main	5255	100yr	4500.00	552.04	567.92	561.42	568.48	0.001507	5.98	757.58	86.54	0.34
main	5254.9		Lat Struct									
main	5125	50yr	4000.00	552.39	567.10		567.76	0.001459	6.64	664.23	109.31	0.34
main	5125	100yr	4497.97	552.39	567.50		568.26	0.001614	7.14	708.30	111.90	0.36
main	5058	50yr	3997.51	552.36	566.97		567.66	0.001661	6.85	665.47	114.25	0.36
main	5058	100yr	4463.10	552.36	567.38		568.14	0.001774	7.25	712.85	114.89	0.37
main	5038	50yr	3977.74	552.11	567.03		567.59	0.001423	6.13	688.72	76.73	0.32
main	5038	100yr	4422.29	552.11	567.44		568.07	0.001562	6.54	720.25	77.20	0.34
main	5012	50yr	3925.16	550.11	567.12	560.48	567.51	0.000899	5.03	822.25	112.29	0.26
main	5012	100yr	4336.20	550.11	567.55	561.10	567.98	0.000962	5.33	873.62	128.46	0.27
main	5000		Bridge									
main	4961	50yr	3925.16	551.22	562.26	562.26	566.96	0.011434	17.39	225.72	61.00	1.00
main	4961	100yr	4336.20	551.22	562.87	562.87	567.93	0.011307	18.04	240.42	63.47	1.00
main	4915	50yr	3925.16	551.14	562.60		563.73	0.005330	8.53	460.16	70.57	0.59
main	4915	100yr	4336.20	551.14	563.15		564.32	0.005306	8.67	499.94	74.18	0.59
main	4886	50yr	3925.16	551.89	562.39		563.57	0.005350	8.71	450.71	68.13	0.60
main	4886	100yr	4336.20	551.89	562.93		564.16	0.005414	8.87	488.70	72.06	0.60
main	4787	50yr	3925.16	552.31	562.17		563.00	0.004140	7.32	536.27	86.59	0.52
main	4787	100yr	4336.20	552.31	562.73		563.58	0.004095	7.39	586.41	92.01	0.52
main	4732	50yr	3925.16	550.18	562.06		562.76	0.003349	6.72	584.48	88.42	0.46
main	4732	100yr	4336.20	550.18	562.62		563.35	0.003155	6.83	635.98	92.28	0.45
main	4691	50yr	3925.16	549.88	561.47		562.55	0.005084	8.36	469.50	70.14	0.57
main	4691	100yr	4336.20	549.88	562.02		563.15	0.005006	8.52	508.79	72.72	0.57
main	4649	50yr	3925.16	550.61	561.22		562.35	0.004581	8.50	461.52	64.21	0.56
main	4649	100yr	4336.20	550.61	561.75		562.94	0.004632	8.74	495.90	66.33	0.56
main	4468	50yr	3925.16	549.25	560.10		561.45	0.005041	9.35	420.08	54.88	0.59
main	4468	100yr	4336.20	549.25	560.55		562.03	0.005120	9.77	445.74	59.07	0.60
main	4270	50yr	4000.00	548.92	559.52		560.44	0.003963	7.70	519.43	77.89	0.53
main	4270	100yr	4500.00	548.92	559.94		560.97	0.004243	8.14	552.77	79.94	0.55
main	4103	50yr	4000.00	548.27	558.46		559.67	0.005085	8.84	465.93	109.60	0.60
main	4103	100yr	4500.00	548.27	558.74	556.79	560.13	0.005560	9.50	500.27	132.73	0.63
main	3873	50yr	4000.00	547.03	558.06		558.77	0.002245	7.15	780.46	339.02	0.42
main	3873	100yr	4500.00	547.03	558.51		559.17	0.002082	7.12	940.14	365.45	0.41
main	3410	50yr	4000.00	546.33	557.16	553.63	557.75	0.002003	6.56	755.12	334.17	0.40
main	3410	100yr	4500.00	546.33	557.60	554.08	558.21	0.002002	6.78	830.74	335.80	0.40

