DRAFT

Feasibility Study

Rumsey Bridge Project
County Project No. 4576
Federal Project No. BRLO 5922-(077)
Bridge No. 22C-0003

Prepared For:

Yolo County
Public Works Division

Prepared By:

QUINCY ENGINEERING

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EXECUTIVE SUMMARY

The Rumsey Bridge is a 311 foot long structure that provides access over Cache Creek. The bridge was built in 1930, and lengthened in 1949. The original bridge consists of concrete through arch spans that are historic in nature. Quincy Engineering (Quincy) has been commissioned to evaluate the seismic and scour vulnerability of the existing bridge and perform a feasibility study to evaluate viable bridge rehabilitation and replacement alternatives. Following approval of the preferred alternative by Caltrans, Quincy will develop Plans, Specifications, and Estimate for construction.

Results from a preliminary seismic assessment indicate that most of the arch span members are severely deficient and are incapable of resisting forces from the design earthquake. To meet current design standards, the bridge must be able to remain standing after the design earthquake, defined as a 1,000 year return period earthquake. In another words, a bridge should be able to withstand an earthquake that has 5 percent probability of occurrence in a 50-year period. For the Rumsey site, the nearby Mysterious Ridge fault could produce an Earthquake up to a 7.0 Maximum Moment Magnitude. Due to its deficiencies, the Rumsey Bridge will likely collapse under a much lower earthquake that could occur on a much more frequent basis. The bridge has also been classified by Caltrans as scour critical, meaning it is vulnerable to collapse during extreme flows in the creek. Consequently, significant retrofitting of the existing structure is required to make the bridge resilient to both seismic and high flow events in the creek. Simply doing nothing has been rejected and the bridge must either be strengthened or replaced.

Given the seismic and hydraulic vulnerabilities, and the structural deterioration of the bridge, five build alternatives and a Do Nothing alternative have been identified and evaluated for either the retrofit/rehabilitation or replacement of the bridge. Retrofit is being considered because the bridge is eligible for inclusion into the national historic record.

Alternative 1 – Retrofit/Rehabilitation of existing bridge

Alternative 2 – Cast-in-Place (CIP) Concrete Box Girder Bridge (keep existing bridge)

Alternative 3 – CIP Concrete Box Girder Bridge (remove existing bridge)

Alternative 4 – New CIP Concrete Box Girder Bridge on Downstream Alignment (Rejected)

Alternative 5 – New CIP Concrete Box Girder Bridge on the Existing Alignment (Rejected)

Alternative 6 – Do Nothing (Rejected)

Alternatives 4, 5, and 6 were not considered viable for various reasons. As a result, detailed analyses of these alternatives were not completed.

The cost to retrofit/rehabilitate the existing bridge is much higher than the cost of a replacement structure. Because of the age of the existing bridge and the high cost to retrofit/rehabilitate it, and to assist with evaluating the various other alternatives, a life cycle cost analysis was completed for the first three alternatives. These costs, along with the advantages and disadvantage for each alternative are provided within this Feasibility Study for use in selecting the appropriate alternative.

Alterative	Advantages	Disadvantages
Alternative 1 Retrofit & Rehabilitation Existing Bridge $10,800,000	Provides safe river crossing	Requires extensive superstructure retrofit
	Preserves historical significance & aesthetics of the original Rumsey Bridge (although retrofitted details would have visual impacts)	Requires extensive substructure retrofit
		High bridge retrofit & rehabilitation cost
		High risk of construction cost overruns while working on existing historic structure
		Lower design life than replacement alterative
		Higher maintenance costs to maintain fiber wrap and seismic retrofit materials
	Lower permanent Right-of-Way impact	More impact on traffic during construction
		Temporary detour requirement
		Does not improve hydraulic conveyance and address scour vulnerability
		Not designed to current vehicular loading requirements
		Non-standard roadway width
Alternative 2 Conventional CIP Concrete Box Girder Replacement & Close existing bridge $3,900,000	Provides separate safe river crossing	Does not address the hydraulic performance under the existing bridge
	Provides a long term, low maintenance structure	Existing bridge is still vulnerable to hydraulic and seismic events
	Improved hydraulic capacity by raising the profile and reducing supports in the creek for the new bridge (does not improve the hydraulic capacity for the existing bridge)	Public safety at risk if existing bridge collapses while in use
	Traffic would use the existing structure during construction	High liability risk to the County
	Improved intersection geometry with SR16 because the skew angle is reduced	No funding mechanism for future maintenance or removal, after collapse of the original non-retrofitted bridge
	Improved location of Abutment 1	More Right-of-Way and environmental impacts
	Preserves existing bridge aesthetics (although new structure would be a visual impact)	
	Lowest construction cost	
Alternative 3 Conventional CIP Concrete Box Girder Replacement, Remove existing bridge $4,500,000	Provides one safe river crossing	Loss to the local community who values the existing bridge aesthetics
	Provides a long term, low maintenance structure	More Right-of-Way and environmental impacts
	Improved hydraulic capacity by raising the profile and reducing supports in the creek	
	Traffic would use the existing structure during construction	
	Improved intersection geometry with SR16 because the skew angle is reduced	
	Improved location of Abutment 1	
	Lower construction cost compared to retrofit alternative	

Costs do not include right of way acquisition, professional engineering, construction support or construction costs. Professional engineering includes Plan Specifications & Estimates preparation (PS&E).

1. INTRODUCTION

Rumsey Bridge viewed south-west from Abutment 5

Yolo County is proposing to rehabilitate or replace the Rumsey Bridge. The Rumsey Bridge is located on County Road 41 (CR 41) over Cache Creek. County Road 41 is located near the town of Rumsey off of State Route 16 (SR 16).

The Rumsey Bridge is classified by Caltrans as structurally deficient (sufficiency rating of 37.7 out of 100). As a result of this classification, the bridge is eligible for rehabilitation or replacement under the Highway Bridge Program (HBP) administered by Caltrans. Caltrans and FHWA have authorized federal funds for the preliminary engineering phase of the project. For this initial phase of the project, HBP funds will reimburse 88.53% of the project costs, with the remaining 11.47% funded by toll credits, a relatively new Caltrans-administered program available for local agency bridges only since 2010. Prior to 2010, federally funded County bridge projects required an 11.47% local match; in the early 2000's the County match requirement for bridge projects was 20%. Until the passage of the most recent federal transportation bill (MAP-21) in 2012, there was a separate federal funding program for local bridges, however MAP-21 eliminated this program, and the use of federal funds for County bridge projects in California is now dependent on State policy. The recent availability of toll credits, and changes to funding programs over time, highlight that the funding environment is not static. The current availability of toll credits, along with current state policy to utilize federal funds to continue the Caltrans HBP, both of which can and will change over time, are significant considerations in regards to the need to proceed in correcting the deficiencies in what will be an expensive local bridge project. Yolo County does not otherwise have local funds to complete such projects if federal funding is eliminated, nor even provide the local match that would have been required 4 years ago, and that may again be required in the future.

Due to the potential historic nature of the Rumsey Bridge, the environmental process is expected to be lengthy. The County has requested a feasibility study in order to determine viable alternatives, historical impacts, and relative costs for the rehabilitation and replacement alternatives. Lifecycle replacement costs have been developed for each alternative in order to determine which alternative provides the best economic value to the public.

Quincy has developed and evaluated three alternatives as described below:

Alternative 1 – Retrofit/Rehabilitation of Existing Bridge

Alternative 2 – Cast-in-Place (CIP) Concrete Box Girder Bridge (Keep existing bridge)

Alternative 3 – CIP Concrete Box Girder Bridge (Remove existing bridge)

Three other alternatives were considered but each was deemed to be infeasible. Therefore detailed analyses of these alternatives were not completed. These alternative are described below.

Alternative 4 – New CIP Concrete Box Girder Bridge on Downstream Alignment (Rejected)

Alternative 5 – New CIP Concrete Box Girder Bridge on the Existing Alignment (Rejected)

Alternative 6 – Do Nothing (Rejected)

This Feasibility Study also defines the design criteria to be used for the final design of the bridge and associated roadway.

2. BACKGROUND

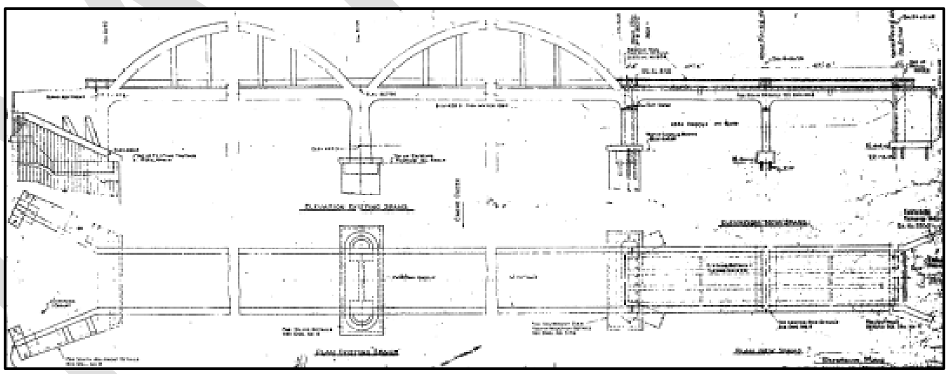
Existing Facility

The original Rumsey Bridge was built in 1930 and was constructed as a two-span reinforced concrete through tied arch. The bridge was founded on reinforced concrete piers and abutments.

Original main Arch spans

The original two-span structure consists of the main concrete tied-arch spans; each span is 108 feet long, for a total bridge length of 216 feet. (See picture to the left). The lowest bridge soffit elevation is located in the main arch spans with an elevation of approximately 434.94 (NAVD88) based on deck survey shots and calculated soffit elevation from the As-Built cross section of the superstructure.

In 1949, a flood damaged the north abutment, so the bridge was rehabilitated and extended with two additional cast-in-place (CIP) reinforced concrete "T"-girder spans on the north end of the bridge. The original Rumsey Bridge was also partially retrofitted with sheet piles to address scour issues at the interior pier and south abutment.



Rumsey Bridge Retrofit As-Built Plan

The added spans are each approximately 47.5 feet long and 3.5 feet deep. These modifications result in the current bridge length of approximately 311 feet. The newer constructed Pier 4 and Abutment 5 are both supported on driven H-piles. These newer spans were designed for H15-44 truck live loads, which does not meet the current *LRFD Bridge Design Specifications*. The steel piles supporting the newer spans are estimated to be 25± feet in length. However, actual pile tip elevations are unknown since they were not recorded on the As-Built drawings.

Rumsey Bridge Spans 3 and 4 – added in 1949 after flood damage to original bridge

In addition to the two additional north spans, both Pier 2 and Pier 3 were retrofitted with sheet piles to protect the pier foundations from scour. Notes on the General Plan indicate that sheet-piles may be as much as 20±ft below the bottom of the original footings. However, actual depths of sheet-pile tip elevations were also not recorded in the As-Built drawings so their exact length is not known.

Aerial view of Rumsey Bridge at the town of Rumsey
Abutment 1 at south-west side
Abutment 5 at north-east side

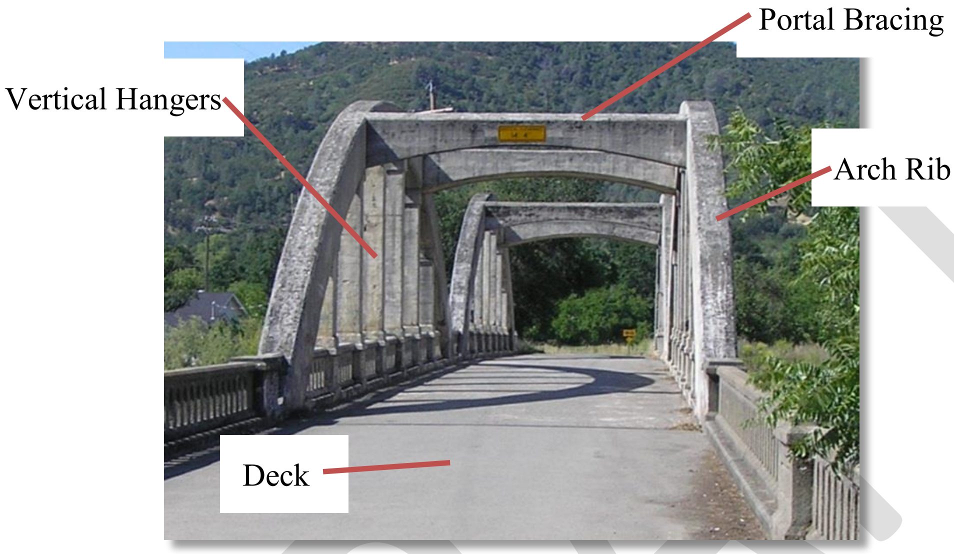
The current bridge configuration is shown in the adjacent aerial photograph. The horizontal roadway alignment is on a tangent that runs from southwest to northeast.
The two arch spans are located on the southwest side of the project site.

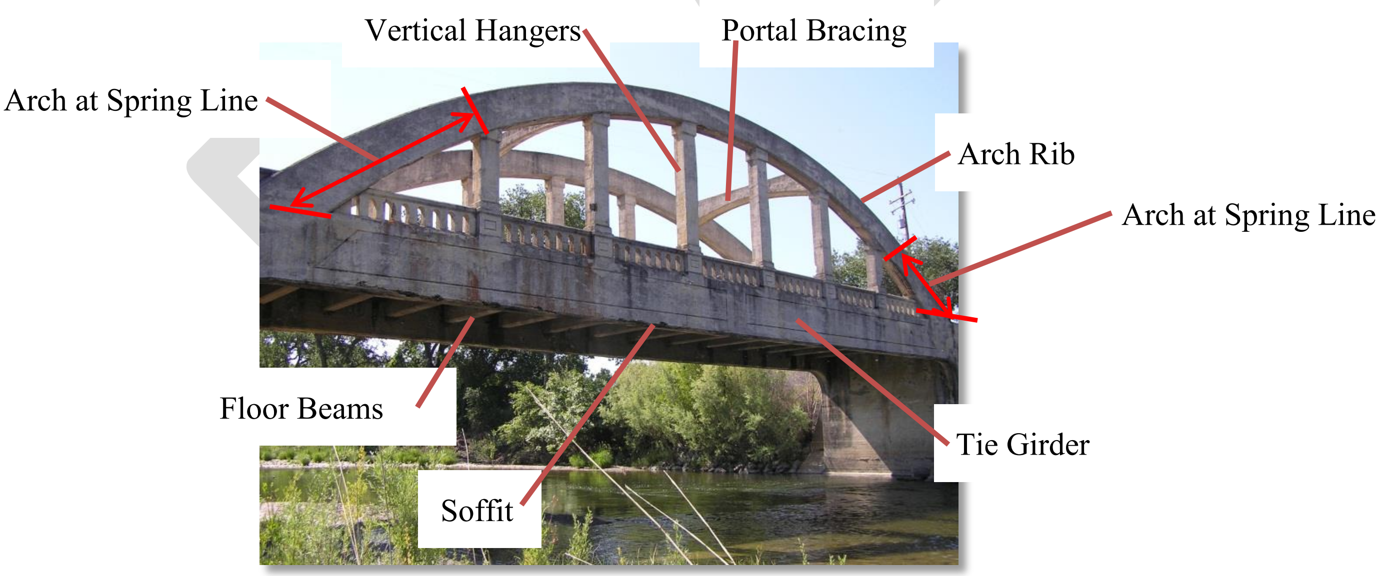
Both the arch spans and the approach spans are on a relatively flat vertical profile. Span 1 has an approximate grade just less than +1%. Spans 2, 3, and 4 have an approximate grade just less than –1%.

The existing roadway width matches the current bridge width of 20.5 feet. There is no pavement delineation/striping on either the road or the bridge.

Naming Convention

The tied-arch portion of the bridge contains a number of structural components that are unique to this style of bridge. Past reports have referred to these elements using different names. For the purpose of maintaining consistency in this report, the naming convention for these structural elements is illustrated in the photos below.

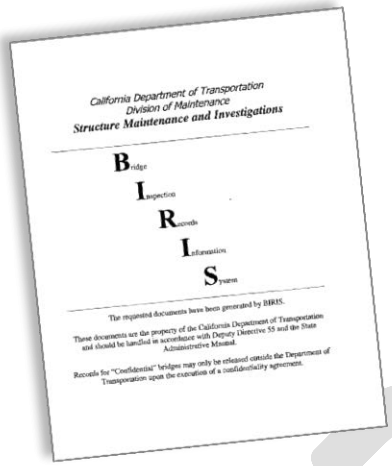




Project History & Bridge Inspection Reports

Over the years, Caltrans has completed evaluations of the bridge and produced Bridge Inspect Reports on a regular basis. One of their earliest inspection reports was prepared in 1964. Based on that report, the bridge had many exposed reinforcing bars and rock pockets in the transverse concrete Floor Beams and Tie Girder members of the arch spans. The report noted that "the mat of the steel was so close that it would be extremely difficult to do a good job getting concrete through or around it." The likely cause of the spalls was poor placement of the concrete and close spacing of the reinforcement, which prevented proper distribution of the aggregates and slurry during the concrete pour. The report further noted numerous cracks in the vertical hangers and scour around Pier 2.

Severe Tie Girder Spalls



California Department of Transportation
Division of Maintenance
Structure Maintenance and Investigations

Bridge
Inspection
Records
Information
System

The requested documents have been generated by BIRIS.

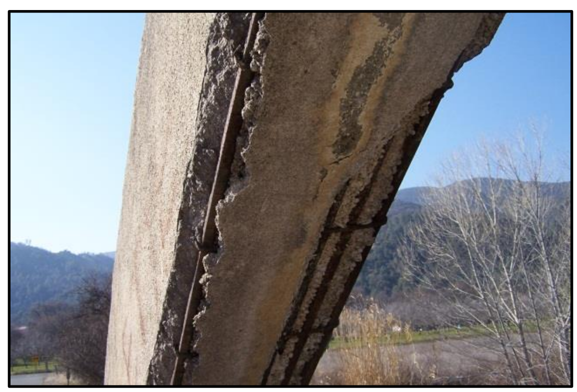
These documents are the property of the California Department of Transportation and should be handled in accordance with Deputy Directive 55 and the State Administrative Manual.

Records for "Confidential" bridges may only be released outside the Department of Transportation upon the execution of a confidentiality agreement.

Caltrans Maintenance Report

In the 1980 Caltrans inspection report, more reinforcing bars were reported as being exposed in the Tie Girders, Arch Rib, and Vertical Hangers. The deck was observed to be worn, with heavy transverse cracks, and exposed aggregate. The 1981, 1985, 1987, 1989, and 1991 Caltrans inspection reports continued to observe further deterioration of the loss of concrete cover and more exposure of reinforcing bars. The reinforcing bars have also showed signs of gradual corrosion. This deterioration will eventually lead to a reduction of allowable load capacity.

In 1992, a Caltrans Supplementary Bridge Report noted that the deterioration of the concrete and reinforcement had progressed to the point that roughly one-third of the total area of the Vertical Hangers and Arch Ribs had either spalled or had impending spalls over corroding rebar. Both Tie Girders had spalled or had impending spalls over the full length and width of the member surface at the bottoms of these girders. Some large spalls were also noted in the Floor Beams. Caltrans stated that the concrete on Rumsey Bridge, after 62 years of service, was low in strength. The aggregate and rebar were very poorly bonded possibly due to the use of a poorly graded river aggregate. (At that same time it was noted, on the other hand, that the approach span was in fairly good condition.) At that time, Caltrans recommended either replacement of the arch spans or extensive repairs.



