Implementing Ultra High Performance Concrete (UHPC) with Dowel Bars in Longitudinal Joints (Shear Key) in an Adjacent Box Beam Bridge

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Abstract
Adjacent precast prestressed box beam bridges are widely used in the United States for medium and short spans due to ease of construction, economics, and low profile. However, the longitudinal connections between adjacent box beams (shear keys) tend to crack. The cracking allows leakage of water carrying corrosive road salts between the joints, which can accelerate corrosion of the beams' reinforcement. Cracking of the joints may also reduce load transfer between beams. Different grouted materials and shear key configurations combined with transverse post-tensioning with or without a composite deck have been employed to reduce cracking. However, some adjacent box beam bridges still exhibit longitudinal cracks in the shear keys. In this research, a new partial depth shear key configuration grouted with ultra-high performance concrete (UHPC) and equally spaced dowel bars were used for the longitudinal connections. No transverse tie bars or intermediate diaphragms were utilized. This first adjacent box beam bridge in the USA containing UHPC shear keys and transverse dowel bars was constructed in the summer of 2014 in Fayette County, Ohio. The bridge was instrumented and monitored. This paper presents the static truck load testing results of the shear keys and the dowel bars. Finite element modeling was also created and calibrated with the field data. The UHPC connection with dowel bars performed satisfactory in load transfer with no cracking in longitudinal connection to date.

Keywords: Adjacent Box-Beams, Dowel Bars, UHPC, Finite Element Modeling, Shear Keys Connection

1. Introduction

Adjacent precast prestressed box beam bridges are popular for medium and short spans. However, longitudinal cracks in the shear keys and reflective cracks in the overlays have been a problem. The longitudinal cracks allow water and deicing chemicals to penetrate into the joint between box beams, causing the corrosion of steel reinforcement and deterioration.

Ultra-high performance concrete (UHPC) represents a new class of concrete, which has high strength, bond, and durability characteristics. The first use of UHPC in the shear key of an adjacent box beam bridge was noted by Perry et al. (2010). The UHPC was successfully placed and the required strength was obtained in the longitudinal and transverse joints after the curing period of the 3-span side-by-side Eagle River Bridge in Canada. However, there was no data available on the bridge performance. Recently, the UHPC with lap-spliced reinforcement was employed in the shear key connection of a pair of full-scale adjacent box beams (Yuan and Graybeal, 2014). The new shear key was larger than a typical shear key and used dowel bars, which were threaded into the beams and were staggered at a 4 in. (102 mm) spacing. Testing at Turner Fairbank Highway Research Center (TFHRC) involved connecting the two box beams together and applying a concentrated load. The results showed that the new design was sufficient to make the bridge behave as a unit and no cracks were recorded in the shear keys even after numerous cyclic loads. Finite Element Modeling of the laboratory testing, along with a parametric analysis, was also performed and it was showed that larger spacing of the dowel bars could be used (Ubbing, 2014). However, the UHPC with dowel bar shear key was utilized in a lab environment and only included two adjacent box beams.

Design was already far along and field behavior can sometimes greatly differ from laboratory conditions. Therefore, a conservative decision was made to utilize the same dowel spacing in the bridge for this study as utilized in the TFHRC testing. A full scale bridge in a field environment utilizing the UHPC and dowel bar shear key connection would further help understand the behavior of the new shear key design. The first adjacent box beam bridge in the USA containing UHPC shear keys and transverse dowel bars was constructed in the summer of 2014 in Fayette County, Ohio. In this bridge, the partial depth shear key configuration and equally spaced dowel bars was the same as that used in the TFHRC testing. The bridge was instrumented and monitored for both truck load and environmental load. Steinberg et al. (2015) explains the fabrication, instrumentation, and testing of the bridge. This paper presents the results of the shear keys and the dowel bars. Finite element modeling was also created and calibrated with the field data.

2. Structure and Instrumentation

The bridge was constructed on Sollars Road in Fayette County, Ohio near the town of Washington Court House. The bridge consisted of seven box beams adjacent to one another. The bridge length was 61 ft (18.59 m) and had a width of 28 ft (8.53 m). Each beam was 48 in. (1219mm) wide and 21 in. (533mm) deep as shown in Figure 1. The bridge did not have, transverse post-tensioning, a composite deck, or transverse tie rods, which is common practice in Ohio.

