Evaluation of Fatigue-Sensitive Details Used in Moline Viaduct, Illinois

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A comprehensive study was undertaken to assess the performance of fatigue-sensitive details used in the Moline Viaduct. This 26-span, 872-m (2,860-ft) structure constructed in 1973 is located on Interstate 74 in Rock Island County, Illinois. The complex bridge superstructure includes variations in span length and width along with curved and superelevated geometries. The superstructure is fabricated from ASTM A36 steel and includes longitudinal plate girders supported by box-shaped cross-girders at the mainline piers. The cross-girder extends continuously through the web plates of the longitudinal girders. Full-penetration welds are used to complete the girder to cross-girder connection. This detail is considered potentially fracture sensitive by an FHWA notice dated April 24, 1978. This notice was issued following the brittle fracture of several steel support bents of the Chicago Transit Authority's Dan Ryan Transit Structure in January 1978. A brief discussion of these brittle fractures is presented to introduce the fatigue behavior characteristics of slotted member bridge details. Examination of the structure revealed a number of cracking problems at the girder to cross-girder connection. In addition, various fatigue-sensitive conditions were identified in the cross-girder interior. Field testing indicated that nominal stress ranges in the vicinity of fatigue-sensitive details were below the crack growth threshold and crack growth should not occur. However, preventive retrofit recommendations and a surveillance program were recommended to address cracking and nonconformance items.

A comprehensive study of the Moline Viaduct was initiated in February 1990 for the Illinois Department of Transportation (IDOT) to address concerns regarding the fabrication of the box-shaped cross-girder and its connection to the longitudinal girders. The details used to complete the cross-girder to girder connection are considered potentially fracture sensitive by an FHWA notice dated April 24, 1978. This notice was issued to alert departments of transportation of potential cracking problems at locations where primary bridge members are slotted to receive other primary members and welding is used to complete the connection. Major brittle fractures that occurred in steel support bents of the Chicago Transit Authority's (CTA's) Dan Ryan Transit Structure in January 1978 prompted the FHWA notice. A brief discussion of these brittle fractures is presented here to introduce the fatigue behavior characteristics of slotted member details. Interestingly, the CTA structures that experienced brittle fracture were designed and fabricated in 1967–1970 by the same two firms that designed and fabricated the Moline Viaduct in 1970–1973.

The Moline Viaduct study included in-depth field inspection, instrumentation and field testing, structural analysis, and development of recommended retrofit procedures. Instrumentation and field testing was performed to determine live-load stress ranges in representative longitudinal girders, cross-girders, and adjacent cross-girder to longitudinal girder connections. This paper summarizes the information that was collected and outlines recommendations for retrofitting the Moline Viaduct.

Description of Moline Viaduct

The 26-span Moline Viaduct includes two separate mainline roadways, northbound and southbound, and four ramps along Interstate 74 in Rock Island County, Illinois. The 872-m (2,860-ft) structure carries traffic to and from the south end of the Twin Memorial Bridge that crosses the Mississippi River. The complex bridge superstructure includes variations in span length and width along with curved and superelevated geometries. The superstructure is fabricated from ASTM A36 steel and includes longitudinal plate girders and box-shaped cross-girders. The viaduct was designed in the early 1970s by using AASHO and IDOT specifications in effect at the time. Construction was completed in 1973. A view of the superstructure framing is shown in Figure 1.

The box-shaped cross-girders extend continuously through the longitudinal girder web plate and are supported by concrete piers to form bents. Cross-girder flange plates are typically 76.2 cm (30 in.) to 91.4 cm (36 in.) wide and 5.1 cm (2 in.) thick, whereas the web plates are typically 116.8 cm (46 in.) deep and 1.9 cm (0.75 in.) thick. Complete-penetration, single-bevel groove welds with backing bars are used to fabricate the box-shaped members. Backing bars are attached to the box interior with short intermittent fillet welds. Interior diaphragms are provided and align with longitudinal girders that frame into the box. Interior diaphragms and bearing stiffeners are fillet welded to the cross-girder web plates and compression flange. The cross-girder web plates and compression flange are connected to the longitudinal girder web with complete-penetration, double-V groove welds reinforced with fillet welds from both sides. A tight fit is provided between the cross-girder tension flange and girder web. A 2.5-cm (1-in.)-radius, half-circle stress relief hole is furnished in the girder web near the cross-girder tension flange. Note that stress relief holes for northbound Piers 1 through 4 were omitted during fabrication. Typical cross-girder fabrication details are shown in Figure 2.

FIGURE 1 Superstructure framing.

The cross-girders were fabricated and shipped to the site with short segments (typically 5 ft beyond the web plate of the cross-girder) of the longitudinal girders attached. Longitudinal girders were then bolted to the shorter segments in the field. The number of longitudinal girders varies from 5 to 11 because of the changing roadway width. Girders occur in two-, three-, or four-span units between hinges, and in positive-moment regions girders are composite with the 20.3-cm (8-in.)-thick concrete deck. Interior girders are straight between bents, whereas fascia girders are curved.

Behavior of Slotted Member Details

Cracking in three rigid frame bents of the CTA Dan Ryan Transit Structure in January 1978 drew national attention to the poor fatigue characteristics associated with slotted member details. The brittle fracture in the Dan Ryan Transit Structure, shown in Figure 3, initiated at the welded junction of the longitudinal girder bottom flange tip and the box-shaped cross-girder. Cracking extended so as to completely fracture the cross-girder bottom flange and much of the web plates. The girder to cross-girder connection was fabricated such that the longitudinal girder bottom flange passed continuously through flame-cut slots in the cross-girder web. The flange to web connection was completed by groove welding around the perimeter of the bottom flange. Examination of the fracture surface indicated that brittle fracture had occurred after fatigue cracks had developed from unfused regions in the welded flange connection. The unfused regions form cracklike embedded defects that at very low stress range levels are sensitive to fatigue crack propagation. Poor-quality welds, fatigue crack growth, low temperatures, and stress concentrations at this highly restrained joint detail contributed to the brittle fractures (*1*).