Severe Arch Rib Spalls

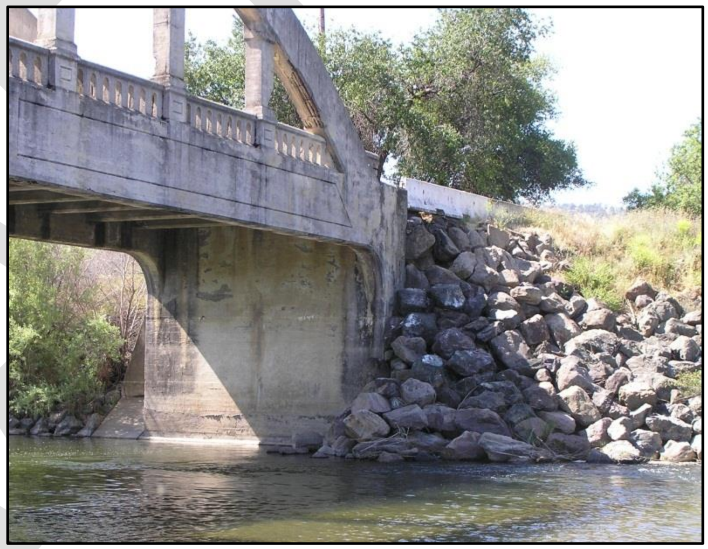
In that same year (1992), Yolo County applied for federal Highway Bridge Replacement and Rehabilitation Program funding to repair concrete spalling identified in Caltrans inspection reports. Caltrans responded that "All major structural deficiencies must be addressed as a part of the rehabilitation project.", and committed to fund rehabilitation only up to the estimated total cost of a replacement bridge, estimated at the time to be $1,080,000. The requirement to address all structural deficiencies, plus the dollar limit on federal participation at the cost of a replacement structure, along with the lack of 20% County match funding for the more extensive federal requirements, resulted in the County abandoning the spalling repair project.

Severe Vertical Hanger Spalls

The 1994 Caltrans inspection noted that the bridge seemed to show a gradual increase in the level of deterioration. The same recommendation as the previous reports was made to either repair or replace the bridge. Caltrans reported a phone conversation with the County's Assistant Director of Public Works, who indicated that the County did not have plans to repair or replace the structure at that time, due to lack of County funds.

In 1995 Yolo County applied for federal Transportation Enhancement Activity Program funds to rehabilitate the bridge but was not successful.

The 1996 inspection report documented further structural deterioration. After a storm event in 1995, the Abutment 1 upstream wingwall and approach rail were undermined and washed away. After the storm event, large rock slope protection (RSP) was placed to stabilize the embankment, but nothing was done to repair the washed-out wingwall (see photos). In addition to the RSP, erosion countermeasures also included installation of rip-rap spur dikes 800± feet upstream of the bridge site. K-rail was placed along the roadway on top of the RSP. The repair was generally satisfactory except that there is a six foot gap between the end of the K-rail and the bridge rail. Due to this gap in the bridge rail, Caltrans recommended the temporary K-rail be replaced with metal beam guard rail.



Abutment 1 Left wingwall washed out and backfilled with RSP

K-Rail placed on top of Abutment 1 RSP backfill

The 1999 inspection report referenced two reports on bridge foundation scour and bridge seismic condition prepared as part of the Local Agency Seismic Retrofit Program. At that time, during an underwater investigation up to 9 feet of sheet piling was found to be exposed below the bottom of the footing on the downstream side of Abutment 1. Sheet piles were also exposed below the wingwall and wingwall footing. Approximately 4 feet of the footing at Pier 2 was also exposed. Caltrans therefore recommended making repairs to the bridge to address the scour at these supports.

In the 2000 inspection report, Caltrans stated that scour was still a concern and recommended that the County take corrective measures to mitigate scour vulnerabilities that could threaten the stability of the bridge. At that time, the bridge's seismic vulnerability was also raised as a serious concern. These two issues were reinforced in a report by Northwest Hydraulic Consultants entitled *Rumsey Bridge Investigation Report* dated October 2, 1995, and a report entitled *Structural Evaluation for the Rumsey Bridge* by an unknown author.

Existing Spur Dike Locations

In 2001, a detailed report by Caltrans Structures Hydraulics Branch, resulted in the change to the National Bridge Inventory Scour Status Code (Item 113) from a 6 to a 3. This meant the condition had changed from "[bridge] scour calculation/evaluation [had] not been made" to "[the] Bridge is scour critical; bridge foundations determined to be unstable for assessed or calculated scour conditions." The bridge was rated scour critical because Abutment 1 was considered at risk due to the potential vulnerability of the RSP placed after the 1995 storm event. If the abutment RSP were lost, the instability of the abutment could endanger the stability of the entire structure. Furthermore, if the thalweg of the creek were to migrate toward Pier 2, the bottom of the creek could reach below the protective sheet piling, thereby threatening that pier's stability as well. Based on the 1949 repair plans, the piles are 25' long, and the sheet piling constructed to protect against scour is only 20' long. During a scour event, because the sheet piling is relatively short, there is a high probability the sheet piling would be washed away. The Caltrans Structures Hydraulics Branch recommended that the County provide appropriate scour countermeasures to mitigate these scour problems.

By 2002, piles of concrete debris were found below the Tie Girders in Spans 1 and 2 near Pier 2, where existing spalls continued to deteriorate along the soffit of the Tie Girders.

Debris built up Pier 2

The 2004 inspection report documented that the Abutment 1 RSP comprised 500 lb to 750 lb rock, while the left side comprised 100 lb to 200 lb rock. Despite the size of the RSP, Caltrans reiterated that the RSP placed at Abutment 1 was insufficient. The condition of the structure noted earlier also continued to deteriorate.

In summary, the Caltrans inspection reports document that the concrete in the tied-arch section of the Rumsey Bridge has been deteriorating more than would be expected for at least the past 50 years. It is likely that this process began soon after the bridge was constructed in 1930. The most recent October 2013 Caltrans inspection report classifies the bridge as "Scour Critical", "Structurally Deficient", with a "Sufficiency Rating" of 37.7 out of 100. The inspection report recommends that corrective measures be taken to avoid the threat that scours poses to the stability of the structure. It recommends that the structure be scheduled for extensive rehabilitation of the arches and girders, and that consideration should be given to replacing the structure.

Bridge Scour Evaluation Plan of Action and Memorandums

In 2005 Yolo County was notified by Caltrans that nine county bridges classified as scour critical, including Rumsey Bridge, required a Plan of Action under federal regulations. The County circulated a Request for Proposals, and selected Quincy Engineering to evaluate the bridges and prepare the Bridge Scour Evaluation - Plan of Action reports required by Caltrans. Quincy Engineering, along with hydraulic specialist Avila and Associates, and geotechnical specialist Taber Consultants, developed a Plan of Action for Rumsey Bridge dated 2007.

2006 Team Field Visit

The Plan of Action scope of work included a review of previous Inspection Reports and As-Built plans, and a site visit and field review meeting by the Quincy Team and Yolo County to observe the condition of the bridge and its surroundings. The Plan of Action entailed qualitative hydraulic, geotechnical, and structural assessments based on judgment and experience.

During the team's field review in 2007, significant structural deterioration and structural deficiencies were observed as noted in the earlier Caltrans Bridge Inspection Reports. Due to the low amounts of confinement reinforcement in all the structural components, condition of the structure, and age of construction, the structure was deemed vulnerable to a significant seismic event.

Scour Plan of Action (POA)

The team also observed significant scour issues. The spur dike, constructed in 2000, intended to reduce embankment and sheet-pile erosion and protect the RSP at Abutment 1, appeared to be marginally effective. The team therefore concluded that the abutment would continue to be vulnerable to scour in a future significant storm event. If the abutment were to be undercut by scour, the abutment footing could settle significantly, which could compromise global structural stability. The team also noted that the lateral resistance capacity of the structure might not be sufficient to resist the lateral stream flow forces in a large storm event that could scour the soil around the sheet piles.

The Quincy Engineering Team provided the following short-term and long-term recommendations:

Structural Assessment Memo

Short-term:

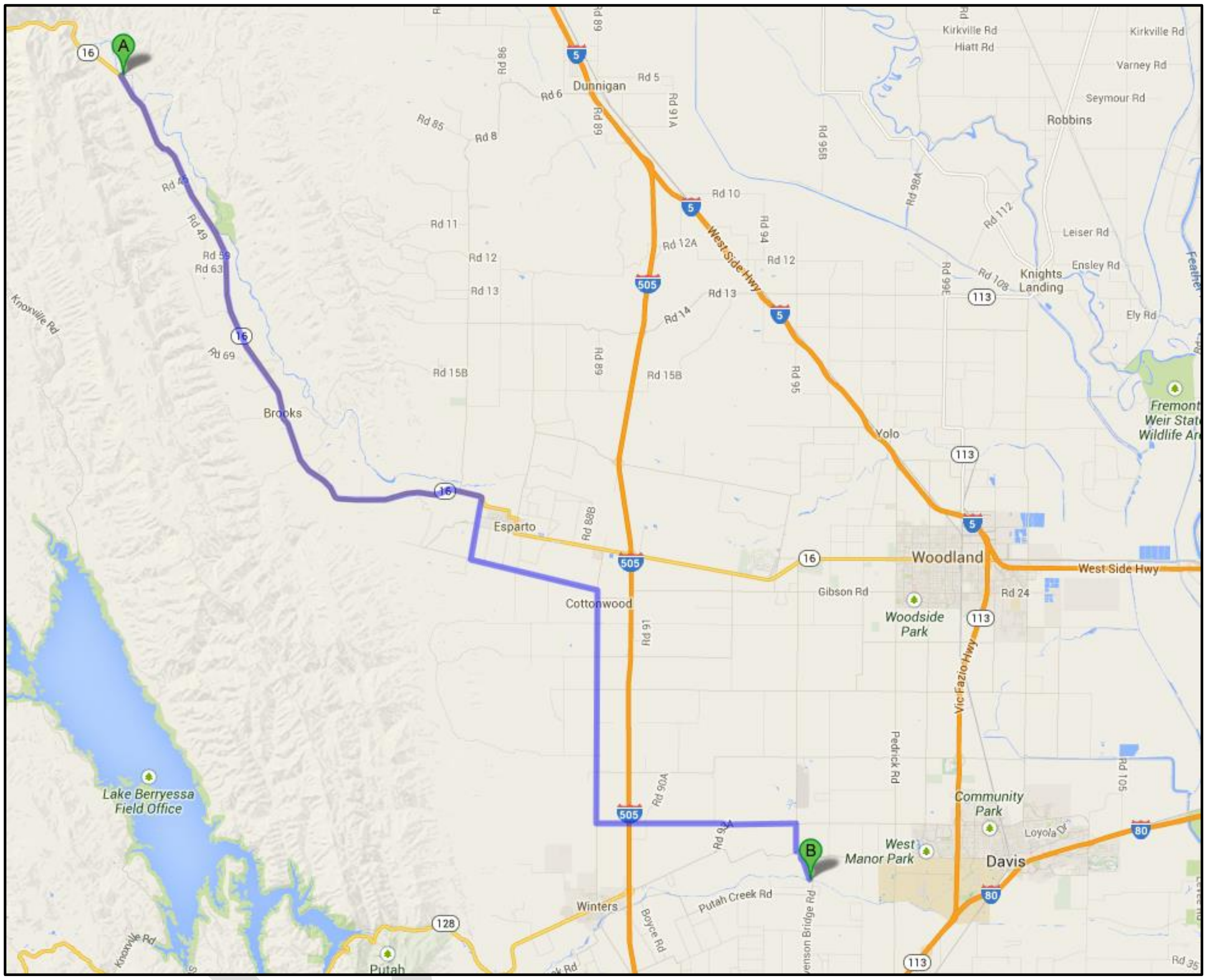
- Monitor the structure during and after significant storm events.
- Consider installing float indicators in the creekbed that, if they were to float to the surface, would indicate that the supports were being scoured and trigger to County to close the bridge.
- Continue efforts to evaluate and implement rehabilitation or replacement of the structure.

Long-term:

- Replace the structure.

Similar Structure (Stevenson Bridge) Study

There is another bridge in the general area that is nearly identical to the Rumsey Bridge, the Stevenson Bridge (23C-0092), also known as the Graffiti Bridge, which is located approximately 40 miles away between Yolo County and Solano County. The Map below shows the Rumsey Bridge at location "A" and the Stevenson Putah Creek Bridge at location "B".



Google Map of Rumsey Bridge at Location A and the similar structure Stevenson Bridge at Location B

The Stevenson Bridge was reviewed and its condition evaluated for similarities that might be useful in the assessment of the condition and vulnerabilities of the Rumsey Bridge. The main superstructure arch spans of the Stevenson Bridge are nearly identical to that of the Rumsey Bridge. Both of the main tied-arch spans for these bridges were actually constructed using the same design drawings. In fact, portions of the Rumsey Bridge's microfilmed As-Built plans are actually titled the Stevenson Putah Creek Bridge. Because these bridges are nearly identical, they likely share similar vulnerabilities.

Stevenson Bridge, standing at Abutment 1, looking north

Rumsey Bridge, standing at end of bridge, T-girder spans side, looking south-west

The Stevenson Bridge is a four-span bridge with two arches spanning between Piers 2 and 4, and a T-beam approach span on either end of the bridge. As stated earlier, the Rumsey Bridge was originally a two-span bridge comprised of just the two arch spans, with the T–beam approach spans constructed north of the arch spans at a later time and of a different design than the Stevenson Bridge approach spans. Therefore, the arch spans are likely similar between the two bridges, while the approach spans may have fewer similarities.

As shown in the following photos, another noticeable difference between the two bridges is that the Stevenson Bridge has four Portal Braces per arch, while the Rumsey Bridge has just two Portal Braces per arch. While they share the same design drawing that calls for only two Portal Braces per arch, two additional Portal Bracings were added to the Stevenson Bridge during construction. Portal Bracing serves as the primary lateral force resisting element when the bridge experiences lateral loads from wind or seismic events. Because it has two additional portal braces, the Stevenson Bridge will have more lateral resistance that the Rumsey Bridge and therefore is likely more seismically resilient to lateral loads than the Rumsey Bridge.

Stevenson Bridge

The Rumsey Bridge (shown below) shows several deteriorated members that could result in structural deficiencies, while the Stevenson Bridge (shown above) appears to be in better condition, with few if any concrete surface spalls and rusting rebar that could exacerbate structural vulnerabilities.

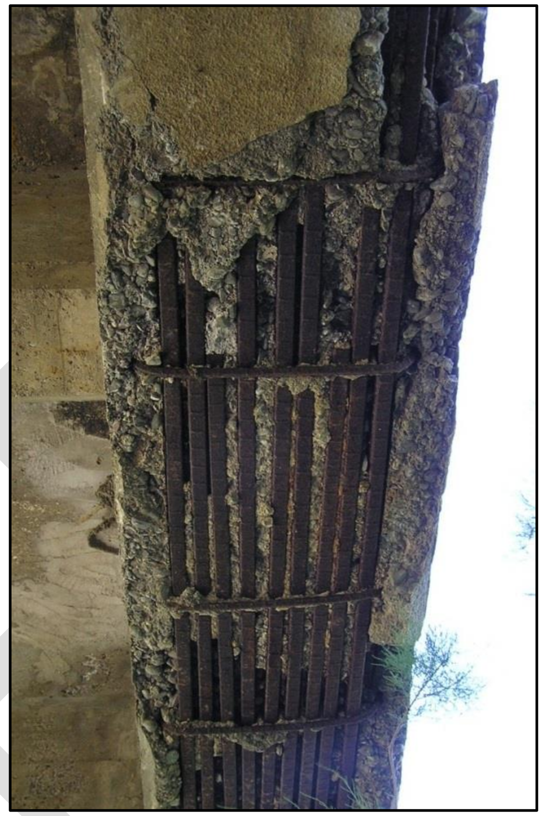
Rumsey Bridge

Stevenson Bridge

These photos from the roadway of the two bridges also show the Rumsey Bridge deterioration (below) that appears to be much less of an issue for the Stevenson Bridge (above).

Rumsey Bridge

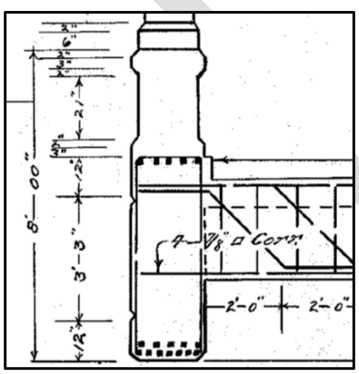
The most significant difference in the condition of the two structures is in the concrete surfaces. As seen in the previous photos, the Rumsey Bridge has extensive spalling on the Arch Rib, Vertical Hanger, and Tie Girder elements, whereas the Stevenson Bridge does not. Some locations on the Rumsey Bridge such as the Tie Girder soffit (see adjacent photo) are in such a deteriorated state that the surface concrete has spalled off over almost the full length and width of the element.



Rumsey Bridge Tie Girder Soffit Severely Spalled

For two very similar structures, these differences raise the question of why the Stevenson Bridge, which was built seven years earlier and with the same design drawings, would be in significantly better condition than the Rumsey Bridge. One answer can be found in the Tie Girder. This photo shows nine square reinforcing bars while, the design drawing only call out for seven bars in the bottom mat, followed by five bars above the seven bars as the second layer of the bottom reinforcement. Regardless of the cause for the additional reinforcement, the result was poor concrete placement where the larger aggregate could not pass through the tightly spaced bars. Only small aggregate and fine sand were able to pass below the rebar to provide concrete cover. As a consequence, the poorly graded concrete did not bond well to the rebar and has spalled overtime.

Other structural elements also have spalled due to similar causes. Based on the field observation where the Arch Rib has spalled, the Arch Rib has seven bars as opposed to the six per design drawing. The rebar congestion along with poor concrete mix and narrow clear cover were likely the causes of Arch Rib cover spall. The Vertical Hanger's cover spall also appears to be related to the narrow clear cover and poor concrete aggregate mix.



Rumsey Bridge Tie Girder As-Built

Other possible causes for concrete spalling:

- Insufficient clear cover – without sufficient clear cover, larger aggregate cannot fill the clear cover space, resulting in poor concrete bond.
- Corroded reinforcement – if the reinforcing steel were already corroded before concrete was placed, then the corroded steel would not bond well with concrete.
- Poor concrete mix quality or design – without proper gradation, aggregate cleanness, and sufficient shape factor, concrete will lose strength and functionality quickly over time.
- Reactive aggregate – in most concrete, aggregates are more or less chemically inert. However, some aggregates react with alkaline hydroxides in concrete causing expansion and cracking over a period of many years.

Based on the Bridge Inspection Reports, the Stevenson Bridge has a "Satisfactory Condition" (6 out of 9) rating for the superstructure, compared to the Rumsey Bridge's rating of "Serious Condition"

(3 out of 9). For the overall Sufficiency Rating, which is based on a scale of 100 points, Stevenson was rated 46 and Rumsey was rated 37.7. The difference between the two is significantly influenced by the deteriorated condition of the concrete, surface spalling, and associated corroding rebar.

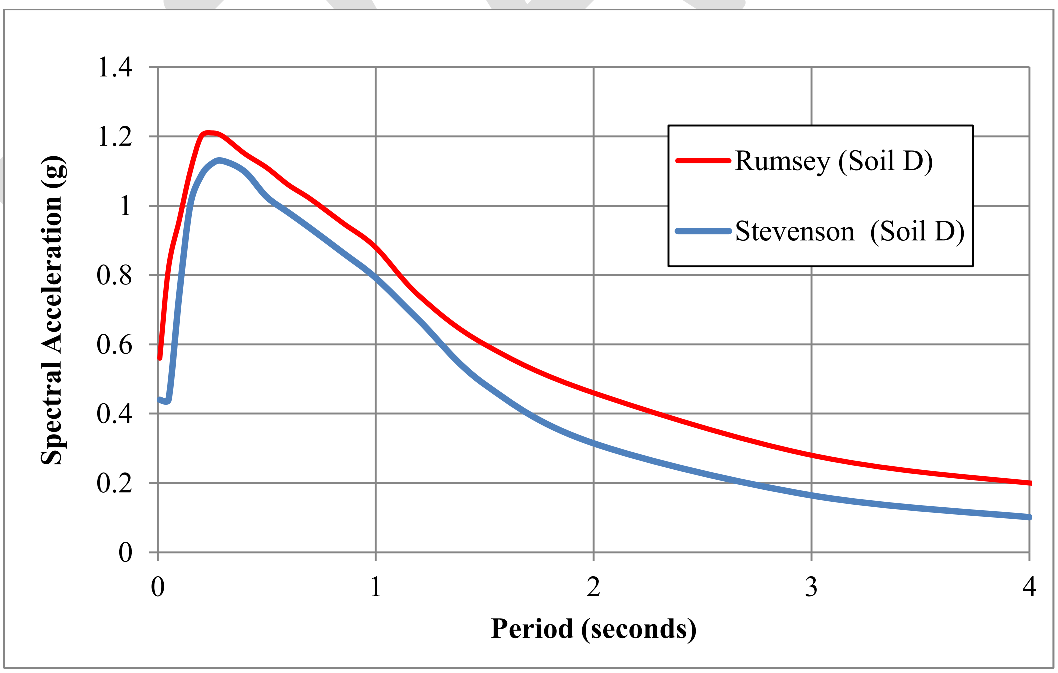
In 2007, TRC Imbsen (Imbsen) prepared a Feasibility Study for the Stevenson Bridge for the Solano County Department of Resource Management. At the time of the report, Imbsen determined the following:

- The bridge was capable of carrying legal loads (HS 20-44).
- The bridge had already passed its probable design life span.
- The bridge had started showing signs of deterioration.
- The load carrying capacity of the bridge would have to be reduced over time.

