Bill Williams River Concrete Bridge Fire Damage Assessment

By Martha Davis, P.E., Paul Tremel, P.E., and Alex Pedrego

The purpose of this Arizona Department of Transportation (ADOT) project was to assess the damage to the Bill Williams River Bridge from a July 2006 tanker accident and fire. The Bill Williams River Bridge, Structure #1272, is located on SR 95 at milepost 161.73 between Parker and Lake Havasu City in Arizona.

At approximately 3 pm on Friday, July 28, 2006 a tanker truck overturned in the middle of the concrete bridge and spilled its load of diesel fuel, which caught on fire. All that remained of the truck after the fire was extinguished was the steel frame of the cab and the axles. The melted aluminum tank was reduced to a layer about 3 inches thick.

The majority of the fire occurred on the bridge deck, lasted about two and a half hours, and damaged three spans including the concrete deck and barrier. The fire under the bridge damaged the pier beam, concrete girders, and underside of the deck. ADOT visited the site on July 31, 2006 to perform a damage assessment inspection, and reopened the bridge to traffic thereafter. The wildlife refuge manager estimated that less than 10% of the 7600 gallon load entered the Bill Williams River through deck drains and expansion joints in the bridge deck. The fire spread east of the bridge and burned for two and a half weeks, affecting 385 acres of the Bill Williams National Wildlife Refuge.

July 28, 2006 Fire – Looking North at West Side of Concrete Bridge. Courtesy of Bill Williams National Wildlife Refuge, U.S. Fish and Wildlife Service.

Existing Structure

The existing Bill Williams River Bridge was built in 1967. It consists of fourteen 76-foot simple spans and two 30-foot concrete slab approach spans. The fourteen main spans are composed of precast, prestressed concrete AASHTO Type III girders with a 6½-inches cast-in-place concrete deck. The bridge is approximately 1125 feet long and over 35 feet wide. The original concrete curb and H-2-1 (two-tube) railing were replaced with concrete barriers in 1986. The concrete deck does not have an overlay.

The Bridge has deck drains on both sides of the bridge, at the face of the concrete barrier. There are four drains along each side of the 76-foot spans and one drain on each side of the 30-foot spans. The drains consist of 4-inch by 6-inch formed holes in the deck, and drain directly into the river below.

Post-Fire Inspection, Materials Testing and Load Rating

Inspection

A detailed inspection of the Bridge was performed by HDR Engineering, Inc. in November 2006. The inspection was limited to the fire-damaged portions, which were in Spans 8, 9, and 10. Due to the remoteness of the site and the lack of a feasible detour, traffic control was utilized to close only one lane at a time during the inspection. Two inspectors accessed the underside of the bridge with an under bridge inspection vehicle. The inspection involved observations of the barriers, girders, pier cap and columns, and top and underside of the deck. These areas were also sounded with a hammer to detect locations of delamination in the concrete. Locations of concrete spalls and exposed mild and prestressed reinforcing were noted.

Girders are numbered 1 through 6 from west to east. This numbering scheme is consistent with the original construction drawings. Span 9, where the fire was concentrated, is located between Piers 8 and 9. A partial plan and typical section are shown in *Figures 1* and *2*, respectively.

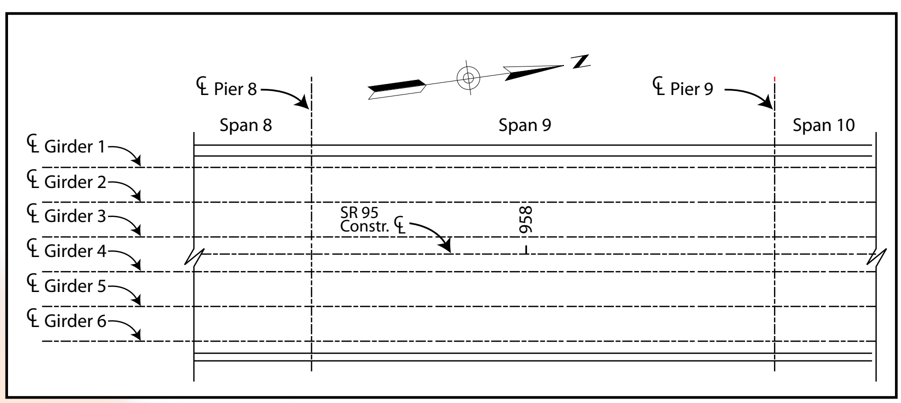

Figure 1: Partial Bridge Plan View.