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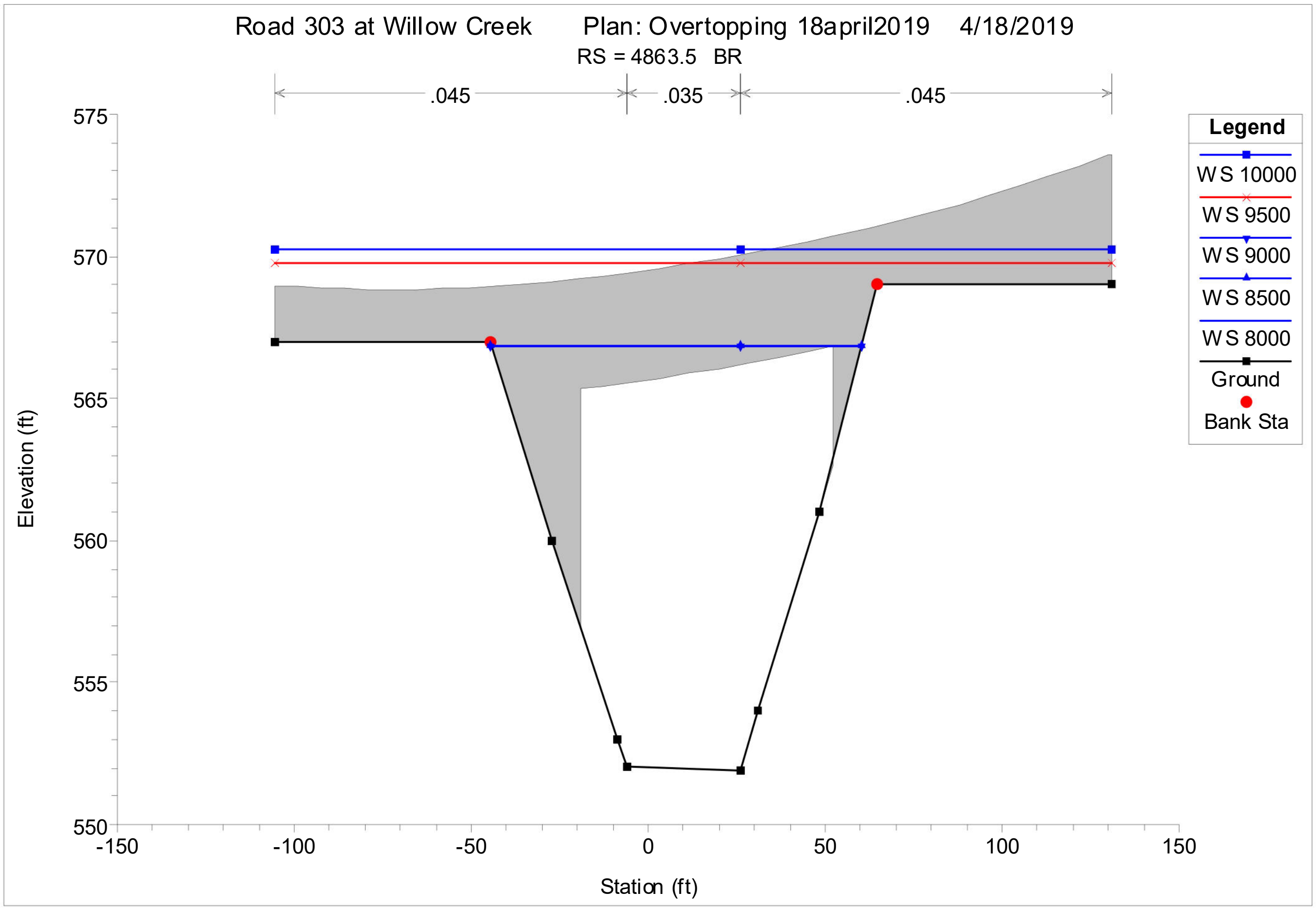
HEC-RAS Plan: prop 23sept2019 River: Willow Creek Reach: main