The bridge utilized the new shear key design as shown in Figure 2. Each dowel bar system consisted of two parts. The first part was embedded in the beam 18 in. (457mm) and contained a female threaded end. The part that was embedded in the shear key had a length of 4.75 in. (121) and had a male threaded end allowing it to be screwed into the part embedded in the beam. For reference, the beams were numbered 1 to 7, from left to right while facing the forward abutment.

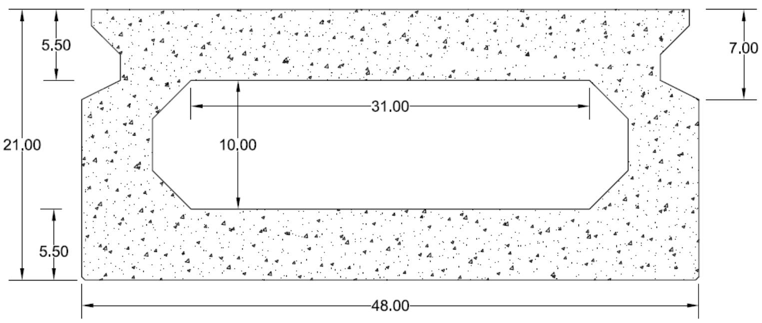

Figure 1. Cross Section's Details, (all dimensions in inches, 1 in. = 25.4 mm)

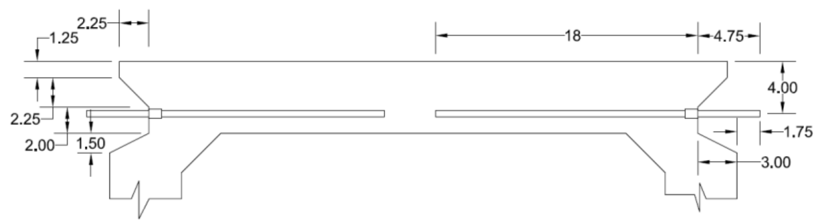

Figure 2. Longitudinal Connection's Details, (all dimensions in inches, 1 in. = 25.4 mm)

The first three box beams were instrumented with vibrating wire strain gages on the dowel bars as shown in Figure 3. The gages were installed at a distance of 2 in. (501 mm) from the threaded end of the dowel bar. Beams 1, 2, and 3 each contained two instrumented dowel bars on the right side of the cross section, one at the quarter span and one at mid span at distances of 204 in. (5182 mm) and 364 in. (9246 mm) from the rear abutment, respectively.

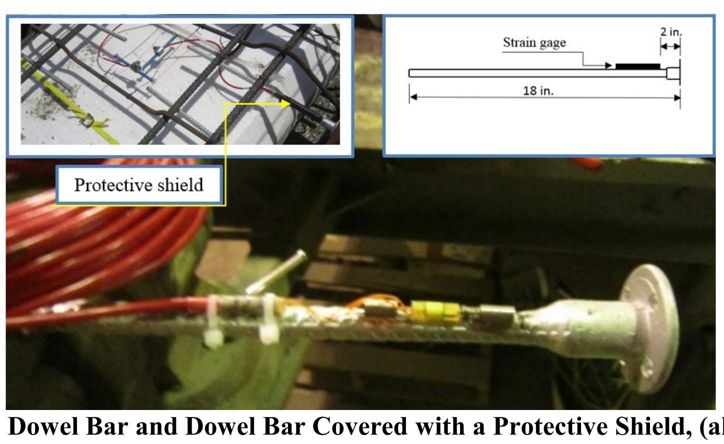

Figure 3. Instrumented Dowel Bar and Dowel Bar Covered with a Protective Shield, (all dimensions in inches, 1 in. = 25.4 mm)

In July 2014, the box beams were erected at the site. A vibrating wire strain gage was installed on six dowel bars at a distance of 1.5 in. (38 mm) from the threaded end as shown in

Figure 4. Two instrumented dowel bars were installed on the left side of Beams 2, 3, and 4, one at the quarter span and one at mid span.

