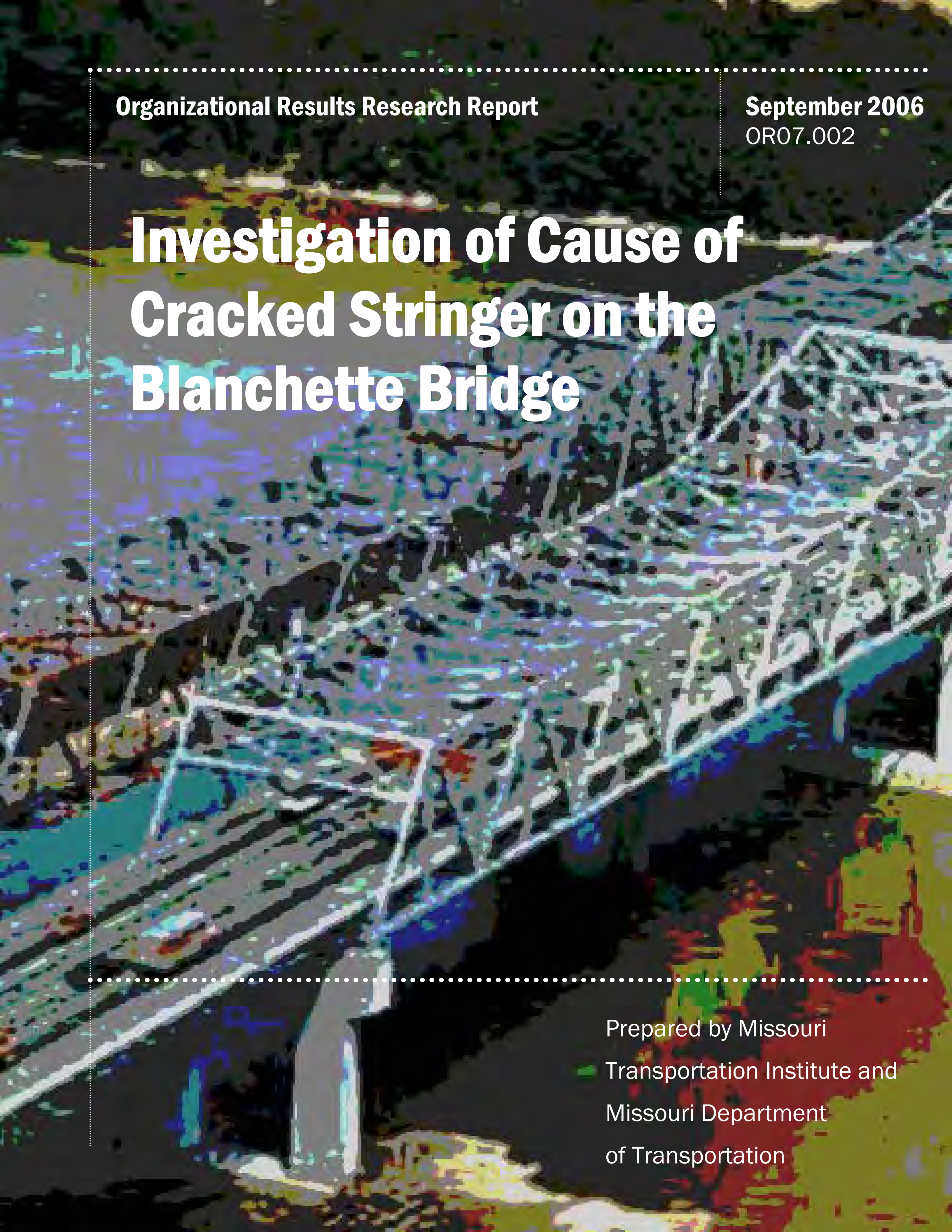

Organizational Results Research Report

September 2006
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Investigation of Cause of Cracked Stringer on the Blanchette Bridge

Prepared by Missouri Transportation Institute and Missouri Department of Transportation

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The Blanchette Bridge carries Interstate 70 across the Missouri River, connecting St. Louis and St. Charles counties in eastern Missouri. The westbound bridge was constructed in 1958. In 1979, the original reinforced concrete roadway deck on the bridge was replaced with a steel grid deck system, welded to supporting girders and stringers. In 1999 and 2005, cracks were discovered in stringers on the bridge approach spans. The stringers were repaired with bolted splices.

The investigation of the cracking included preliminary analyses, review of historical documents, detailed finite element analyses, and field testing of the bridge. The cracking was determined to result from the field weld detail used combined with the load distribution effects of the stiff grid deck. Recommended mitigation measures are included.</td></tr>
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FINAL REPORT

RI05-036

Investigation of Cause of Cracked Stringer on the Blanchette Bridge

Prepared for
Missouri Transportation Institute
and
Missouri Department of Transportation
Organizational Results

by

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September 2006

Executive Summary

The Blanchette Bridge carries Interstate 70 across the Missouri River, connecting St. Louis and St. Charles counties in eastern Missouri. The westbound bridge was constructed in 1958. In 1979, the original reinforced concrete roadway deck on the bridge was replaced with a steel grid deck system, welded to supporting girders and stringers.

In 1999 and 2005, cracks were discovered in stringers on the bridge approach spans. The stringers were repaired with bolted splices. The Missouri Department of Transportation authorized this research project to definitively determine the cause of the cracked stringers. In addition to determining the cause, other areas of the bridge that may be prone to such cracking were to be identified and possible preventative measures suggested.

This report reviews the history of the bridge, discusses the specific details of the cracked stringer discovered in 2005, and presents the investigation approach and findings.

The investigative effort included site visits, review of bridge documents, preliminary fatigue analyses, review of material property information, detailed analyses, and field testing. The conclusions include discussion of the cause of the cracking and proposals for mitigation of the problems.

The following conclusions were reached on the cause of the cracked stringers:

1. The stringer cracking occurred at details that had very high stress concentration factors due to open shim butt joints with fillet welds crossing the joint.
2. Fatigue of the fillet welds due to traffic loading led to cracking of the weld material.
3. High negative bending stresses in the stringer resulted in cracking through most of the section after the weld crack propagated into the stringer.
4. The high negative bending stresses result from construction or temperature forces, or some combination of those, in addition to the effects of continuity on the stringer force distribution.
5. The redundancy and strength of the grid deck and stringers prevented serious distress or failure in the bridge deck.

The detail that led to the cracking was due to decisions made during the design and construction phases of the redecking project.

The following actions are recommended:

1. The open shim butt joints should be retrofit or modified to eliminate the crack initiation area where the fillet welds cross the joint gaps. A suitable retrofit plan uses grinding to remove the welds for a distance of 2” in each direction from the gaps. Grinding should remove all weldment in this area. Ends of welds should be smoothly tapered to reduce stress concentrations. Following grinding, the affected

areas should be visually inspected and tested by dye penetrant or magnetic particle methods to confirm that no cracks exist in the stringer or remaining welds.

2. This case study should be presented to bridge design, construction inspection, and bridge maintenance staff to alert them to the causes of and relevant issues behind the stringer cracking. The objective of this action is to prevent similar details from being used in future projects without proper consideration of potential problems.
3. Drawings of other bridges with grid decks should be reviewed to check if similar details exist on other structures.

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1. INTRODUCTION

The Blanchette Bridge carries Interstate 70 across the Missouri River, connecting St. Louis and St. Charles counties in eastern Missouri. The westbound bridge was constructed in 1958. In 1979, the original reinforced concrete roadway deck on the bridge was replaced with a steel grid deck system, welded to supporting girders and stringers.

In 1999 and 2005, cracks were discovered in stringers on the bridge approach spans. The stringers were repaired with bolted splices. The Missouri Department of Transportation authorized this research project to definitively determine the cause of the cracked stringers. In addition to determining the cause, other areas of the bridge that may be prone to such cracking were to be identified and possible preventative measures suggested.

This report reviews the history of the bridge, discusses the specific details of the cracked stringer discovered in 2005, and presents the investigation approach and findings.

The investigative effort included site visits, review of bridge documents, preliminary fatigue analyses, review of material property information, detailed analyses, and field testing. The conclusions include discussion of the cause of the cracking and proposals for mitigation of the problems.

2. BRIDGE HISTORY

The Blanchette Bridge (see Figure 2-1) carries Interstate 70 across the Missouri River, connecting St. Louis and St. Charles counties in eastern Missouri. The eastbound and westbound lanes of the highway are carried on separate structures. Both bridges are owned and operated by the Missouri Department of Transportation (MoDOT).

The westbound bridge originally carried traffic in both directions. This structure has MoDOT Bridge Number L561. The parallel eastbound structure was completed in 1977, and has MoDOT Bridge Number A3292. The subject of this project is a cracked stringer found in Bridge L561.

I-70 Twin Bridges

Figure 2-1. Blanchette Bridge; westbound structure in background.
(Millstone 1979)

Bridge L561 (see Figure 2-2) was completed in 1958. It consists of a steel through-truss structure with plate girder approach spans. The truss spans cross the navigable channel of the Missouri River.

The bridge was originally designed according to the provisions of the Standard Specifications for Highway Bridges, 1953 edition, with some exceptions and modifications to the standards. The bridge was designed to carry H20-S16-44 live load.

The bridge is currently striped to carry five lanes of traffic. MoDOT records (Missouri Department of Transportation 2005) show that traffic peaked around the year 2002, with an average annual daily traffic (AADT) rate of around 90,000 on the westbound bridge. Site surveys show that trucks make up 10.8 percent of this traffic. Opening of reliever bridges has reduced the volume, and traffic is expected to decline in the future.

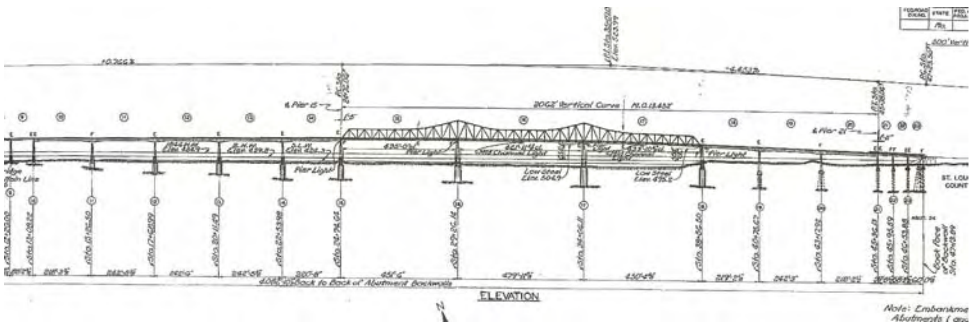


Figure 2-2. Bridge L561 elevation.
(Missouri State Highway Department 1955)

The cracked stringer is located in the east approach spans. The structural system (see Figure 2-3) in these spans consists of three longitudinal plate girders, which support transverse floorbeams. Longitudinal stringers are supported on the floorbeams between the plate girders. Cantilever brackets outside the exterior plate girders support an additional exterior stringer along each edge of the approach spans. Looking across the cross-section from left to right, the deck is supported on an exterior stringer, an exterior plate girder, two stringers, the interior plate girder, two stringers, an exterior plate girder, and an exterior stringer. The stringers are continuous over multiple floorbeams, with a floorbeam spacing of 24’ 3”. A similar set of spans carry traffic on the west approach to the bridge.

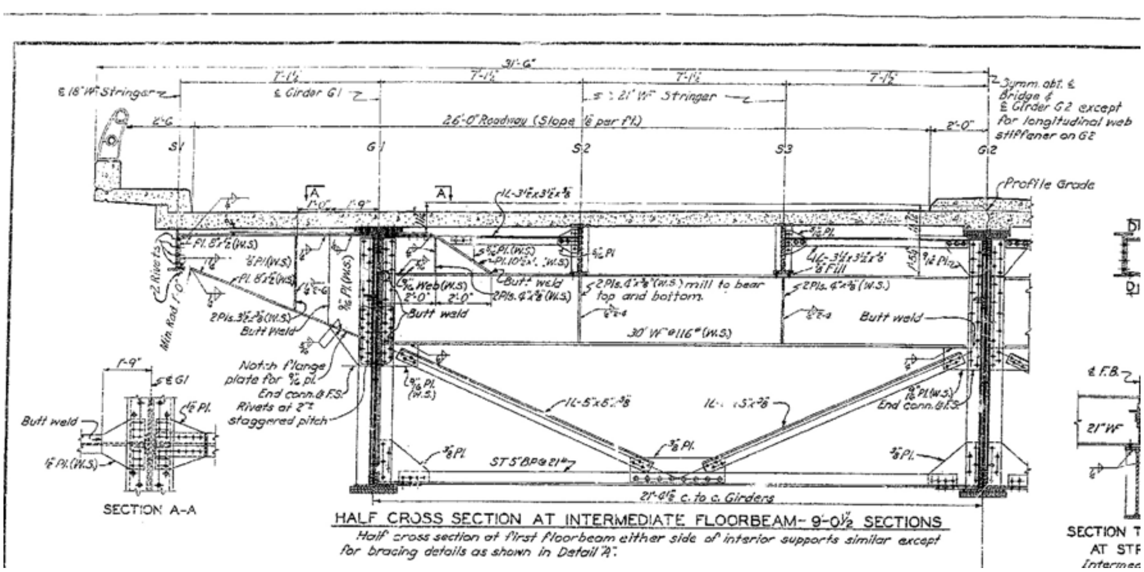


Figure 2-3. Typical superstructure section through approach spans showing original cast-in-place concrete deck.
(Missouri State Highway Department 1955)

The bridge was originally constructed with a reinforced concrete deck. This deck was replaced with a steel grid deck in 1979, following completion of the parallel bridge.