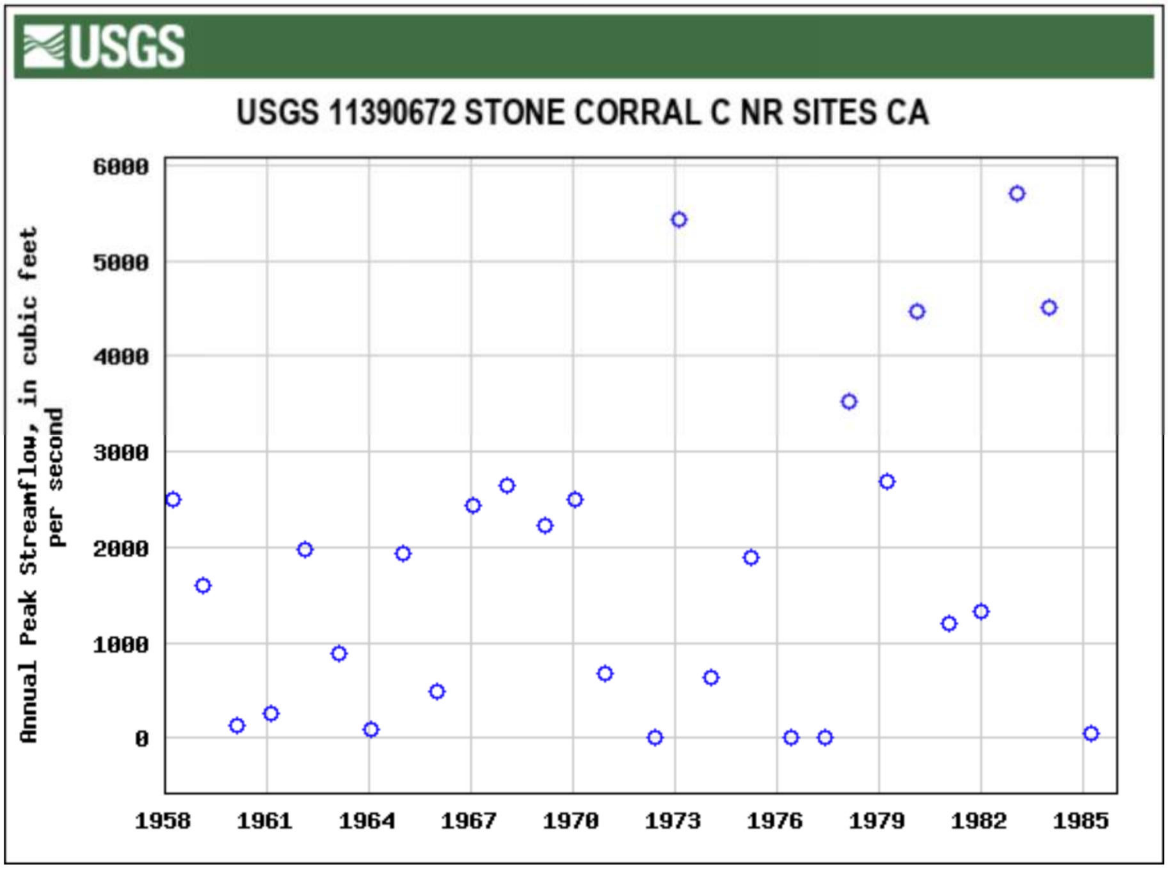
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
main	5923	50yr	4000.00	556.88	568.98		569.37	0.001087	5.35	827.19	98.25	0.30
main	5923	100yr	4500.00	556.88	569.66		570.08	0.001101	5.56	894.75	100.34	0.31
main	5803	50yr	4000.00	555.37	566.82		568.91	0.009660	12.11	362.33	61.37	0.77
main	5803	100yr	4500.00	555.37	567.53		569.62	0.009083	12.20	407.18	65.27	0.75
main	5632	50yr	4000.00	554.16	566.36		567.59	0.004313	8.98	456.00	59.72	0.55
main	5632	100yr	4500.00	554.16	567.04		568.36	0.004294	9.29	497.57	61.78	0.55
main	5509	50yr	4000.00	554.23	565.42		566.95	0.005896	9.91	403.57	51.71	0.63
main	5509	100yr	4500.00	554.23	566.06		567.70	0.006030	10.30	437.03	53.35	0.63
main	5375	50yr	4000.00	555.01	564.99		566.19	0.004173	8.80	454.32	59.32	0.56
main	5375	100yr	4500.00	555.01	565.63		566.93	0.004215	9.12	493.34	60.86	0.56
main	5255	50yr	4000.00	552.04	564.76		565.67	0.003156	7.61	525.40	66.13	0.48
main	5255	100yr	4500.00	552.04	565.42		566.39	0.003228	7.90	569.50	68.47	0.48
main	5125	50yr	4000.00	552.39	563.71		565.12	0.004701	9.54	424.66	57.56	0.58
main	5125	100yr	4500.00	552.39	564.27		565.83	0.004788	10.05	457.69	60.66	0.59
main	5058	50yr	4000.00	552.36	562.98		564.68	0.008320	10.45	382.79	59.79	0.73
main	5058	100yr	4500.00	552.36	563.63		565.39	0.008186	10.65	422.69	63.11	0.73
main	5038	50yr	4000.00	552.23	563.18		564.41	0.005013	8.87	450.71	62.05	0.58
main	5038	100yr	4500.00	552.23	563.82		565.13	0.005048	9.16	491.06	64.39	0.59
main	5012	50yr	4000.00	552.20	563.19		564.24	0.004091	8.19	488.46	66.12	0.53
main	5012	100yr	4500.00	552.20	563.84		564.95	0.004128	8.46	531.91	68.64	0.54
main	4961	50yr	4000.00	552.14	563.12		563.99	0.003583	7.50	533.54	75.16	0.50
main	4961	100yr	4500.00	552.14	563.78		564.70	0.003580	7.71	584.02	78.40	0.50
main	4915	50yr	4000.00	552.08	562.96		563.83	0.003288	7.51	532.42	70.83	0.48
main	4915	100yr	4500.00	552.08	563.60		564.54	0.003330	7.77	578.85	73.40	0.49
main	4886	50yr	4000.00	552.04	562.90	559.39	563.72	0.003322	7.24	552.59	78.21	0.48
main	4886	100yr	4500.00	552.04	563.56	559.89	564.42	0.003270	7.44	604.83	80.76	0.48
main	4863.5		Bridge									
main	4787	50yr	4000.00	551.36	562.43		563.28	0.003493	7.37	542.83	77.60	0.49
main	4787	100yr	4500.00	551.36	563.09		563.98	0.003480	7.56	594.87	81.10	0.49
main	4732	50yr	4000.00	550.23	561.62		562.90	0.005689	9.08	440.50	63.12	0.61
main	4732	100yr	4500.00	550.23	562.22		563.59	0.005732	9.40	478.68	65.16	0.61
main	4691	50yr	4000.00	549.88	561.52		562.63	0.005173	8.45	473.09	70.38	0.57
main	4691	100yr	4500.00	549.88	562.13		563.31	0.005168	8.70	517.10	73.30	0.58
main	4649	50yr	4000.00	550.61	561.26		562.42	0.004687	8.62	464.18	64.38	0.57
main	4649	100yr	4500.00	550.61	561.84		563.09	0.004829	8.96	502.10	66.71	0.58
main	4468	50yr	4000.00	549.25	560.07		561.49	0.005278	9.55	418.88	54.68	0.60
main	4468	100yr	4500.00	549.25	560.50		562.11	0.005627	10.20	442.79	58.61	0.62
main	4270	50yr	4000.00	548.92	559.52		560.44	0.003963	7.70	519.43	77.89	0.53
main	4270	100yr	4500.00	548.92	559.94		560.97	0.004243	8.14	552.77	79.94	0.55
main	4103	50yr	4000.00	548.27	558.46		559.67	0.005085	8.84	465.93	109.60	0.60
main	4103	100yr	4500.00	548.27	558.74	556.78	560.13	0.005560	9.50	500.26	132.72	0.63
main	3873	50yr	4000.00	547.03	558.06		558.77	0.002245	7.15	780.43	339.01	0.42
main	3873	100yr	4500.00	547.03	558.51		559.17	0.002082	7.12	940.16	365.45	0.41
main	3410	50yr	4000.00	546.33	557.16	553.63	557.75	0.002003	6.56	755.12	334.17	0.40
main	3410	100yr	4500.00	546.33	557.60	554.08	558.21	0.002002	6.78	830.74	335.80	0.40

APPENDIX D- OVERTOPPING AND FLOOD OF RECORD

At flows near 9,500 cfs, water first overtops the bridge and roadway as shown below, resulting in a WSE of 569.8 ft at the upstream face of the proposed bridge.



The Flood of Record at the Stone Corral Creek gage (No. 11390672) occurred on January 26, 1983 resulting in a discharge of 5,700 cfs. A basin transfer was completed to the project site which resulted in 2,390 cfs. The HEC-RAS model was re-run with this discharge which resulted in a water surface elevation of 560.4 feet upstream of the proposed bridge.