Imbsen also determined that the bridge's main load carrying members would be loaded beyond their capacities and the bridge would likely collapse during a design level earthquake. As part of their Feasibility Study, Imbsen evaluated two rehabilitation/retrofit options and one replacement option.

In an effort to gain a better understanding of potential issues with the Rumsey Bridge, Quincy has reviewed Imben's Feasibility Report, Design Calculations, Geotechnical Report, and Petrographic Examination Report, and has compared the structural characteristics of the Stevenson Bridge to those of the Rumsey Bridge. The follow observations and assessments were made:

- While the bridges have a nearly identical design, they were constructed at very different sites. The Acceleration Response Spectrum (ARS) curve at the Rumsey Bridge site is higher than the ARS curve used in the Stevenson Bridge Feasibility Report, as shown in the plot below. This means higher seismic demands will be imposed on the Rumsey Bridge during an earthquake.

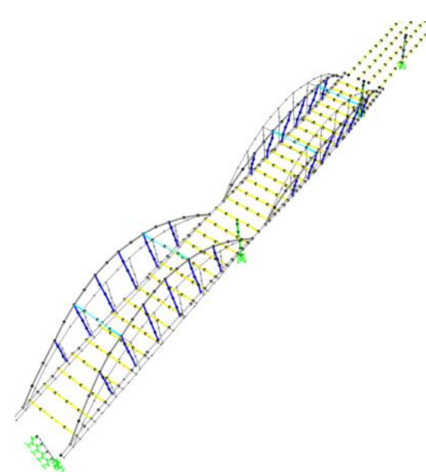


ARS curve comparison

- The hydraulic behavior along Cache Creek (for Rumsey Bridge) is more severe than along Putah Creek (for Stevenson Bridge). The Rumsey Bridge site has a long history of scour issues and lateral channel migration that greatly complicates the proposed rehabilitated/retrofit alternative. Putah Creek, on the other hand has a more predictable and regulated flow that is controlled by a dam upstream from the Stevenson Bridge.
- As previously discussed, the structural condition of the Stevenson Bridge is much better than that of the Rumsey Bridge.
- Based on Quincy's seismic assessment of the Rumsey Bridge, the member force demands far exceed the member capacities. Based on these high Demand-to-Capacity (D/C) ratios, an increase or decrease in the structure concrete strength will not greatly alter the vulnerability of the various bridge members. The baseline assessment model assumed a 2500 psi concrete strength. To verify that the concrete strength is not a significant factor, sensitivity analyses were performed using 1000 psi, 2000 psi and 4000 psi for the concrete strength. This evaluation showed insignificant changes to the force D/C ratios (All D/C ratios were well above 1.0), meaning members were still greatly overloaded. As a result of these sensitivity analyses, Non Destructive Testing (NDT) for material strengths for the Rumsey Bridge is not recommended at this time. In the final design phase, if the retrofit/rehabilitation alternative is chosen, NDT may be justified to potentially reduce the magnitude of repairs to a given member.

  NDT was performed for the Stevenson Bridge. This can be warranted if member demands are close to the member's capacity, in which case a small change in strength can affect whether or not a member needs to be repaired or strengthened. As stated earlier, this is not the case for the Rumsey Bridge so early NDT is not needed.

  Furthermore, the structural analysis performed for the Stevenson Bridge Feasibility Study also used a concrete compressive strength of 2,500 psi. According to the study, the concrete core samples taken from the Stevenson Bridge and tested by ASTM C 42 yielded compressive strengths ranging from 1,920 psi to 4,920 psi, with an average of 3,140 psi. Given that the concrete condition at the Rumsey Bridge is worse than the Stevenson Bridge, along with the sensitivity analysis described above, these results from Stevenson Bridge support the use of 2,500 psi as the concrete compressive strength in the Rumsey retrofit analysis.
- The Stevenson Bridge is supported on taller piers, which lead to a longer structural period of vibration. This means the bridge is more flexible and as such will attract lower seismic forces in the bridge.
  - One of the seismic retrofit strategies for the bridge is to remove the existing curtain wall between the columns of the piers, and then install fiber wrap around the remaining columns. This will accomplish two things.
    - Changing the substructure from a pier wall system to a multi-column bent system will make the substructure more flexible, further lengthening the period of the bridge and further reducing the seismic forces.



Frequency / Period Analysis
Rumsey Bridge

   - The fiber wrap around the columns will increase their ductility and ability to withstand seismic forces.

  This decrease in forces and increase in substructure strength provides a dual benefit, resulting in a great cost/benefit to the bridge.

- This same strategy does not provide the same benefit to the Rumsey Bridge because its structural period is near the peak of the ARS curve. This means that even though the period of the bridge would be increased, it could not be increased enough to dramatically reduce the forces in the bridge. That is, the modified period would still remain near the peak of the ARS curve, attracting high seismic forces.

In summary, the Stevenson Bridge is in better condition and has some site specific characteristics such as lower seismic effects and taller piers that make it a superior candidate for bridge rehabilitation/retrofit. While it is also feasible to retrofit the Rumsey Bridge, the cost will be significantly higher than the Stevenson Bridge because of its poor condition, site specific issues (scour, seismic demands) and its shorter, stiffer piers. The Stevenson Bridge project construction cost estimate, as programmed in the November 2013 Caltrans HBP list, is $6,372,000.

Nearby Bridge Project on Cache Creek (Guinda Bridge)

Approximately 5 miles downstream from the Rumsey Bridge lies the Guinda Bridge, carrying County Road 57 over Cache Creek. In 2010 the County completed construction of a new CIP box girder style bridge, to replace a steel truss bridge that had numerous structural deficiencies. The table below summarizes some key design features of the completed Guinda Bridge, along with corresponding preliminary design parameters as determined to date for the Rumsey Bridge. While these parameters will change if replacement is the selected alternative and the design is refined, the bridge sites have many similarities, and the Guinda Bridge project provides a reasonable benchmark of the cost and timing that could be expected if a CIP box girder replacement project was to be undertaken at the Rumsey Bridge site.

Parameter	Rumsey Bridge	Guinda Bridge
Total Span	311’ (existing)	370’, 3 spans
Design Scour Depth-Pier	27’	34.8’
Design Scour Depth-Abutment	18’	23’
Spectral Acceleration (seismic)	1.2	1.6
Depth of pile: pier 1 / pier 2	105’	98’/ 72’

Nine bids for the Guinda Bridge project were received, ranging from $1,723,000 to $2,480,000. The County awarded a contract to the low bidder, MCM Construction, Inc., and issued the Notice to Proceed on April 27, 2010. Work began at the site on May 12, 2010. The new bridge was completed in October 2010, and approaches were paved November 6, 2010, allowing the bridge to open to the public in the fall of 2010. The project was accepted by the Board of Supervisors on March 29, 2011, with a total construction contract cost of $1,767,900.

3. NEED AND PURPOSE

Problems, Deficiencies, Justifications

The Rumsey Bridge was originally constructed in 1930 and lengthened in 1949, and is classified by Caltrans as structurally deficient, with a sufficiency rating of 37.7 out of 100. As a result of this classification, the bridge is eligible for rehabilitation or replacement under the Highway Bridge Program (HBP) administered by Caltrans. In the past, the HBP program has required a local match of County funds ranging from 11.47% to 20%. Since 2010 Caltrans has allowed toll credits to be used in lieu of matching funds, effectively providing for 100% federal funding for local bridge projects. The toll credit program is significant because many local agencies (including Yolo County) do not have matching funds for large and expensive bridge projects. While the existing tied-arch bridge has provided a functional river crossing for the last 84 years, it is essentially at the end of its service life and this program may be one of the only funding mechanisms available to the County to rehabilitate or replace this bridge.

While the Rumsey Bridge was lengthened in 1949 in an effort to reduce scour impacts to the bridge, the longer bridge has continued to experience scour related problems. Abutment 1 was built at the outward bend of Cache Creek, and the river tends to attack to the outside of this curve during large storm events. As a result, this abutment, its roadway approach, and RSP are still vulnerable. As mentioned earlier, a storm in 1995 caused severe erosion behind this abutment and nearly took out the entire approach roadway as shown in the photo below. After the 1995 damage, Rock Slope Protection (RSP) was placed in attempt to stabilize the bridge from flood events.

Storm Damage behind Abutment 1 in 1995

Approach RSP Repair 1995

However, past field inspections revealed that the RSP had sunk below the bottom of footing at the downstream end of the abutment leaving an approximately 3.3 foot gap between the bottom of abutment footing and the bottom of the channel. In addition, if the RSP is swept away in a flood event, the creek could undermine the abutment footing, and the stream flow could apply significant lateral force to the abutment and remaining wingwall, potentially causing them to be damaged or to collapse.

At Pier 2, the scour depth is estimated between 21 feet and 27 feet without taking potential thalweg migration into consideration. The thalweg is the center of the channel defined by its deepest point and for dynamic rivers like Cache Creek, this deep point can move laterally within the creekbed over time. As-built plans indicate that the existing steel H-piles are 25± feet long and the sheet piles that encase the footing and

H-piles are only 20± feet long. If the thalweg moves closer to Pier 2, it could combine with anticipated scour to further lower the creekbed around the pier and pose a significant threat to its foundation. At a minimum, if the full scour were to occur, the sheet piles would be completely exposed and would wash away, and the H-piles would have only 5 feet of embedment at most. Under this condition, the piles would be unstable and the bridge would surely collapse. If the thalweg migrated toward the pier, all foundation elements would be completely exposed, again resulting in a collapse of this support and likely both arch spans. For these reasons, Pier 2 is also vulnerable to scour.

Due to the unstable embankment condition and the exposed footing at Abutment 1, and the lack of adequate scour protection at Pier 2, the Rumsey Bridge is scour critical. This means that one or more of its support are vulnerable to scour attack that could lead to the loss of support at one or more locations and the potential for a partial or total collapse of the bridge.

Not only is the existing structure vulnerable hydraulically, it is also vulnerable to seismic events. Due to the massive weight of the tied-arch superstructure and stiff pier wall substructure, the bridge attracts very large forces during an earthquake. These forces, combined with poor structural details in the superstructure (Arch Ribs, Vertical Hangers, and Tie Girders), make the bridge susceptible to collapse during a significant seismic event. See Chapter 5 of this report for more information on the seismic vulnerability of the bridge.

For the existing bridge to remain in place and remain serviceable and safe well into the future, it will require rehabilitation, seismic retrofitting, and foundation enhancements to make it resilient to extreme events like floods and earthquakes.

Seasonal Road Closed Sign

This bridge provides access from State Route 16 to the east side of Cache Creek for emergency vehicles, fire protection, and residences, with an average daily traffic of 60. County Road 41 extends northeast from the bridge to Colusa County, where the road becomes Sand Creek Road, which continues over the Coast Range to the Central Valley near Arbuckle. Approximately 1.3 miles northeast of the bridge, County Road 41 becomes dirt surfaced and is closed and not useable in the winter months, as shown on posted sign in the adjacent photo. Without the bridge to provide access between the town of Rumsey and the east side of the creek, the detour is 63 miles long (through the towns of Arbuckle and Williams), and is available only during the dry season.

Under the National Bridge Inventory, this structure has a Historical Bridge Inventory Category Rating of 2, meaning this bridge is eligible for the National Register of Historic Places. Consequently rehabilitation and replacement alternatives must consider the project's impacts to this historic resource.


Local Residence near Rumsey Bridge

This Feasibility Study documents the design and construction considerations for both the rehabilitation and replacement alternatives. The proposed project will improve public safety by providing a safe river crossing for all legal vehicles.

4. DESIGN AND CONSTRUCTION CONSIDERATIONS

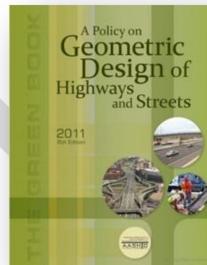
Design Criteria

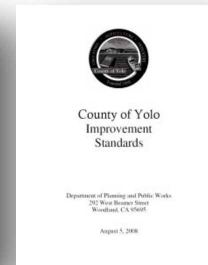
All alternatives have been evaluated based on the following criteria:

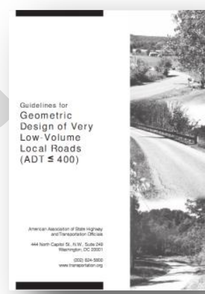
- Roadway Design

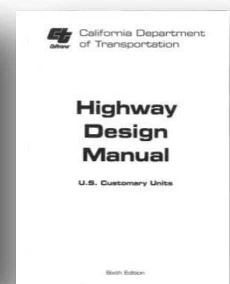
Several documents were used to determine the project design criteria including:

- County of Yolo *Improvement Standards* (County Standard) dated 2008,
- AASHTO's "*A Policy on Geometric Design of Highways and Streets*", 2011 Edition,
- AASHTO's "*Guidelines for Geometric Design of Very-Low Volume Local Roads*", and
- Caltrans "*Highway Design Manual*".









Where there are discrepancies between the design documents, the AASHTO standards shall be used as long as they do not worsen the existing condition. The summary of the project minimum design criteria are as follows:

Functional Classification = Local Rural with Level terrain

ADT = 60 (Year 2013); 335 (Year 2030)

Design Speed = 45 mph

In accordance with the requirements stated in AASHTO's "*A Policy on Geometric Design of Highways and Streets*", 2011 Edition and AASHTO's "*Low-Volume Guidelines*", 2011, the appropriate design speed for Local Rural road with a level terrain classification is 40 miles per hour for design volume less than 400 vehicles per day. Based on the *County Standard*, the rural unposted design speed is 65mph. The operating speed of this facility (CR 41) is 45 mph, and the adjacent facility (SR 16) operating speed is 55 mph. A design speed of 45 mph has been selected.

Maximum superelevation rate = 12% (emax=12%)

Maximum Grade = 5%

For Load Road, Level 45 mph AASHTO allows up to 7%.

However, 5% is selected to meet the ADT standards as pedestrians do utilize the bridge.

Lane Widths

The AASHTO requirements for the approach roadway for a Local Rural Road with a future ADT below 400 is two 10-foot travel lanes with 2-foot graded shoulders, for a total width of 24 feet.

At locations where metal beam guard railing and end treatments are provided, a 4-foot offset from the travel way to the barrier rail is recommended.

A County design exception would be required because the County Standard is two 12-foot lanes with 4-foot paved shoulders, for a total width of 32 feet.

For the Bridge Replacement Alternatives at a new upstream alignment:

The proposed roadway section is two 12-foot lanes with 2-foot paved and 4-foot AB graded shoulders.

The proposed bridge lane width and travel way width comprises two 12-foot lanes and 2-foot shoulders for a barrier to barrier clear width of 28 feet.

For the Bridge Retrofit/Rehabilitation Alternative:

Roadway improvement is not recommended since the existing bridge cannot be widened. The rehabilitation alternatives will require a design exception to keep the existing 20’ width as Yolo County *Improvement Standards* requires a wider width for rural streets.

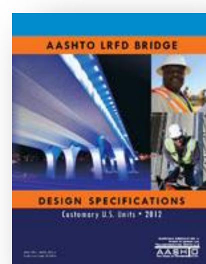
Cut Slopes = 2:1 (h:v)

Fill Slopes = 2:1 (h:v)

See the Design Criteria Memorandum in **Appendix A** for a comparison between County and AASHTO standards.

- Bridge Design

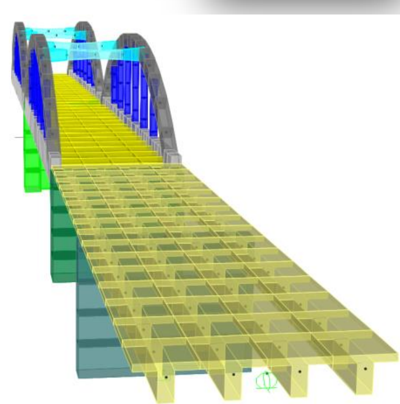
  Final bridge design will be performed in accordance with *AASHTO LRFD Bridge Design Specifications, Sixth Edition, and the latest Caltrans Amendments* (current version is dated November 2011). The latest updated versions of Caltrans bridge design manuals will also be utilized when applicable.



- Seismic Design

  Replacement bridge seismic design will be performed in accordance with the latest Caltrans *Seismic Design Criteria* (current version is Version 1.7 dated April 2013)

  For rehabilitation/retrofit of the existing bridge, seismic assessment and design will be based on a no-collapse criterion. A 3-dimentional finite-element global model will be created to assess seismic force, displacement, and rotation demands. Local



Global 3D Model

nonlinear moment-curvature models for each element type will be used to determine local member force, displacement, and rotation capacities. See Section 5. EXISTING BRIDGE ASSESSMENT & RETROFIT STRATEGY of this report for an in-depth retrofit methodology.

- Hydraulic Performance

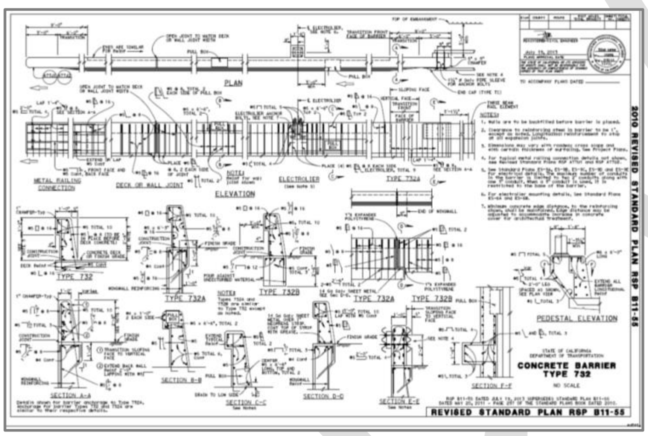
  The Caltrans *Local Program Manual* requires the bridge soffit to be a minimum of 2-feet above the 50-year flood elevation, and that the bridge is capable of conveying the 100-year flood.

  For the bridge replacement alternatives, design will meet these criteria (except slightly less freeboard may be provided at the abutments in order to conform the roadway profile to the adjacent highway, SR16).

  For the bridge retrofit/rehabilitation alternative, a design exception will be required because the existing bridge does not meet the hydraulic freeboard requirements. The soffit of the bridge is actually 2.78 feet below the 100-year water surface elevation.

Bridge Railings

For local agency projects to qualify for federal funding, Caltrans Structures Local Assistance indicates that bridge railings must conform to the full-scale crash-test criteria established in *Manual for Assessing Safety Hardware* (MASH) and *National Cooperative Highway Research Program* (NCHRP 350). For a 45 mph design speed, an appropriate railing should satisfy TL-2 crash test requirements or greater.



Caltrans Standard Plan, Concrete Barrier Type 732 – Proposed Rail on new Bridge

For the bridge replacement alternatives, new Concrete Barrier Type 732 with Tubular Bicycle Railing is recommended. Concrete barriers are rated TL-4. A concrete barrier will also collect the storm water and divert it to a roadway collection system, which will prevent stormwater from flowing directly into the river. Concrete barriers are also beneficial because they require far less maintenance than side mounted metal tube railing or thrie-beam guard railing, which are damaged by vehicular impacts.

For the bridge retrofit/rehabilitation alternative, the existing bridge railing may need to be replaced, with an exterior appearance that replicates the appearance of the original rail. This will be determined in consultation with the State Historic Preservation Officer (SHPO).

Approach Guardrail

Approach guardrailings are typically required on the approach end of the bridge rails to protect oncoming traffic from the blunt end of the bridge rail. Based on the clear width between the barrier rails on the new bridge, guard railing and protective end treatments are required at all four corners of the bridge.

The standard approach guard railing will meet FHWA's MASH and NCHRP 350 requirements. The bridge rail will be connected to a standard stiffened section of Caltrans standard Midwest Guardrail System Transition Railing (Type WB-31) adjoining either a flared or in-line terminal end treatment system.