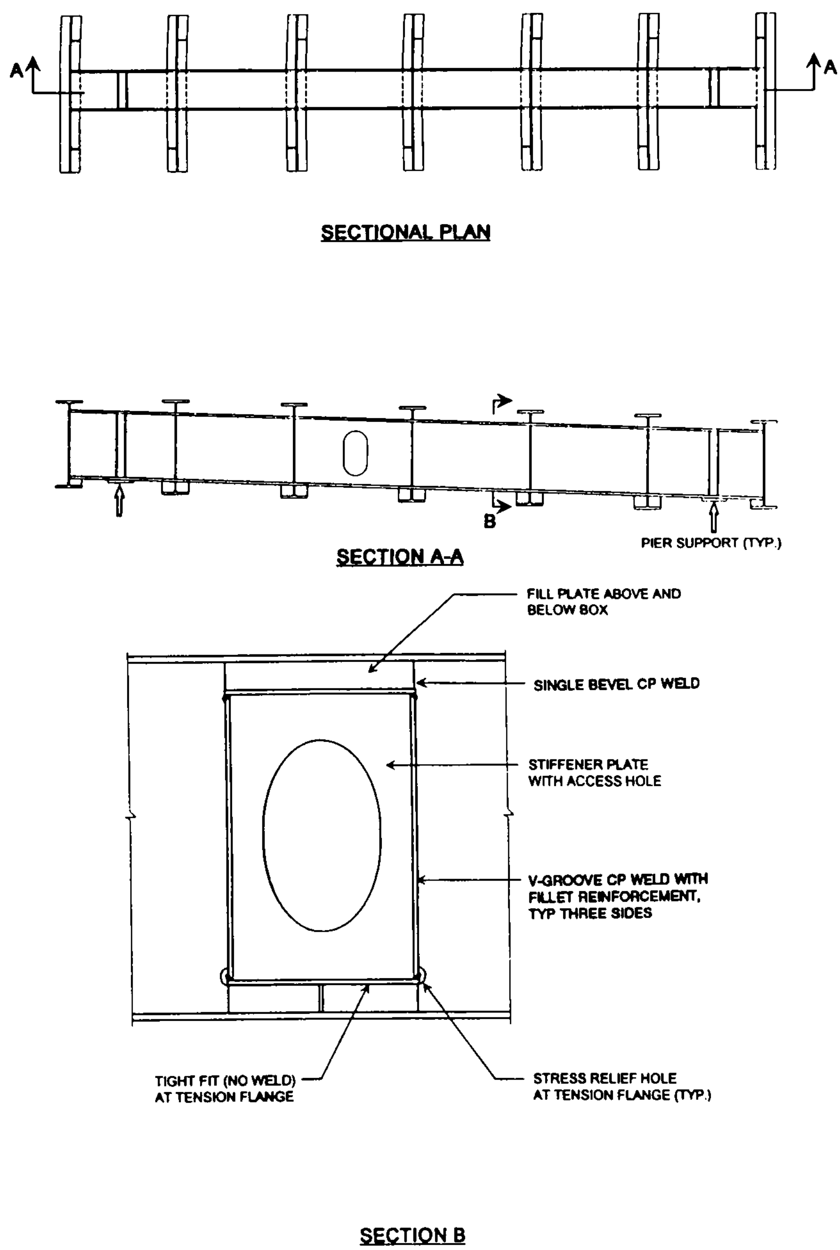
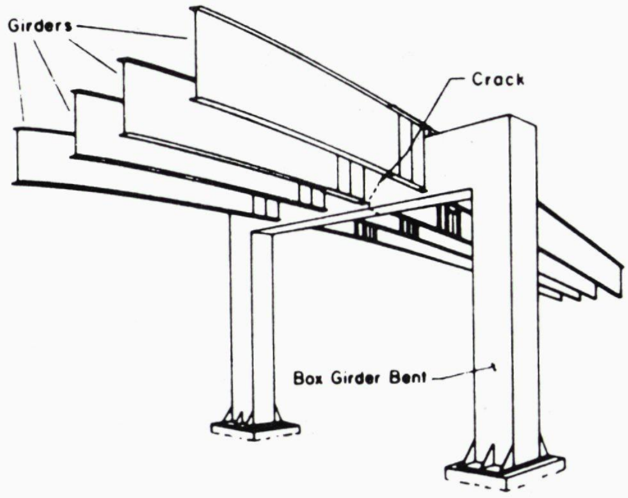


FIGURE 2 Typical cross-girder details.

Brittle fractures in the Dan Ryan Transit Structure, in addition to cracking found in other structures containing similar details, created a need for further research into the fatigue behavior of slotted member details. FHWA responded in April 1978 by issuing a warning to departments of transportation about the fracture-sensitive nature of slotted member details. The warning states that member penetrations located in a tension region exhibit a potential for fracture greater than a Category E detail. Slots that closely approximate the size of the member passing through the slot may contain flame-cut edges and sharp reentrant corners, which may result in high stress concentrations. At the points where welding is provided to close the gap, additional stress concentrations are imposed because of shrinkage of the highly restrained weld around the periphery of the slot. Furthermore, weld construction may result in embedded weld defects at the corners of the slot (*2*). These factors affect the fatigue sensitivity of this type of detail.

Research (*3–5*) reported that the fatigue resistance of slotted member details ranged from slightly better than Category E to approximately one-half of Category E′. Factors affecting the fatigue resistance classification



(a)

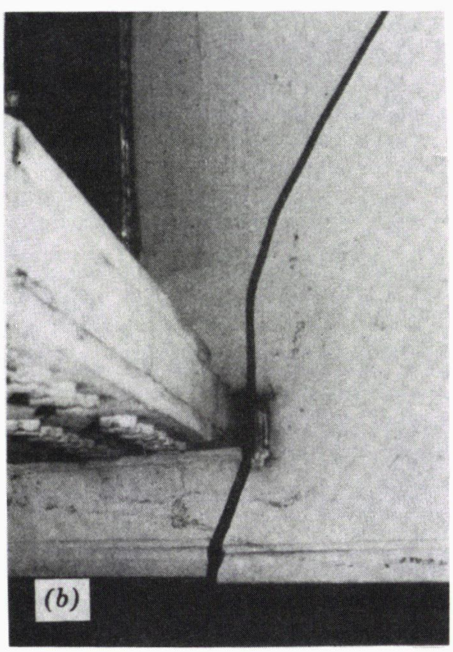


FIGURE 3 Fracture in CTA Dan Ryan Transit Structure, slotted member detail (*1*): (*a*) location of fracture; (*b*) close-up view of cracked box girder web.

of slotted member details include size of the connected components, quality and type of weld, and presence of a stress relief hole at the penetrating flange tip. The fatigue behavior for flanges framing into or piercing through girder webs is comparable to the fatigue resistance of a Category E′ detail when the flange thickness is equal to or greater than 2.5 cm (1 in.). For a flange thickness of less than 2.5 cm (1 in.) fatigue behavior more closely corresponds to a Category E detail. Fatigue resistance for member penetrations welded on each side is significantly improved over that for member penetrations welded from one side only. Member penetrations welded from one side only provide a fatigue resistance approximately one-half that of a Category E′ detail and should be avoided. The predicted fatigue behavior for member penetrations welded on one or both sides is illustrated in Figure 4. Fatigue resistance is also dependent on web plate thickness when fillet welds are used to complete the connection. The ability of achieving complete penetration with the base metal is greatly reduced when the thickness of the member being penetrated is greater than 0.6 cm (0.25 in.). The presence of weld discontinuities creates cracklike defects that are susceptible to fatigue crack growth at reduced stress levels. Providing stress relief holes at the tip of a flange penetration that is attached to the slotted member by fillet welds was found to result in a member fatigue behavior less than Category E′.

The poor performance of slotted member details used in the Dan Ryan Transit Structure, past research, and the similar details used in the Moline Viaduct led to IDOT's decision to carry out an in-depth evaluation of the girder to cross-girder connections.