USGS
USGS 11390672 STONE CORRAL C NR SITES CA
Annual Peak Streamflow, in cubic feet per second
6000
5000
4000
3000
2000
1000
0
1958
1961
1964
1967
1970
1973
1976
1979
1982
1985

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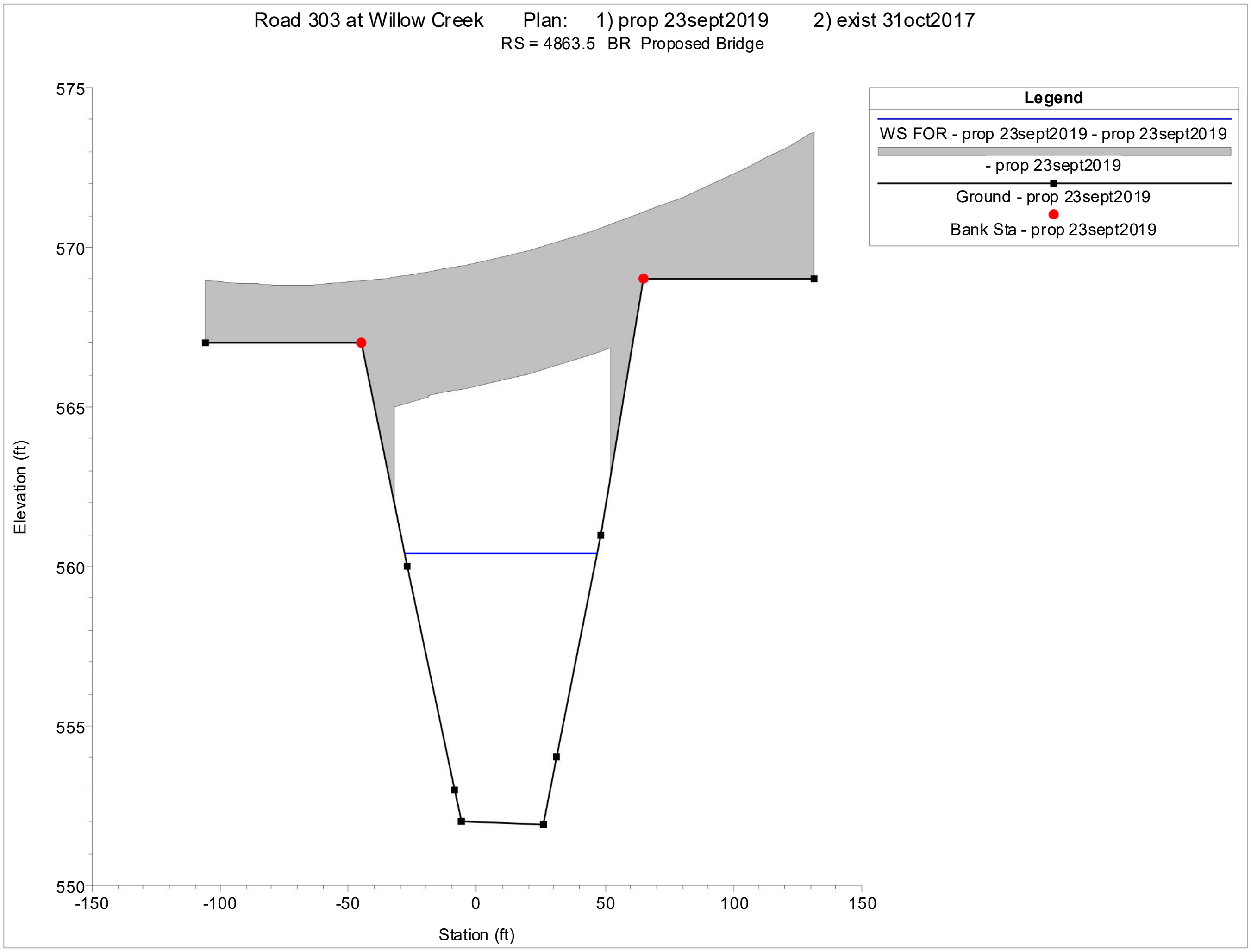
Road 303 at Willow Creek   Plan: 1) prop 23sept2019   2) exist 31oct2017
RS = 4863.5 BR Proposed Bridge
Legend
WS FOR - prop 23sept2019 - prop 23sept2019
- prop 23sept2019
Ground - prop 23sept2019
Bank Sta - prop 23sept2019
Elevation (ft)
Station (ft)
575
570
565
560
555
550
-150
-100
-50
0
50
100
150

APPENDIX E- ROCK RIPRAP SIZING

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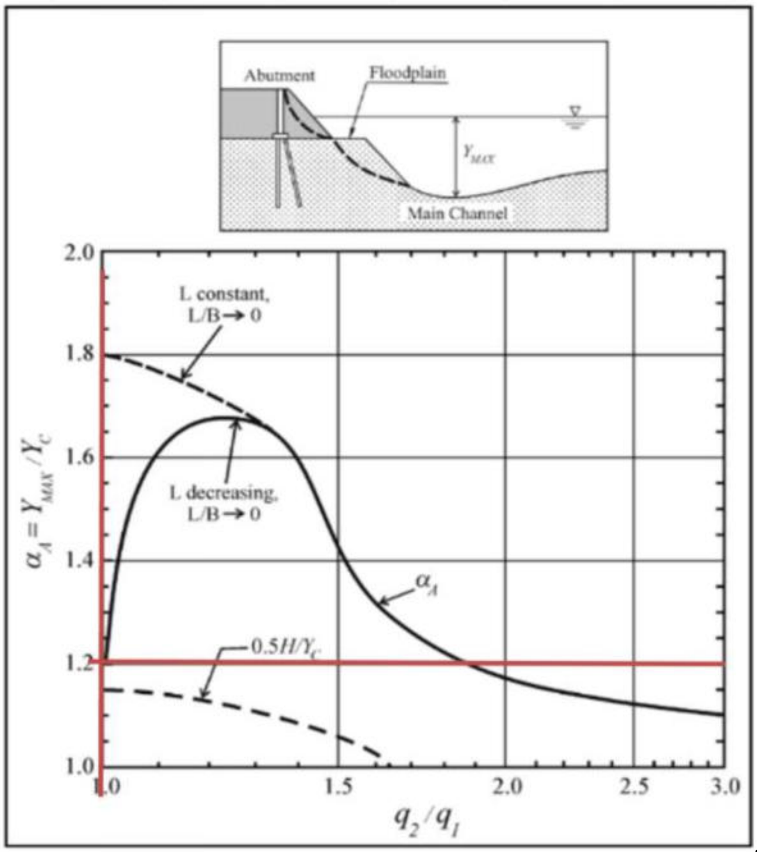
Willow Creek