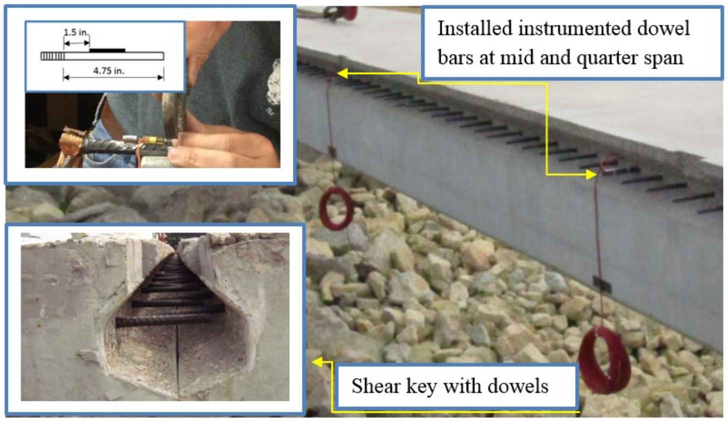

Figure 4. Instrumenting and Installed a Shear Key Dowel Bar and Shear Key with Dowels (all dimensions in inches, 1 in. = 25.4 mm)

On July 16, 2014, the three shear keys between Beams 1 - 4 were instrumented with vibrating wire strain gages. Six short length gages were set in a transverse direction as shown in Figure 5. Each shear key was instrumented with one transverse gage at the quarter span and one at mid span. Shear Keys 1 and 3 were also instrumented with one longitudinal gage at the quarter span and one gage at mid span as shown in Figure 5. After gage installation, excess expandable filler material used to seal the bottom of the keys between the beams was removed.

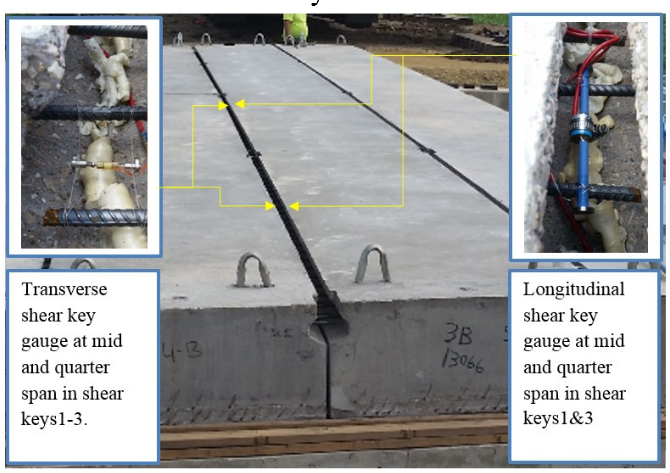

Figure 5. Transverse and Longitudinal Shear Key Gage

On July 17, the shear key joints were cast using (UHPC). Two mixers were used to properly mix the UHPC. The UHPC was moved to the joints in wheelbarrows and placed into chimneys made of plastic buckets located at the larger joint openings as shown in Figure 6. The UHPC flowed into the joints and the filling of the joints was assured by the hydraulic head of the UHPC in the chimneys. Five days later the plywood forms that covered the joints were removed. No cracks in the shear keys were observed from visual inspection.

Figure 6. Placement of UHPC into the Longitudinal Shear Keys

3. Bridge Testing

After installation of a waterproofing membrane and asphalt was paved on the bridge's surface, two loaded trucks were used to test the bridge. The weights of the trucks were 56.1 kips (249.54 kN) and 53.4 kips (237.53 kN) determined using truck scales.

Four static load configurations were used in the tests, and the trucks were positioned to obtain the maximum moment at mid-span. The load configurations were:

1. A single 56.1 kip (249.54 kN) truck load placed in the left lane
2. A single 53.4 kip (237.53 kN) truck load placed in the right lane
3. Two trucks placed side-by-side with a 109.6 kip (487.52 kN) total load
4. Two trucks placed back to back in the left lane for a 109.6 kip (487.52 kN) total load

4. Finite Element Model and Verification

A 3-D linear elastic finite element model of the entire bridge was developed using ABAQUS/CAE software to investigate the behavior of the bridge and to be used in a future parametric study. The Modulus of Elasticity and Poisson's ratio for the model were determined from cylinder tests of the beam and UHPC concretes. The model utilized a 5 in. (127 mm) mesh in the longitudinal direction. The concrete and UHPC were modeled as a host region and the reinforcement modeled as an embedded region. The magnitude and position of the loading applied to the model was the same as the trucks used in the field test. The tire area was measured in the field and a uniform load was applied to elements used to represent the tire area in the model. In order to calibrate the finite element model, three different boundary conditions were used to compare results with field measurements. A pin-roller with a longitudinal 15 kip/in (2.63 kN/mm) spring boundary condition gave results that compared best to field measurements. The maximum difference in the mid span bottom deflection and strain for all seven beams were found to be approximately 4% and 14%, respectively.