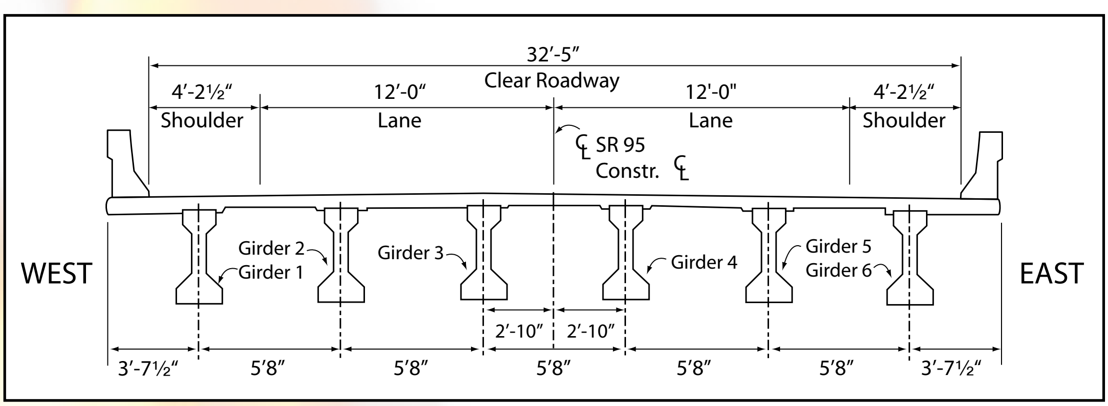

Figure 2: Typical Section.

Based on the pattern of the fire damaged areas, it appears that much of the diesel fuel and fire reached the underside of the bridge through the deck drains on the west side of the bridge. Fuel and fire also came through the drains on the east side of the bridge, though to a lesser extent than on the west side. It appears that the majority of the fire was in Span 9, with evidence of fire limited to the two deck drain locations in Spans 8 and 10 adjacent to Span 9. The wind was blowing west to east during the fire. Therefore, the damage to the girders was localized to the drain locations on Girders 1 and 2, and spread to most of the girder length for the remaining four girders. The east end of the pier cap at Pier 8, the barrier along the east side of the bridge, and Girder 6 in Span 9 exhibited the greatest amount of spalled and delaminated concrete.

The fire-damaged girders do not show visible signs of loss of prestressing force, as they do not appear to sag and flexural cracks are not present. There was significant spalling of the top and bottom flanges of Girder 6 in Span 9, with a majority along the exterior (east) face. The other five girders in Span 9 exhibited more localized spalling. The bottom flange at the ends of the girders at Piers 8 and 9 exhibited varying degrees of spalling, with the most significant spalling on Girder 6 at Pier 8. Varying lengths of the bottom corners of the bottom flange of all girders were spalled. The corner prestressing was exposed at the larger chamfers. Except for Girder 2, there are locations on all girders in Span 9 of exposed prestressing strand, varying in length from 3 inches to approximately 8 feet.

A dull tone was produced when the underside of the east deck over-hang in Span 9 was sounded with a hammer. The surface was easily chipped away. These characteristics indicate delamination and significantly deteriorated internal strength properties.

Materials Testing

Materials sampling and testing was performed by CTLGroup, an independent testing laboratory. Two CTLGroup representatives were on site during the inspection to perform non-destructive evaluation and to take material samples for laboratory testing. The sample locations were selected from areas that appeared to be the most severely damaged, to provide conservative values of estimated material strength.

Six concrete cores were taken for compressive strength testing and petrographic analysis. Three cores were taken from the deck and one core each from the pier cap at Pier 8, web of Girder 6 in Span 9, and the east barrier in Span 9. According to CTLGroup's report, "the overhang concrete has most likely lost a significant portion of its original strength and should not be considered in evaluation of the overhang". Based on these findings and the inspection observations, the east portion of the roadway was closed to traffic.

The concrete strength results, each only from a single test, are as shown in *Table 1*.

The strength of three reinforcing steel samples was tested. A #5 mild reinforcing bar from the deck had yield strength of 52,300 psi.

Table 1: Concrete Strengths.

Element	Strength (f′c) per As-Builts	Post-Fire Test Results
Deck	3,000 psi	3,950 psi
Girder	4,800 psi	5,800 psi
Pier Cap	2,500 psi	5,050 psi

A #4 mild reinforcing bar from an exposed girder stirrup had yield strength of 57,300 psi. A sample of prestrssing steel from the south end of Girder 6 in Span 9 was too short to perform a tensile test, but microhardness testing results roughly convert to tensile strengths between 255 and 292 ksi.