Stone Size (d30)		4961	4915	4886	4863.5 BR U	4863.5 BR D	4787	4732	4691	Units
		Constants and Coefficients								
Safety Factor (typically 1.1)	Sf	1.10	1.1	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	0.3	
Velocity distribution coefficient	Cv	1.19	1.18	1.19	1.19	1.18	1.19	1.17	1.18	
Blanket thickness coefficient	CT	1	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	2.65	
Acceleration due to gravity	g	32.2	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	78.4	73.4	80.76	84.34	77.12	81.1	65.16	73.3	
Average Velocity	Vavg	7.71	7.77	7.44	6.28	6.69	7.56	9.4	8.7	ft/s
Hydraulic Depth	y	7.45	7.89	7.49	8.5	8.72	7.33	7.35	7.05	feet
		Other Inputs								
Slope	fraction	2:1	2:1	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	0.5	0.5	
Bank Angle	theta	26.57	26.57	26.57	26.57	26.57	26.57	26.57	26.57	
Radius of curvature of bend	Rc	240	240	240	240	240	240	240	240	feet
		Calculations								
$K_1 = \sqrt{1 - \left[\frac{\sin(\theta - 14°)}{\sin 32°}\right]^{1.6}}$	Rc/W	3.06	3.27	2.97	2.85	3.11	2.96	3.68	3.27	
	K1	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	
Characteristic velocity	Vdes	11.47	11.44	11.12	9.44	9.93	11.30	13.59	12.81	ft/s
$d_{30} = y(S_f C_S C_V C_T)\left[\frac{V_{des}}{\sqrt{K_1(S_g - 1)gy}}\right]^{2.5}$	**d30**	0.87	0.85	0.81	0.52	0.58	0.85	1.32	1.16	feet
	d50	1.05	1.02	0.97	0.63	0.70	1.02	1.59	1.39	feet
	d50	12.58	12.27	11.64	7.53	8.42	12.21	19.03	16.73	inches
Stone Weight (W)										
	Yw	62.4	62.4	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	2.65	2.65	
	Ys	165.36	165.36	165.36	165.36	165.36	165.36	165.36	165.36	lb/ft3
	d	1.0	1.0	1.0	0.6	0.7	1.0	1.6	1.4	feet
	W	161.80	150.13	128.41	34.73	48.50	147.98	560.22	380.68	pounds
RSP Class by Size	Class	IV	IV	III	II	II	IV	VI	V	
	Size	15	15	12	9	9	15	21	18	inches
	1.5*d50	22.5	22.5	18.0	13.5	13.5	22.5	31.5	27.0	inches
	d100	30	30	24	18	18	30	42	36	inches
	Thickness	30	30	24	18	18	30	42	36	inches
	Method	B	B	B	B	B	B	A or B	A or B	
RSP Class by Weight	Class	IV	IV	III	II	II	III	VI	V	
	Weight	300	300	150	60	60	150	750	500	pounds
	W100	2200	2200	1100	470	470	1100	6000	3800	pounds
	Method	B	B	B	B	B	B	A or B	A or B	

Abutment Scour Condition A:

The abutment scour amplification factor is 1.20.



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The abutment scour is 2 feet.

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

$$y_c = y_1\left(\frac{q_{2c}}{q_1}\right)^{6/7} \qquad y_{max} = \alpha_A\, y_c \qquad y_s = y_{max} - y_0$$

Parameter	Description	Metric Units		US Units		Notes
y_1	Upstream flow depth	3.51	(m)	11.50	(ft)	*Flow area of bridge / W_2*
y_0	Flow depth prior to scour	3.58	(m)	11.73	(ft)	*Data from chosen upstream XS*
α_a	Amplification factor for live-bed conditions	1.20	-	**1.20**	-	*For spill through abutments: Use Figure 8.9* *For wingwall abutments: Use Figure 8.10*
W_1	Width of the upstream channel	24.39	(m)	80.01	(ft)	*Width of Flow upstream of the bridge section*
Q_1	Flow in the upstream channel	127.43	(m^3/s)	4500.0	(ft^3/s)	*Flow upstream of the bridge section*
q_{2c}	Unit discharge in the constricted opening accounting for non-uniform flow distribution	4.98	(m^2/s)	53.57	(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	5.23	(m^2/s)	56.24	(ft^2/s)	*Q_1 / W_1*
q_2/q_1	Ratio of unit discharge	0.95	(m)	0.95	(ft)	*Value used in Figure 8.9 and Figure 8.10 to determine amplification factor*
y_c	Flow depth including live-bed contraction scour	3.36	(m)	11.03	(ft)	*Equation Above*
y_{max}	Max flow depth resulting from abutment scour	4.03	(m)	13.24	(ft)	*Equation Above*

Live Bed Abutment Scour Depth (y_s)	**1.5**	**(ft)**
	0.5	(m)

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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 θ > 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	2.59	(m/s)	8.50	(ft/s)	*Obtained from (BR U) Flow Distribution Table; Bridge Information Macro*
Fr_1	Froude Number directly upstream of the pier	0.45	-	0.45	-	*$Fr_1 = [V_1 / (gy_1)^{1/2}]$*

HEC-18 Equation Maximum Pier Scour Depth (y_s)	**5.7**	**(ft)**
	1.7	(m)

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

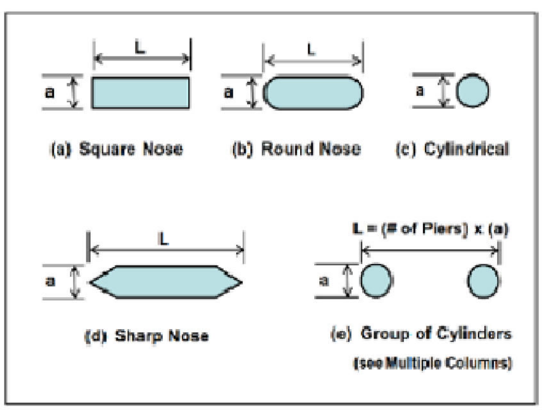

Figure 7.3. Common pier shapes.