In-Depth Inspection

The detailed field inspection of the Moline Viaduct superstructure was subdivided into two parts: (a) high-quality fatigue crack inspection of the welds used to connect longitudinal girders to the cross-girders and (b) inspection of the welds used to fabricate the cross-girders. Considerable time was also given to the inspection of other fatigue-sensitive details including lateral gusset plates and details susceptible to out-of-plane distortion, such as offset cross-frames.

Girder to Cross-Girder Connections

The inspection and nondestructive testing efforts concentrated on the girder to cross-girder connections in the vicinity of the cross-girder top and bottom flanges. At these locations vertical welds either terminate at a stress relief hole or intersect a horizontal weld [Figure 2(*b*)]. All 2,440 weld terminations or intersections were carefully inspected. For interior girders eight weld terminations or intersections, four on each side of the longitudinal girder, were inspected. At fascia girders four weld intersections were inspected. In general, the welds used to connect the longitudinal girders to cross-girders exhibit satisfactory workmanship. Typically, the vertical welds looked better than the horizontal welds.

A total of 82 (3 percent) of the 2,440 weld terminations or intersections exhibited cracking. The number of crack locations in the northbound bridge (52 cracks)

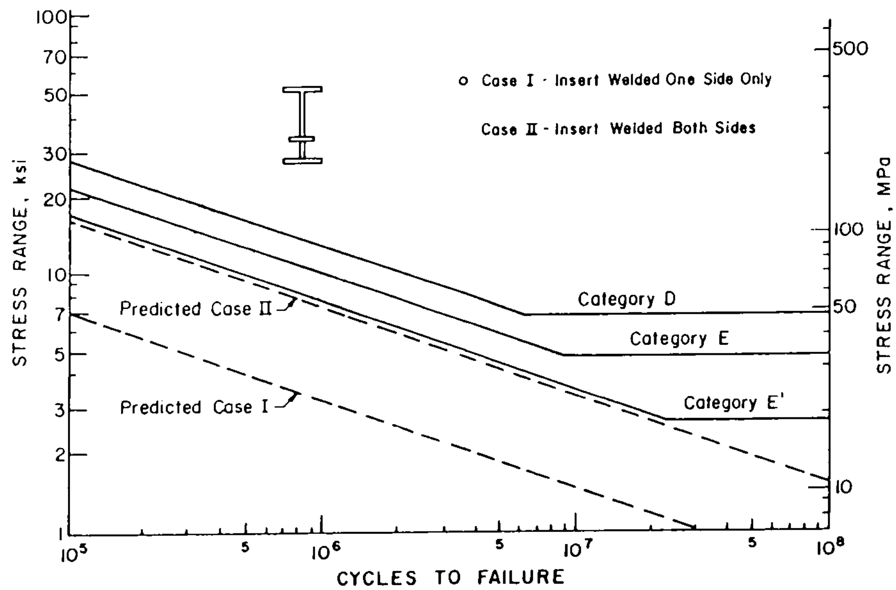


FIGURE 4 Comparison of fatigue life prediction of web penetration with unfused areas in the web (5).

was almost double that found in the southbound bridge (30 cracks). However, 21 of the 52 northbound bridge cracks were located at Piers 1 through 4, where stress relief holes had been omitted during fabrication. Excluding northbound Piers 1 through 4, cracking in the northbound and southbound bridges was almost equal; 31 cracks were observed in the northbound bridge, 23 at the cross-girder top flange and 8 at the bottom flange weld terminations or intersections, whereas 30 cracks were observed in the southbound bridge, 21 at the top flange locations and 9 at the bottom flange locations. A summary of the crack inspection findings is provided in Table 1.

Because many of the 82 cracks were similar in orientation and location, it was possible to categorize the cracks into several types. The most predominant cracking occurred at weld terminations (Figure 5) located at the top corners of the longitudinal girder to cross-girder connection or at weld intersections (Figure 6). In total, 50 of the 82 cracks were located at the top corner connection. Crack lengths varied from 0.3 cm (0.12 in.) to 5.1 cm (2 in.), with an average length of 1.3 cm (0.5 in.). Fifteen of the 32 cracks discovered at the bottom corner weld termination or intersection were located in northbound Piers 1 through 4, where prefabricated stress relief holes had been omitted (Figure 7). Cracks originated from the tight-fit gap between the cross-girder tension flange and girder web and extended horizontally along the groove weld toe. Several cracks were observed to turn and extend downward. Crack lengths ranged from 0.3 cm (0.12 in.) to 2.5 cm (1 in.) and averaged 1.6 cm (0.63-in.).

Subsequently drilled 1.6-cm (0.63-in.)-diameter stress relief holes were provided in northbound Piers 1 through 4. The holes were poorly positioned and did not intercept the crack tip. From Figure 7 it can be observed that crack propagation is not likely to intersect the drilled hole.

Interior Examination of Cross-Girders

All 49 cross-girders were opened and their interior surfaces were inspected. Several details used in the cross-girder fabrication including discontinuous backer bar joints and connecting tack welds, flame-cut gouges, weld remnants, and welded erection aids are categorized as fatigue sensitive. The objective of the inspection was to observe and identify any condition that potentially may have an adverse effect on the long-term performance of the bridge and to find evidence of crack growth if it existed. Heavy corrosion in 10 cross-girders impaired the inspection effort.

Discontinuous backer bars represent the most fatigue-sensitive condition observed. The full-penetration weld used to construct the box is fused to the backer bars on each side of the backer bar butt joint. The butt joint gap represents a built-in cracklike defect. Close inspection of this detail revealed no crack extension. Short intermittent fillet welds, used to hold the backer bar in position, exhibited poor profiles, porosity, and significant undercutting. In general, about 160 short fillet welds were present in 2.4 m (8 ft) of cross-girder (the space between two girder diaphragms). A

TABLE 1 Crack Inspection at Girder to Cross-Girder Connections

Northbound Bridge					Southbound Bridge				
	No. of Cracks					No. of Cracks			
Pier	Top	Bottom	Total	Max. Length cm (in.)	Pier	Top	Bottom	Total	Max. Length cm (in.)
1*	1	3	4	1.9 (¾)	1	2	0	2	1.3 (½)
2*	0	4	4	2.5 (1)	2	0	0	0	--
3*	3	7	10	5.1 (2)	3	3	1	4	1.9 (¾)
4*	2	1	3	1.3 (½)	4	1	2	3	1.3 (½)
5	0	4	4	2.5 (1)	5	2	0	2	2.5 (1)
6	0	0	0	--	6	2	0	2	0.3 (1/8)
7	1	1	2	1.6 (5/8)	7	1	0	1	2.5 (1)
8	2	0	2	1.9 (¾)	8	0	0	0	--
9	2	0	2	1.3 (½)	9	0	3	3	0.9 (3/8)
10	4	1	5	4.2 (1 5/8)	10	1	0	1	3.2 (1¼)
11	1	0	1	1.0 (3/8)	11	0	0	0	--
12	0	0	0	--	12	0	0	0	--
13	1	0	1	0.6 (¼)	13	0	0	0	--
14	0	0	0	--	14	0	0	0	--
15	0	0	0	--	15	1	1	2	1.3 (½)
16	0	0	0	--	16	0	0	0	--
17	1	0	1	1.3 (½)	17	1	0	1	1.0 (3/8)
18	0	1	1	5.1 (2)	18	0	0	0	1.0 (3/8)
19	1	0	1	2.5 (¼)	19	2	0	2	1.3 (½)
20	0	0	0	--	20	1	0	1	0.6 (¼)
21	1	0	1	1.3 (½)	21	0	0	0	--
22	2	1	3	3.8 (1½)	22	1	2	3	1.9 (¾)
23	0	0	0	--	23	3	0	3	2.5 (1)
24	7	0	7	1.3 (½)	24	0	0	0	--
25	0	0	0	--	25	0	0	0	--
TOTAL	29	23	52		TOTAL	21	9	30	

* Stress relief holes omitted during fabrication.