Petrographic analysis of the cores identified the depth of: paste color change, paste softening, microcracks, cracks, and carbonation. According to the report, "all of the concrete specimens exhibited abnormal microcracking at various depths and some level of paste alteration due to the fire."

Quoting CTLGroup's report: "Paste softening, carbonation, and microcracking can adversely affect the strength and durability of concrete and contribute to accelerated corrosion of the reinforcement and perhaps shorten service life. Abnormal microcracking is one indicator of depth of damage in the bridge components and results in reduced section properties for the fire-affected girders and deck that will occur over time with cyclic environmental and traffic loading. Therefore, the calculation of structural capacities for an ultimate limit state condition should not include the depth of concrete that exhibits this abnormal microcracking since that portion of the concrete may be prone to accelerated deterioration in the future. However, for near term assessment of the structural capacity (load rating) of the damaged concrete sections, the depth of concrete exhibiting paste alteration (softening and/or color change) should be excluded from the section properties of the element being evaluated."

A summary of the findings from the petrographic analysis included in the report is provided in *Table 2* below.

Table 2: Summary of Petrographic Analysis.

Core	Location	Maximum Depth of Paste Alteration		Depth of Microcracks	
		Top	Bottom	Top	Bottom
C1	Deck, Span 9	0.6"	0.2"	0.8"	0.4"
C2	Deck, Span 8	0.8"	0"	0.7"	1.7"
C3	Deck, Span 9	0.8"	0.1"	1.2"	0.4"
C4	Girder G6 Web, Span 9	0.4" (Interior)	0.6"	0.4"	0.8"
C5	Pier 8 Cap	1.0" (Exterior)	N/A	0.8"	N/A
C6	Barrier, East Span 9	0.8" (Exterior)	N/A	1.6"	N/A

Color change in the paste of the concrete is indicative of the temperature to which the concrete is exposed. The paste color begins to change at 230°C (450°F). The petrographic analysis of the concrete samples taken from the various structural elements of the bridge show color change to a depth not greater than one inch. While color change for each core was different, they all showed a gradual change from gray on the surface to medium beige, and in two cores pinkish-orange at a depth of no more than an inch. The depth of reinforcing steel is more than one inch; therefore reinforcing steel was not exposed to temperatures greater than 230°C. Since the yield strength of steel begins to decline at 425°C (800°F), the yield strength of the reinforcing steel does not appear to have been affected by the fire, as confirmed by the reinforcing steel tensile tests.

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Load Rating Analysis

Based on the findings from the inspection and materials testing, a load rating analysis was performed using reduced girder and deck cross sections. The thickness of concrete cover neglected for section properties was determined by actual spall depths and the depth of microcracks listed in the petrographic analysis. While there are many uncertainties in the extent of the damage caused by the fire, the assumptions made to develop the reduced section properties were deemed to be conservative.

The girder cross section that was analyzed combined all the damage that was observed during inspection and materials testing. The reduced section was applied to the entire length of the girder in the fire damaged area. The concrete neglected from the top and bottom flanges was based on spalls and apparent delamination. The thickness of the web was reduced by ½ inch on each side, based on observations in the field and the depth of microcracks listed in the materials testing report. One prestressing strand on each end of the bottom row was ignored since it was exposed in several locations on multiple girders. See *Figure 3* for a sketch of the reduced girder cross section used in the load rating calculations. The girders were rated for flexure and shear. Due to the deck geometry and girder spacing, the distribution factor for the exterior girder was higher for dead load but not for live load. The girder ratings are based on loading to an interior girder with HS20-44 live load with an allowable tension of $3\sqrt{f'c}$ in accordance with ADOT requirements.

The concrete deck between girders was rated for flexure. The deck thickness used was 6½ inches as measured inside the core hole, which matches the deck thickness shown on the as-built plans. Following the guidance in the materials testing report, the deck thickness used in the near term post-fire rating calculations was reduced by the maximum depth of paste alteration: 0.8 inches on top and 0.2 inches on bottom. The deck was also rated for the long term condition using the maximum depth of microcracks: 1.2 inches on top and 1.7 inches on bottom.