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 ≥ 2	1.1
Medium Dunes	30 > H ≥ 10	1.2 to 1.1
Large Dunes	H ≥ 30	1.3

APPENDIX G – LOCATION HYDRAULIC STUDY FORM

LOCATION HYDRAULIC STUDY FORM

Dist.___3___Co.__Glenn___Rte. ___County Road 303___Project ID: _______Bridge #11C0163
Federal-Aid Project Number: BRLO 5911(059)

Floodplain Description:

The South Fork of Willow Creek flows northerly through the project site through the central part of Glenn County (County). It drains an approximate 14 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 60 ft to 100 ft through the project reach. The channel is sparsely vegetated and the overbanks are 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 by FEMA.

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 (11C0163) on County Road 303 over the South Fork of Willow Creek and construct the minimum approach work to accommodate the project. This will consist of approximately 1,000 feet of roadway reconstruction at the northerly approach and 500-ft on the southerly approach to the bridge. The bridge and roadway are being shifted approximately 100-feet downstream to bring the roadway geometrics up current design standards *(AASHTO Very Low Volume Roads, ADT ≤ 400)*. 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= 4,500 CFS
WSE100= ___563.6 ft
The flood of record, if greater than Q100:
Q=___n/a___CFS WSE= ___n/a___
Overtopping flood Q= 9,500 CFS WSE= ___569.8 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.