The interaction between shear keys and precast prestressed concrete beams is an important factor, which affects the calibration between the finite element and the field results. The interaction was modeled as a surface-to-surface interaction. The tangential behavior was defined using a friction formulation (penalty) by defining the coefficient of friction. The normal behavior was defined using linear contact by defining the contact stiffness. Two shear friction coefficients, 1 and 0.8, were used. The beam- shear key interaction also was modeled using tied constraints. In this type of constraint, no slip was allowed between two surfaces by assuming the full bond between them.

The maximum principal tensile strains and maximum principal tensile stresses in the shear keys and dowel bars for different type of constraints are shown in Table 1. The results show that the increase in the friction coefficient slightly decreased the maximum principal tensile strains and stresses in both the shear keys and dowel bars. The tie constraints led to an increase in the maximum principal tensile strain and stress in the shear keys about twice the value as using the surface-to-surface interaction. The increase in these values might be related to preventing the movement between the two surfaces, which caused the shear keys to exhibit higher stresses. However, there was a high decrease in the maximum principal tensile strain and stress in the dowel bars because no slip was allowed at the interface and the total load only transfers by the shear keys. Therefore, the interaction between the shear keys and the precast element was modeled as a surface-to-surface interaction with friction coefficients equal to 1 because it led to results that were more accurate. In addition, this value was recommended to use as a friction coefficient by Russell and Graybeal (2013).

Table 1. Maximum Principal Strains and Maximum Principal Stresses in the Shear Keys and Dowel Bars for Different Types of Constraints, (1 ksi= 6.895 MPa)

Type of Constrains	Shear Keys		Dowel Bars	
	Maximum Principal Tensile		Maximum Principal Tensile	
	Strains (με)	Stresses (ksi)	Strains (με)	Stresses (ksi)
S-to-S, friction coefficient =0.8	16	0.10	227	7.49
S-to-S, friction coefficient =1	15	0.10	225	7.42
Tie constraints	50	0.28	24	0.66

5. Results and Discussions

5.1. Strain in Shear Keys in the Longitudinal and Transverse Directions

The measured strains in the shear keys in longitudinal and transverse directions from the field test were compared with the finite element results for both mid and quarter span locations. The longitudinal strains in Shear Keys 1 and 3 are shown in Table 2. Finite element results agreed fairly well with field results. The maximum strain from the field results was observed in Shear Key 3 at mid span when the bridge was loaded with two trucks at mid span. However, the maximum strain from the finite element model was observed in Shear Key 1 at mid span when the bridge was loaded with two trucks back to back in the left lane. The shear key was subjected to bending in the longitudinal direction, therefore compressive strains were recorded as the beams and shear keys behaved together.

Table 2. Longitudinal Strain in the Shear Keys 1 and 3 at Mid and Quarter Span

Load Configuration	Results Types	Shear key 1		Shear key 3	
		Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)
One Truck on Left (1)	Field	-35	-18	-35	-29
	FEM	-41	-30	-35	-25
One Truck on Right (2)	Field	-23	-12	-30	-23
	FEM	-20	-16	-27	-19
Two Trucks on Mid Span (3)	Field	-58	-29	**-71**	-52
	FEM	-66	-46	-60	-44
Two Trucks on Left (4)	Field	-64	-35	-65	-46
	FEM	**-74**	-52	-57	-41

The maximum longitudinal strain values from the finite element model for different load configurations were higher at locations other than that of the gages. The results show that the maximum longitudinal compressive strain was 83 με for load configuration 1 on the top of Shear Key 2 at mid span, 77 με for load configuration 2 on the top of Shear Key 5 at mid span, 122 με for load configuration 3 on the top of Shear Key 2 at mid span, and 122 με for load configuration 4 on the top of Shear Key 2 at mid span.

The transverse strains at mid and quarter span from the field measurements and the finite element modeling for different load configurations are shown in Table 3. Consistent results were found between the field and finite element results. The maximum transverse strain from both field and finite element modeling was observed when the bridge was loaded with Configuration 4. Furthermore, the maximum transverse field strain was observed in Shear Key 3 with a value of 13 με at quarter span. However, the maximum transverse strain from finite element model was observed on Shear Key 1 with a value of 16 με at mid span.