The standard flared approach treatment is feasible at all four corners of the bridge. The adjacent photo shows the north end existing approach roadway (looking from the end of bridge). There is sufficient roadway length for the placement of the 25-foot stiffened transition section, the 37.5-foot long alternative flared terminal system, or the 50-foot long alternative in-line terminal system.

Northside Approach Roadway (from End of Bridge)

Southside Approach Roadway (before Beginning of Bridge)

The photo to the left shows the south end existing approach roadway looking toward the bridge. There is sufficient room for either a standard guardrail system with stiffened transition section, or an alternative flared terminal system or alternative in-line terminal system.

Aerial View – Proposed alignment at upstream of existing bridge

Design Exceptions

While the design standards are a mixture of AASHTO and County Standards **(see Appendix A)**, it is important to note that all replacement alternatives will match or improve the existing condition. Design exceptions for various standards and alternatives are summarized below:

All Alternatives require the following County design exceptions:

- Not meeting County's 65 mph rural/unposted design speed standards
- Not meeting County's 8-foot shoulders standards (4-foot full paved, 4-foot AB)
- Not meeting County's 32-foot bridge rail-to-rail clear width
- Not meeting County's 4-foot bicycle lane width

Additional design exceptions will be required for the following alternatives:

Alternative 1 – Retrofit/Rehabilitation of existing bridge

- FHWA freeboard criteria for passing the 50-year storm with 2 feet of freeboard for the existing bridge

Alternative 2 – Add new CIP Concrete Box Girder Bridge, keep existing bridge

- FHWA freeboard criteria for passing the 50-year storm with 2 feet of freeboard for the existing bridge

Alternative 3 – CIP Concrete Box Girder Bridge Replacement

- No additional design exceptions

Contractor Access

For both replacement and retrofit alternatives, the channel access at this site could occur from both the north and south bank of the existing bridge.

Area of Potential Effect (APE) Map

Staging Areas

Staging areas are anticipated in the red shaded area shown above. Depending on which alternative is selected, the existing roadway may be closed; therefore some staging could take place on the existing roadway. Environmental restrictions typically prevent the storage of materials and equipment within the river banks. While a large flat area in the southwest quadrant appears ideal for a staging area, the contractor may have to negotiate with private landowners for a temporary construction easement to use this space.

Right-of-Way

Depending on the alignment alternative selected, permanent right-of-way takes may be required. Temporary Construction Easements (TCE) are anticipated in order to establish temporary detours or provide adequate room for construction. Bridge replacement alternatives will require acquisition of additional right-of-way. TCE's will be required for contractor's staging areas.

Community Interaction

The County attended several community meetings in 2013 and one on January 6, 2014. At these meetings, the County informed residents in the Rumsey area about the early planning stages of the Rumsey Bridge Project, shared apparent alternatives, and received initial public feedback. During the meetings the County also answered questions pertaining to funding, the consultant selection process, the project schedule, and also identified opportunities for future public involvement during the environmental phase of the project. The County asked the community to provide their initial feedback on three alternatives:

1. Bridge Retrofit/Rehabilitation of the existing structure;
2. Bridge replacement on an upstream alignment with removal of the existing bridge; and
3. Bridge Replacement on an upstream alignment and closure of the existing structure.

The majority of participants attending these meetings expressed support for bridge rehabilitation. The historic character of the bridge is valued by residents, and the Rumsey Bridge is considered by many to be a defining landmark in the upper Capay Valley.

Utilities

Few utilities exist within the project limits. Depending on the alternative selected, utility relocations may be necessary. The two known utility that may be impacted are listed below:

- All the design alternatives avoid the overhead shown to the right.
- The stream gage also shown in the photo will need to be relocated for a new bridge.
- Telephone phone conduit mounted onto the south side of the bridge that would need to be temporarily relocated and then reattached for a retrofit, or relocated into a new bridge.

Overhead Utility on Eastside of existing bridge

Environmental/Permits

The design of the proposed project will minimize environmental impacts as much as possible for both the replacement and rehabilitation/retrofit alternatives. Environmental studies are currently being prepared in compliance with Caltrans standards and with federal and state requirements for National Environmental Protection Act (NEPA) and California Environmental Quality Act (CEQA), respectively. These studies can be completed after a preferred alternative is selected. Yolo County will prepare the CEQA document, which is expected to be an Initial Study/Mitigated Negative Declaration. Either Caltrans or the County will prepare the NEPA documentation, which might be in the form of a Categorical Exclusion. The draft Preliminary Environmental Study (PES) and the Area of Potential Effect (APE) map have been completed (see **Appendix G**) and approved by Caltrans.

The foundations for all bridge alternatives will consist of either large diameter cast-in-steel-shell (CISS) or cast-in-drilled-hole (CIDH) piles. For new bridge construction, installation of these foundation types minimizes impacts by eliminating shored/cofferdam excavations that are required for existing bridge pile footing foundation retrofit. The bridge rehabilitation/retrofit alternative would require large shored excavation to strengthen the existing foundation and tie the large CISS or CIDH foundation to the existing footings. The nature of this work will require stream diversion for all alternatives.

One major component to project environmental documentation is for compliance with Section 106 of the National Historic Preservation Act and its regulations under Title 36, Code of Federal Regulations, Part 800 (36 CFR 800). Section 106 requires Caltrans (on behalf of the Federal Highway Administration) to identify whether this project (or undertaking) will cause an adverse effect to historic properties (i.e., resources listed in or eligible for listing in the National Register of Historic Places [NRHP]). As part of the Section 106 process, Caltrans must consult with the State Historic Preservation Officer (SHPO) and work to get concurrence on conclusions regarding the project's effect on historic properties. Caltrans has established detailed instructions for the Section 106 process in its Standard Environmental Reference (SER). For Local Assistance Program projects, Caltrans delegates much of the Section 106 compliance documentation to the local agency. The conclusions and outcome of the Section 106 process will also be used for the project's compliance with Section 4(f) of the Department of Transportation Act.

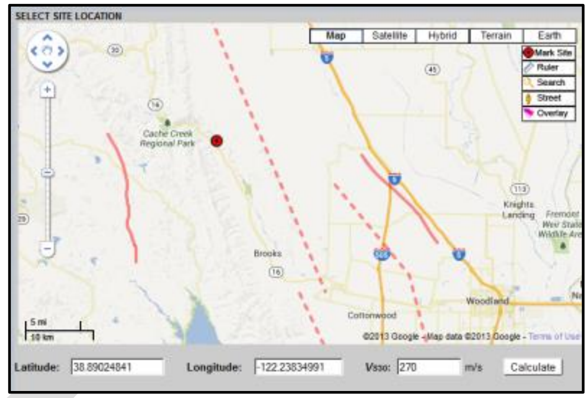
According to Caltrans' historic bridge inventory, the Rumsey Bridge / County Route 41 over Cache Creek (Bridge No. 22C0003) is eligible for listing in the NRHP (status designation Category 2), and thus it is considered a historic property for purposes of Section 106 compliance. (it is also considered a historical resource for purposes of CEQA compliance, as per CEQA Guidelines Section 15064.5). Once the preferred alternative is selected, the County is responsible for preparing a Finding of Effect (FOE) to assess whether the project will have an adverse effect on the historic bridge, applying the Criteria of Adverse Effect (36 CRF 800.5) and assessing the project's compliance with the Secretary of Interior's Standards for the Treatment of Historic Properties. Caltrans will submit the FOE to SHPO for concurrence. Potential FOE conclusions of various alternatives are discussed below. If the FOE concludes that the preferred alternative will cause an adverse effect, a Memorandum of Agreement (MOA) will be required to establish measures to mitigate the adverse effect(s). The MOA (which may be prepared by Caltrans) would be between Caltrans and SHPO, with the County as a concurring party. The Section 106 consultation process would be completed when the MOA (if needed) is signed. Caltrans would then likely delegate responsibility for completion of mitigation measures to the County. This process could be lengthy and will be on the environmental critical path once a preferred alternative is selected.

The FOE's conclusion will depend on the preferred alternative selected. Caltrans is likely to require additional information presented in the FOE about other alternatives considered, but rejected. Demolition of the historic Rumsey Bridge will be an adverse effect. It would also be considered a use for Section 4(f). Alternatives that rehabilitate / retrofit the bridge, with or without a new adjacent bridge, might not cause an adverse effect. It may be possible that alterations to the bridge could be completed in a manner that meets the Secretary of Interior's Standards for the Treatment of Historic Properties. Coordination between the project architectural historian and project engineers could help identify whether this scenario is possible.

Geotechnical/Foundations

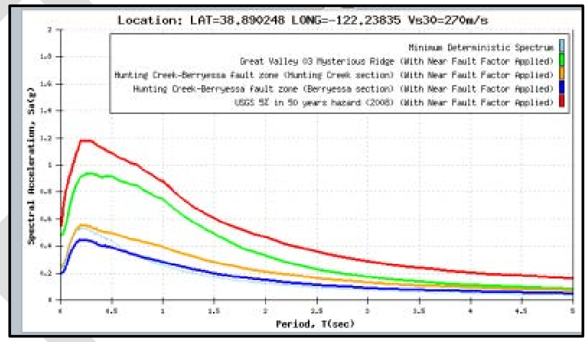
Taber has drilled two borings, one at each of the existing bridge abutments. The Log of Test Borings can be found in Appendix E. Taber has also provided the Acceleration Response Spectrum curve for the site.

Since the project site is within 4 miles of an active fault (Great Valley 03 Mysterious Ridge), the spectral acceleration is magnified for the near site effects in accordance with the Caltrans *Seismic Design Criteria*.



Seismic faults near the Rumsey project site

Based on the available data, the team recommends either larger diameter cast-in-drilled-hole (CIDH), cast-in-steel-shell (CISS) or driven piles. Larger diameter piles are desirable because they allow for the removal of cobbles and boulders that are present at this site. Shallow foundations such as spread footings were not considered feasible due to the scour issues and high seismicity at the site.



Enveloped Acceleration Response Spectra based on various faults near the project site and Seismic Hazard Map, using deterministic and probabilistic methods.

For replacement alternatives, the team recommends single column bents supported on large diameter CIDH, CISS, or driven steel piles at the piers. Large diameter piles are very cost effective and perform well in high seismic and scour sites. Single column bents will also reduce issues with debris thereby minimizing future maintenance costs related to debris removal.

For the retrofit/rehabilitation alternative, the team recommends installing two large diameter CIDH concrete pile shafts adjacent to the existing footing, at each pier. The large diameter piles will be tied to the existing footing cap by the means of installing a larger out-rigger footing that encapsulated the existing footing.

Large Diameter Drilled Shaft example

Falsework

Depending on which alternative is selected, the falsework requirements at the site may vary from falsework placed in the creek for a new bridge, to falsework or work platforms suspended from the existing bridge for the rehabilitation/retrofit alternative. There are no known environmental restrictions or mitigation measures on this project that would preclude the use of falsework in the creek; however creek flows may need to be considered during the falsework design. The allowable time the falsework can remain in the Creek may also be subject to creek flows and environmental permit requirements. Based on likely permitting requirements the construction window for work in the creek is restricted to between June 1st and October 31st. This four-month window may not provide an adequate amount of time to construct all new bridge replacement alternatives in one season, and could result in a multiple season construction timeline for the project. Depending on the nature of the retrofit/rehabilitation alternatives, multi-season construction may also be required.

Temperature

Maximum Temperature: 113° F
Minimum Temperature: 15° F

Deck Protection and Corrosion

The project is located in Environmental Area II (Moderate climate) based on Caltrans Memo to Designers. Based on geotechnical borings corrosive soils are not present.

Rock Slope Protection

Rock Slope Protection at the bridge abutments will be designed using the Caltrans "*California Bank and Shore Rock Slope Protection Design*" and FHWA's "*Hydraulic Engineering Circular No. 18*" (HEC-18).

For the bridge replacement alternatives, the location of Abutment 1 will be shifted back up the slope of the bank to increase the flow area beneath the bridge. RSP at both abutments will be designed to handle to 100 year storm event.

For the bridge rehabilitation alternative, the existing RSP at Abutment 1 will be removed and replaced with a stronger RSP (layered) system after the reconstruction of the broken and missing wingwall.

Abutment 1 left – RSP backfill after a storm event took out the bridge wingwall

5. EXISTING BRIDGE SEISMIC ASSESSMENT & RETROFIT STRATEGY

Existing Bridge Seismic Assessment Methodology

For the assessment of the existing bridge, the following documents and information were used:

- The 1930 design/As-Built plans
- The 1949 retrofit/As-Built plans
- All available Caltrans Inspection reports
- Field visual inspection of the bridge for deficiencies
- Scour Plan of Action in 2006 for Yolo County by Quincy
- Feasibility Study, Design Calculations, and miscellaneous reports for Stevenson Bridge

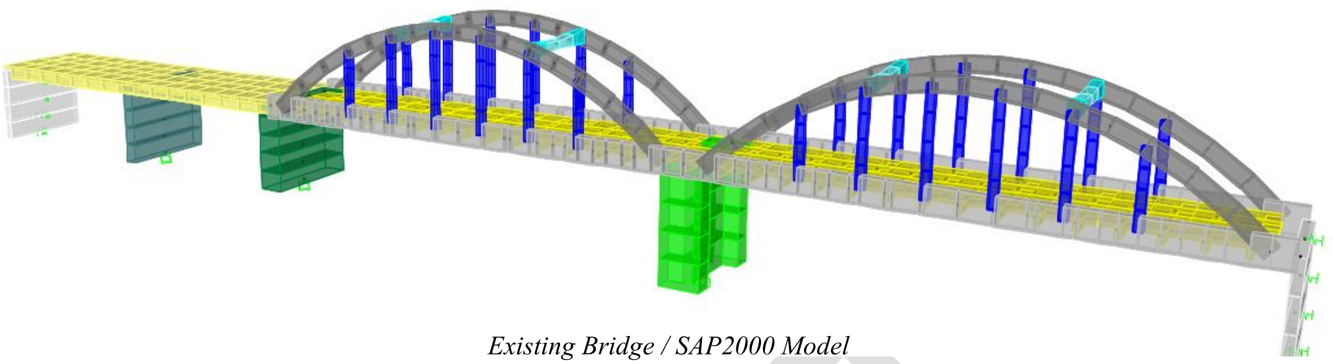
The Rumsey Bridge has been evaluated to meet the performance requirement of "No-Collapse", which means that the bridge could be significantly damaged during an earthquake, but would not collapse. This conforms to the Caltrans design methodology and industry practice for bridge seismic design in California. Afterward an earthquake, the bridge may require extensive repairs, or may have to be replaced entirely, but it would remain standing through an earthquake to minimize the threat to public safety during the event.

Elevation View of the Existing Bridge

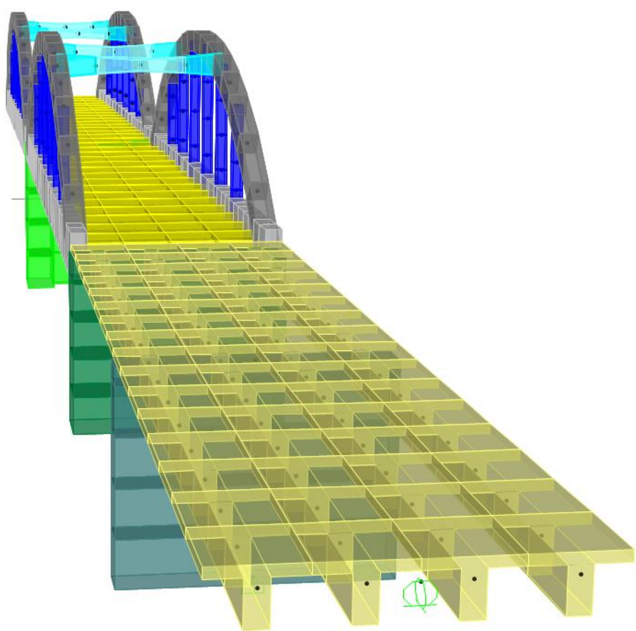
The Rumsey Bridge is considered to be a Non-Standard Bridge by the Caltrans *Seismic Design Criteria* due to its unique superstructure type. To capture its complex seismic response, this bridge requires a more detailed analysis than typically prescribed. Therefore, an explicit elemental level dynamic analysis model was created to capture the effects on individual structural elements, including the Arch Ribs, Tie Girders, Vertical Hangers, Portal Bracing, Floor Beams, and other members. This analysis was completed in the computer program SAP2000 (SAP), a Structural Analysis Program created by Computers and Structures, Inc. A multimodal linear elastic dynamic analysis was performed with a Soil Type-D acceleration response spectrum (ARS) curve with a 5% damping ratio.

Body constraints and rigid elements were utilized to model diaphragm and deck rigidity. The structure was modeled explicitly with boundary restraints and releases, and longitudinal springs were iterated for force and displacement convergence to capture the behavior of the abutment-soil interaction.

At the pier locations, the bottom of vertical support members were modeled as fixed (translation and rotation in all degrees of freedom) because footing retrofits are required at all locations to address scour concerns. For instance, the Pier-2 H-piles embedment is so short that the piles would become unstable under the maximum scour condition. Under this condition, the footing would be unstable and would have to be retrofitted just to maintain stability, regardless of seismic considerations. Thus the bottoms of the piers were fixed.



Existing Bridge / SAP2000 Model

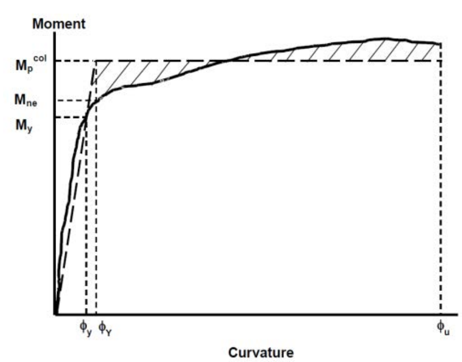


T-Girders at Span 3 & 4 at End of Bridge

As discussed earlier, a parametric study determined that the seismic vulnerabilities would not be sensitive to the concrete strength of the arch spans. With that finding, a concrete compressive strength (f_c') of 2,500 psi was used in the baseline assessment model. A yield strength (f_y) of 40 ksi was used for reinforcing steel based on historical material properties of rebar from the 1930's.

In an earthquake, the primary superstructure members such as Arch Ribs, Vertical Hangers, Tie Girders, and Portal Bracing are critical members that must remain elastic to prevent collapse of the arch spans. Therefore gross section properties were used for all superstructure structural elements to obtain force demands. If demands were found to exceed the elastic capacity of an element, this indicated that the member would need to be retrofitted in order to remain intact during an earthquake. The detailed element modeling for specific bridge members is described in **Appendix I** of this report.

Since the superstructure (arch elements) must remain elastic for structural stability, moment-curvature analyses were performed on the piers to determine their capacity. This analysis took into account the non-linear material behavioral characteristics of the concrete and rebar in the pier. This analysis was used to determine the strength and displacement/rotational capacities of the pier. A push-over analysis was then performed to determine the global structural displacement capacity of the pier.



Moment-Curvature Analysis Curve

The primary collapse mechanisms for the Rumsey tied-arch portion of the bridge would be the failure of the primary load carrying members, listed below.

Primary members:

- Arch Ribs
- Tied Girders
- Portal Bracing
- Piers

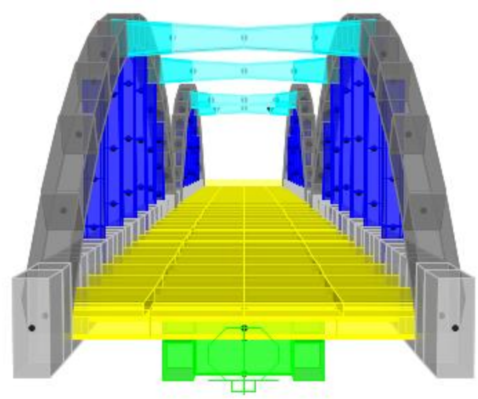
Since these members contain a very limited amount of confinement reinforcement, the forces in these members must be limited to their yield curvatures based on expected material properties. In other words, inelastic, ductile behavior cannot be permitted in primary members. This will ensure that primary load carrying elements behave essentially elastic during a seismic event thereby preventing the spans from collapsing. The primary member acceptance criterion is force Demand-to-Capacity ratio of less than 1. That is to say, the demand on the member must be less than its capacity.