The redecking project (see Figure 2-4) was designed by the Missouri State Highway Department and performed by Millstone Construction, Inc. The bridge was closed to traffic during the deck replacement, and the newly opened parallel bridge carried two-way traffic as a bypass. (Note that Figure 2-1 actually shows the two structures during the redecking work.)

The general construction sequence of the redecking project was as follows:

1. Removal of the existing concrete deck.
2. Attachment of shim plates to top of plate girders and stringers. Shims in the negative moment areas of the plate girders were bolted to the top flange angles. Shims on stringers were field welded to the top flanges.
3. Placement of grid deck.
4. Field welding of grid deck to top of shims.
5. Placement of wearing surface on grid deck.

Steel shims were needed on top of the plate girders to allow the steel grid deck (see Figure 2-7) to be set at the correct elevation and bear fully on the girders and stringers. The grid deck transverse bars were welded to these shims. A plan change was made during the redecking project, adding steel plate shims to the tops of the stringers in the plate girder spans as well.

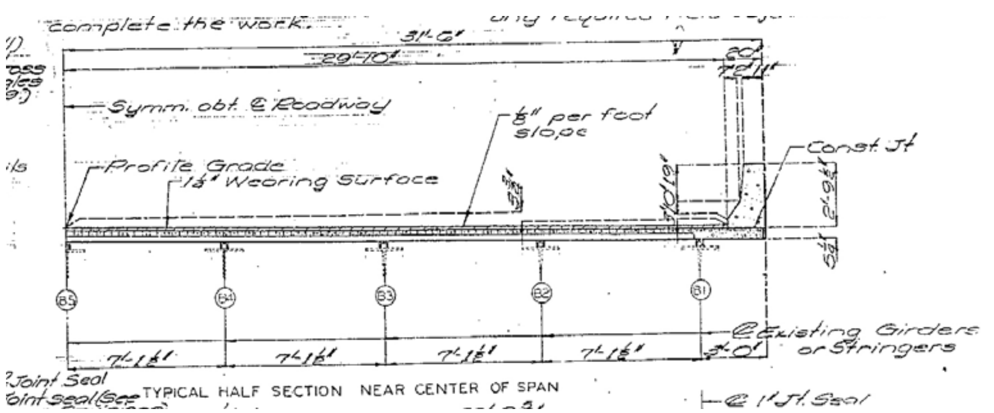


Figure 2-4. Typical section for redecking on beam span approaches.
(Missouri State Highway Department 1978)

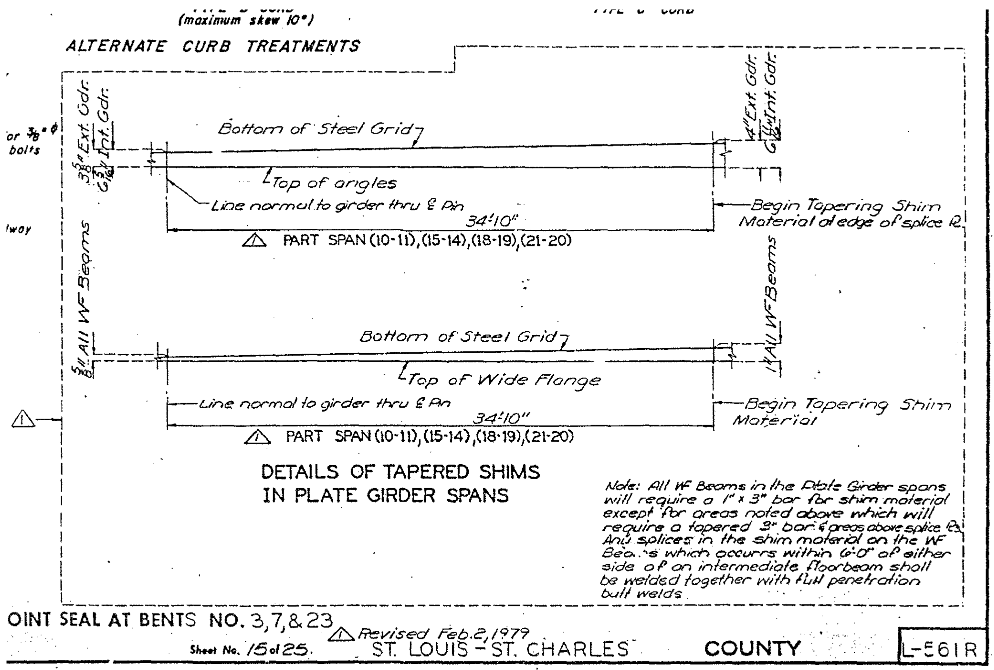


Figure 2-5. Details of tapered shims in plate girder spans.
(Missouri State Highway Department 1978)

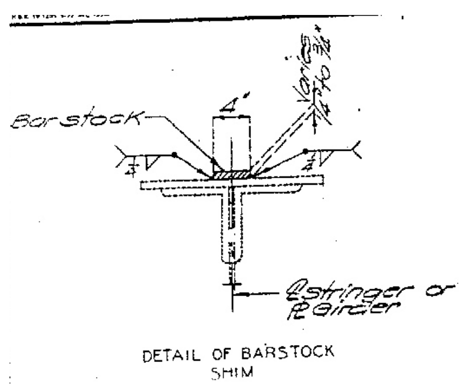


Figure 2-6. Detail of shim to top flange weld.
(Missouri State Highway Department 1978)

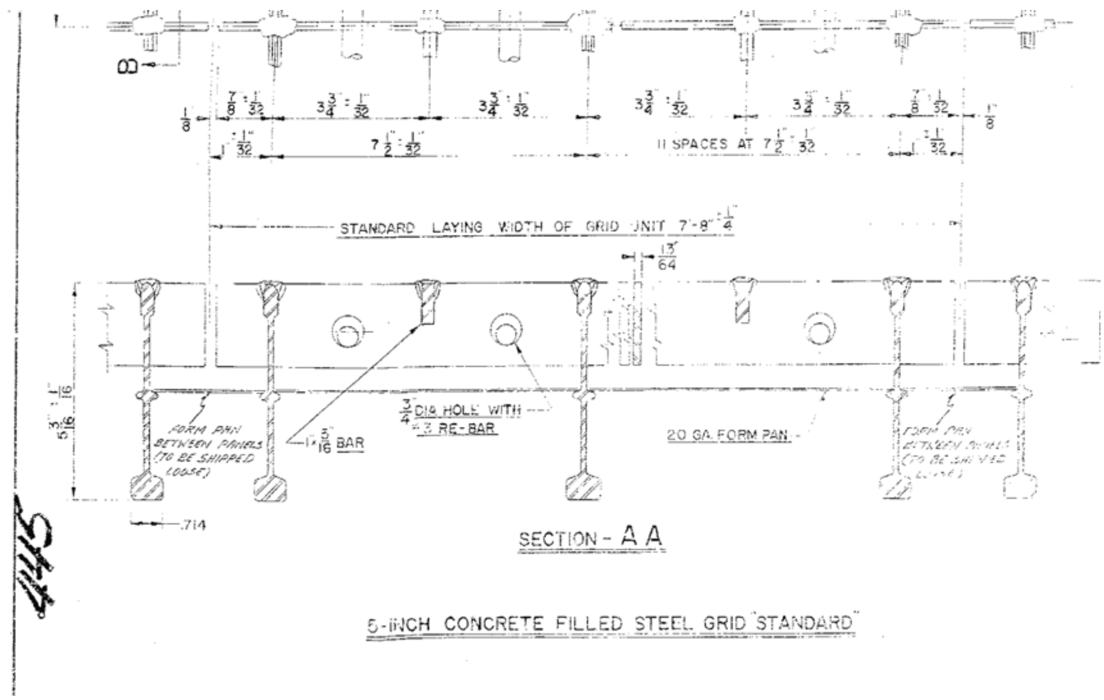


Figure 2-7. Grid deck typical section.
(Greulich 1978)

Several aspects of the redecking project are notable in relation to the later stringer cracking. The first is the note on sheet 15 of the redecking plans, added as part of the plan change. This note can be seen in Figure 2-5 and states the following:

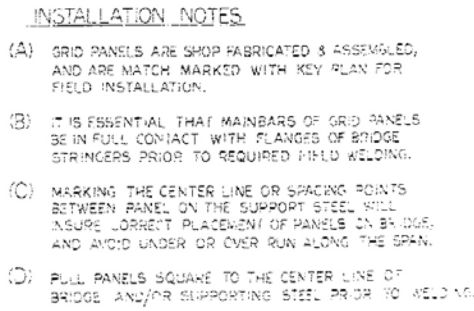
> All WF Beams in the Plate Girder spans will require a 1"x3" bar for shim material except for areas noted above which will require a tapered 3" bar and areas above splice plates. Any splices in the shim material on the WF Beams which occurs within 6'-0" of an intermediate floorbeam shall be welded together with full penetration butt welds.

This change, as shown in Figure 2-8, prevented open butt joints from occurring in the end quarters of the stringer spans.

Figure 2-8. Transverse view of grid deck welded to shim plate on top flange of stringer in plate girder spans.

A second significant detail is the treatment of the fillet welds connecting the shim plate to the stringer top flange. Field inspection of the open butt joints in the shim plates shows that the fillet welds frequently were run across the gap between the shims. Field observations also show that the fillet welds are typically 3/8” to 1/2” in size, compared to the 1/4” size shown in the project drawings.

A third factor relates to the fit of the grid deck to the structure. The installation notes, as shown in Figure 2-9, for the grid deck state that it is essential for the bearing bars of the grid deck to be in full contact with the stringer flanges prior to welding. The grid deck is a relatively stiff element. Two deck panels are used to cross the bridge deck width. Each deck panel would be supported by an exterior stringer, the exterior plate girder, two interior stringers, and the center plate girder. Any variations in the relative top elevations of these elements will cause a “misfit” or gap between the bottom of the grid deck and the top of the shim material at one or more of the support locations. Misfits could also be caused by variations in the depth of the grid deck (tolerances of +/- 1/16” are shown for the bearing bar depth on the shop drawing). Grid decks are welded, and distortion from these misfits can occur. This distortion would result in the deck warping, so the affected panel would no longer be planar, thereby providing an additional source for misfits.


INSTALLATION NOTES

(A) GRID PANELS ARE SHOP FABRICATED & ASSEMBLED, AND ARE MATCH MARKED WITH KEY PLAN FOR FIELD INSTALLATION.

(B) IT IS ESSENTIAL THAT MAINBARS OF GRID PANELS BE IN FULL CONTACT WITH FLANGES OF BRIDGE STRINGERS PRIOR TO REQUIRED FIELD WELDING.

(C) MARKING THE CENTER LINE OR SPACING POINTS BETWEEN PANEL ON THE SUPPORT STEEL WILL INSURE CORRECT PLACEMENT OF PANELS ON BRIDGE, AND AVOID UNDER OR OVER RUN ALONG THE SPAN.

(D) PULL PANELS SQUARE TO THE CENTER LINE OF BRIDGE AND/OR SUPPORTING STEEL PRIOR TO WELDING.

Figure 2-9. Grid deck installation notes.

Discussions with one of the contractor’s project engineers revealed that a number of these misfits occurred during the construction and that the typical solution to the problem was parking a piece of construction equipment on the panel, forcing it to seat itself on all of the shim plates for welding (Tallman 2006). Apparently this solution was not a common occurrence, but was not rare, either.

3. CRACKED EAST APPROACH STRINGER

MoDOT bridge inspectors found a severely cracked stringer (see Figure 3-1) on July 19, 2005. The two inspections prior to this one were on January 29, 2003, and March 15, 2004, with no indications of any cracking at this location identified. The crack had occurred in stringer S3, between the third and fourth floorbeam west of Bent 20 in the east approach plate girder approach spans.

Figure 3-1. Cracked stringer found during 2005 inspection.

As shown in Figure 3-2, the crack initiated at the shim plate butt joint and propagated downward through the stringer top flange and web. It arrested in the bottom of the stringer web, near the top of the stringer bottom flange.

Figure 3-2. Close-up view of shim plate butt joint and crack.

The photographs show that the stringer crack remained open, with the stringer cambered upward.

MoDOT bridge personnel repaired the crack with splice plates (see Figure 3-3). Both the top and bottom flanges and the web of the stringer were spliced.

No samples were taken from the crack area at the time of repair. The crack surface was not visible following the repair, as it was covered with splice plates and paint, as shown in Figure 3-4.