-- No crack found

number of these welds were cracked through the weld throat, with the cracks oriented parallel to the primary stress flow of the box.

Numerous welds that were made as a temporary aid during fabrication and without regard to good welding procedures were found. A number of these welds were cracked. Several cracks represented an extremely poor condition because the crack was oriented perpendicular to the primary stress flow of the box. No crack extension into the cross-girder plates was observed. In addition, flame-cut gouges were found throughout the cross-girders. Flame cutting was used to cut access holes and to remove defective welds and temporary welded attachments. Gouges and rough surfaces in areas subjected to tensile stress were carefully examined for signs of crack extensions; none were found.

Instrumentation and Field Testing

An instrumentation and field testing program was carried out to measure the Moline Viaduct's response to dynamic loadings provided by control vehicles and normal traffic. IDOT provided two 21,800-kg (48,000-lb) six-wheel dump trucks as the control vehicles and arranged a police escort to control traffic. The control vehicles were driven side-by-side across the bridge at 72 km/hr (45 mph). The test program objectives were as

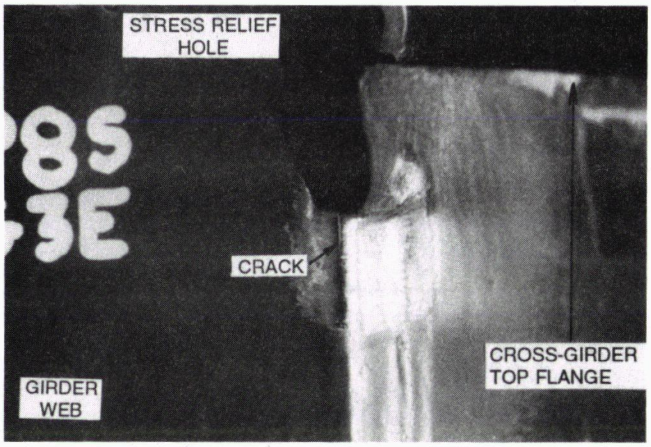


FIGURE 5 Typical crack at top flange stress relief hole.

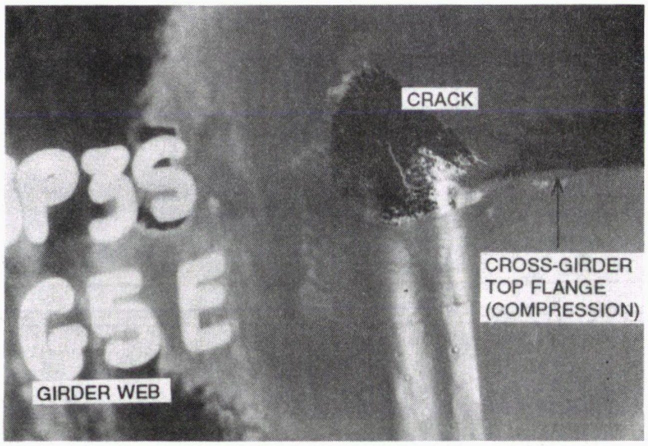


FIGURE 6 Typical crack at top flange weld intersection.

follows: (a) to measure strain levels to compare and verify the structural analysis, (b) to determine stress range at girder to cross-girder connections, and (c) to determine stress range in the box-shaped cross-girder.

Location of Instrumentation

A total of 54 single-element strain gauges were installed. Figures 8 and 9 show the gauge layout and numbering system. Strain gauges 1 through 6 and 49 through 54 were installed on cross-girders at southbound Pier 3 and northbound Pier 6, respectively, to measure the strains in the box-shaped member at maximum moment locations. Gauges 7 through 10 were installed on the girder web at southbound Pier 3 to measure strains in the vicinity of two cracks found at girder to cross-girder connections. Two longitudinal girders in northbound Spans 4–5, 5–6, and 6–7 were instrumented at maximum positive- and negative-moment locations. All gauges were aligned with the longitudinal axis of the member that was instrumented.

The measured stress ranges are summarized in Table 2. In general, the control loading produced the maximum stress ranges. The data obtained during 6 hr of normal traffic provided only a minimal number of vehicles that produced stress levels comparable to those from the control loading. Normal traffic data were recorded continuously during business hours on a typical weekday.

Measured Stresses in Cross-Girders

Representative strain gauge plots are given in Figure 10 for gauges installed on the northbound Pier 6 cross-girder. The maximum tensile stress ranges at the bottom flange of southbound Pier 3 and northbound Pier 6 cross-girder were 10.3 MPa (1.5 ksi) and 12.4 MPa (1.8 ksi), respectively. Gauge 3, located just below the flame-cut access opening, indicated a tensile stress range of 9.6 MPa (1.4 ksi). The strain gradient across the member cross section suggests that the neutral axis is located at the midheight of the member.

Measured Stresses Adjacent to Cracked Connections

Representative strain gauge responses for Gauges 7 and 10 are given in Figure 11. In general, these gauges indicated tensile stress ranges equal to or less than 3.4 MPa (0.5 ksi). However, Gauge 8 indicated a tensile stress range of 16.5 MPa (2.4 ksi). Additional strain gauge work is required to determine the reasons for this

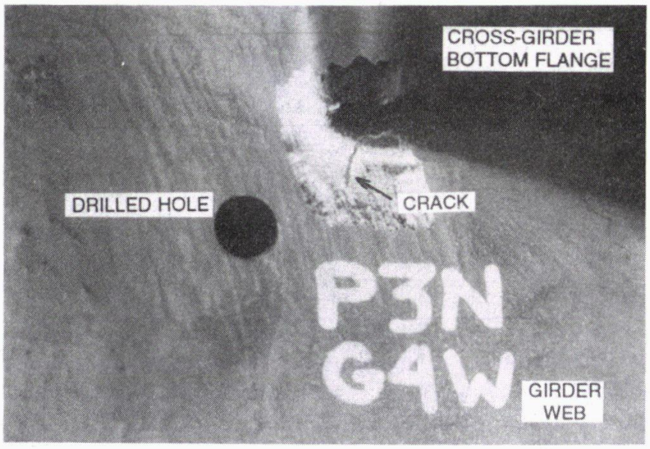


FIGURE 7 Typical crack at bottom flange stress relief hole.