Secondary elements are also important to prevent structural collapse. However, secondary elements may be allowed to behave inelastically. The following elements were considered to be secondary elements.

Secondary members:

- Vertical hangers
- Transverse floor beams

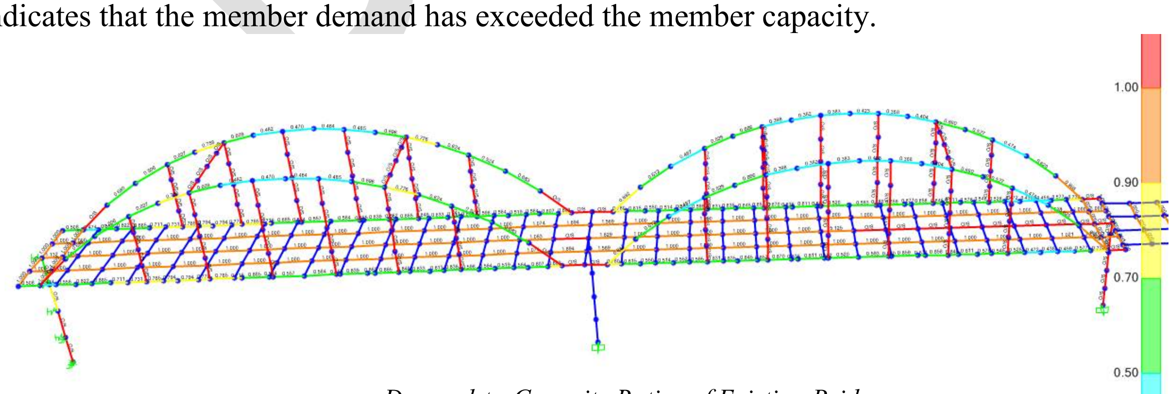
Flexural moment Demand-to-Capacity ratios may be greater than 1 for secondary elements, provided that both the rotational capacity is greater than the rotational demand, and the shear capacity is greater than the shear demand.



Arch Spans View at Beginning of Bridge

Existing Bridge Vulnerabilities

Once the capacities of existing members were determined and compared to demands from the dynamic analysis, it was determined that nearly all superstructure elements do not have enough capacity to withstand seismic forces as shown in the Demand-to-Capacity (D/C) ratio color schematic below. A D/C ratio greater than 1 indicates that the member demand has exceeded the member capacity.



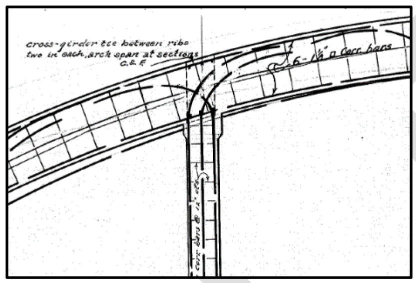
Demand-to-Capacity Ratios of Existing Bridge

The maximum demand-to-capacity ratios from the analysis are summarized in the following sections:

<u>Arch Ribs</u>

The existing arches are 36 inches deep by 27 inches wide. According to the As-Built plan, the arch ribs should be reinforced with six $1\frac{1}{8}$" square bars. However, based on field inspection, seven bars are visible in the arch as observed in multiple locations where much of the concrete cover on the bottom side of the Arch Ribs has spalled off.

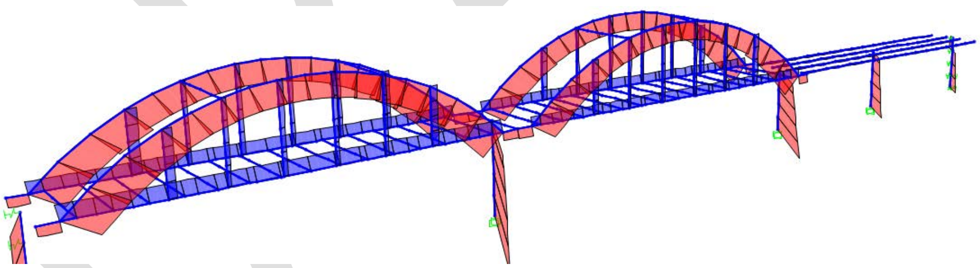
Arch Rib Spalls
(Span 1 left arch, near Pier 2)



Arch Rib As-Built

Arch Rib Spalls
(Span 2 right arch, near Pier 3)

For the seismic assessment, only six longitudinal bars were used to determine member capacity because it is possible that the additional bar exists only at lap splice locations and is not continuous throughout the length of the arch. Under normal gravity load (dead load), the arch ribs are under compression as they hold the arch span up, as shown in schematic below.



Dead Load axial effects – Red indicates Compression & Blue indicates Tension

For confinement, 3/8" square bars at 18 inch spacing are provided. The existing amount of confinement is very light and does not meet today's minimum transverse reinforcement requirement. Without the minimum amount of transverse reinforcement, the arch is unable to restrain the growth of diagonal cracking and is unable to provide much ductility in a seismic event.

Under seismic loads, the arch members do not have sufficient capacity to meet the seismic bending and shear demands in both major and minor axes. The maximum moment demand occurs near the "spring line" adjacent supports of the arch. Based on the existing condition of the bridge, the maximum Demand-to-Capacity ratio of the Arch Rib elements for combined axial and flexural is 4.98. The shear strength comes mostly from the concrete since the shear reinforcement is minimal. The shear D/C = 1.65.

<u>Tie Girders</u>

The Tie Girder is a tension element that acts like a bow string that prevents the arch from flattening out.

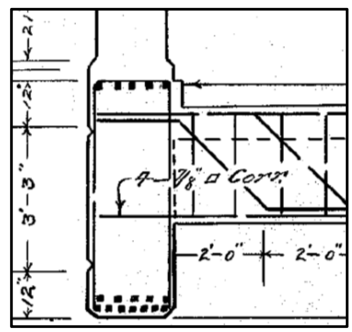
Span 1 elevation view – Tie Girder at span 1 left side

These members have also experienced significant loss of cover along the bottom of the member, much like the arches themselves.

As shown in the photographs below, the concrete has spalled so much that the second layer of the main reinforcement is exposed.

Tie Girder Spall – Pier 2 left isotropic view

Tie Girder Spall – Pier 2 left looking from bottom



Tie Girder As-Built Reinforcement

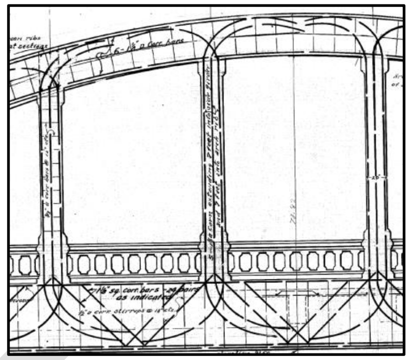
The Tie Girders are 63 inches deep by 23 inches wide. Similar to the Arch Ribs, the Tie Girder was also constructed with more main longitudinal rebar than specified. The plans called for seven $1\frac{1}{8}$" square bars but based on a field inspection up to nine bars are visible from the underside of the girder at several locations. Similar to the Arch Ribs, it is likely that some locations have rebar lap splices that result in cover loss over time. For confinement, 1/2" square bars at 18 inch spacing are provided, which also does not meet the minimum transverse reinforcement requirements. This results in poor seismic performance.

Under earthquake loads, the Tie Girders experience both tension and compression due to the transverse ground motion bending the spans in the plane of the deck. The maximum bending for this behavior occurs near Pier-2. Because these girders were designed to handle vertical loads, and not the lateral loads induced by an earthquake, they have very little strength in this direction and cannot handle lateral seismic forces. The maximum Demand-to-Capacity ratio of the Tie Girders for combined axial and flexural loads is 2.71.

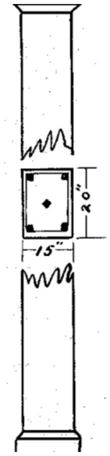
The maximum shear D/C = 4.45, which occurs near Pier-2, where the arch meets the Tie Girder.

Vertical Hangers

The Vertical Hangers are 20 inches thick by 15 inches wide. Each hanger has five $1\frac{1}{8}$" square longitudinal reinforcing bars that extend seven feet into the Tie Girder, and seven feet into the Arch Rib. For confinement, 3/8" square bars at 12 inch spacing are provided. The Vertical Hanger confinement does not meet the minimum transverse reinforcement requirements. The existing Vertical Hangers already have significant cover loss in various locations, as shown in photos below. Based on the As-Built plans, it is unclear if lap-splices were allowed during construction.



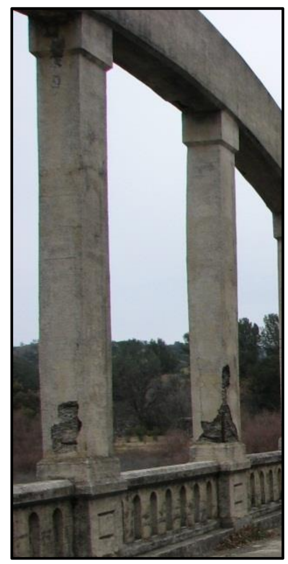
Vertical Hanger As-Built Reinforcement limits



As-Built Reinforcement size

The Vertical Hanger behaves as a tension element, suspending the deck from the arch. Because these members are in tension, only rebar provides load carrying capacity because concrete does not provide tensile strength. When in tension, concrete also does not contribute to the shear capacity of the member so it was not included in the member shear capacity.

Vertical Hanger Elevation View (Span 1 right side)



Vertical Hanger (Span 1 left side)

Under seismic loading, the Vertical Hangers remain in tension. Because the hangers are in tension, their flexural capacity is very low. The rebar are already stressed under dead load, so any additional bending stress easily over stresses the reinforcement.

The maximum Demand-to-Capacity ratio of the Vertical Hangers for combined axial and flexural loads is extremely overstressed, with a D/C = 404.08.

The shear Demand-to-Capacity ratio is also extremely overstressed with maximum D/C = 30.00.

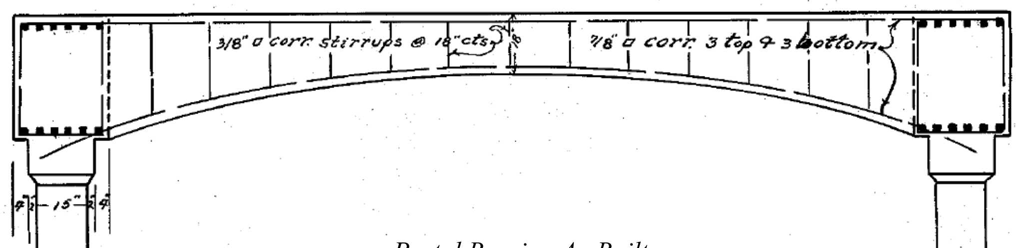
Vertical Hanger Zoomed in view (Span 1 left side)

Portal Bracing

The Portal Bracing varies in depth between 34 inches deep at the end to 18" deep in the middle of the member. The braces are 15" wide. Each brace is reinforced with six $\frac{7}{8}$" square longitudinal bars. These bars are not adequately developed into the arches due to both the low concrete strength of the arch and the short distance the bars extend into the arch. This means the bars will pull out of the arch before they reach their maximum strength.

Portal Bracing (Span 1, 2nd portal)

Confinement reinforcement consists of 3/8" square bars at 18 inch spacing. Like most of the elements in the bridge, this rebar does not meet the minimum transverse reinforcement requirements.



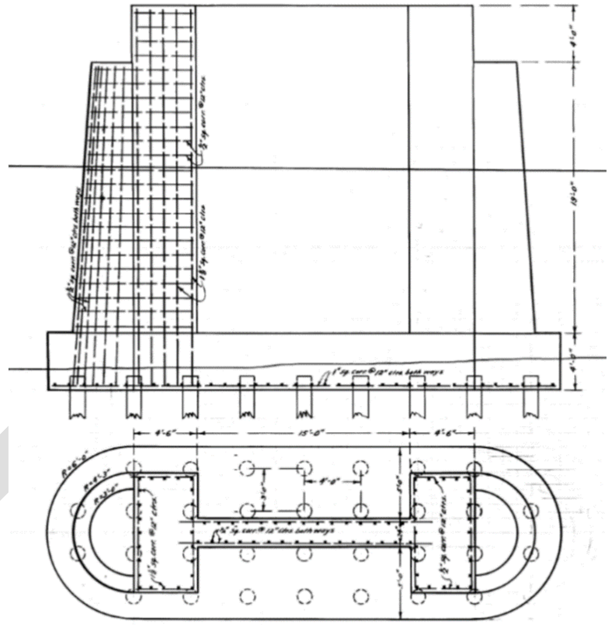
Portal Bracing As-Built

The maximum Demand-to-Capacity ratio of the portal bracings for combined axial and flexural is highly overstressed with D/C = 20.67.

The shear Demand-to-Capacity ratio is also overstressed with maximum D/C = 4.43.

<u>Substructure</u>

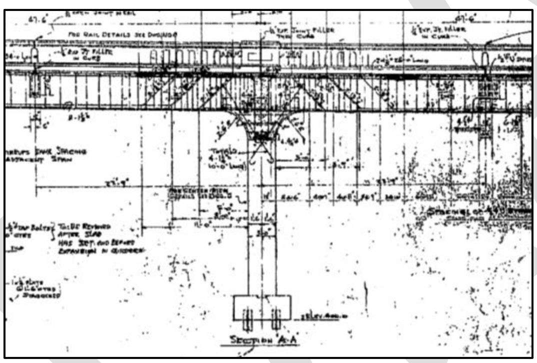
Pier 2 is approximately 23' tall. It consists of two primary columns each 4'-6" thick connected by a 2' thick shear wall between the main columns, and two built-out semi-circular shapes outside the columns that taper from a radius of 3' at the top to 4'-6" at the base. The main column width tapers from approximately 6' near the soffit to 9' at the footing. The columns have $1\frac{1}{8}$" square longitudinal bars, and ½" square confinement bars at 12 inch spacing. The shear wall only has ½" square bars for both the main and confinement reinforcement.



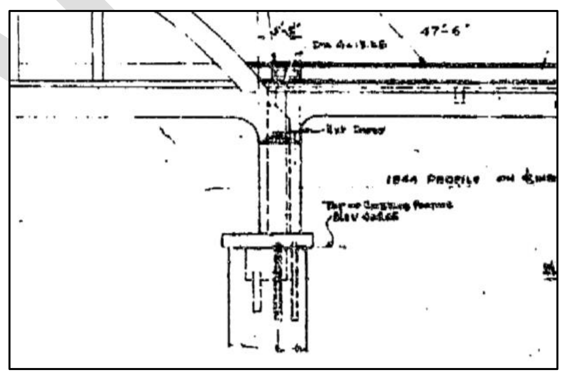
Pier 2 As-Built

Structurally, this pier behaves as a column in the longitudinal direction, i.e., along the bridge. In the transverse direction, the pier behaves as a shear wall .

Pier 3 consists of a lightly reinforced abutment stem (with ½" square bars) and a thickened pier wall built-out with semicircular ends, shown to the in left photo below.
Pier-4 consists of a new pined-pined rebar detail to eliminate moment transfer between the superstructure and substructure (see Retrofit Plan reinforcement detail).



Pier 4 As-Built



Pier 3 As-Built

Pier 3 and Pier 4 (left side of bridge)

Abutment 1 and Pier 2 (left side of bridge)

Seismic demands are reported in the table below.

For Pier 2, the D/C in the longitudinal direction is 1.39, and 0.71 in the transverse direction. This means under the earthquake loads the pier behaves inelastically along the direction of the bridge, but behaves elastically along the strong direction of the pier.

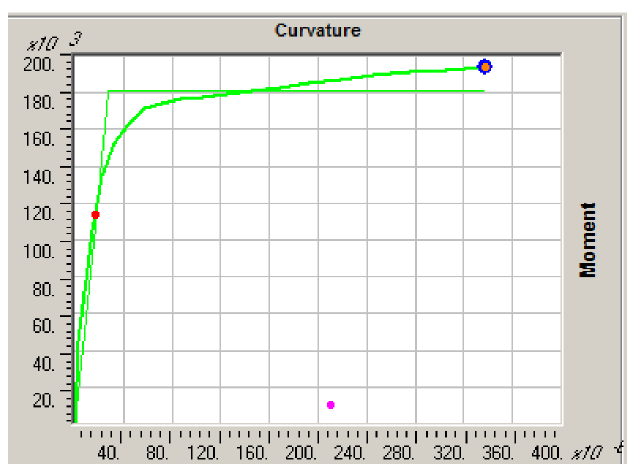
Pier 2 Results						
	Force Demand [kips-inches] @ pier bottom					
	Soil Profile D				Governing Demand Results	
	Case I, Trans + DL 100% Transverse + 30% Longitudinal		Case II, Long + DL 30% Transverse + 100% Longitudinal			
Longitudinal Transverse	63,000 k-in 299,000 k-in		182,000 k-in 89,600 k-in		182,000 k-in 299,000 k-in	
	Force Capacity [kip-inches] @ pier bottom					
	Axial Load, P	M-yield	Mp	Icrack [in^4]	M-yield	D/C
Longitudinal	1322 k (comp)	131,000 k-in	197,000 k-in	2,010,000	131,000 k-in	1.39
Transverse	1322 k (comp)	422,000 k-in	613,000 k-in	20,684,000	422,000 k-in	0.71

Pier 2 Existing Bridge Seismic Analysis Results

Since the pier behaves inelastically in the longitudinal direction only, a Moment-Curvature M analysis was completed for that direction. The maximum moment capacity is determined when either the ultimate compressive strain $_{cu}$ or the reduced ultimate tensile strain $^{R}_{su}$ of reinforcement steel is reached. The As-Built plans do not identify if lap splices were used in the main reinforcement (typically at the top of the footing), a conservative concrete strain limit of 0.002 was used, and is the controlling factor in determining the displacement/curvature capacity for the nonlinear assessment. A reduced ultimate tensile strain $^{R}_{su}$ of 0.06 was used for the reinforcement and is based on historical properties of reinforcement from the 1930s.

To determine the curvature capacity, a Moment-Curvature (M) analysis was performed in the longitudinal direction of the pier. The pier reached a concrete strain limit of 0.002 at a curvature of 0.0001063, which is defined as the curvature capacity of Pier 2.

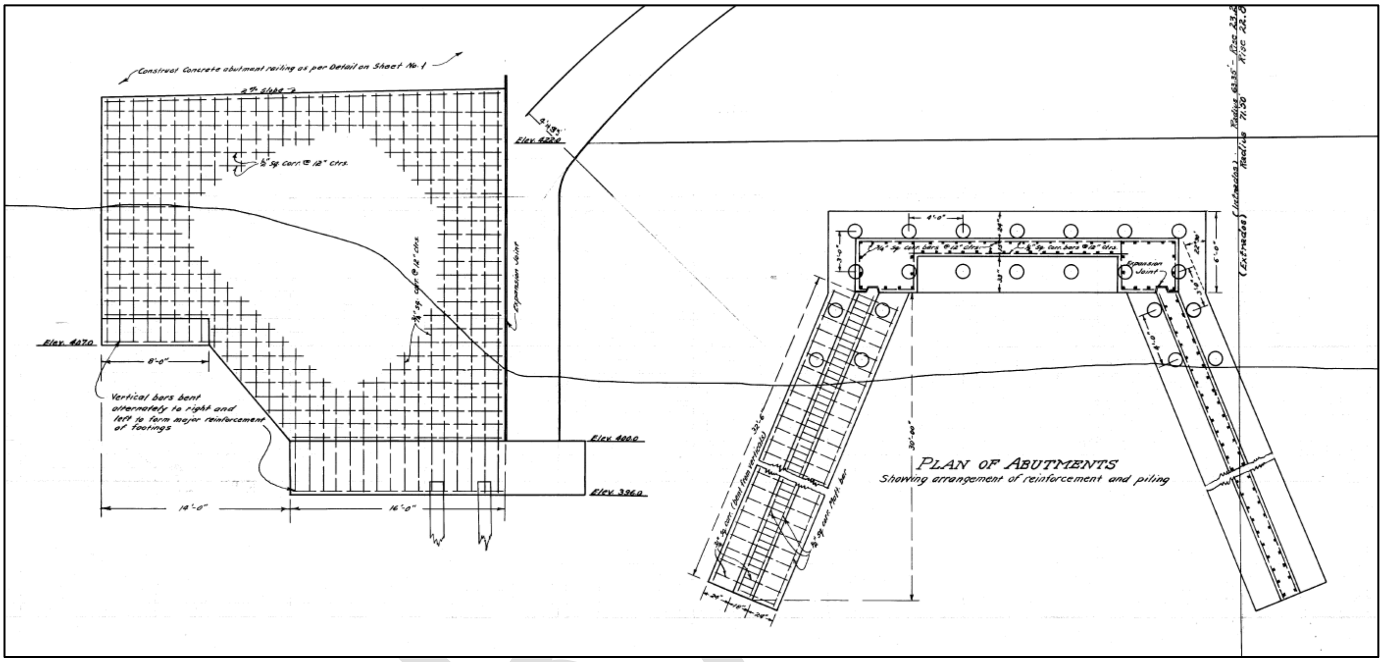
The curvature demand $_{demand}$ can be derived from the equation $\theta_{demand} = L_p \; _{demand}$, where the rotation demand θ_{demand} is obtained from the ARS analysis. The analytic plastic hinge length L_p is determined as a function of the pier length, the expected yield, and the diameter of the steel reinforcement. Comparing curvature capacity to the pier longitudinal curvature demand, of 0.0000348, the pier has sufficient curvature capacity, yielding an inelastic curvature D/C = 0.33.