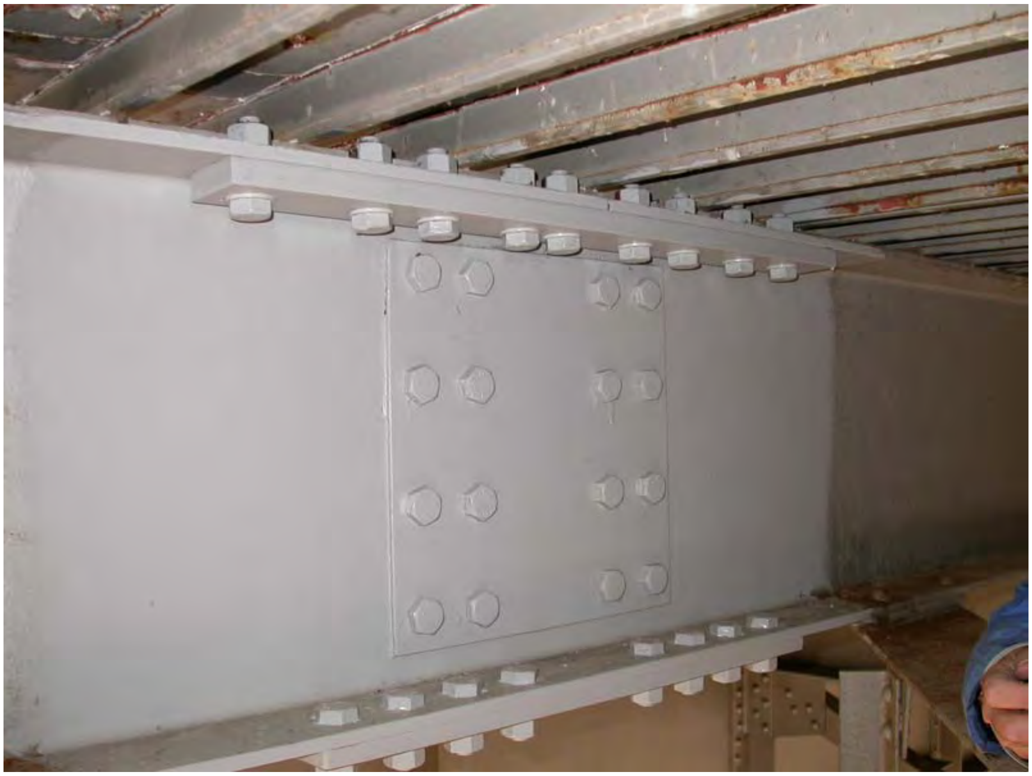


Figure 3-3. Splice repair of cracked stringer.

Figure 3-4. Close-up view of crack area after repair.
*Note that fillet weld runs across shim plate butt joint.

MoDOT bridge maintenance staff reported that a similar crack was discovered in August 1999 in the west approach spans of the bridge. The crack was located just west of bent 12 in the northern stringer off of the north catwalk. The inspection report describes it as a fairly straight vertical crack right through the stringer. (Martens 2006)

Other butt joints on the bridge have similar fillet weld treatments (see Figure 3-5). The shop drawings previously discussed show one butt joint per stringer bay between floorbeams, with an additional joint at each expansion joint. This would result in 180 shim open butt joints in the west approach spans and 300 in the east approach spans.

Figure 3-5. Similar butt joint weld detail located near cracked stringer on east approach.

4. COMPONENT AND MATERIAL PROPERTIES

The original bridge drawings show that the interior stringers are 21WF62 sections (see **Table 4-1. Section properties of 21WF62 Beams.**Table 4-1), composed of structural carbon steel conforming to ASTM Specification A7 (see Table 4-3).

Table 4-1. Section properties of 21WF62 Beams.

Area	Depth	Flange Width	Flange Thickness	Web Thickness	Moment of Inertia (Axis x-x)	Section Modulus (Axis x-x)
in^2	in	in	in	in	in^4	in^3
18.23	20.99	8.24	0.615	0.4	1326.8	126.4

(AISC 1957)

The shim plates and grid deck (see Table 4-2) are composed of ASTM A36 steel (see Table 4-4), with the exception that the main bearing bars of the grid deck are composed of ASTM A588 steel.

Table 4-2. Section properties for grid deck.

Type of Steel	Weight Per sq. ft.	Section Properties				Maximum Span for Allowable HS-20 Load (Continuous)
		Positive		Negative		
		$S_{composite}$	S_{steel}	$S_{composite}$	S_{steel}	
	psf	in^3	in^3	in^3	in^3	
A36	44.56	5.78	3.412	3.344	3.333	5'-11 ½"
A588	44.56	5.78	3.412	3.344	3.333	7'-9"

(Greulich 1978)

Notes on the shop drawings show that shop welds were to be made using E70 low hydrogen electrodes. No information on welding was found in the redecking project plans.

Table 4-3. Material properties of historic bridge steels.

Bridge Type	Type of Member	Decade of Manufacture	Material Specification	Yield Strength (ksi)	Ultimate Strength (ksi)	RA (%)
Simple-Span Truss	Rolled Beam	1950	A 7	36.3	69.3	42
Stringer Bridge	Welded Girder	1950	A 373	37.5	66.8	-
Continuous-Stringer Bridge	Welded Girder	1940	A 374	36.8	60.9	43
Stringer Bridge	Welded Girder	1950	A 375	36.5	58.1	-

(Chen et al. 2005)

Table 4-4. Material properties of modern bridge steels.

Material Specification	Min. Yield Strength (ksi)	Ultimate Strength (ksi)
A36	36	58-80
A588	50	70 (min.)

(Barsom 1994)

No testing was performed on material from the stringers. MoDOT staff did not have any components from the bridge available for testing, and no areas were suitable for removal of samples from the stringers.

A7 steel has typically shown lower fracture resistance than modern bridge steels. It is generally considered weldable (Ricker, 1988). Inspection of the fillet welds show no indication that problems were experienced in welding of shim plates to the bridge stringers.

5. APPROXIMATE STRUCTURAL ANALYSES

Several approximate analyses were performed to better understand the behavior of the bridge and its components. The results of these analyses also helped direct later efforts in the investigation.

The open butt joints in the shim plates can occur throughout the middle portion of the span between floorbeams. The calculations described in this section typically check forces at mid-span between floorbeams.

<u>Stringer continuity</u>

The bridge stringers are continuous over multiple floorbeam supports. The span length between floorbeams (24’ 3”) is shorter than the distance between the rear axles of the fatigue truck (30’). These two factors cause negative moment at midspan of the stringer spans during truck passage.

A line girder model of a 4-span stringer unit was developed and run using the RISA 3-D structural analysis program. A fatigue truck was run across the unit. Maximum negative moment values of 25.6 ft-kip per lane and maximum positive moment values of 87.2 ft-kip per lane were calculated.

<u>Grid deck interaction with stringers</u>

A typical design (for a concrete deck, for example) would assume that the weight carried by each supporting member varies according to the width of the deck it carries. In other words, the support load is proportional to the “tributary area” of the deck. This approach is valid in cases where the deck has little or no stiffness when placed, as is typical with cast-in-place concrete decks.

The grid deck has significant stiffness in both the longitudinal and transverse directions. The most significant impact to this study results from the transverse stiffness. Each grid deck panel is supported by the exterior stringer, exterior plate girder, two interior stringers, and the interior plate girder. The plate girders are much stiffer than the stringers. Therefore, when uniform loads are applied to the grid deck, the plate girders carry more of the load than the stringers. Practically, this significantly reduces the dead load carried by the stringers.

A RISA 3-D model was developed to study the resulting load distribution. Transversely, the model considered half the width of the bridge and excluded the exterior stringer. In the longitudinal direction, the model considered only the section of the bridge from floorbeam to floorbeam. The model and deflected shape are shown below in Figure 5-1 and Figure 5-2, respectively.

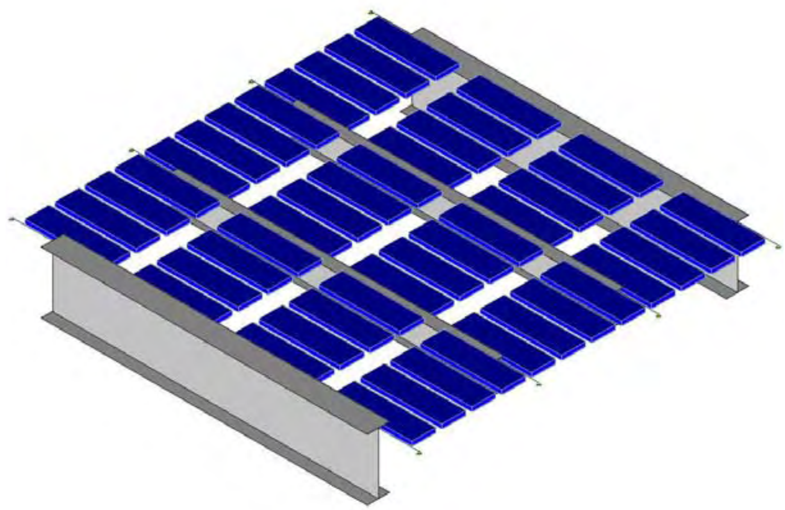


Figure 5-1. RISA model of section of bridge deck.

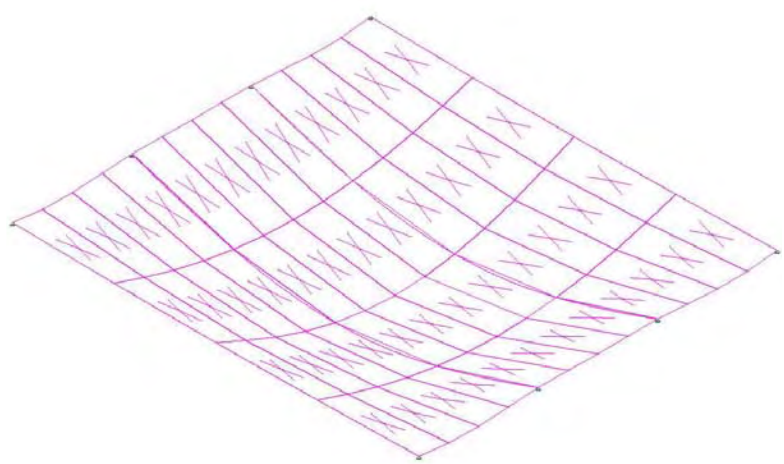


Figure 5-2. Deflected shape of model under uniform load.

Results of the deck-stringer interaction model show that the distribution of dead load to stringers at the midspan (between floorbeams) can be significantly different than the assumptions of the tributary area approach. Approximate calculations showed the stringers with grid deck carrying about 25 percent of the load calculated by the tributary area method.

Construction misfits

As discussed earlier in this report, misfits occurred during installation of the grid deck that prevented the deck from seating on all supports for welding. This condition was addressed by parking construction equipment on the deck, forcing it to seat, and keeping the equipment in place until the welds of the deck panel to the stringer were completed. When the equipment was removed, it can be anticipated that the grid deck will rebound upward, carrying the stringer with it and inducing negative bending at mid-span.

Approximate analyses of this situation were made assuming the stringers had various end conditions. With fixed ends, the mid-span moment resulting from a 1/8" upward deflection of the stringer was -147 ft-kip. With simple supports, the mid-span moment resulting from a 1/8" upward deflection of the stringer was -46 ft-kip. These values are quite significant in relation to the calculated dead load mid-span moment of 25 ft-kip based on the tributary area method.

Fatigue

An approximate fatigue analysis was performed using the procedures described in National Cooperative Highway Research Program Report 299, *Fatigue Evaluation Procedures for Steel Bridges* (Moses et al. 1987). This general approach is similar to that included in the AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating (LRFR) of Highway Bridges* (AASHTO 2003).

Bridge details subjected to repetitive loading are classified into categories based on fatigue resistance, as shown in Figure 5-3.

The base metal at the shim butt joint was considered to fall into Category E for the initial analysis, since it involves the termination of a welded cover plate. The detail as a whole, including the weld metal, is actually significantly worse than a Category E. This factor will be discussed more fully later in this report.

The limiting stress range given in this procedure (corresponding to a infinite life at a constant stress range amplitude) is 1.6 ksi for a Category E detail.