Table 3. Transverse Strain in Shear Keys 1-3 at Mid and Quarter Span

Load Configuration	Results Types	Shear Key 1		Shear Key 2		Shear Key 3	
		Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)
One Truck on Left (1)	Field	6	6	5	3	8	8
	FEM	10	7	9	6	9	6
One Truck on Right (2)	Field	2	8	7	7	5	7
	FEM	4	3	6	4	7	5
Two Trucks on Mid Span (3)	Field	8	8	7	6	10	12
	FEM	14	9	13	8	13	7
Two Trucks on Left (4)	Field	10	7	6	4	11	**13**
	FEM	**16**	12	14	10	14	10

The transverse tensile strain and maximum principal tensile strain from FEM for load configuration 4 are shown in Figure 7. This load configuration was chosen because it exhibited maximum principle tensile strain. The results show that the maximum transverse tensile strain of 23 με for load configuration 4 was on the top of Shear Key 2 at mid span. The left rear tire of the truck, which was on Shear Key 2, might have caused the high tensile strain in the transverse direction at this position because the applied load created a negative moment in the transverse direction. The maximum principal tensile strain of 28 με was on the end of Shear Key 3 for this load configuration. The results show that the maximum principal tensile stress was 0.19 ksi (1.31 MPa) at the end of Shear Key 3.

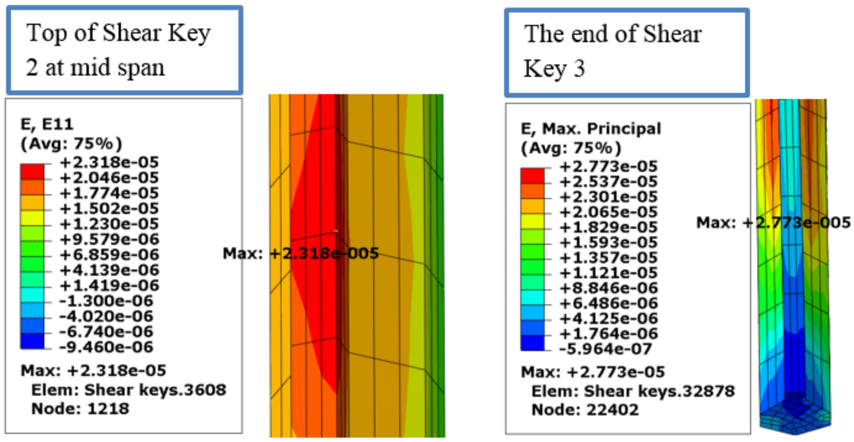

Figure 7. Transverse Strain and Maximum Principal Tensile Strain for Load Configuration 4

The allowable tensile strength (f_{ct}) in the shear key was calculated by using Equation (1) (Russell and Graybeal, 2013).

$$f_{ct}=6.7\sqrt{f_c'} \qquad (1)$$

where f_c' is the cylinder strength of UHPC

Since the maximum principal tensile stress of 0.19 ksi (1.31 MPa) from the modeling was lower than the estimated cracking stress of 0.99 ksi (6.83 MPa), the shear key was not expected to exhibit any cracking. The estimated allowable tensile strength in the shear key was about five times greater than the stress in the shear key determined from modeling due to the two trucks loaded back to back on the bridge.

5.2. Strain in the Dowel Bars

The strain in the dowel bars embedded in the shear, keys from the field test and the finite element model for all load configurations are shown in Tables 4. The field and finite element results compared well even though strains were low.

Table 4. Axial Strain in the Dowel Bars Embedded in Shear Keys 1-3 at Mid and Quarter Span

Load Configuration	Results Types	Dowel in Shear Key 1		Dowel in Shear Key 2		Dowel in Shear Key 3	
		Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)	Mid Span (με)	Quarter Span (με)
One Truck on Left (1)	Field	7	4	6	3	10	9
	FEM	10	6	8	6	9	6
One Truck on Right (2)	Field	3	3	4	6	6	6
	FEM	5	4	6	5	8	6
Two Trucks on Mid Span (3)	Field	7	6	8	4	13	10
	FEM	14	9	13	9	13	9
Two Trucks on Left (4)	Field	9	7	8	3	**15**	12
	FEM	**15**	10	14	10	13	10