Pier 2 Moment-Curvature about the Weak Axis

P effects are negligible since the relative pier displacement times the dead load is small when compared to the column idealized plastic moment.

Abutment 1 is 22’ tall with a 6’ wide footing. In addition to the abutment stem, the wingwalls are also partially founded on an elevated footing. The wingwalls are constructed with expansion joint that is not connected to the abutment stem as a traditional wingwall would be. Due to the expansion joint between the abutment stem and abutment wingwall, the wingwalls can be dislocated from the abutment by an externally applied force, much like the storm event that washed out the abutment wingwall in 1995.



Abutment 1 As-Built

Abutment 1 and Abutment 5 are also vulnerable because their moment capacity is less than the moment demand. The abutment piles and pile/footing connection are also inadequate under the seismic loads.

<u>Approach Spans</u>

The approach span superstructure in general has few vulnerabilities. The primary concern is the potential to unseat at the Pier 3 location, which has only 1 foot of seat width. Without restraining movement at Abutment 1 and Abutment 5, Span 3 could unseat and collapse during an earthquake.

At Pier 4 the superstructure is connected to the substructure with a lapped bar connection (X-shaped) that acts as a pin. This detail limits the amount of seismic force that will transfer from the Pier 4 substructure to the superstructure, so the superstructure is not adversely affected during an earthquake.

Summary of Deficiencies

In summary, the bridge has numerous deficiencies as shown below:

1. Arch Ribs
2. Vertical Hangers
3. Tie Girders
4. Portal Bracing
5. Piers 2, 3 & 4
6. Abutments 1 & 5

Each of these members must be retrofitted, and the retrofit strategy is discussed in the following pages.

Seismic Retrofit Strategy

Several retrofit measures must be incorporated in order to address the deficiencies summarized above. The arch is an unusual, complex structure that does not lend itself to common retrofit measures such as strengthening members by encasing in concrete or steel jackets, or constructing in-fill walls between members. Rather, in order to maintain the general appearance of the bridge, each deficient member can be strengthened by wrapping it with a fiber material that provides additional strength and confinement with a minimal change to the dimensions of the bridge. With that in mind, the seismic retrofit strategy for the arch spans includes removing and patching unsound concrete, patching spalled surface areas, and fiber-wrapping superstructure elements to improve ductility and increase strength.

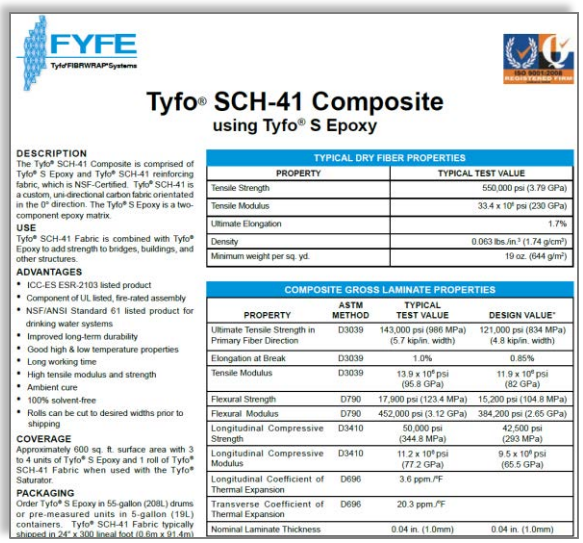
Substructure retrofit measures include the retrofit of all abutment and pier footings to resist both scour and seismic deficiencies. Large diameter piles would be added to the outside of the existing footing outline and a new footing cap would tie the new piles to the existing footing. At the abutments, large diameter piles would be added behind the existing abutment wall to address both scour and seismic deficiencies.
The retrofit of each element is discussed in further detail below.

Arch Span Retrofit - Fiber Reinforced Polymer

Fiber Reinforced Polymer (FRP) provides additional strength and ductility to bridge elements. Caltrans has approved FRP for use in jacketing various structural members to increase their strength, and FYFE Company LLC is one of the companies that have been preapproved by Caltrans to do such work. Below is a FYFE product specification for the SCH-41 Carbon system (CT system 9) approved by Caltrans. The retrofit strategy mentioned in the following pages utilizes this carbon fiber wrap system to strengthen various arch elements.

FRP is commonly been used to provide confinement, and axial and shear capacity enhancement for existing members. FRP can also provide additional flexural capacity to members.

The design guidelines for FRP strengthening are presented in ACI 440.2R-08 *"Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures"*. Additional information on the criteria for evaluation fiber wrap systems can be found in International Code Council's ICC-ES AC 125 *"AC125 Concrete and Reinforced and Unreinforced Masonry Strengthening Using Fiber-reinforced Polymer (FRP) Composite Systems"*.


FYFE
Tyfo® FIBRWRAP® Systems

ISO 9001:2008 REGISTERED FIRM

Tyfo® SCH-41 Composite
using Tyfo® S Epoxy

DESCRIPTION
The Tyfo® SCH-41 Composite is comprised of Tyfo® S Epoxy and Tyfo® SCH-41 reinforcing fabric, which is NSF-Certified. Tyfo® SCH-41 is a custom, uni-directional carbon fabric orientated in the 0° direction. The Tyfo® S Epoxy is a two-component epoxy matrix.

USE
Tyfo® SCH-41 Fabric is combined with Tyfo® Epoxy to add strength to bridges, buildings, and other structures.

ADVANTAGES
- ICC-ES ESR-2103 listed product
- Component of UL listed, fire-rated assembly
- NSF/ANSI Standard 61 listed product for drinking water systems
- Improved long-term durability
- Good high & low temperature properties
- Long working time
- High tensile modulus and strength
- Ambient cure
- 100% solvent-free
- Rolls can be cut to desired widths prior to shipping

COVERAGE
Approximately 600 sq. ft. surface area with 3 to 4 units of Tyfo® S Epoxy and 1 roll of Tyfo® SCH-41 Fabric when used with the Tyfo® Saturator.

PACKAGING
Order Tyfo® S Epoxy in 55-gallon (208L) drums or pre-measured units in 5-gallon (19L) containers. Tyfo® SCH-41 Fabric typically shipped in 24" x 300 lineal foot (0.6m x 91.4m)

TYPICAL DRY FIBER PROPERTIES	
PROPERTY	**TYPICAL TEST VALUE**
Tensile Strength	550,000 psi (3.79 GPa)
Tensile Modulus	33.4 x 10^6 psi (230 GPa)
Ultimate Elongation	1.7%
Density	0.063 lbs./in.^{3} (1.74 g/cm^3)
Minimum weight per sq. yd.	19 oz. (644 g/m^2)

COMPOSITE GROSS LAMINATE PROPERTIES			
PROPERTY	**ASTM METHOD**	**TYPICAL TEST VALUE**	**DESIGN VALUE***
Ultimate Tensile Strength in Primary Fiber Direction	D3039	143,000 psi (986 MPa) (5.7 kip/in. width)	121,000 psi (834 MPa) (4.8 kip/in. width)
Elongation at Break	D3039	1.0%	0.85%
Tensile Modulus	D3039	13.9 x 10^6 psi (95.8 GPa)	11.9 x 10^6 psi (82 GPa)
Flexural Strength	D790	17,900 psi (123.4 MPa)	15,200 psi (104.8 MPa)
Flexural Modulus	D790	452,000 psi (3.12 GPa)	384,200 psi (2.65 GPa)
Longitudinal Compressive Strength	D3410	50,000 psi (344.8 MPa)	42,500 psi (293 MPa)
Longitudinal Compressive Modulus	D3410	11.2 x 10^6 psi (77.2 GPa)	9.5 x 10^6 psi (65.5 GPa)
Longitudinal Coefficient of Thermal Expansion	D696	3.6 ppm./°F	
Transverse Coefficient of Thermal Expansion	D696	20.3 ppm./°F	
Nominal Laminate Thickness		0.04 in. (1.0mm)	0.04 in. (1.0mm)

Caltrans pre-approved Carbon Fiber Wrap
(Tyfo SCH-41 Composite system)

Surface Preparation

Before installing FRP, the surface of the member must be prepared. First, unsound concrete must be removed and replaced. Highly corroded bar reinforcing steel should also be removed, and replaced with new reinforcement, which can be spliced to existing bars by welding. Light corrosion on other bars should be removed by abrasive blasting. Then the surface is repaired by injecting epoxy into any cracks in the concrete. Next, all concrete surfaces to be wrapped with FRP should be abrasive blast cleaned or ground to provide a rough bonding surface. Corners of the FRP retrofitted members must also be rounded to a minimum radius of 1½" so that a sharp corner does not induce high stresses in the FRP that could cause it to fail.

Extensive surface preparation is required for spalled surface area

Maintenance/Appearance

The FRP carbon fiber system itself is susceptible to decay due to ultraviolet exposure. To mitigate this effect and prolong the retrofit system, the FRP must be painted. The paint system is also susceptible to ultraviolet exposure and weathering, so it is necessary to repaint the FRP every 10 years to maintain the protective coating. Without intermittent maintenance, the FRP will eventually lose structural capacity. Because of the need to provide ongoing maintenance of the protective coating, the County's future cost to maintain the retrofitted bridge is higher compared to maintaining a new concrete bridge. In addition, the FRP is susceptible to damage from vehicles hitting/scraping the areas exposed to traffic on the narrow bridge. This is especially a concern with the wide agricultural equipment moving throughout the County.

The FRP and the paint will affect the appearance of the retrofitted bridge. Technically, only the portions of the bridge that have FRP installed require painting, which will result in an inconsistent appearance of the bridge. This could be addressed by painting the entire bridge. Applying the FRP system to elements such as the vertical hangers will require the alteration of architectural column cap and base details, as well the guardrail, to fully wrap the structural element. Additionally, it should be noted that the corners of any FRP wrapped elements will have to be rounded (to approximately two inch radius) to apply the fiber wrap, which will also alter the appearance of the bridge. Finally, the FRP will also cover portions of the architectural detailing (grooves) in the exterior face of the Tie Girder. The detailed analysis of the effects that fiber wrapping will have would be undertaken as part of the environmental clearance phase of the project.

Potential Retrofit Risks

The appearance of some architectural features of the bridge will potentially be adversely affected. The extent of the visual impacts are generally understood, but will not be fully known until the actual details are finalized in the developed design phase of the project.

With the relative newness of the proposed retrofit technology for this type of structure, it is possible that project costs could increase significantly as the details are developed during the final design phase of the project.

In addition, the long-term durability of fiber wrapped structures is not well defined because these materials do not have extensive historical performance data in bridge applications. Having only been used on bridges over the past 25 years, there is an element of risk in estimating the design life of a bridge retrofitted with this technology.

Furthermore, as stated previously, long term performance of the retrofitted bridge could deteriorate without proper County maintenance. The fiber wrapped portions of the bridge will need to be repainted periodically at a future cost to the County. In addition, fiber wrap repair cost for repairs to damage caused by vehicular impacts would be an added expense to the County.

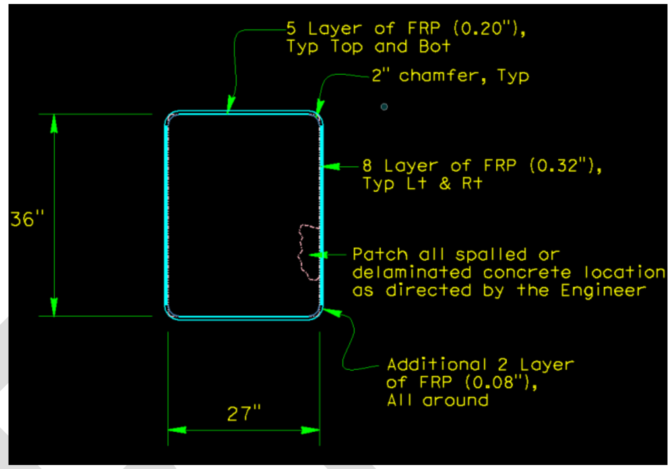
<u>Arch Rib Retrofit</u>

The Arch Rib retrofit is required in the Spring Line section of the arch, and will comprise several components of the FRP system as follows:

- On the top and bottom of the arch, 5 layers of 0.04" of FRP (Total 0.20").

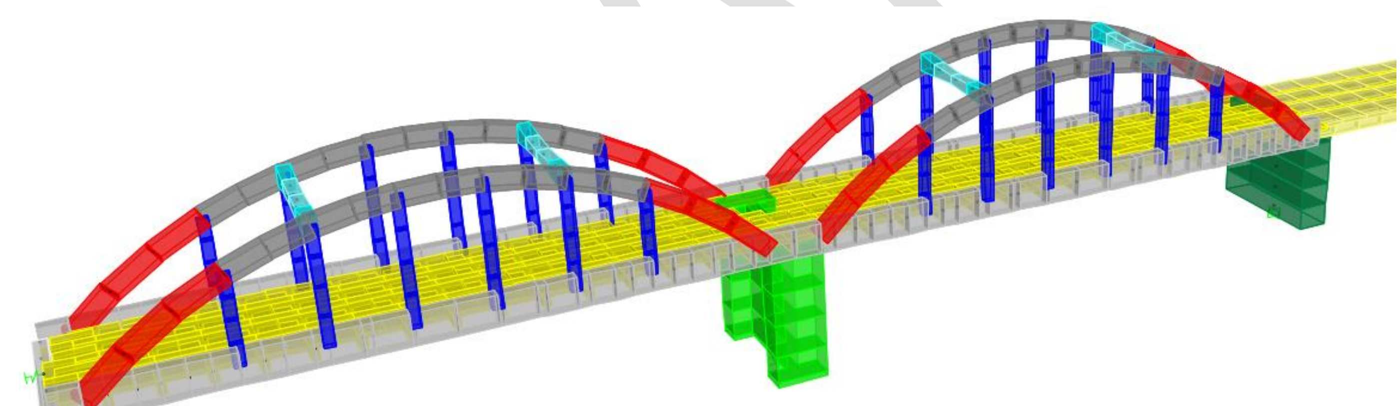
- On the side (inside and outside) faces of the arch, 8 layers of 0.04" FRP (Total 0.32").

- At the spring line outside face (exterior of bridge), the FRP must be bonded to the Tie Girder and lapped with the Tie Girder FRP. At the spring line inside face (interior of bridge), holes will be drilled through the deck to allow the FRP to be passed through the deck so that it can be bonded to the Tie Girder.



Conceptual Arch Rib Retrofit Strategy

- Lastly, the arch will be wrapped with 2 layers of FRP to provide additional confinement. Red elements in the figure below indicate the approximate location of where the Arch Rib will be retrofitted with the FRP system.



Limits of Arch Rib Retrofit shown in Red

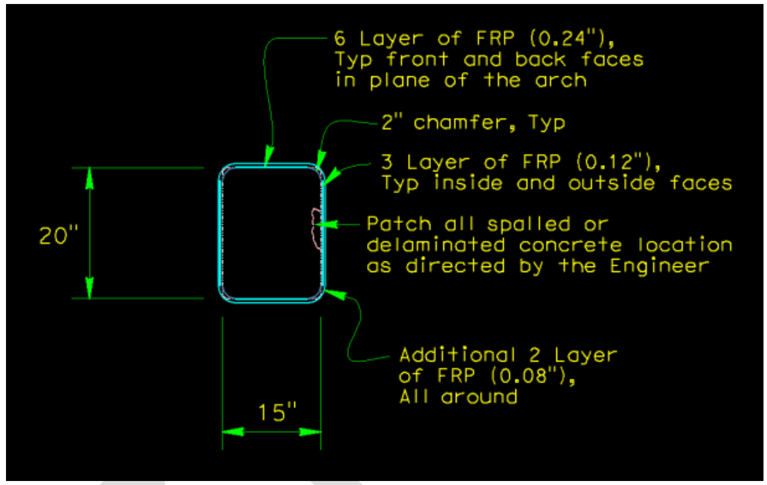
Another strategy considered but not selected included enlarging the arch cross section by encasing it in 6" to 8" of concrete and new reinforcement. This would have the same effect as the FRP. Although the arch only requires strengthening in the spring line section, the concrete encasement would have to be done over the full length of the arch to avoid changing its appearance because the encasement is so much thicker than the FRP. This would also add significant additional mass to the bridge, which would further increase seismic demands. Therefore this alternative is not recommended.

Vertical Hanger Retrofit

The Hanger retrofit includes 6 layers of 0.04" FRP (Total 0.24") on each face of the hanger in the plane of the arch, and 3 layers of 0.04" FRP (Total 0.12") on the inside and outside faces.

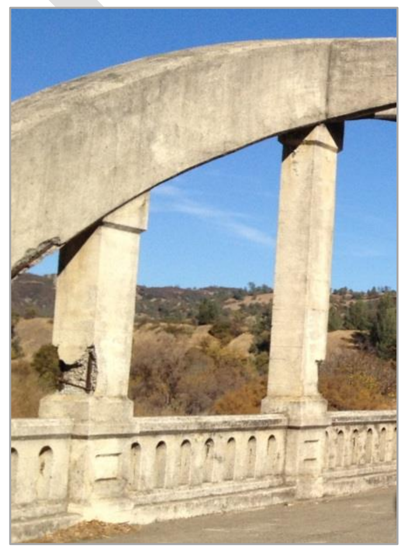
Additionally, 2 layers of FRP will be wrapped around the arch section to provide additional confinement.

As indicated by the red elements in the figure below, all hangers will be retrofitted.

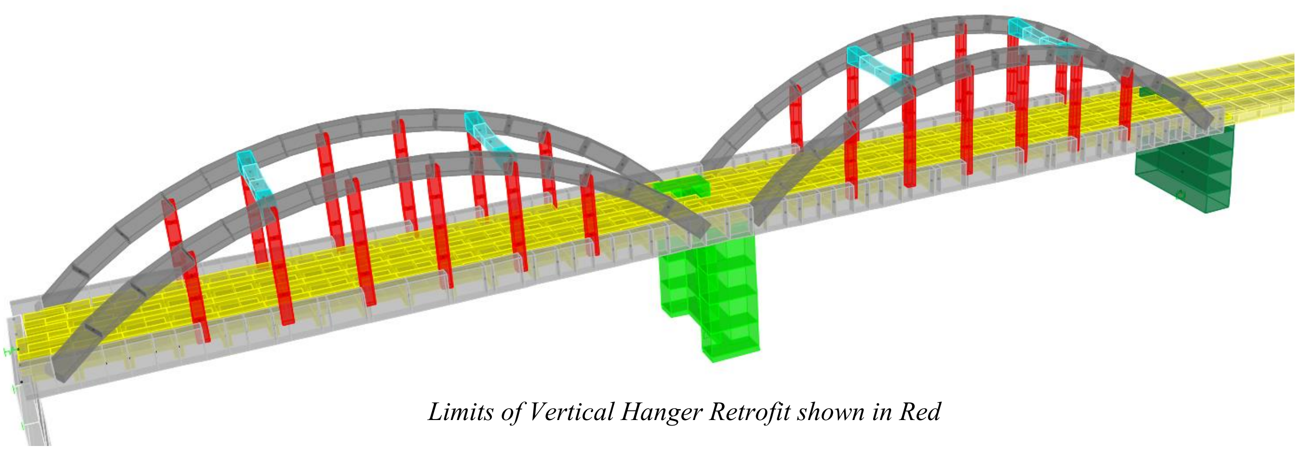


Conceptual Vertical Hanger Retrofit Strategy

The FRP will be wrapped over the full height of the Vertical Hanger. FRP will extend to the top of the Hanger and onto the Arch. FRP will also be wrapped around the Hanger, including at the top where the Hanger is enlarged as an architectural feature. At top and bottom, where the Hanger has architectural features, a 4:1 slope of epoxy/mortar will be constructed to create a smooth transition for the FRP to extend onto the wider Arch or Tie Girder. At the bottom of the Hanger, where the member is integral with the Tie Girder and bridge railing, the bridge railing will have to be removed (except for horizontal bars) so that the FRP can be wrapped to the bottom of the vertical member where it meets the bridge deck. Holes will be drilled through the deck to feed the FRP material through the deck and to wrap around the Tie Girder. After the Hanger is wrapped, the bridge railing will be reconstructed.