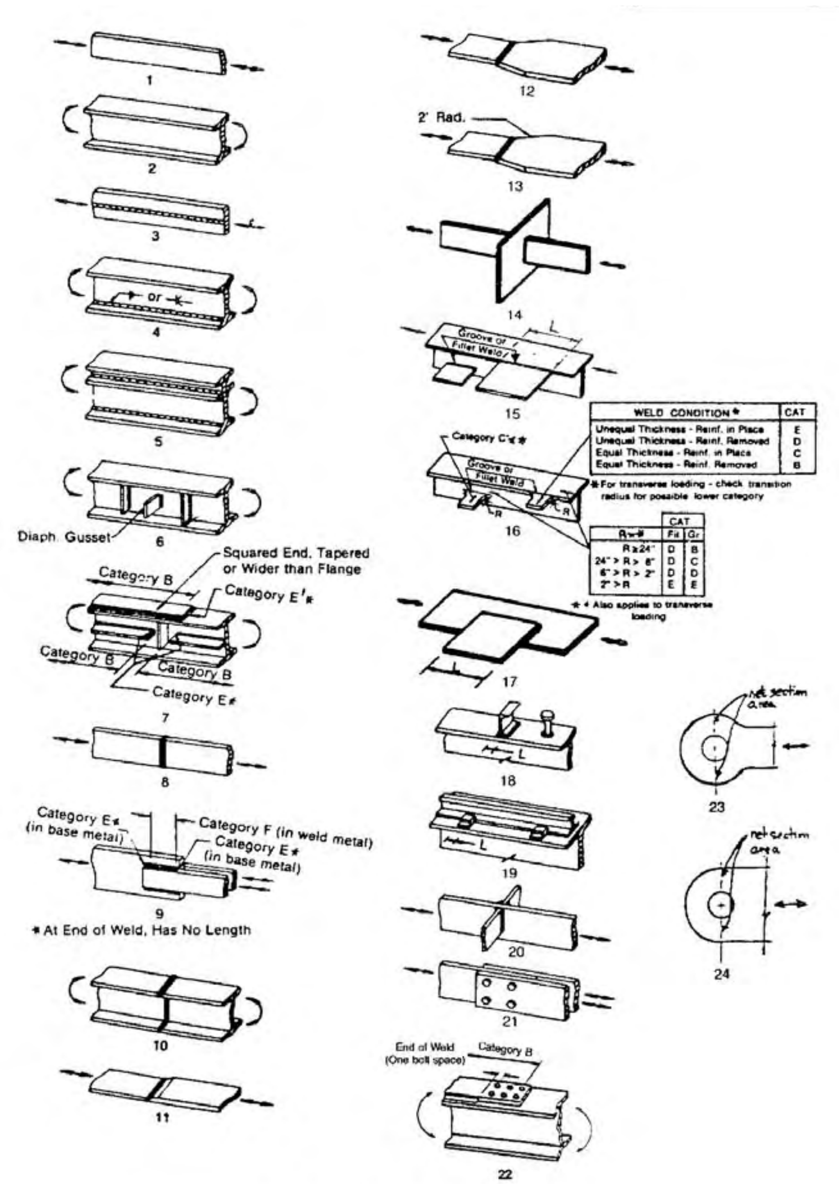


FIGURE 10.3.1C Illustrative Examples

Figure 5-3. Examples of fatigue categories for various details; Blanchette detail is most similar to Category E in Example 7.
(AASHTO 2002)

The evaluation procedure loads the member under consideration with a fatigue truck (see Figure 5-4). The stress range at a specific detail due to the passage of the fatigue truck is calculated and compared to allowable values for the appropriate fatigue category.

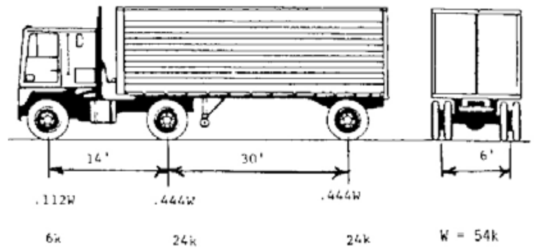


Figure 6.2.2A. Fatigue truck. (Note: A variable spacing of 14 to 30 ft can be used instead of the 30-ft main axle spacing, but this will significantly reduce the calculated remaining life.)

Figure 5-4. Fatigue truck.
(Moses et al. 1987)

Using the live load results from the RISA run, the calculated load range for the fatigue track passage is 25.6 + 87.2 = 112.8 ft-kip per lane. An impact value of 10 percent is recommended for typical cases. The calculated distribution factor is 0.42 lanes per girder, which results in a calculated load range (live load plus impact) per stringer of 52.1 ft-kip. This load range then results in a stress range of 4.9 ksi at the butt joint detail.

The portion of the stress range producing tension in the top flange of the stringer is 1.1 ksi. The calculated dead load stress in the top flange is 2.3 ksi of compression (using the tributary area calculation). According to the recommended procedure, no further analysis should be needed, since the calculated compression is more than two times the calculated tension portion of the stress range.

However, the earlier calculations described in this section demonstrate the uncertainty of the actual dead load compressive stress at the butt joint detail. It is probable that in some cases, the actual dead load stress is significantly less than the calculated value.

For investigation purposes, the fatigue life calculation was continued. An AADT of 90,000 with a 10.8 percent truck volume was used. It was assumed that 10 percent of the trucks crossing the structure impact the stringer under consideration. One and a half (1.5) load cycles per truck passage are used in the calculation, in accordance with the Report 299 recommendations. For the Category E detail, a mean life of 9.3 years resulted from these calculations.

This approximate analysis showed that the detail was vulnerable to fatigue cracking at an early age if the stress range due to applied loading (*i.e.*, other than dead load) ever causes tension in the area of the butt joint detail. Calculations described earlier in this section show that several possible reasons exist for the dead load stress to be low enough that this condition occurs in the stringers.

6. DETAILED STRUCTURAL ANALYSIS

The configuration of the shim plate open butt joints (see Figure 6-1(a)), where the fillet welds run continuously across the joint, produces evident geometrical discontinuities. When such sections are subjected to negative bending moments and tensile stresses in the direction parallel to the fillet welds, maximum local tensile stresses are induced at the notch (butt joint) of the weld tips that are greater than the applied nominal stress (*i.e.*, the remote tensile stress away from the stress singularity zone).

The magnitude of the stress concentration factor directly affects the fatigue life of a construction detail, either by correcting the corresponding *S-N* curves to be used in stress-life analysis, which include a reduced endurance limit defined as the stress level below which the material has infinite life, or when adopting a strain-life approach to account for notch root plasticity if deemed relevant (Bannantine et al. 1990), e.g. in Chen et al. (2005).

The fatigue life of a construction detail is directly affected by the magnitude of the stress concentration factor. When conducting a fatigue life estimate, this effect is accounted for either by correcting the corresponding S-N curves to be used in stress-life analysis, which include a reduced endurance limit (defined as the stress level below which the material has infinite life), or by adopting a strain-life approach to account for notch root plasticity if deemed relevant (Bannantine et al. 1990, Chen et al. 2005).

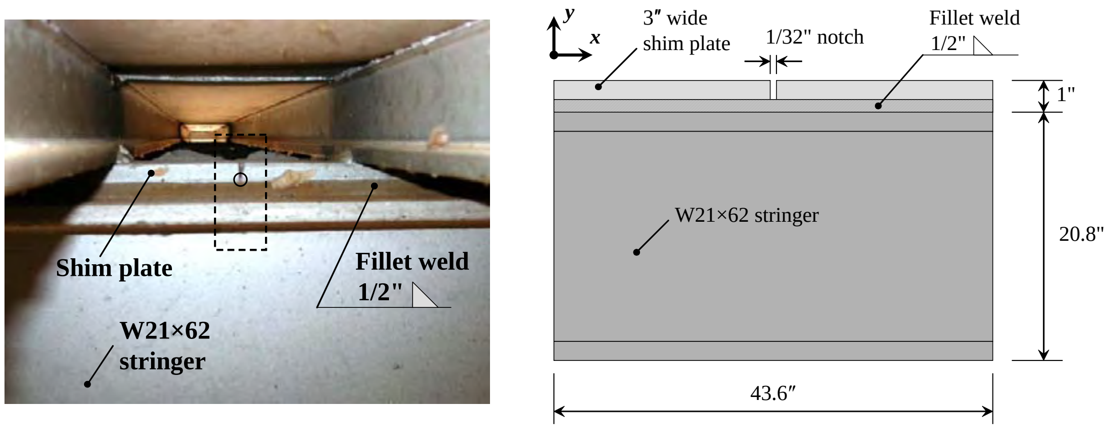
Due to the rather unusual geometry of the detail under investigation, the evaluation of the stress concentration factor was conducted by means of finite element analysis (FEM).

<u>Finite Element Analysis</u>

The commercial finite element (FE) analysis code, ABAQUS/Standard v. 6.5-1, was used to conduct the nonlinear stress analysis of the model depicted in Figure 6-1(b), as per measurements taken in the field during a preliminary inspection. The total length of the stringer portion considered was taken as 43.6 in > 2*d* = 41.5 in, where *d* = height of the W21 rolled stringer section, thereby enabling the effective evaluation of both the tensile (along the *x*-axis) stress at the tip and the remote stress at increasing negative bending moment. Symmetry with respect to the *x-y* plane was used to model half of the stringer and shim plate subassembly.

Three-dimensional solid elements (4-node linear tetrahedron C3D4) were used, with a discretization that was progressively refined in the vicinity of the shim plate butt joint section, as illustrated in Figure 6-2. To accurately characterize the stress concentration, the mesh implemented was selected upon verification of standard cases of discontinuous three-dimensional geometry with known stress concentration factors. A typical ASTM A36 stress-strain response was used as input in the model. Yield strength F_y = 36 ksi, elastic modulus E = 28.5 msi, and Poisson's ratio ν = 0.3 were assumed for both the parent material and the fillet welds (neglecting the slight difference with the minimum value F_y = 33 ksi prescribed for A7 steels, starting from 1933; A7 and A9 steels were consolidated in the A36 specification in 1965).

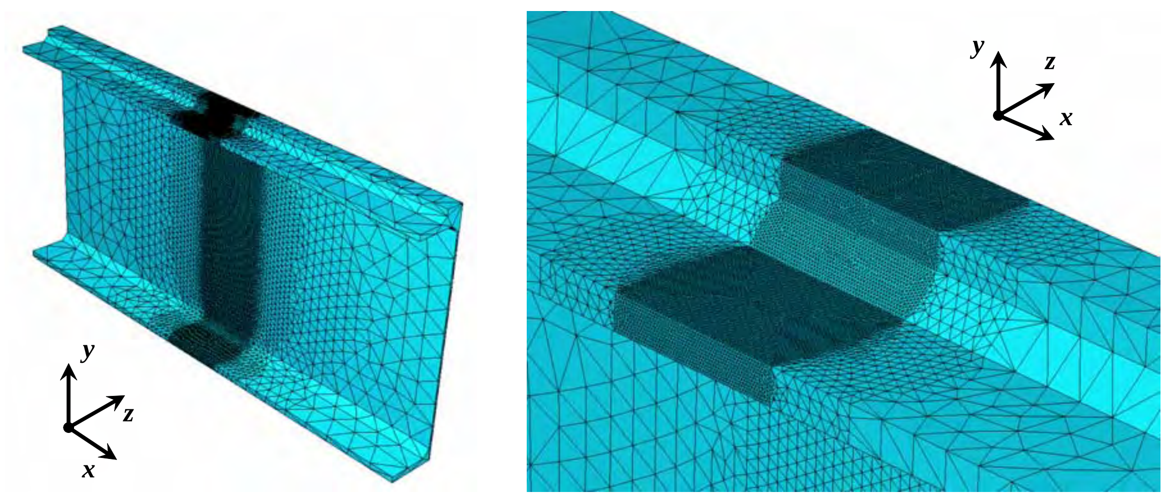
Since the stress singularity becomes of concern primarily under the effects of negative bending moments that tend to open the butt joint, the model was subjected to increasing bending moments that were simulated by means of equivalent tensile and compressive pressures. These pressures were applied at the free faces at one end, perpendicular to the *y-z* plane. The opposite end was constrained at discrete nodes such that fixed conditions were rendered, thereby accounting for the continuity of the stringer.



(a) (b)

Figure 6-1. (a) Photo of typical open shim plate butt joint*; (b) geometric characteristics of three-dimensional FE model.

*Dashed rectangle and solid circle indicate butt joint area and stress singularity zone, respectively.



(a) (b)

Figure 6-2. (a) FE model for stress concentration characterization: discretization; (b) close-up view of refined mesh at stress singularity zone.

A second FE model was also developed by simply eliminating the fillet welds in the first model. This model aims at evaluating the reduced stress concentration in a case where the welds had not been made continuous across the butt joint, while adopting a worst-case scenario approach (*i.e.*, complete lack of weld material in proximity to the notch section).

Results and Discussion

The stress concentration factors from both the FE analyses, defined as σ_{tip} / σ_{remote}, where σ denotes the combined Von Mises stress (versus the nominal stress in the outer face of the shim plate, away from the notch area, are shown in Figure 6-3(a). The σ_{tip} levels with respect to the associated σ_{remote} are provided in Figure 6-3(b).

The theoretical stress concentration factor in the elastic range, $K_t = \sigma_{tip} / \sigma_{remote}$ (σ_{tip} and $\sigma_{remote} \leq F_y$), which depends on the geometry and mode of loading only, reaches the extremely high value of 39.1 in the actual detail where the crack occurred. This value is explained by using an uncommon configuration for the shim plate butt joint, which produces a rather severe three-dimensional geometric discontinuity evidently prone to crack initiation, as graphically illustrated by the stress contours in Figure 6-4.