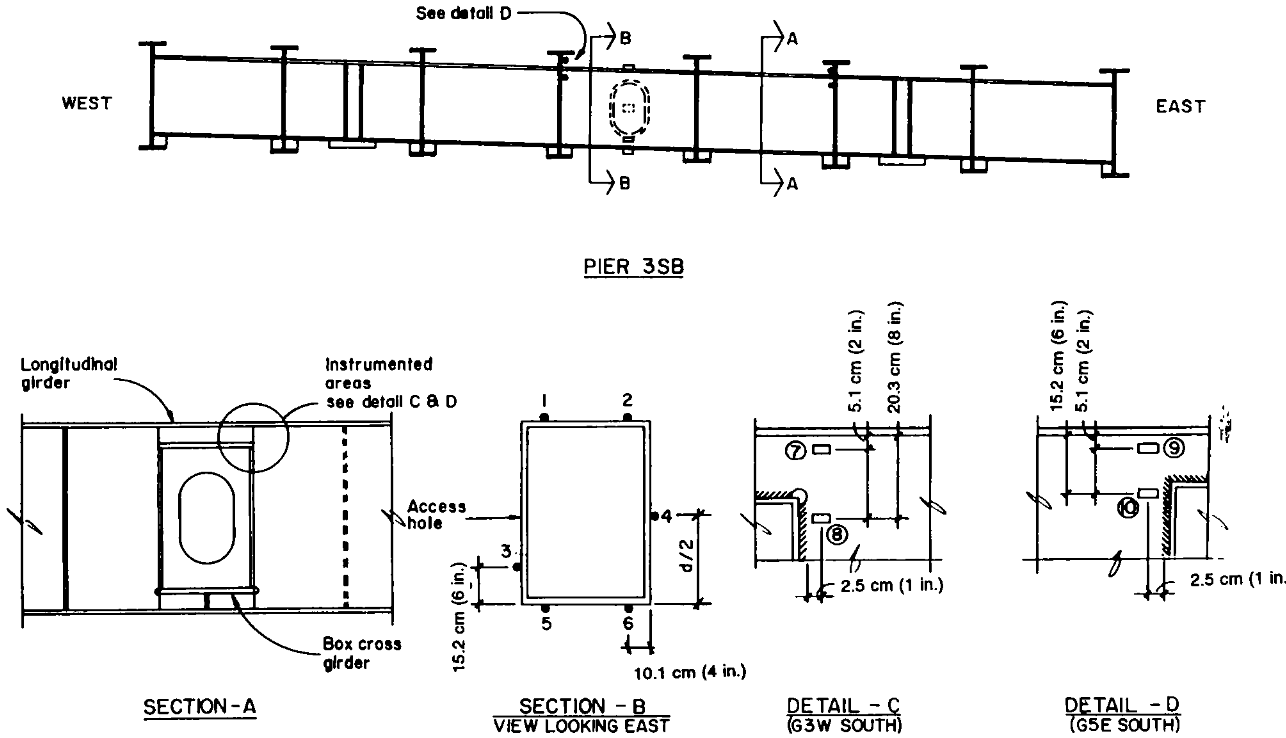


FIGURE 8 Strain gauge layout on cross-girder at southbound Pier 3.

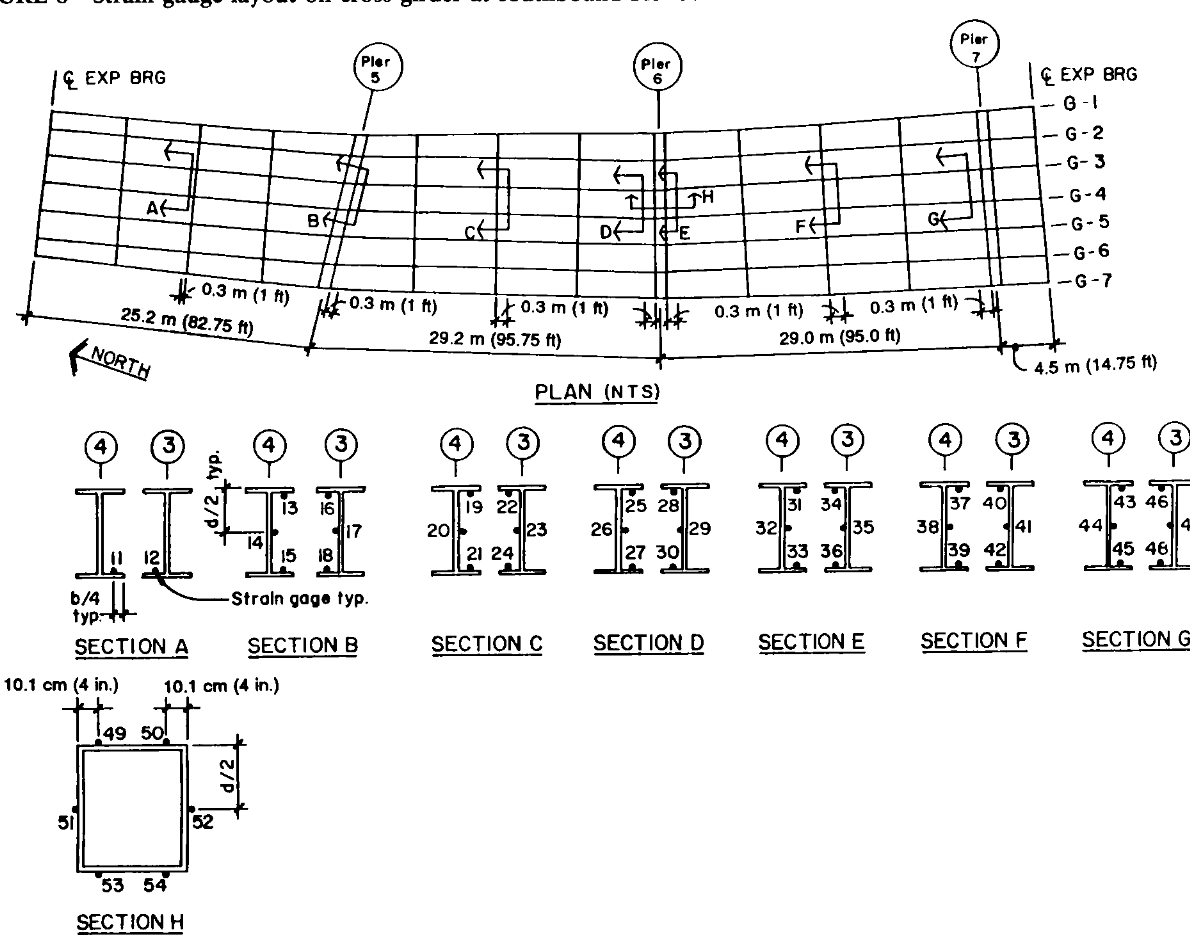


FIGURE 9 Strain gauge layout on longitudinal girders in northbound Spans 4–5 through 6–7.