Vertical Hanger architectural detail



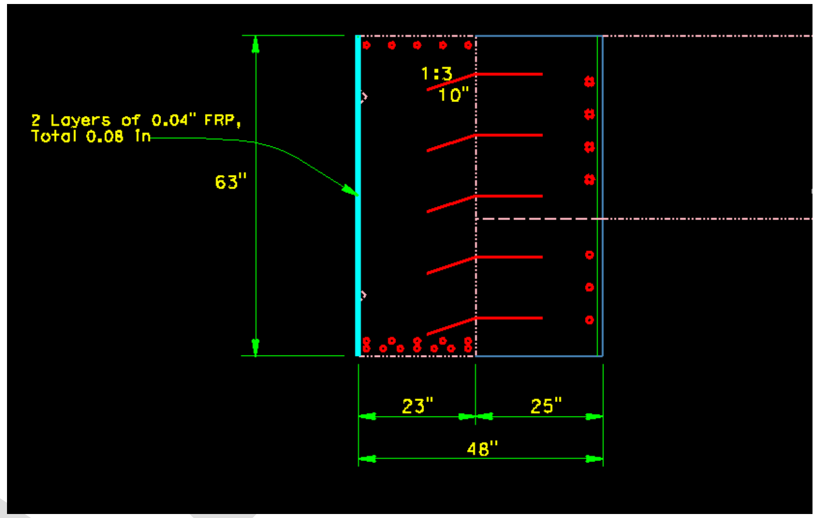
Limits of Vertical Hanger Retrofit shown in Red

Tie Girder Retrofit

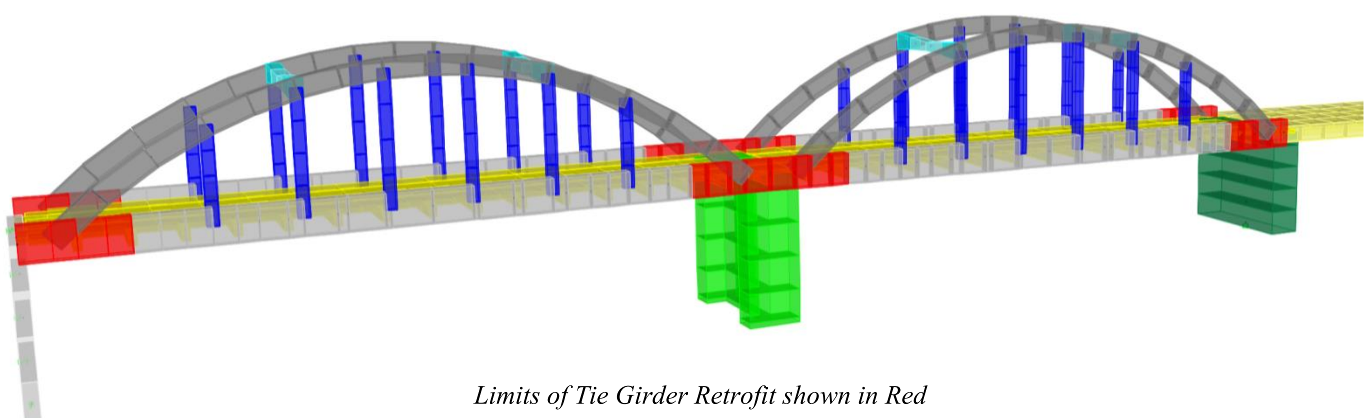
The Tie Girders are deficient in flexure at the supports. They are not "thick" enough so must be made thicker. They must also be made stronger since there is little reinforcement in their side (vertical) faces. Thus their retrofit consists of the following:

- Enlarge the Tie Girder with a concrete bolster to the inside of the girder at each support (abutments and pier)
- Apply FRP to the outside face of the Tie Girder, 2 Layers of 0.04" FRP (Total 0.08") will be applied to increase bending strength in the other direction.

This strategy allows the limit of retrofit to be applied where it is necessary. Red elements in the figure below indicate the approximate location of where the Tie Girder will be retrofitted with FRP and section enlargement.



Conceptual Tie Girder Retrofit Strategy



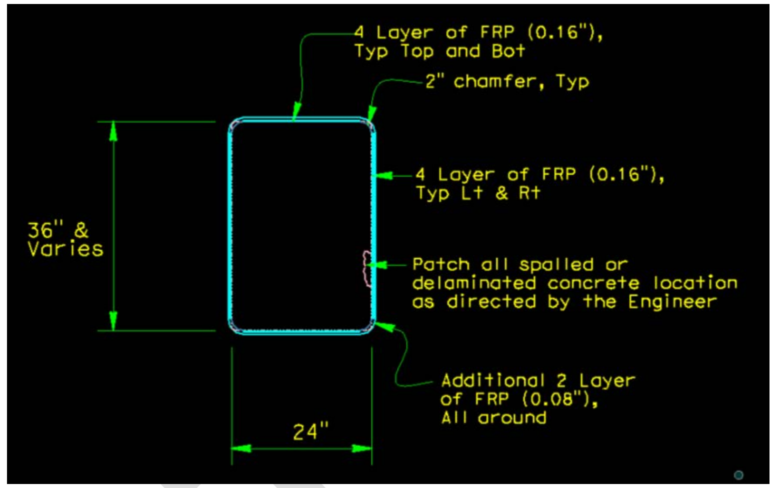
Limits of Tie Girder Retrofit shown in Red

Another strategy considered but not selected included constructing a concrete bolster on both the interior side and exterior side of the Tie Girders. Similar to the Arch Rib alternate retrofit, this alternative would require the Tie Girder to be enlarged from beginning of the bridge to the end of the arch span in order to not significantly alter the appearance of the bridge. Again, this strategy would add additional mass to the bridge, which would increase the seismic demands. This alternative is not recommended.

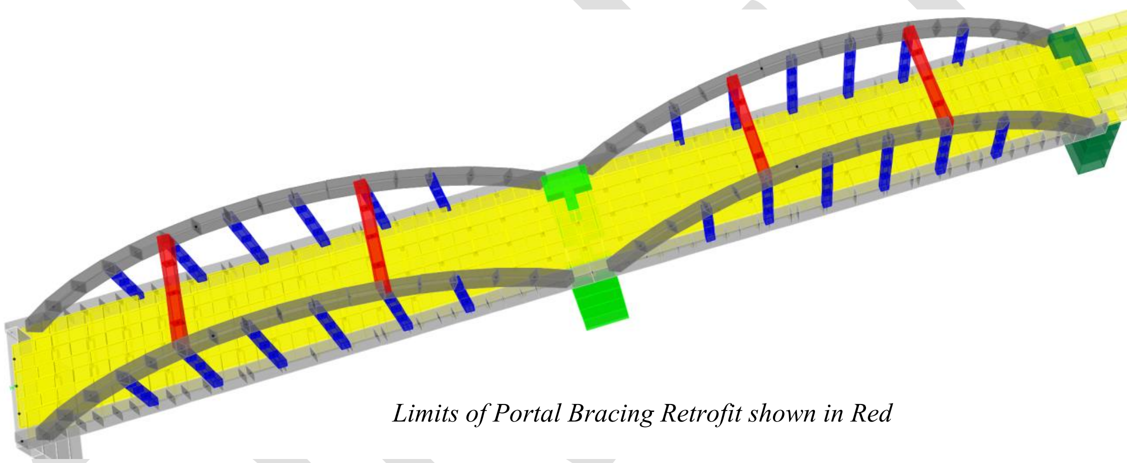
<u>Portal Bracing Retrofit</u>

The Portal Brace retrofit entails applying 4 layers of 0.04” FRP (Total 0.16”) to each face along the length of the member. In addition, the brace will be wrapped with 2 layers of FRP to provide additional confinement.

Red elements in the figure below indicate the approximate location of where the Portal Bracing will be retrofitted.



Conceptual Portal Bracing Retrofit Strategy

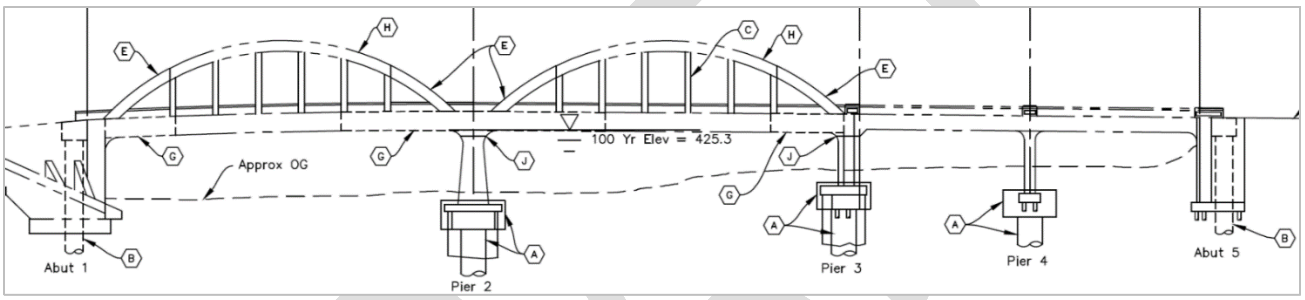


Limits of Portal Bracing Retrofit shown in Red

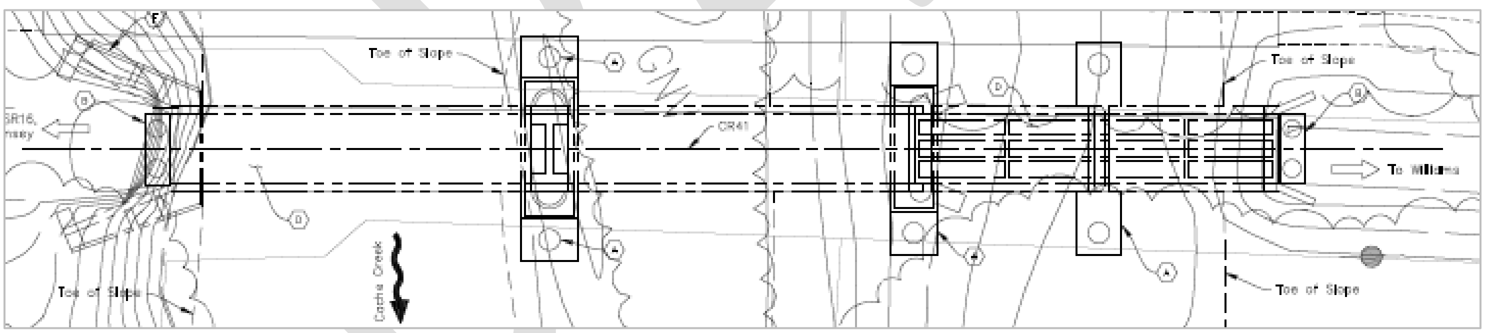
Substructure Retrofit

The retrofit of the footings and foundations of all supports is required to maintain stability under scour and seismic events. The scour issues discussed earlier are a significant threat to the bridge and the foundations must be strengthened to resist this condition. At the same time the foundations must also be strengthened for seismic demands. While these two conditions do not occur at the same time, the retrofit will provide for both cases, with the more severe of the two conditions controlling the design.

Deep foundations consisting of large diameter CIDH piles will be installed and connected to the existing footings. The piles will provide both vertical and lateral support to supplement and/or replace the existing foundations depending on the load condition. The seismic analysis also shows the existing footing to be inadequate so these must be enlarged both to strengthen the footing and to link the CIDH piles to the existing bridge. Preliminary analysis indicates that two 60” piles will be required behind the existing abutments, while each pier will require two 84” large diameter piles placed outside of existing pier footing footprint.



Elevation View – Conceptual Substructure Retrofit Strategy



Plan View – Conceptual Substructure Retrofit Strategy

Another strategy considered included separating the superstructure from the substructure by the means of “Base Isolation”. These bearings would decrease the seismic demands in the entire bridge by effectively lengthening the structural period, i.e., making the bridge more flexible. This strategy was not considered feasible because of a host of structural complications associated with disconnecting the arch spans from their supports. In addition, it would not address scour issues that threaten the stability of the supports. Therefore this strategy is not recommended.

Other Minor Retrofit & Rehabilitation

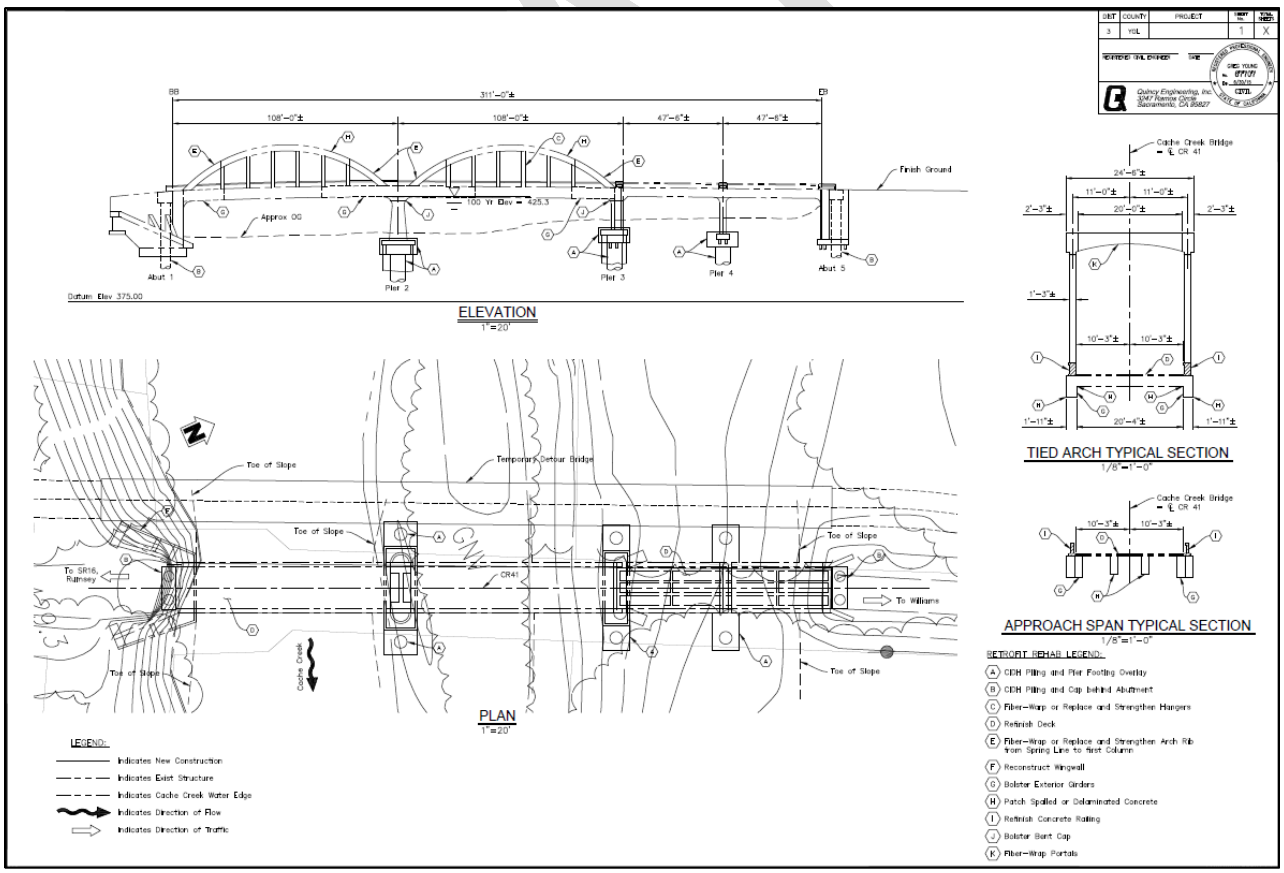
While the bridge is being retrofitted, other items of work that should be completed include reconstructing the missing wingwall (washed out in 1995), reinstalling a more robust RSP system, refurbishing the existing bridge railing (if SHPO requires keeping the existing rail) or replacing it, and placing a polyester concrete overlay on the deck.

6. ROADWAY AND BRIDGE LAYOUT ALTERNATIVES

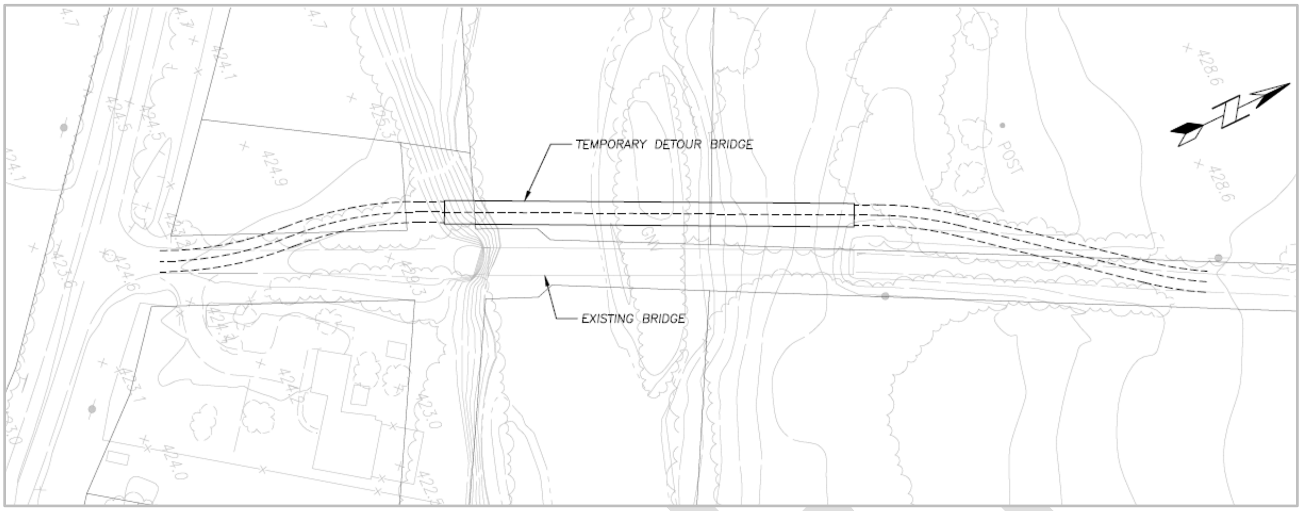
Multiple alignment alternatives were developed as part of this study. Five alternatives were identified and evaluated. Three warranted further consideration, while two were dismissed out of hand. Finally, a "Do Nothing" alternative was also considered. The advantages and disadvantages for each viable alternative are listed below. All alignments were developed to minimize environmental, traffic, and right-of-way impacts to the fullest extent possible.

Alternative 1 – Existing Bridge Retrofit/Rehabilitation

For an alternative that maintains the existing bridge, the roadway alignment would remain the same. A detour via a temporary bridge alignment would be required during the construction duration because the existing bridge is not wide enough to allow both retrofit equipment/operations and public traffic at the same time. A longer construction duration would be required for installation of a temporary detour bridge. The detour right-of-way and environmental impacts would be temporary when compared to the permanent impacts associated with a new roadway alignment. Since the existing bridge would be rehabilitated, no improvement to the vertical profile could be addressed, nor could the hydraulic clearance deficiency be improved. Finally, keeping the existing bridge would not provide a wider roadway and bridge that would meet the route's functional classification, based on the Yolo County Improvement Standards.