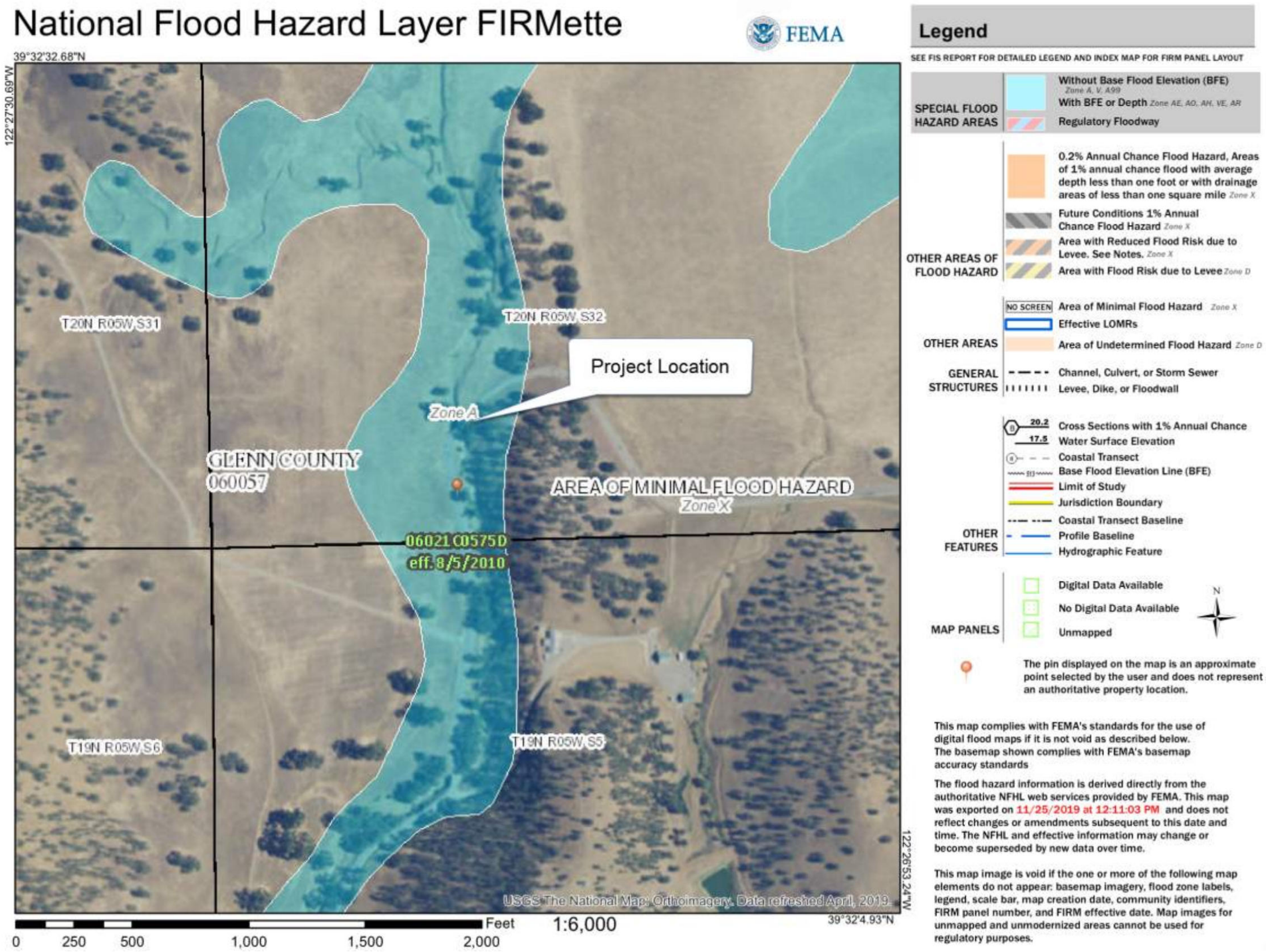

Figure 1. FEMA FIRMette of Map Number 06021C0575D dated August 5, 2010

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 and Figure 3~~Figure 4~~ the water surface elevation is lowered nearly 4 feet upstream of the proposed bridge, slightly higher between the locations of the existing (to be removed) and proposed bridges, and unchanged farther downstream with the proposed bridge. 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 higher water surface elevations upstream and lower water surface elevations downstream. With the proposed bridge, the hydraulics have improved and this improvement results in approximately 1 ft higher water surface elevations between the location of the existing and proposed bridges. As shown in Figure 3~~Figure 4~~, this slight increase in the water surface elevation will result in no adverse impacts to any insurable structures or floodplain value.

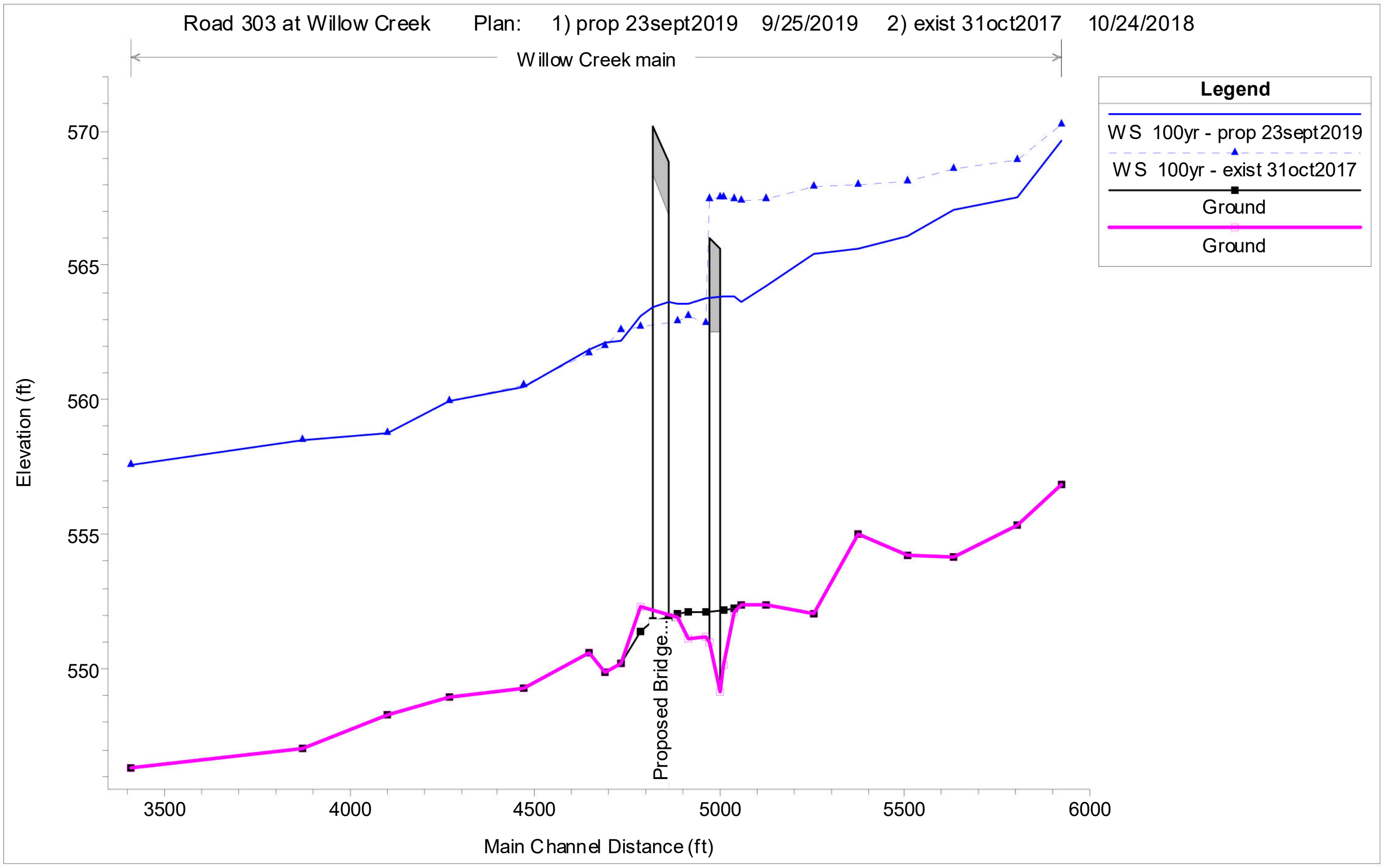

Figure 2. WSE comparison between existing (dashed) and proposed (solid) conditions for the 50-yr and 100-yr discharge.