TABLE 2 **Maximum Measured Stress Ranges***

Gage	Stress Range MPa (ksi)	Gage	Stress Range MPa (ksi)
1	-6.9 (-1.0)	28	2.1 (0.3)
2	-6.9 (-1.0)	29	1.4 (<0.2)
3	9.6 (1.4)	39	-8.3 (-1.2)
4	1.4 (<0.2)	31	2.8 (0.4)
5	10.3 (1.5)	32	-4.8 (-0.7)
6	10.3 (1.5)	33	-8.9 (-1.3)
7	3.4 (0.5)	34	1.4 (<0.2)
8	16.5 (2.4)	35	1.4 (<0.2)
9	3.4 (0.5)	36	-6.2 (-0.9)
10	3.4 (0.5)	37	1.4 (<0.2)
11	23.4 (3.4)	38	8.9 (1.3)
12	20.7 (3.0)	39	24.8 (3.6)
13	-6.9 (-1.0)	40	1.4 (<0.2)
14	-8.3 (-1.2)	41	8.3 (1.2)
15	-14.4 (-2.1)	42	18.6 (2.7)
16	-8.3 (-1.2)	43	2.1 (0.3)
17	-10.3 (-1.5)	44	-2.1 (-0.3)
18	-12.4 (-1.8)	45	-10.3 (-1.5)
19	-2.8 (-0.4)	46	1.4 (<0.2)
20	22.0 (3.2)	47	2.1 (0.3)
21	28.9 (4.2)	48	-9.6 (-1.4)
22	1.4 (<0.2)	49	-12.4 (-1.8)
23	8.3 (1.2)	50	-12.4 (-1.8)
24	16.5 (2.4)	51	1.4 (<0.2)
25	2.1 (0.3)	52	1.4 (<0.2)
26	-8.9 (-1.3)	53	12.4 (1.8)
27	12.4 (1.8)	54	12.4 (1.8)

* Negative values represent compression

higher response. However, secondary effects associated with out-of-plane bending or distortion of the girder web most likely account for the increased response measured at Gauge 8.

Measured Stresses in Longitudinal Girders

The maximum tensile stress range was 28.9 MPa (4.2 ksi), measured by Gauge 21, located in the positive moment region of Span 5–6. In general, tensile stress ranges in positive-moment regions averaged 24.1 MPa (3.5 ksi), whereas tensile stress ranges measured in negative-moment regions were less than 3.4 MPa (0.5 ksi). Girder 4 experienced larger stress ranges than did Girder 3. This behavior is due to the traffic lane positions above the girders and the lateral distributions of loads across the bridge. Test data revealed composite action between the concrete deck and girders in the negative-moment region, even though shear studs were not provided.

Discussion of Results

The cross-girders used in the Moline Viaduct would be classified as nonredundant members because their failure may result in collapse of the bridge, whereas the longitudinal girder framing system is a redundant structure providing multiple load paths. The detail used to connect the two members is considered potentially fracture sensitive by an FHWA notice dated April 24, 1978. On the basis of this notice and previous research (3–5), the girder to cross-girder connection would be classified as Category E′ detail. Several factors were used to justify this determination: (a) connecting welds were made from both sides of the girder web plate, (b) welds were complete-penetration groove welds, (c) the tension

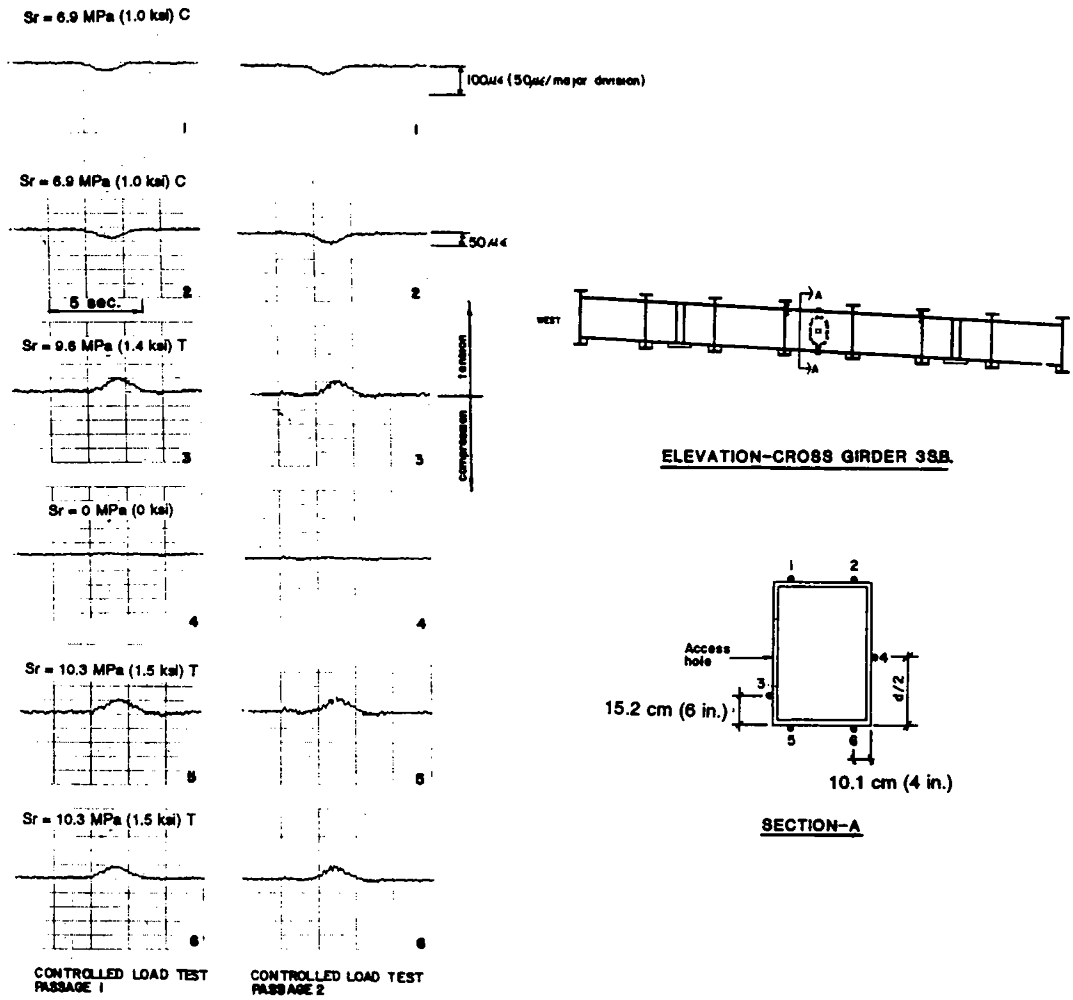


FIGURE 10 Representative strain response for cross-girder at southbound Pier 3.

flange was not welded, (d) slots were not flame cut, and (e) stress relief holes were provided at weld terminations adjacent to the cross-girder tension flange.

Welded details in the cross-girders included Category B′, Category C, and Category E details. Bearing stiffeners and diaphragms would be classified as Category C, whereas groove-welded backer bar details were Category B′ conditions. The discontinuous backer bar detail, however, created a built-in cracklike defect that was probably more severe than Category E. In addition, a number of conditions such as flame-cut gouges and temporary fabrication weldments exhibited poor workmanship and were not in compliance with current AASHTO specifications.