Alternative 1 – Advanced Plan Study (APS)



Detour Route - Plan View for Retrofit/Rehabilitation Alternative

Advantages:

- Provides safe river crossing
- Preserves historical significance & aesthetics of the original Rumsey Bridge (although retrofitted details would have visual impacts)
- Lower permanent Right-of-Way impact

Disadvantages:

- Requires extensive superstructure retrofit
- Requires extensive substructure retrofit
- High bridge retrofit & rehabilitation cost
- High risk of construction cost overruns while working on existing historic structure
- Lower design life than replacement alterative
- Higher maintenance costs maintain fiber wrap seismic retrofit materials
- Higher impact on traffic during construction & Temporary detour requirement
- Does not improve hydraulic conveyance and address scour vulnerability
- Not designed to current vehicular loading requirements
- Non-standard roadway width

Alternative 2 – New CIP Concrete Box Girder Bridge on Upstream Alignment

A detour around the site is 63 miles long and has been determined to be unacceptable. Therefore a creek crossing at this location must be maintained at all times, regardless of the bridge alternative chosen. For any bridge replacement alternative, the roadway alignment options are limited to either following the existing alignment or an upstream alignment. In the former scenario, a temporary detour bridge would be required to allow a new bridge to be constructed on the existing alignment. For the upstream alignment the roadway could diverge from the existing roadway on a skew to the existing roadway, rather than a parallel alignment.

By doing so, the new roadway would simply pass through a gentle curve to a new tangent alignment that intersects Highway 16 northwest of the existing intersection. On the other hand, a bridge constructed parallel to the existing would require a reversing "S" curve alignment at the northeast end of the bridge. Constructing a new bridge would enable the southwest abutment to be shifted farther up the bank to allow more flow to pass underneath the bridge and greatly reduce potential scour issues at that support. This roadway alignment option would also allow the existing bridge to remain in service during construction which would eliminate the need for a costly temporary bridge.

A key element of this alternative would be keeping the existing bridge in place in case it is determined by SHPO that the bridge should not be removed. However, this alternative would not provide for any rehabilitation or other work to extend the life of the existing bridge. As a result, the existing bridge would continue to deteriorate and at some point would become a hazard for the County to address, eventually requiring the closure of the bridge. Eventually the bridge could collapse or be so close to collapse after an extreme seismic or hydraulic event that the County would need to remove the bridge due to safety and liability concerns. Since the County will have already used federal funds to replace the bridge, no funding mechanism would be available to maintain or even remove the collapsed structure. Keeping the existing historic bridge in place would still allow the historic resource to be enjoyed. However, there could still be a visual impact to the historic bridge because the new structure would affect the views to and from the existing bridge.

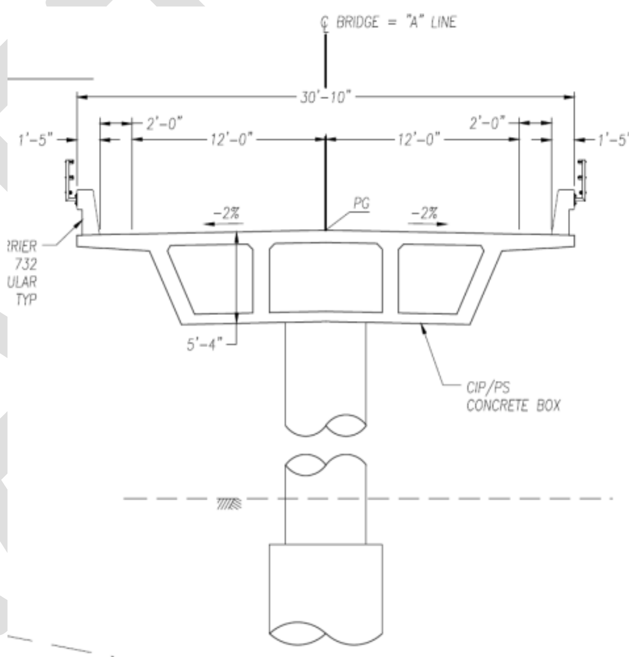
Bridge Replacement Upstream Option – Aerial View

New Bridge

Cast-in-place (CIP) concrete box girders are the most common bridge type in California. They are also the most cost effective structure where falsework can be employed to construct the bridge and there are no major time constraints for work within the waterway.

Several span configurations have been considered for the replacement bridge. Longer spans reduce the number of supports, which would reduce the hydraulic impacts in the creek. On the other hand, longer spans require a deeper superstructure, which would require the roadway profile to be raised. The amount the profile can be raised is controlled by the conform to Highway 16, and the desire to keep the maximum grade to no more than 5% for ADA compliance. A three-span configuration best balances these two competing requirements.

A CIP Concrete Box Girder bridge would be supported by single column bents on large diameter CIDH or CISS piles. Large diameter piles are very cost effective and perform well in high seismic and scour zones. Single column bents would also reduce issues with debris and therefore result in lower maintenance costs associated with debris removal around the bridge.



Replacement Typical Section

Advantages:

- Provides a separate safe river crossing
- Provides a long term, low maintenance structure
- Improved hydraulic capacity by raising the profile and reducing supports in the creek
- Traffic would use the existing structure during construction
- Improved intersection geometry with SR16 because the skew angle is reduced
- Improved location of Abutment 1
- Preserves existing bridge aesthetics (although new structure would be a visual impact)
- Lowest construction cost

Disadvantages:

- Does not address the hydraulic performance under the existing bridge
- Existing bridge is still vulnerable to hydraulic and seismic events
- Public safety at risk if existing bridge collapses while in use
- High liability risk to the County
- No funding mechanism for future maintenance or removal, after collapse of the original non-retrofitted bridge
- More Right-of -Way and environmental impacts

Alternative 3 – New CIP Concrete Box Girder Bridge on Upstream Alignment

This alternative is the same as Alternative 2, providing for a new Cast-in-Place Concrete Box Girder Bridge. The only difference is that the existing bridge would be removed.

Advantages:
- Provides one safe river crossing
- Provides a long term, low maintenance structure
- Improved hydraulic capacity by raising the profile and reducing supports in the creek
- Traffic would use the existing structure during construction
- Improved intersection geometry with SR16 because the skew angle is reduced
- Improved location of Abutment 1
- Lower construction cost compared to retrofit alternative

Disadvantages:
- Loss to the local community who values the existing bridge aesthetics
- More Right-of -Way and environmental impacts

Alternative 4 – New CIP Concrete Box Girder Bridge on Downstream Alignment (Rejected)

A downstream alignment is not considered feasible due to the conflict with the property on the southeast side of the existing bridge. Therefore, a bridge replacement on the downstream alignment is rejected.

Alternative 5 – New CIP Concrete Box Girder Bridge on the Existing Alignment (Rejected)

A bridge replacement on existing alignment was considered uneconomical. Benefits from smaller right-of-way cost would not outweigh the benefits of bridge replacement on a tangent upstream alignment. This alternative would require a costly temporary detour bridge and could delay the construction schedule. Furthermore, replacement on existing alignment with a raised profile for hydraulic conveyance would require extensive fill which could impact the adjacent southeast property. Hence, a bridge replacement on the existing alignment is rejected.

"Do Nothing" Alternative (Rejected)

This alternative must be considered for the environmental process for historic bridges. Taking no action does not meet the purpose and need of the project. As previously mentioned, the Rumsey Bridge is vulnerable to collapse during a major hydraulic or seismic event. Not taking any actions puts the public at risk, and is unacceptable. This alternative has been rejected from further consideration.

7. CONSTRUCTION COSTS

Construction costs have been developed based on preliminary quantities and unit costs for similar projects. A 10% mobilization and 25% contingency are included in the total costs to account for uncertainty in the preliminary phase. All costs are presented in 2014 dollars.

Alternative 1 – Retrofit/Rehabilitation of existing Rumsey Bridge.

The estimated construction cost is $10,800,000.

(The Stevenson Bridge project construction cost estimate, as programmed in the November 2013 Caltrans HBP list, is $6,372,000. The Rumsey Bridge retrofit cost (excluding the roadway cost) is higher because the Rumsey Bridge is in worse structural condition and is in a higher seismic zone as described in the *Similar Structure (Stevenson Bridge) Study* in Chapter 2.)

Alternative 2 – Add a new CIP Concrete Box Girder Bridge on an upstream alignment and close the existing Rumsey Bridge (Keep existing bridge).

The estimated construction cost is $3,900,000.

This Alternative has the lowest overall cost today. However, it does not account for future cost to remove the bridge, if and when the bridge requires demolition due to earthquake or storm damage.

Alternative 3 – CIP Concrete Box Girder Bridge Replacement on upstream alignment (Remove existing bridge).

The estimated construction cost is $4,500,000.

This alternative costs the same as Alterative 3 but includes the cost to remove the existing bridge.

Following completion of the 95% design, the engineer's estimate will be updated utilizing final bridge design quantities and updated unit prices that reflect the most current historical cost information available at the time.

These estimates do not include costs associated with right of way acquisition, environmental clearance, professional engineering, construction support or construction engineering. Professional engineering includes Plan Specifications & Estimates preparation (PS&E).

8. LIFECYCLE COST

Based on preliminary construction estimated costs, the replacement alternatives cost less than the retrofit/rehabilitation alternative cost. However, these construction costs alone do not consider the life cycle costs associated with the remaining shorter service life of the retrofitted existing bridge in Alterative 1, nor the future cost of the bridge removal required in Alternative 2. Therefore, a life cycle cost analysis was completed to determine normalized cost in 2017 dollars. Typically a life cycle analysis brings the future construction and maintenance costs to today's present value. However, since this project requires a lengthy environmental process and the earliest anticipated construction award date is in 2017, a future value analysis was performed to normalize costs to 2017 dollars for each of the alternatives.

For the life cycle analysis, the following assumptions were made:

- New bridges designed to current standards are designed with a 75 year life expectancy.
- All replacement alternatives assume that the structure will be relatively maintenance free for their service life of 75 years.
- A 75 year cycle analysis is applied to each alternative to form a common baseline timeframe.
- To account for inflation, construction costs were estimated to increase at a rate of 3% per year, based on Caltrans Construction Cost Index information, see **Appendix D**. The future value equation as for the analysis is as follows. $FV = PV * (1 + i)^N$
- The expected service life for the Alternative 1 retrofitted bridge is estimated between 50 to 75 years. Conservatively, a 50 year retrofitted life expectancy is used in the analysis. The carbon fiber and the exterior paint system are susceptible to decay due to ultraviolet (UV) exposure. Without intermittent maintenance (repaint everything 10 year), the fiber wrap will eventually lose structural capacity. The cost of repainting the FRP every 10 years is estimated to be $100,000 in year 2027 (for permit, bid, award, mobilize, prep). This does not include the cost of any repairs that may be necessary due to damage to the FRP from vehicular impacts. The repainting and FRP repair cost escalation is estimated at an additional $25,000 every 10 years to account for inflation and vehicle impact and damage to the FRP system.
- A discount rate of 1% per year is utilized when calculating Present Value. $PV = FV / (1 + i)^N$

  The Present Value equation is used to calculate the 2017 costs required for future tasks to construct a bridge replacement in **Alternative 1**, and to remove the damaged un-retrofitted Rumsey Bridge in **Alternative 2**. A 1% discount rate is conservatively used to reflect the current low market savings interest rate.
- For **Alternative 2**, the original 1930 arch spans of the bridge are 83 years old and have already exceeded their design service life. Removal is assumed to be required in 10 years for the analysis, assuming an earthquake or scour event will cause the bridge to collapse. While it is unknown when a significant earthquake or scour event will occur, due to the age of the structure and the rate of deterioration, collapse could occur at any time during the analysis period.

Detailed lifecycle cost analyses for each alternative are detailed in the following pages:

Life Cycle Costs

Net Future Value adjusted to the proposed 2017 construction year (in 2017 dollars)

Alternative 1 - Bridge Retrofit/Rehabilitation of existing structure

Design Life of Retrofitted Bridge (50-75yrs) = 50 years

Cost to Retrofit Bridge if retrofitted today = $10,800,000 (see quantities & estimates)

$FV = PV * (1 + r)^n$

*Future Cost = Initial Cost * (1 + Annual % Increase)^{years}*

Annual % increase in Construction Cost = 3 %

Future Cost in 2017 to retrofit existing bridge = $11,801,000

Cost of a future bridge if constructed today = $4,500,000 (assumed cost for CIP Conc Box Girder in Alt 3)

$FV = PV * (1 + r)^n$

*Future Cost = Initial Cost * (1 + Annual % Increase)^{years}*

Annual % increase in Construction Cost = 3 %

Future Cost in 2067 to construct a new bridge = $21,557,000

Prorated fraction cost to a common 75 year baseline timeframe analysis. (Prorate cost of new bridge to first 25 years of its life.)

Proration multiplier = (75-50)/75 = 25/75 = 0.333

$7,185,667

Initial Deposit Required

$PV = FV / (1 + r)^n$

where,

P = Future Value = Cost of prorated 2067 bridge = $7,185,667

r = discount/interest rate = 1 %

n = # years discounted 50

Solving for initial Deposit Required = $4,369,000 (amount needed in 2017 to fund a future prorated New Bridge in year 2067)

Initial Deposit Required

$PV = FV / (1 + r)^n$

where,

P = Future Value = Cost of paint and/or FRP repair from vehicle impact/damage in 2027

$100,000

r = discount/interest rate = 1 %

n = # years discounted 10

Solving for initial Deposit Required = $91,000

Initial Deposit Required

$PV = FV / (1 + r)^n$

where,

P = Future Value = Cost of paint and/or FRP repair from vehicle impact/damage in 2037

$125,000

r = discount/interest rate = 1 %

n = # years discounted 20

Solving for initial Deposit Required = $102,000

(Alternative 1 cost estimate continued on following page)

Initial Deposit Required

$PV = FV / (1 + r)^n$

where,

P = Future Value = Cost of paint and/or FRP repair from vehicle impact/damage in 2047
$150,000

r = discount/interest rate = 1 %

n = # years discounted 30

Solving for initial Deposit Required = $111,000

Initial Deposit Required

$PV = FV / (1 + r)^n$

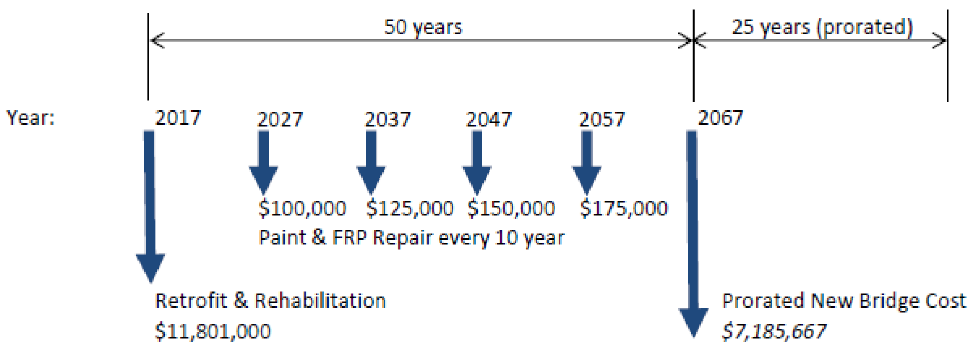
where,

P = Future Value = Cost of paint and/or FRP repair from vehicle impact/damage in 2057
$175,000

r = discount/interest rate = 1 %

n = # years discounted 40

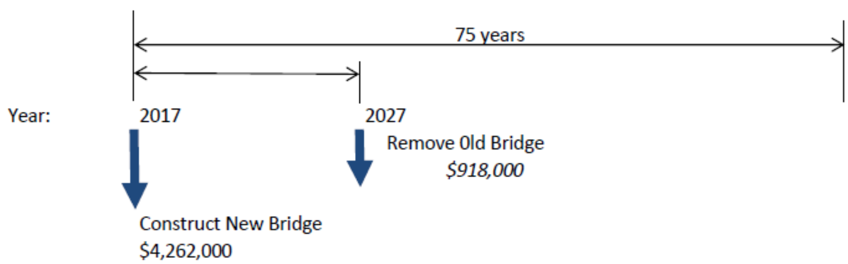
Solving for initial Deposit Required = $118,000



Net Future Value Cost =	$ 11,801,000	
+	$ 4,369,000	Amount needed in 2017 to fund a future prorated New Bridge in year 2067
+	$ 91,000	Paint & Repair in 2027
+	$ 102,000	Paint & Repair in 2037
+	$ 111,000	Paint & Repair in 2047
+	$ 118,000	Paint & Repair in 2057
Net Future Value Cost =	**$ 16,600,000**	**for Alternative 1** (rounded to nearst $100,000)

Alternative 2 - Add New CIP Concrete Box Girder Bridge on an upstream alignment and keep existing Rumsey Bridge (no retrofit)

Design Life of New Bridge (75-100yrs) =	75 years	
Cost of New Bridge if constructed today =	$3,900,000	(Bridge Replacement without existing bridge removal cost)
$FV = PV * (1 + r)^n$		
*Future Cost = Initial Cost * (1 + Annual % Increase)^{years}*		
Annual % increase in Construction Cost =	3 %	
Future Cost in 2017 to construct new bridge =	$4,262,000	
Assumed when Bridge Removal is Required =	10 years	
Cost to Remove Bridge if removed today =	$625,000	(Slightly higher cost than Alt 1's, since Contractor needs to remobilize)
$FV = PV * (1 + r)^n$		
*Future Cost = Initial Cost * (1 + Annual % Increase)^{years}*		
Annual % increase in Construction Cost =	3 %	
Future Cost in 2027 to remove old bridge=	$918,000	
Initial Deposit Required		
$PV = FV / (1 + r)^n$		
where,		
P = Future Value = Desired future value =	$918,000	
r = discount/interest rate =	1 %	
n = # years discounted	10	
Solving for initial Deposit Required =	$831,000	(amount County needs to have in year 2017 to remove existing old bridge in year 2027)



Net Future Value Cost = $4,262,000 + $831,000

Net Future Value Cost = $5,100,000 for Alternative 2 (rounded to nearst $100,000)

Alternative 3 - CIP Concrete Box Girder Bridge Replacement on an upstream alignment with removal of the existing bridge

Design Life of New Bridge (75-100yrs) = 75 years

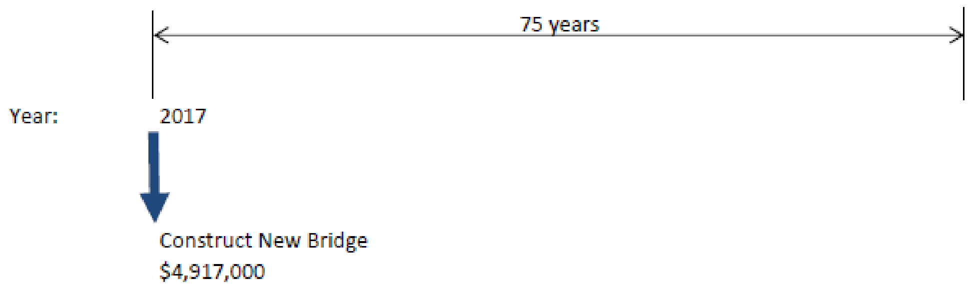
Cost of New Bridge if constructed today = $4,500,000 (Bridge Replacement <u>with</u> existing bridge removal cost)

$FV = PV * (1 + r)^n$

$\text{Future Cost} = \text{Initial Cost} * (1 + \text{Annual \% Increase})^{\text{years}}$

Annual % increase in Construction Cost = 3 %

Future Cost in 2017 to construct new bridge = <u>$4,917,000</u>



Net Future Value Cost = $4,900,000 for Alternative 3 (rounded to nearst $100,000)

Base on the Life Cycle Cost analysis, the future cost (in year 2017) for Alternative 1 is $16.6 million, for Alternative 2 is $5.1 million, and for Alterative 3 is $4.9 million, as shown in the figures and calculations above.

9. CONCLUSION

Quincy Engineering evaluated a Retrofit/Rehabilitation Alternative and four Replacement Alternatives for the Rumsey Bridge. Advantage and disadvantages along with construction costs for each Alternative were tabulated in the Executive Summary. This Feasibility Study serves as a tool for Caltrans and Yolo County to make a decision on how to proceed next with the project.

The decision of whether to rehabilitate/retrofit or replace the existing Rumsey Bridge cannot be made solely on cost. It will also have to consider other factors, such as the historical character of the existing bridge, impacts to the local community, environmental considerations, and future costs. No recommendation is provided at this time. A final recommendation will be made after receiving supplementary public feedback and going through Caltrans review process.