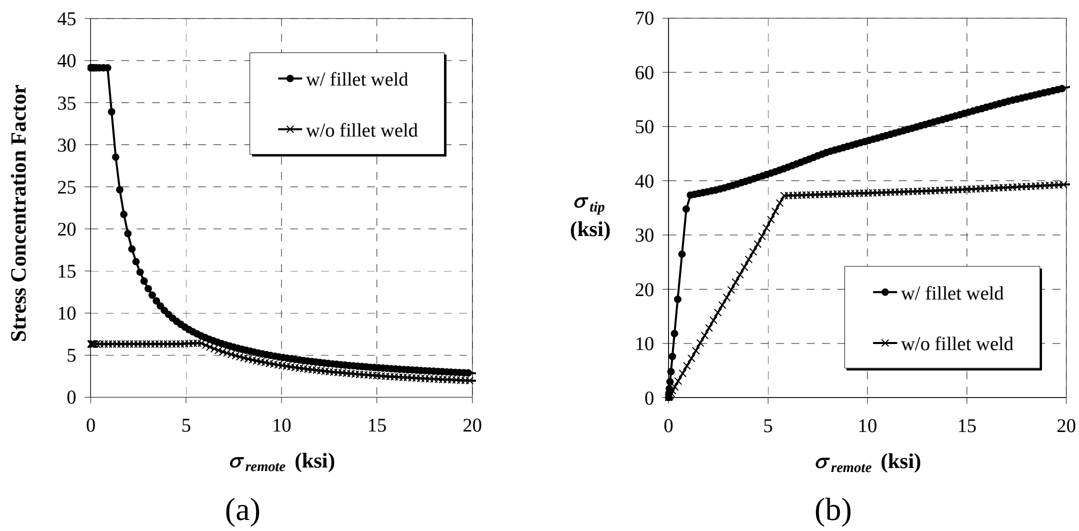
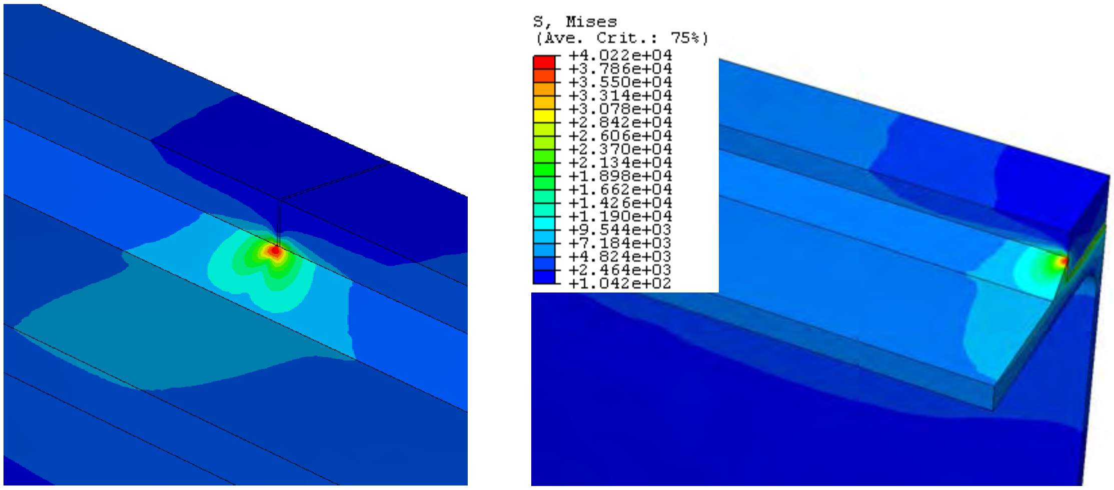


Figure 6-3. (a) Stress concentration factor in fillet weld at butt joint notch section versus remote stress; (b) combined Von Mises stress at notch tip versus remote stress.



(a) (b)

Figure 6-4. Graphical rendering of stress analysis results at given negative bending moment.

*Sample contours of (a) Von Mises stress (in psi) at shim plate butt-joint and (b) thru-notch section view. Note stress concentration at weld tip at notch location in red.

Figure 6-3(b) shows that even under such a relatively ideal condition, remote stresses in the range 0.5–1 ksi (strain ~18–35 με), fairly commonly encountered in similar bridge members under service loads, may result in tip stresses approaching or exceeding the yield strength. The yield strength typically lies slightly above the endurance limit approximated as $0.5F_u$, where F_u = ultimate strength (Bannantine et al. 1990). Hence, the fatigue life of the detail inevitably falls far below all of the seven primary *S-N* fatigue curves (Cat. A through E') from the AASHTO, AREMA, AWS, and AISC specifications (see Figure 6-5), which are based on the lower bounds of full-scale fatigue test data with a 97.5 percent survival limit.

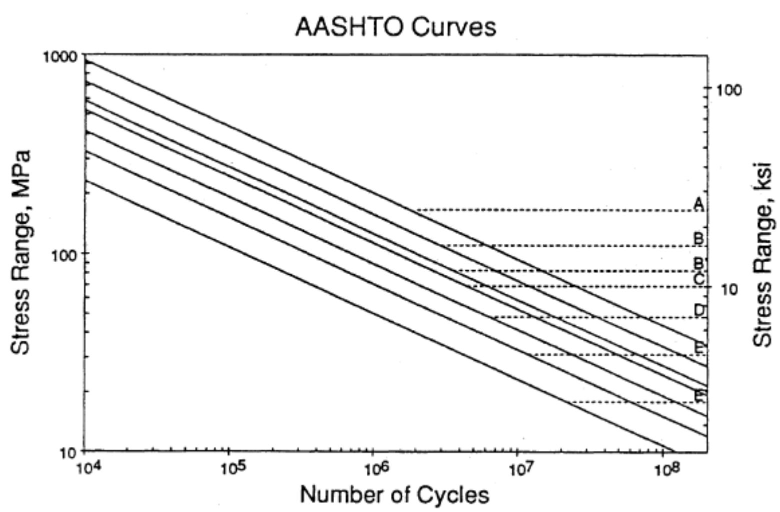


Figure 6-5. Fatigue life (*S-N*) curves from the AASHTO (2004) design specifications.

*Horizontal lines represent constant amplitude limits that indicate the detail category

In reality, when using either a stress-life (*e.g.*, Juvinall approach) or a strain-life (*e.g.*, Neuber's rule), K_t may produce conservative fatigue life estimates and may be replaced by the generally smaller fatigue stress concentration factor, K_f, which is dependent on geometry, mode of loading, and material type. In most cases, a 5 to 6 limiting value on K_f has been observed. This limiting value is generally attributed to the blunting effect of local yielding and/or initiation/propagation effects in very sharp notches, where the total life is more dependent on crack propagation (Bannantine et al. 1990).

In the present case, and considering the exceptionally high value of K_t, the assumption of a less conservative factor, as well as accounting for the positive effects of material plasticity, should not be recommended. Early steels, including the A7 steel used for the bridge stringers, had either no specified level or a high level of carbon content, typically resulting in poor to fair weldability (Stout and Doty 1953). The presence of a relatively high carbon content poses the issue of brittleness because of the formation of martensitic interphases with diffused reduced material toughness.

Indeed, the crack under investigation has propagated in a relatively short period of time between two annual inspections, thereby supporting the hypothesis of brittle fracture, as opposed to larger plastic strains possibly resulting in low-cycle fatigue.

7. FIELD TESTING

Field measurements of strains in girder stringers were taken in July and August, 2006. A battery powered data acquisition system was used to collect strain measurements under routine traffic. This information was analyzed to determine the typical stress ranges occurring in the stringers.

It is important to note that the field measurements do not indicate the initial stress state in the bridge members. The measurements only show changes from the conditions when the gages were applied. Practically, this means that the dead load stress and residual stress level in the members and welds cannot be measured by this instrumentation.

Traffic control concerns prevented testing of known weight trucks. This information would have been helpful in comparing calculated stringer stress values with field measurements. However, since the focus of this testing is on measurement of the stresses actually experienced by the stringers, the lack of known weight test data does not impact the usefulness of the findings.

Data Acquisition System

The data acquisition system used for the measurement of the strain data is a stand alone data acquisition and logging unit (see Figure 7-1). This unit is commercially available under the name DataTaker DT800 (Ref).

Figure 7-1. Stand Alone Data Acquisition System.

The DT800 has 42 analog inputs, giving 42 separate single ended channels or 24 differential channels. These are isolated and over-voltage protected, with measurement across 12 auto-scaling ranges from 10mV to 13V full scale.

All common measurement types are supported, including DC and AC (RMS) voltage, current, resistance, temperature, bridges, strain gauges, 4-20mA loops and frequency. Adjustable excitation and triggering are provided on all channels. A Serial Sensor Port is also included.

Digital I/O consists of 8 digital input channels, and 8 digital I/O channels. Two of the digital inputs have adjustable thresholds for the monitoring of low level signals. Digital state, counts at up to 10kHz and triggering are supported on all digital channels.

An RS232 port, a 10baseT Ethernet port and a PC card port are provided as standard for dataTaker programming and data retrieval. Data can either be returned in real time or stored to internal RAM or a memory card. The DT800 stores programs and data DOS format enabling full compatibility with Windows.

The DT800 has modem dial-in and dial-out capability. TCP/IP is supported, which means that the DT800 can communicate over a local area network. In addition, an on-board FTP server is provided so that files can easily be transferred via the Ethernet or RS232 ports.

The DT800 systems come with comprehensive software suite enabling setup, graphical programming, mimics, plotting and spreadsheet views of the collected data.

Strain Gages Location and Installation

Strain gages were mounted on the stringers at top and bottom flanges, at the mid-height of the web and on the side of the shim plate welded on the top flange of the stringers (see Figure 7-2).

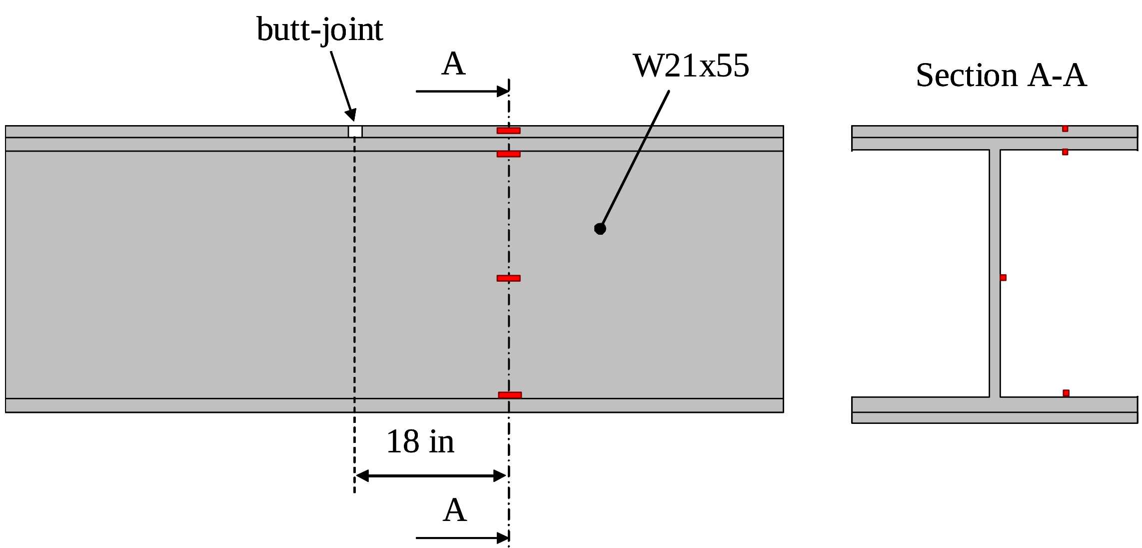


Figure 7-2. Strain gage layout.

Figure 7-3 shows the location of the monitored sections. Please note that for simplicity the monitored stringers were indicated as “Stringer 1” to “Stringer 4”, being Stringer 2 the one where the crack formed. Also the sections in which the strain gages were installed were coded as “Section 1” to “Section 3”. In the same figure, the red dots represent locations on the four stringers corresponding to the cracked section. Finally, at all locations the strain gages were mounted at a distance of 18 in from the butt-joint except for stringer 1 where three additional strain gages were placed at the top and bottom flange and at the mid-height of the web in the section near the butt-joint.

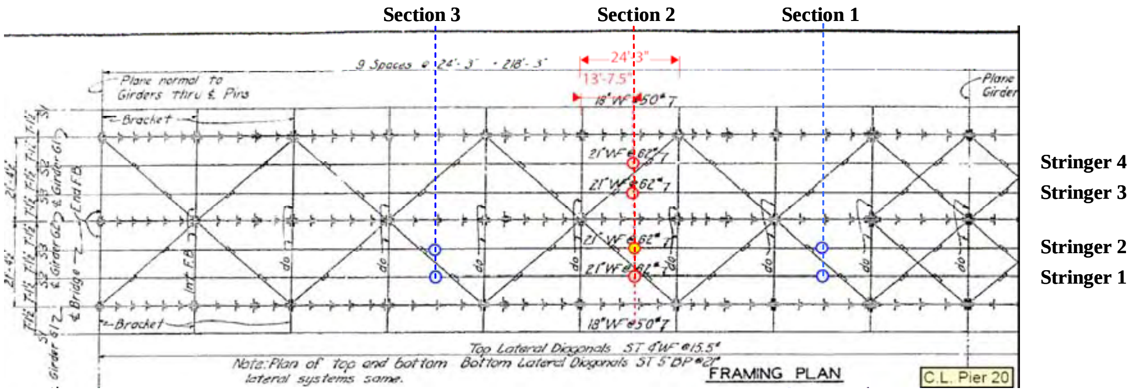


Figure 7-3. Strain gage locations (red and blue dots indicate locations).

Figure 7-4 and Figure 7-5 show the installation of the strain gages and the typical strain gages layout on the stringers after installation, respectively.

The data acquisition system was mounted in a box mounted on the floor beam as showed in Figure 7-6. The box was locked after the sensors were installed and the data acquisition system was programmed.

Figure 7-4. Strain gage installation.