Girder to Cross-Girder Connections

The girder to cross-girder connection detail was exposed to a very complex state of stress resulting from member interaction at the joint, the geometry of the structural framing, welding of residual stresses, and the forces induced during erection. Cracking has occurred in the longitudinal girder web plate adjacent to corners of the cross-girder along the welds used to join the cross-girder to the longitudinal girder. No cracking was found to extend into the nonredundant cross-girder. A total of 82 (3 percent) of the 2,440 corner locations exhibited cracks. A significantly higher incidence of cracking occurred at connection locations in northbound Piers 1 through 4 where stress relief holes had been omitted. Excluding these locations, cracking was observed at 57 (2.3 percent) of the corner locations.

On the basis of inspection findings, review of project welding specifications, and field testing, cracking at the girder to cross-girder connection is believed to have resulted from several conditions that include, but are not limited to, restrained shrinkage of the large welds around the cross-girder perimeter and the forces experienced during erection of the complex curved and su-

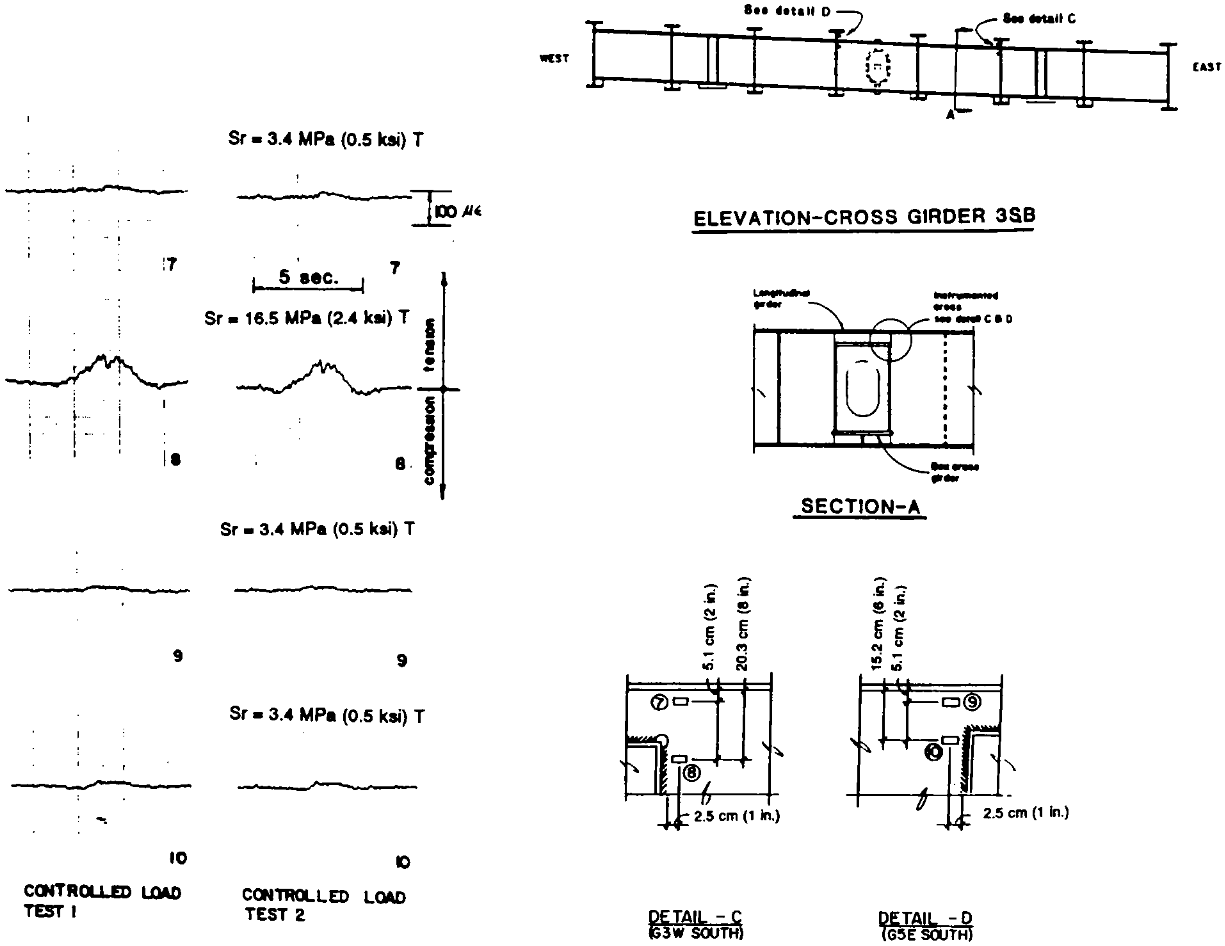


FIGURE 11 Representative strain response adjacent cracked girder to cross-girder connections.

perelevated bridge structure. This conclusion is supported by the following observations: (a) omission of stress relief holes in northbound Piers 1 through 4 resulted in a significantly higher incidence of cracking at these locations, and (b) stress levels in the connection region adjacent to the corners of the cross-girder were measured to be less than 6.9 MPa (1.0 ksi) of tension, which is at a level at which crack growth is not expected.

The fatigue resistance of a Category E′ detail is represented by the following equation:

$$N = A \cdot S_r^{-3.0}$$

where

N = estimated minimum number of cycles to failure,
S_r = allowable stress range [MPa (ksi)], and
A = constant $26.926 \cdot 10^8$ MPa ($3.908 \cdot 10^8$).

This equation is based on a statistical evaluation of available test data (5) and represents the lower bound of failures for the tested details. The maximum stress range at which no fatigue crack growth will occur under constant-amplitude load conditions is called the constant-amplitude fatigue limit (CAFL). The CAFL for a Category E′ detail used in a redundant load path structure is given as 17.9 MPa (2.6 ksi). A 20 percent reduction for 100,000, 500,000, and 2,000,000 cycles is generally applied to allowable stress ranges for redundant members to obtain allowable values for nonredundant load path structures. However, for lower fatigue strength details, a more substantial reduction is taken to discourage their use. For example, the CAFL for the nonredundant Category E′ detail has been reduced 50 percent to 8.9 MPa (1.3 ksi). Note that failure of a Category E′ detail used in the fabrication of a nonredundant structure will perform in accordance with the S-N curve for that detail without any safety factor applied. In other words, the detail behavior is not influenced by the redundant or nonredundant nature of the structure.

The maximum stress range measured in the longitudinal girder web adjacent to a crack was 16.5 MPa (2.4 ksi) of tension. At other locations the maximum measured tensile stress range was between 2.1 and 3.4 MPa (0.3 and 0.5 ksi). On the basis of a finite-element model the calculated stress ranges were between 4.8 and 6.2 MPa (0.7 and 0.9 ksi) for the control loading (6). A stress histogram was not developed to determine an effective stress range. However, observations of the 6 hr of data obtained during a normal business day and review of the average daily truck traffic suggest that an effective stress range of approximately one-half of the maximum would not be unrealistic. Considering the redundant nature of the longitudinal girder and using the maximum stress range instead of an effective stress range, it can be shown that the measured stress levels are below the Category E′ CAFL. Comparing the Category E′ CAFL with the maximum stress range is conservative and suggests that crack growth would not be expected.