Load configuration 4 produced the highest strains. The maximum recorded strain for the part of the dowel bar embedded in Shear Key 3 at mid span from the field test was 15 με. This strain corresponds to a stress of 0.44 ksi (3.03 MPa) when a Modulus of Elasticity of 29,000 ksi (200,000 MPa) for the dowel bar was utilized. The maximum strain for the part embedded in Shear Key 1 at mid span from the finite element model was also 15 με (0.44 ksi) (3.03 MPa). The dowel bars embedded in the beams exhibited slightly higher strains than the part embedded in the joints. The low recorded strains referred to the fact that no cracks occurred in the shear key to engage the dowel bars to carry stress.

Load configuration 4 produced the largest strains in the dowel bars. The maximum axial tensile strain was 299 με and the maximum principal tensile strain was 370 με for the dowel bar embedded in Shear Key 3, which located between loaded and unloaded beams. This corresponds to a maximum principal tensile stress of 12 ksi (82.74 MPa), which is lower than the yield strength of 60 ksi (414 MPa). The maximum principal tensile stress is shown in Figure 8. The maximum stress was located at the interface between the UHPC and precast element. The dowel bars had reserve capacity to resist the applied load with the specified dowel bar spacing.

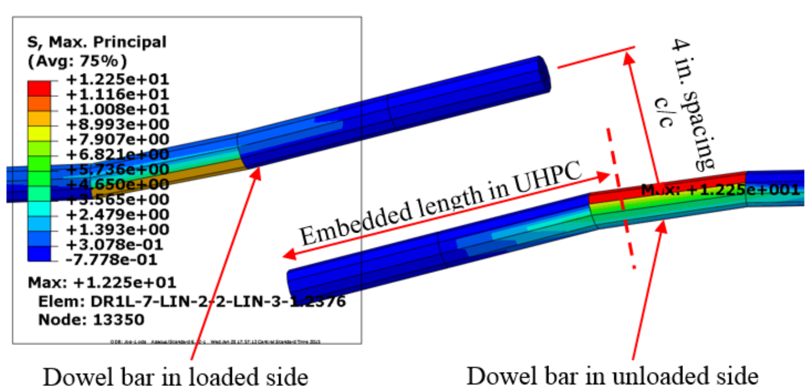

Figure 8. Maximum Principle Tensile Stresses in the Dowel Bars for Load Configuration 4, (1 ksi= 6.895 MPa & 1 in.=25.4 mm)

6. Conclusions

The behavior of the shear keys and the dowel bars due to static truck load were investigated. Based on the results, the following conclusions were drawn:

- UHPC was used successfully in the shear key in an adjacent prestressed concrete box beam bridge. The plywood forms were removed from the joints 5 days after placement and no cracks were observed from inspection of the shear keys.
- The field and FEM model results for the strain in the shear keys in the longitudinal direction compared well. The high bond between the beams and shear keys resulted in continuity. Therefore, the shear key in the longitudinal direction was subjected to bending causing the recorded compressive strains.
- The field and finite element results for the transverse strain in the shear key also showed consistent results. The maximum tensile transverse strain from both field and finite element model results were observed when the bridge was loaded with two trucks back to back on the left lane. However, these strains were relatively low.
- The maximum principal tensile stress in the UHPC from FEM was 0.19 ksi (1.31 MPa) at the end of Shear Key 3. This stress was less than the estimated cracking stress of 0.99 ksi (6.83 MPa). Therefore, the stress in the shear key was not high enough to cause any cracks from the loading. The allowable tensile strength in the shear key was about five times greater than the stress in the shear key due to the two trucks load back to back on the bridge.
- The field and finite element results for the strain in the dowel bars compared well. The magnitudes of the strains recorded in the dowel bars were low. The low recorded strains were due to two reasons. The shear key did not crack and therefore little stress was carried by the dowel bars. Secondly, the strain gages were not positioned at the critical location.
- The dowel bar exhibited a maximum principle tensile stress of 12 ksi (82.74 MPa) when the bridge was loaded with two trucks back to back according to the FEM modeling. This value was lower than the yield strength of 60 ksi (414 MPa). The dowel bars have enough capacity to resist the applied load with this dowel bar spacing.
- A finite element model was successfully developed to predict the behavior of the bridge and can be used further in parametric studies.

7. References

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