Figure 7-5. Typical strain gage layout on stringer.

Figure 7-6. Data acquisition system mounted on bridge.

Strain measurements were taken continuously from July 10 to July 20, 2006, and from July 31 to August 15, 2006, using a frequency of 5 Hz.

<u>Stringers Stress Range Data</u>

For fatigue analysis purposes, the strain/stress range frequency data (strain spectrum) is used, such data define the frequency (number of occurrences) of various strain ranges. The development of strain range frequency data may be made by means of a data compression / reduction process called “cycle counting.” Various methods for cycle counting are available. The one used here is the “rainflow” process that is described in the standard of the American Society for Testing and Materials (ASTM).

The previously section described layout and location of the elecrical resistance strain gauges. Figure 7-7 to Figure 7-15 show the strain ranges data in the form of bar graphs of the tensile stress range frequencies occurring on top of the shim plate. It should be noticed that the sign of the stresses can be found by looking at the data recorded during the nights between 2 AM and 3:30 AM when the traffic is much reduced and it is possible to determine short periods of times with little or no load on the Bridge.

Additionally, the maximum variation of the average strain recorded between day and night was of 16 με (about 0.46 ksi) due to temperature variations in the deck. The variations in temperature cause the deck to move, displacing the stringer and inducing stress in it. This maximum was recorded for stringers 2 and 3 at section 2, and it results in a tensile stress at the top of the shim plates.

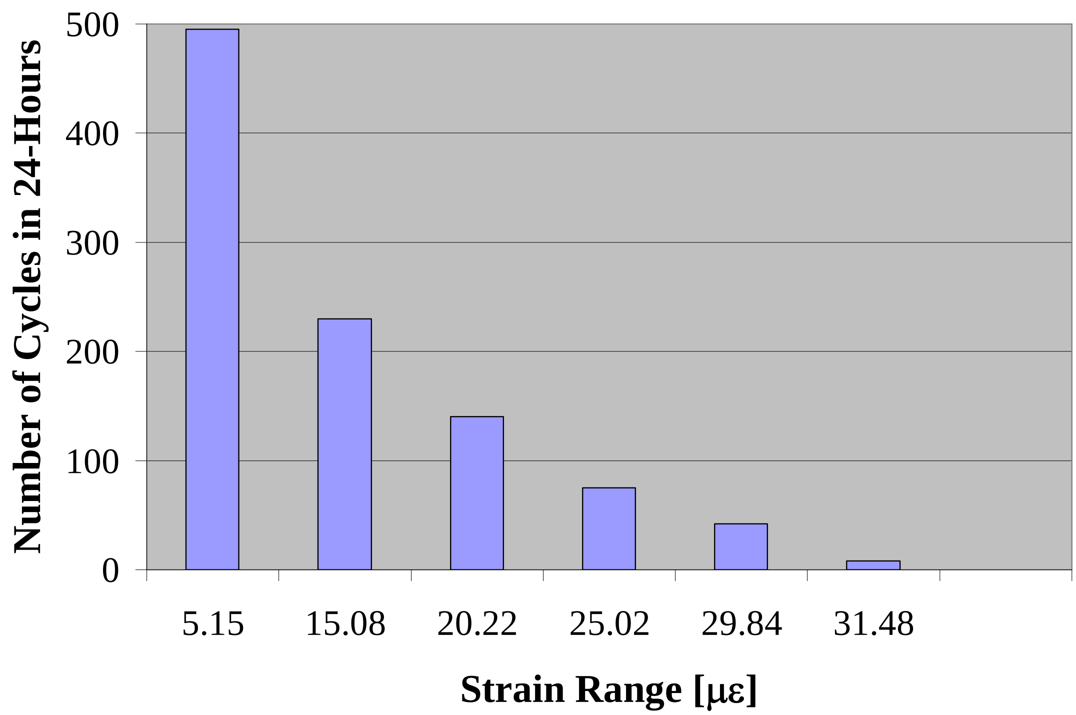


Figure 7-7. Strain range stringer 1 section 1.

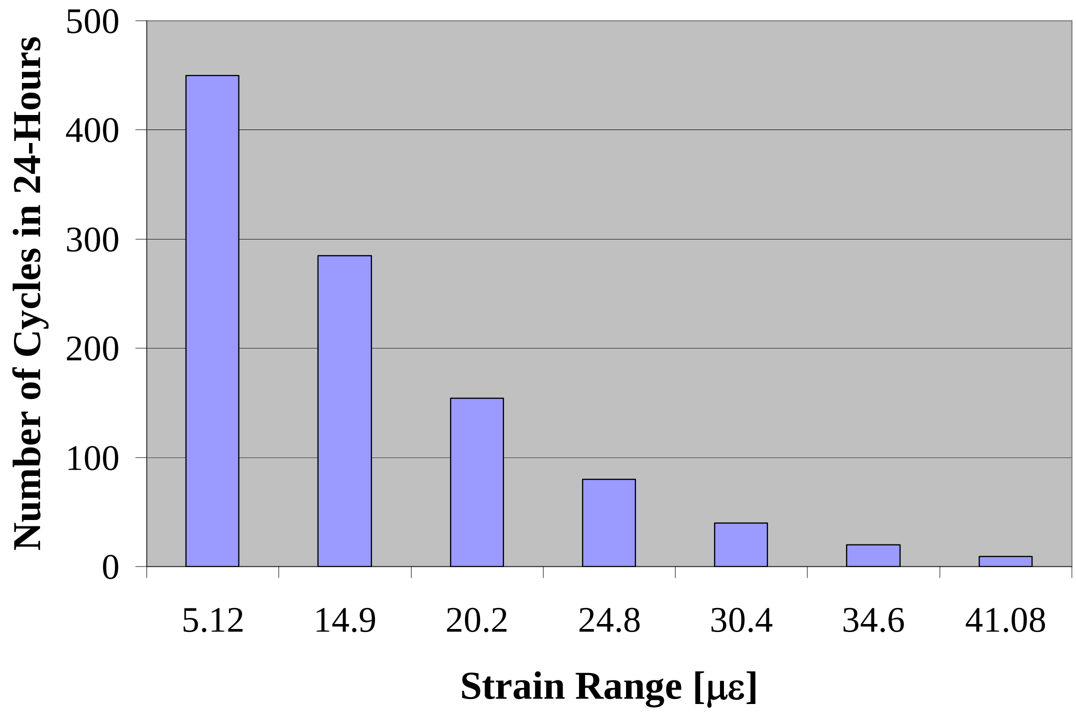


Figure 7-8. Strain range stringer 1 section 2 at the butt-joint.

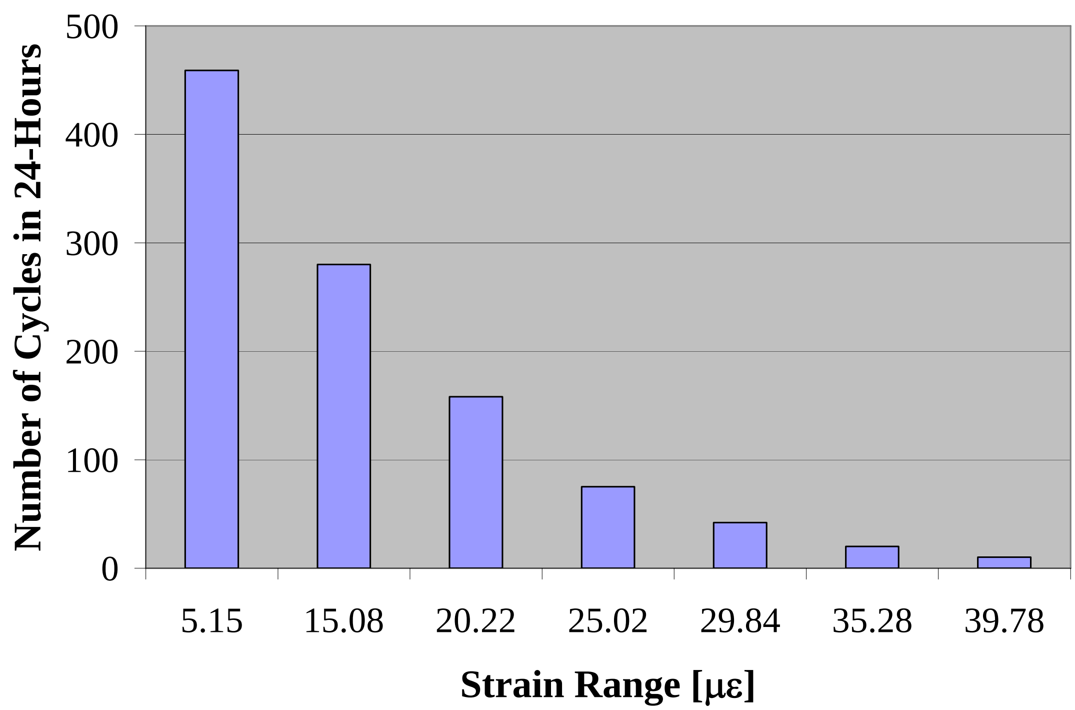


Figure 7-9. Strain range stringer 1 section 2.

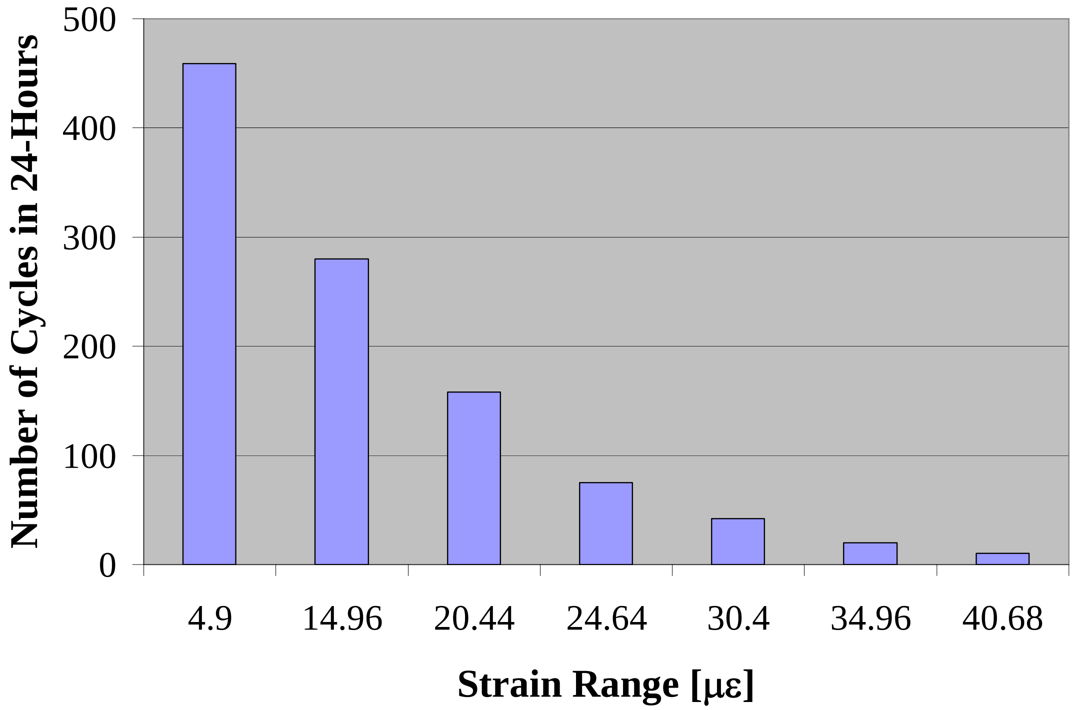


Figure 7-10. Strain range stringer 1 section 3.

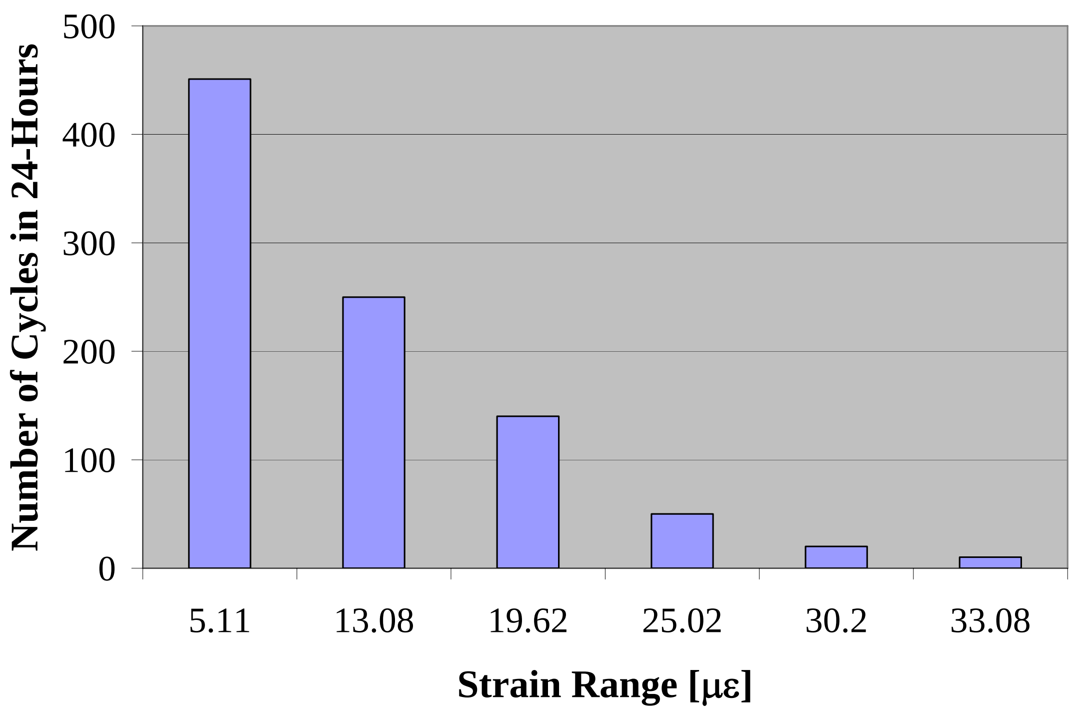


Figure 7-11. Strain range stringer 2 section 1.