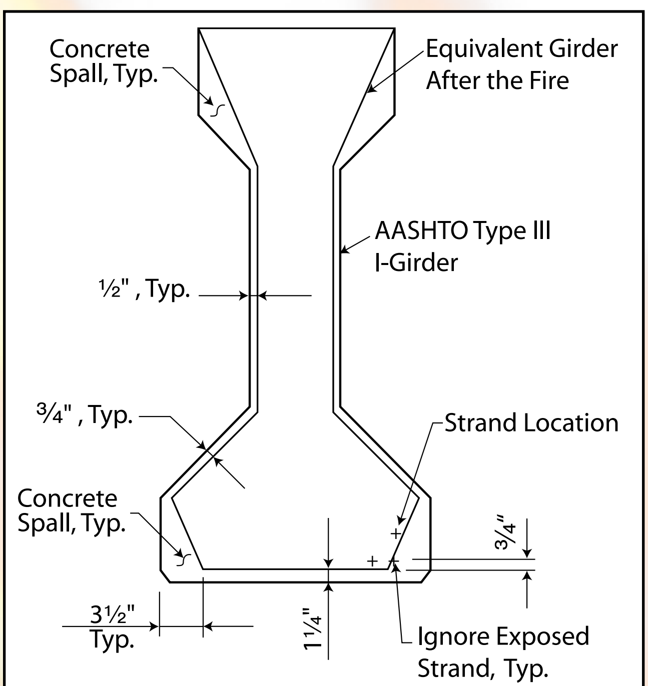

Figure 3: Reduced Girder Section.

Table 3: Inventory Ratings.

Element	Mode	Pre-Fire HS	Pre-Fire Tons	Post-Fire: Short Term HS	Post-Fire: Short Term Tons	Post-Fire: Long Term HS	Post-Fire: Long Term Tons
Deck	Flexure – LFD	28.2	50.8	26.2	47.2	***16.9***	30.4
Girder	Flexure – LFD	33.6	60.5	25.0	45.0	22.6	40.6
Girder	Flexure – ASD	24.9	44.8	20.4	36.8	***18.1***	32.6
Girder	Shear – LFD	***19.9***	35.8	***18.8***	33.9	***17.5***	31.5

Note: LFD = Load Factor Design, ASD = Allowable Stress Design

Table 4: Operating Ratings.

Element	Mode	Pre-Fire HS	Pre-Fire Tons	Post-Fire: Short Term HS	Post-Fire: Short Term Tons	Post-Fire: Long Term HS	Post-Fire: Long Term Tons
Deck	Flexure – LFD	47.0	84.6	43.7	78.7	28.2	50.8
Girder	Flexure – LFD	56.0	100.8	41.7	75.1	37.7	67.8
Girder	Shear – LFD	33.2	59.8	31.5	56.6	29.2	52.6

For comparison purposes, a load rating analysis was also done for the bridge in its pre-fire condition. The section properties were based on the as-built drawings. The post-fire concrete strengths listed previously were used for both the pre-fire and post-fire ratings, since they were higher than the 28-day compressive strengths shown on the plans. The same reinforcing steel yield strengths were also used for both ratings: 52,300 psi for the deck, 57,300 psi for the girder shear stirrups, and 270 ksi for the prestressing strand. The results of the load rating calculations are summarized in *Tables 3* and *4*.

Due to the presence of abnormal microcracking and paste alteration, along with numerous other factors, the long term post-fire ratings decreased from the pre-fire ratings by about a third. The controlling inventory rating for the near term is HS18.8, which is 33.9 tons, a 6% reduction from the pre-fire rating.

Conclusions

The concrete cover, coupled with the durable behavior of prestressed, precast concrete girders, protected the mild and prestressing reinforcing from the heat of the fire. The reserve flexural capacity in the girders and deck along with higher material strengths, as determined by testing, allowed the post-fire operating load ratings to be above HS20. The Bill Williams River Bridge, and particularly the prestressed, precast concrete girders, performed well during and after the fire.▪

Martha Davis, P.E. is a structural engineer with HDR. She is a registered professional engineer in Arizona and California.

Paul Tremel, P.E. is a vice president with HDR with over 20 years of bridge design experience. He is a registered professional engineer in Arizona and Nevada.

Alex Pedrego is a structural designer with HDR. He is a certified engineer-in-training in Arizona.

References

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2. CTLGroup, "Fire Damage Evaluation of Bill Williams Bridge", February 13, 2007.
3. HDR Engineering, Inc, "Final Project Assessment: Bill Williams River Bridge, Structure #1272, Fire Damage Assessment & Repair," August 23, 2007.