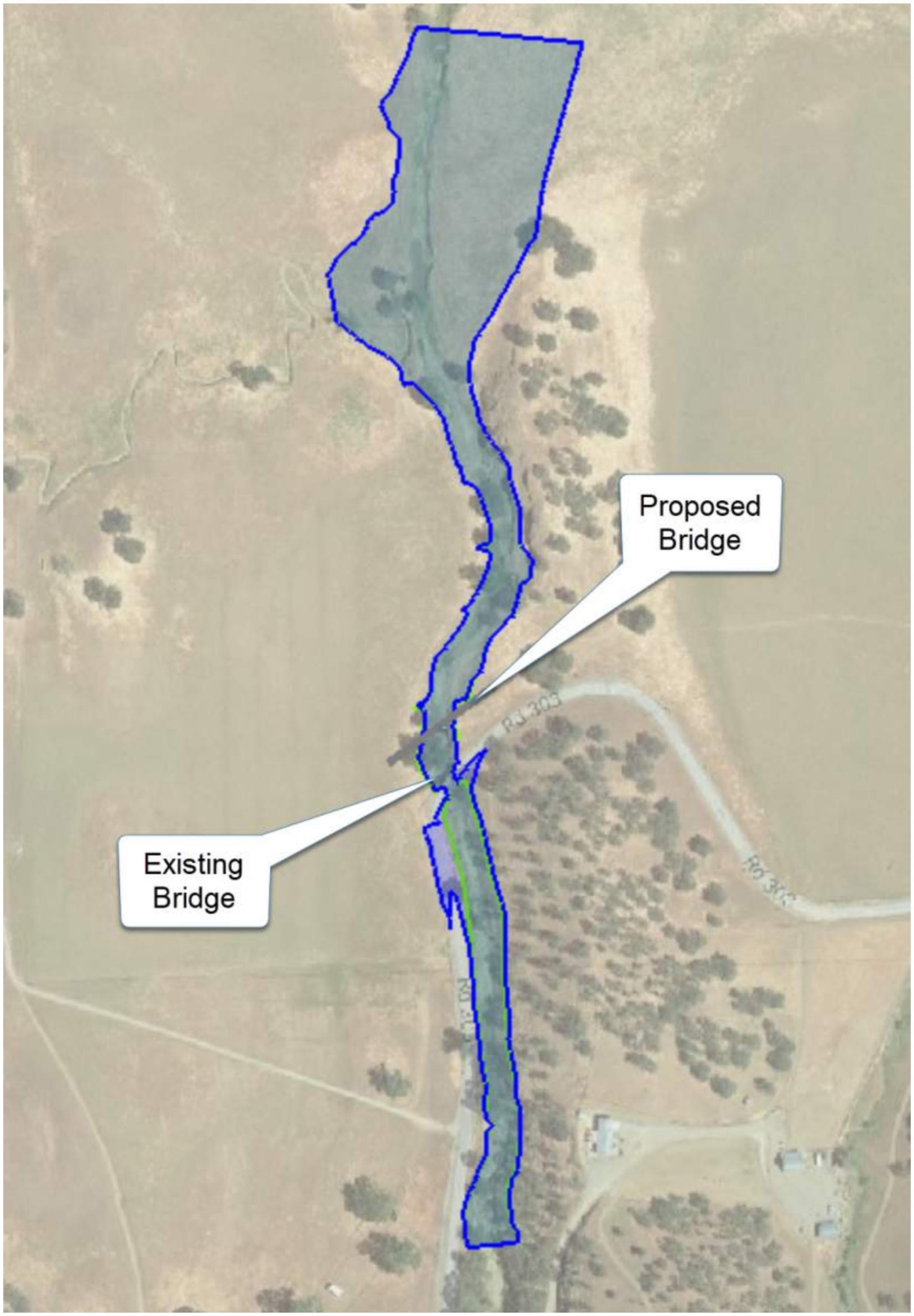

Figure 34. Water surface extents comparison between existing (blue) and proposed (green) conditions for the 100-yr discharge.

Potential Q100 backwater damages:

A. Residences? NO____X_____YES_________

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

B. Other Bldgs? NO____X_____YES_________

There are no buildings adjacent to the creek that are impacted by 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 4 feet upstream and only increased 1 ft between the existing and proposed bridges 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___X_____YES__________
B. Emergency vehicle access?	NO__________YES___X_____
C. Practicable detour available?	NO__________YES___X_____
D. School bus or mail route?	NO__________YES __X_____

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	$______0______
B	Property	$______0______
	Total	$______0______

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 303_____P.M.___Federal-Aid Project Number: BRLO 5911(059)
Project ID____Bridge No. 11C0163

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.

____ *Date* __________________
District Hydraulic Engineer *(capital and 'on' system projects)*

____ *Date* __________________
Local Agency/Consulting Hydraulic Engineer *(local assistance projects)*

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:

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

____ *Date* __________________
Local Agency Project Engineer (*local assistance projects)*

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.

____ *Date* __________________
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).*

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.

____ *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.

APPENDIX H- SUMMARY FLOODPLAIN ENCROACHMENT REPORT

SUMMARY FLOODPLAIN ENCROACHMENT REPORT

Dist.____3____Co.____Glenn____Rte.____County Road 303______K.P. ____________
Federal-Aid Project Number: BRLO 5911(059)
Project No.: __________________________ Bridge No._____11C0163

Limits:

The County proposes to replace the existing structure (11C0163) on County Road 303 over the South Fork of Willow Creek and construct the minimum approach work to accommodate the project. This will consist of approximately 1,000 feet of roadway reconstruction at the northerly approach and 500-ft on the southerly approach to the bridge. The bridge and roadway are being shifted approximately 100-feet downstream to bring the roadway geometrics up current design standards *(AASHTO Very Low Volume Roads, ADT ≤ 400)*. The new structure is anticipated to be a 1-span, cast-in-place, post-tensioned, box girder bridge.

Floodplain Description:

The South Fork of Willow Creek flows northerly through the project site through the central part of Glenn County (County). It drains an approximate 14 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 60 ft to 100 ft through the project reach. The channel is sparsely vegetated and the overbanks are 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 by FEMA.

		No	Yes
1.	Is the proposed action a longitudinal encroachment of the base floodplain? *The proposed bridge is not a longitudinal encroachment.*	x	
2.	Are the risks associated with the implementation of the proposed action significant? *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 74 feet longer, thus improving the hydraulics through the proposed structure.*	x	
3.	Will the proposed action support probable incompatible floodplain development? *The proposed bridge replacement will make the bridge 74 feet longer and will not support incompatible floodplain development.*	x	
4.	Are there any significant impacts on natural and beneficial floodplain values? *The proposed construction will have only minor impact to the existing riparian habitat in the creek at the bridge site.*	x	

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.