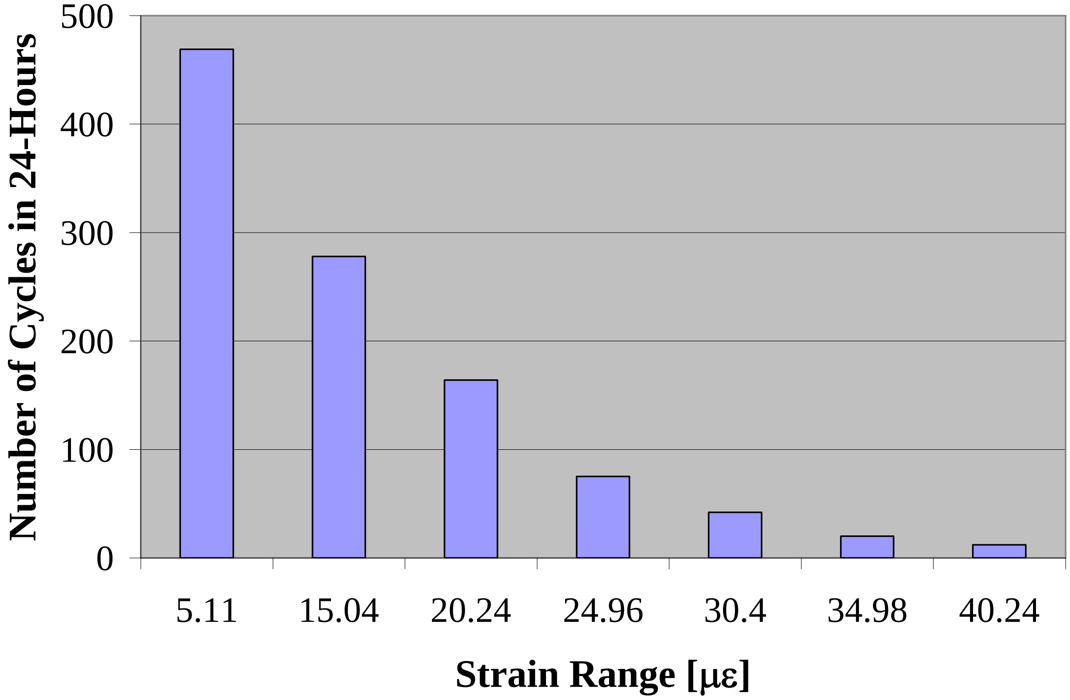


Figure 7-12. Strain range stringer 2 section 2.

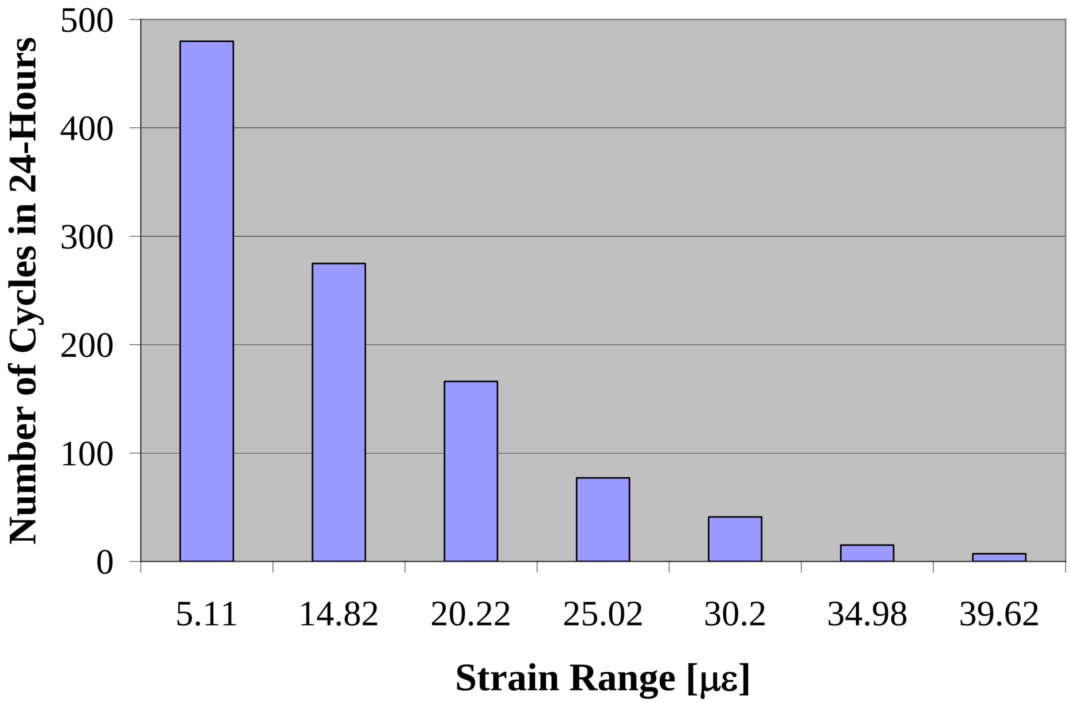


Figure 7-13. Strain range stringer 2 section 3.

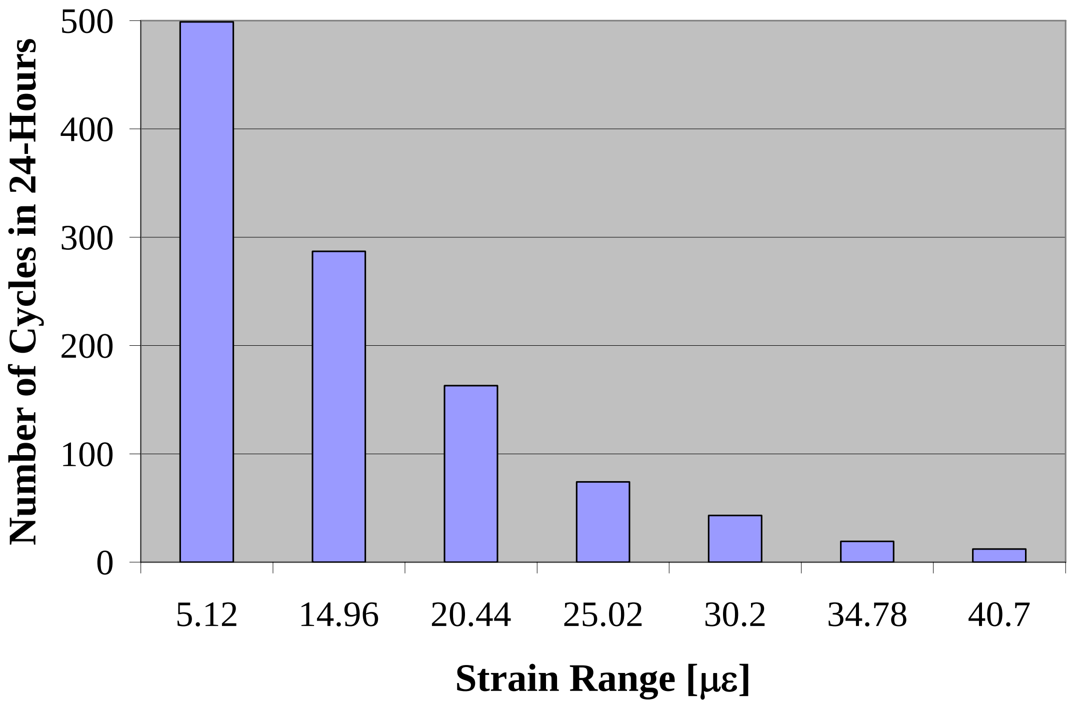


Figure 7-14. Strain range stringer 3 section 2.

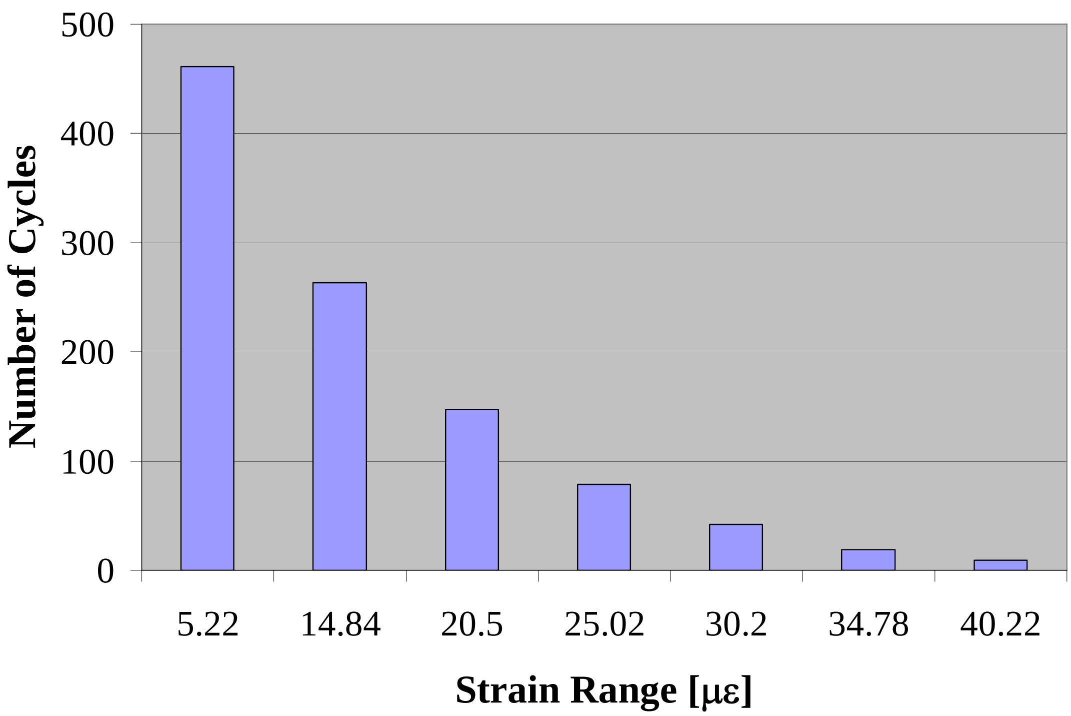


Figure 7-15. Strain range stringer 4 section 2.

Fatigue Evaluation Procedure

The evaluation of the effective stress range was conducted according to the Alternative 1 of the NCHRP 299 procedure (Section 6.2.1) (Moses et al. 1987). The effective stress range for each histogram is given by:

$$S_r = \left(\sum_{i=1}^{n} \left(f_i \cdot S_{ri}^{\ 3} \right) \right)^{1/3} \qquad (1)$$

Where f_i is the fraction of stress ranges within an interval and S_{ri} the stress range at the mid-width of the interval. The midpoints of intervals and the number of cycles in each interval are as shown in the figures summarizing the strain gage results.

Table 7-1 summarizes the effective stress recorded in correspondence of all monitored sections. The effective stress ranges are below the constant amplitude fatigue limit. However, as discussed earlier in this report, the stress concentration factor for this weld detail produces significant fatigue effects even with this measured effective stress range. It can be observed a lower effective tensile stress range for the sections closer to the pier (Section 1 for Stringers 1 and 2).

Table 7-1. Effective Stress Ranges at the Monitored Sections

Stringer	Section	Effective Stress Range [ksi]
Stringer 1	Section 1	0.482
Stringer 1	Section 2 at the butt-joint	0.522
Stringer 1	Section 2	0.523
Stringer 1	Section 3	0.524
Stringer 2	Section 1	0.447
Stringer 2	Section 2	0.527
Stringer 2	Section 3	0.509
Stringer 3	Section 2	0.521
Stringer 4	Section 2	0.509

8. ASSESSMENT FINDINGS

Visual inspection shows that the stringer cracks initiated in the weld material as the fillet welds cross the gap in the shim plates at open butt joints. The condition of the east approach stringer following cracking implies that the stringer was initially experiencing negative moment at the crack location instead of the positive moment that would normally be anticipated.

Research has shown that cracking can occur in members that experience mostly compression stress range cycles if some (relatively few) cycles cause tension at the location, especially if a residual tensile stress is present in the material. This type of residual tensile stress can be expected in welds.

An approximate fatigue analysis showed that cracking could be expected in a relatively short period after the redecking project if live load stress ranges caused tension in the stringer top flange or weldment.

The detailed structural analysis showed a very high stress concentration factor in the weld at the shim plate butt joint. This joint causes a very significant geometric discontinuity, which has been noted as a major source of weld failures, particularly in members experiencing tensile stresses (Ricker, 1988).