Cross-Girder Fabrication

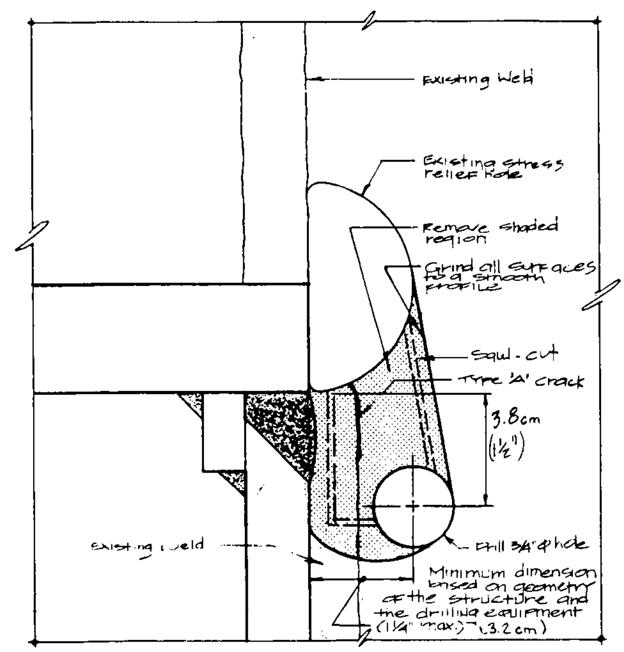
A number of fatigue-sensitive conditions and instances of poor workmanship including discontinuous backer bar joints and poor-quality connecting tack welds, flame-cut gouges, weld remnants, and welded erection aids were identified in the 49 cross-girders. However, no cracks were found to extend into the cross-girder plates. The most significant condition is represented by the discontinuous backer bar joints, which were oriented perpendicular to the tension stress field and which represented a built-in cracklike defect. No cracking was observed at the discontinuous backer bar detail; however, crack growth would not be visually notable until the crack extended beyond the backer bar.

Field testing of representative cross-girders indicated a maximum stress range of about 12.4 MPa (1.8 ksi) of tension. This stress level, although conservative since it is greater than the effective stress range, is below the crack growth threshold for a Category E detail. The CAFL for a Category E detail used in a nonredundant member is 17.2 MPa (2.5 ksi). Although crack growth is not expected, the severity of the backer bar joint may be such that a fatigue resistance lower than Category E could occur, in which case crack growth may develop.

Recommendations and Conclusions

The study described here confirmed the concerns of IDOT about the fatigue-sensitive nature of the girder to cross-girder connections and cross-girder fabrication details used in the Moline Viaduct. On the basis of findings of the in-depth inspection and field testing program, the following recommendations are provided for retrofitting the girder to cross-girder connections.

1. All existing cracks should be removed by coring. Recommended details were developed for all crack types observed. An example repair detail is shown in Figure 12. The function of the repair is to remove the entire crack and adjacent weld metal. This modification will also provide an excellent surface for future inspection.
2. The sections of material removed during retrofitting should be subjected to fractographic examination to confirm the findings of the present study. The tough-



DETAIL-B

PROCEDURE

1. Drill a 1.9 cm (¾ in.) diameter hole through the girder web plate as close as possible to the web plate of the cross-girder, as shown in Detail B.
2. Remove material in shaded area by sawcutting. Grind edges to provide smooth transitions between material removed and existing stress-relief hole.
3. Remove all burrs from cut edge and grind surface to obtain a surface roughness (R_a) of 1000 or less. Grinding operation shall use 1.3 cm (½ in.) diameter or larger carbide burrs.
4. Obtain approval of Engineer before proceeding. All ground and drilled surfaces shall be checked for cracks by magnetic particle or dye penetrant testing.
5. Clean exposed steel surfaces to remove any contaminants or rusting.
6. Paint surfaces with undercoat and final coat.

FIGURE 12 Recommended retrofit for crack Type A.

ness and metallurgical properties of the removed samples should also be determined.

3. New 5.1-cm (2-in.)-diameter half-circle stress relief holes should be provided in northbound Piers 1 through 4 where stress relief holes were omitted during fabrication.

4. An on-going surveillance program should be developed and implemented to identify possible crack growth.

Recommendations for retrofitting the cross-girder are as follows:

1. All discontinuous backer bar joints located within the tension regions of the cross-girder shall be removed by coring. A 3.8-cm (1.5-in.)-diameter core hole shall be cut to encompass the groove weld and discontinuous backer bar detail. The core hole shall be checked for cracklike defects. If no defects are found, a steel plate and gasket shall be placed over the hole and held in place with a high-strength bolt.

2. Several removed cores should be subjected to metallographic examination to ensure that crack growth had not occurred at the detail.

3. Interior surfaces that have experienced corrosion should be cleaned and painted.

4. Removal of a substantial number of fatigue-sensitive details within the cross-girders is not warranted at this time. However, a 5-year surveillance program should be set up to monitor cracked backer bar tack welds, flame-cut gouges, weld remnants, welded erection aids, and so forth for possible crack growth. On the basis of field testing measurements of stress, these details are unlikely to experience crack growth.

The cracking that was observed in the Moline Viaduct appears to have occurred mainly during fabrication and erection, and because stress levels are low, little or no crack growth is expected. Retrofits were developed to eliminate cracking defects at the potentially fracture-sensitive girder to cross-girder connections. No crack extension into the nonredundant cross-girder was observed. Severe cracklike defects associated with the discontinuous backer bar are to be removed. These recommendations along with a surveillance program will ensure many more years of useful service for the Moline Viaduct.

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REFERENCES

1. Fisher, J. W. *Fatigue and Fracture in Steel Bridges, Case Studies.* Wiley-Interscience, New York, 1984, pp. 116–132.
2. *Weld Details.* Federal Highway Administration Notice, N5140.13. FHWA, U.S. Department of Transportation, April 24, 1978, 2 pp.
3. Fisher, J. W., H. Hausammann, M. D. Sullivan, and A. W. Pense. *NCHRP Report 206: Detection and Repair of Fatigue in Welded Highway Bridges.* TRB, National Research Council, Washington, D.C., 1979, 85 pp.
4. Fisher, J. W., B. H. Barthelemy, D. R. Mertz, and J. A. Edinger. *NCHRP Report 227: Fatigue Behavior of Full Scale Welded Bridge Attachments.* TRB, National Research Council, Washington, D.C., 1980, 47 pp.
5. Keating, P. B., and J. W. Fisher. *NCHRP Report 286: Evaluation of Fatigue Tests and Design Criteria on Welded Details.* TRB, National Research Council, Washington, D.C., 1986, 66 pp.
6. Koob, M. J., R. A. Walther, and G. T. Blake. *Evaluation of Girder to Cross-Girder Connections and Bearing Problems of the Moline Viaduct.* Wiss, Janney, Elstner Associates, Inc., 1991, 60 pp.