Field measurements of actual stress ranges under traffic show an effective stress range of about 0.5 ksi. Also the maximum tensile strain induced by thermal effects was 16 $\mu\varepsilon$, corresponding to an initial cyclical stress of about 0.46 ksi. These measured stresses, combined with a dead load force in the stringer less than expected in design, can induce tension in the area of the shim butt joints.

Based on these findings, the stringer cracking occurs due to a combination of effects:

- The most critical factor is the fillet weld crossing the shim open butt joints, resulting in a very high stress concentration in the weld.
- Stringer top flanges may be in tension at this location, despite the intuitive sense that the stringers in the middle half of the span should be in positive bending. Negative bending causing top flange tension can result from vehicular live loads, temperature forces, reduced actual dead loads, and initial construction misfits.
- Welds will likely experience tension during some live load cycles when the combination of negative bending due to the moving load on the stringer and the likely residual stress in the weldment exceeds the dead load compression at the detail.
- The repetitive loading, with some cycles causing tension in the weldment, led to fatigue cracking of the weld. This cracking propagated into the stringer, which was under negative bending at this location, resulting in the crack progressing through the stringer until it arrested as it neared the bottom flange. The relatively rapid growth of the crack, as well as the camber of the girder when the crack was found, imply that the stringer was carrying significant negative moment.

Two cracked stringers have been found in routine inspections of the bridge. No visible signs of distress at the deck level were reported related to these cracks, and no other significant damage to the bridge was found associated with the cracks. The strength and stiffness of the grid deck, combined with the redundancy of the stringers due to their continuity across several floorbeams, provides alternate paths for vehicular loads to be transferred around the crack location to other portions of the structure and then carried to the supports.

The two stringers cracked after 20 and 26 years in service following the redecking project. The age of the detail at the time of the first crack shows that even though the detail creates a very high stress concentration, other factors (particularly the low live load stress ranges) must be mitigating this condition. On the other hand, since fatigue damage is constantly accumulating, more cracks can be expected in the future. Therefore, repair, retrofit, or mitigation efforts to address these details are warranted. Specific strategies for these will be discussed later in this report.

The factors leading to cracking (*i.e.*, the use of this construction detail) have several sources. Most of these factors occurred in the design phase.

The designers apparently did not realize the significance of the stiffness of the grid deck in relation to the dead load forces in the stringers. While they did eliminate open butt joints in the areas within 1/4 of the span between floorbeams (a typical approximation of the inflection points in the span), they did not foresee the possibility of reduced dead loads, due to live loads, leading to tensile stresses in the midspan area. In addition, the weld detail shown did not give direction on how to handle the welds at the shim plate open butt joints. Finally, the issues of construction tolerances and the possibility that the grid deck would not seat on the stringers were not foreseen, leading to the misfits described earlier.

Construction inspectors apparently either did not notice the field welds crossing the butt joints, or more likely, they did not realize the adverse effects the welds could cause.

In fairness to all parties involved in the redecking project, it should be emphasized that the resulting problems have been limited to date and took twenty years or more to occur.

During preparation of this report, MoDOT staff provided a list of Missouri bridges with grid decks.

Table **8-1** shows this list; no similar cracking has been reported in these bridges.

Table 8-1. Bridges with open and filled grid decks in Missouri.

District	Bridge	County	Route	Year Built	Year Rehabbed
1	K0697	Buchanan	US59	1938	1975
2	G0069	Saline	MO 240	1922	1986
2	K0999	Carroll	MO 41	1939	1983

3	K0932	Pike	US 54	1928	1981
3	L0099	Marion	US 24	1930	1982
4	K0108	Clay	US 69	1933	1990
4	K0392	Jackson	US 24	1934	1977
4	K0456	Platte	US 69	1935	1979
5	S0391	Camden	J	1932	1997
6	A4856	St. Louis City	MO 799	1951	1989
6	L0561	St. Louis	I-70	1958	1995
6	J1000	St. Louis	US 40	1935	1992
6	L0667	St. Louis City	I-64	1956	1977

(Foster 2006)

9. RETROFIT AND REMEDIATION STRATEGIES

The other locations on the Blanchette Bridge that have similar shim butt joints have the potential for fatigue cracking of welds. Depending on the stress state of the stringer, the crack may propagate into the top flange or may arrest at that point.

A variety of repair and mitigation strategies and procedures can be considered to address fatigue damage and vulnerability (Byers et. al. 1997). Response strategies that are applicable to the Blanchette stringers include the following:

- Perform no repairs or retrofits, while continuing normal inspections. This option could be considered the "do nothing" approach.
- Perform no repairs or retrofits, but increase inspection frequency. This option would reduce the exposure time between a stringer cracking and its discovery and repair.
- Perform limited retrofits, aimed at removing crack initiation locations. This approach has been described as being particularly effective in connections, such as this one, that are fatigue sensitive and prone to imperfections (Byers et. al. 1997).
- Perform more extensive retrofits, aimed at strengthening the stringers and protecting against the consequences of stringer cracking.

Repairs or retrofit schemes should recognize the limited space available for access to the weld area. The distance between the top of the stringer top flange and the bottom of the concrete fill in the deck is about 3", as shown in Figure 9-1. In addition, the shim plate is centered on the stringer top flange, placing the fillet weld about 2 1/2" from the edge of the stringer top flange. (One impact of the limited space for retrofit can be seen in Figure 3-3, where the bolts in the top flange splice are located to fit between the ribs of the grid deck.)

Retrofits aimed at removing crack initiation locations could include welding the open butt joints, grinding of the existing welds, or peening of the welds.

In many locations, the shim plates were butt welded prior to installation of the shims. No cracking has been found at this location, and the continuous fillet welds have much better fatigue performance than at locations where the open butt joints occur. Therefore, one possible retrofit would be to weld the open butt joints. This option would remove the discontinuity that produces the high stress concentration. The weld would have to both fill the gap between the ends of the shim plates and remelt the fillet weld at this location to eliminate any existing cracks. Practically, however, the access to the joint (see Figure 9-1) is so limited that performing high quality welds, particularly along the stringer centerline, would be virtually impossible. Because of this factor, this retrofit option was eliminated from further consideration.

Figure 9-1. Limited space available for work on joints.

The purpose of grinding (see Figure 9-2) would be to remove the fillet welds in the immediate vicinity of the butt joints. The detailed structural analysis showed that removal of the welds would reduce the stress concentration effect at this location by a factor of about six. To maximize the effectiveness of this approach, grinding must remove all of the fillet weld material at the shim butt joint and leave the remaining weld ends and stringer top surface in a smooth condition. Fillet welds should be removed approximately to a distance of about 2” in each direction from the shim butt joint. The ends of the welds should be ground at a taper from full-thickness to zero at the 2” clear point. Past research has shown good results from a 1:3 taper ratio (weld thickness to length of taper) (Simon and Albrecht, 1981).

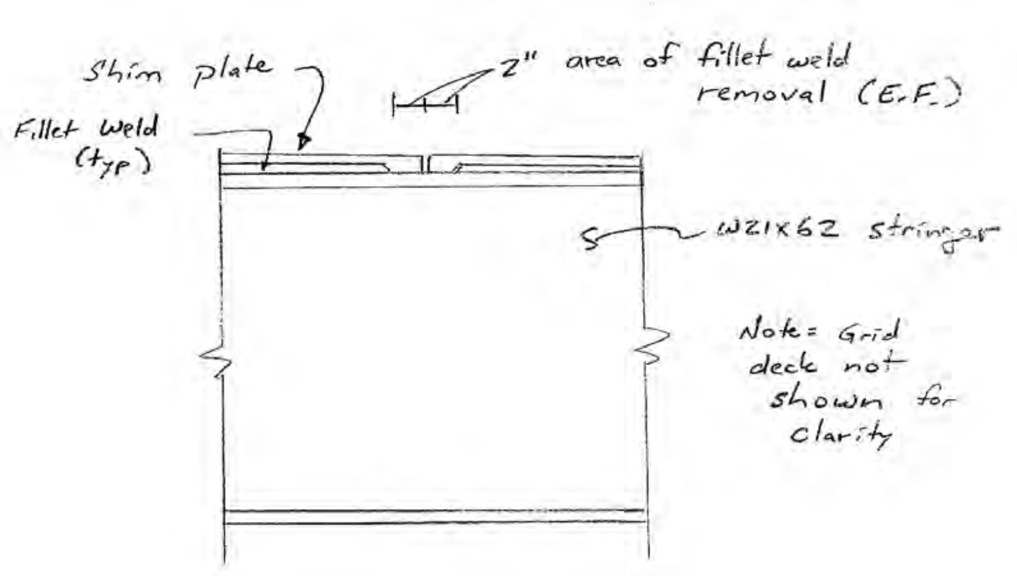


Figure 9-2. Grinding retrofit.

Peening of the weldment induces compressive residual stresses and can close very small cracks. Hammer peening of the fillet welds in this location may be awkward because of the limited space available. Ultrasonic Impact Treatment (UIT) is a technology (see Figure 9-3) developed in the Soviet Union and relatively recently introduced in the United States. It uses ultrasonic waves to produce residual compressive stresses in weldments. Tests on girders after UIT showed significant improvement in fatigue performance of structural welds (Takamori and Fisher 2000). UIT is a possible retrofit method for the shim butt joint welds. Note that peening of the welds will improve their fatigue resistance, but will leave the fillet weld across the butt joint gaps in place.

Figure 9-3. Ultrasonic impact treatment equipment.
(Applied Ultrasonics, 2006)

Strengthening retrofits would reduce the stress range at the joint by adding material. This material could also provide an alternate load path in case of a stringer crack. Conceptually, these retrofits are similar to the spliced repair of the stringer (shown in Figure 3-3). Flange splice plates would be attached to the top and bottom flanges. Web splice plates would be placed on each side of the web. The splice plates would be bolted to the stringer.

Table 9-1 shows a number of strategies for addressing this issue. For comparison purposes, each strategy is listed with its incremental cost, advantages, and disadvantages.

Table 9-1. Comparison of response strategies for addressing stringer shim joints.

Strategy	Incremental Cost (over current procedures)	Advantages	Disadvantages
1 – "Do nothing"	None	No change in current procedures	Relies on redundancy for safety between inspections
2 – Increase inspection frequency	Minor	Little change in procedures, reduces "exposure period" between inspections	Relies on redundancy for safety between inspections
3—Mitigate crack initiation sites	Moderate	Removes crack initiation sites	Does not provide additional load paths in case of cracking
4—Strengthen structure	Significant	Provides additional load paths in case of cracking	Most complex and costly

The AASHTO *Manual for Condition Evaluation and Load and Resistance Factor Rating of Highway Bridges* (AASHTO 2003), Section 7.4, states that bridges fabricated prior to 1978 may have lower fracture toughness levels than are currently acceptable. The commentary points out that propagating fatigue cracks in bridges of questionable fracture toughness are very serious. Therefore, strategies 1 and 2 from Table 9-1 cannot be recommended, as they do not improve the physical performance of the bridge or reduce the risk of cracks occurring.

Strategies 3 and 4 do improve the performance. Practically speaking, implementation of strategy 4 (strengthening) would probably include the work described in strategy 3 (removal of crack initiation sites), as the installation of the strengthening would obstruct access and prevent later work on the weldment.

Considering the inherent redundancy of the stringer and grid deck system and the performance of the bridge after prior cracks, strategy 3 is the approach that best balances structural safety and economy.

Grinding, rewelding, and peening (either hammer or UIT) are possible techniques for removing crack initiation sites. Rewelding will be hampered by the limited accessibility to the butt joints. Peening will improve the performance of the weld across the butt joint, but leaves the weldment in place. Only grinding actually removes the weld material from the critical location. Therefore, grinding to remove the weldment is recommended.

10. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions can be reached on the cause of the cracked stringers:

1. The stringer cracking occurred at details that had very high stress concentration factors due to open shim butt joints with fillet welds crossing the joint.
2. Fatigue of the fillet welds due to traffic loading led to cracking of the weld material.
3. High negative bending stresses in the stringer resulted in cracking through most of the section after the weld crack propagated into the stringer.
4. The high negative bending stresses result from construction or temperature forces, or some combination of those, in addition to the effects of continuity on the stringer force distribution.
5. The redundancy and strength of the grid deck and stringers prevented serious distress or failure in the bridge deck.

The detail that led to the cracking was due to decisions made during the design and construction phases of the redecking project.

The following actions are recommended:

1. The open shim butt joints should be retrofit or modified to eliminate the crack initiation area where the fillet welds cross the joint gaps. A suitable retrofit plan uses grinding to remove the welds for a distance of 2" in each direction from the gaps. Grinding should remove all weldment in this area. Ends of welds should be smoothly tapered to reduce stress concentrations. Following grinding, the affected areas should be visually inspected and tested by dye penetrant or magnetic particle methods to confirm that no cracks exist in the stringer or remaining welds.
2. This case study should be presented to bridge design, construction inspection, and bridge maintenance staff to alert them to the causes and relevant issues of the stringer cracking. The objective of this action is to prevent similar details from being used in future projects without proper consideration of potential problems.
3. Drawings of other bridges with grid decks should be reviewed to check if similar details exist on other structures.

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