Review of Field Measurements for Distortion-Induced Fatigue Cracking in Steel Bridges

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Through a review of existing field inspection results and bridge details, many bridges have been identified as susceptible to out-of-plane, distortion-induced fatigue cracking. The review includes floor beam girder, multiple-girder diaphragm, and floor beam tie girder box systems. Consideration is given to primary live-load stresses on the global scale, whereas live-load stress ranges, rotations, and out-of-plane distortions are investigated at the local level. The details of plate attachment, the manner of force transmittal between components, and the specific geometry at gaps are shown to be influential factors in determining the magnitude of out-of-plane distortions and corresponding stresses. Cases are cited in which secondary members imposing out-of-plane displacements of only thousandths of an inch have resulted in cyclic bending stresses at the level of allowable stresses at details. Specific behavior for each detail is discussed.

Forces in secondary members such as cross bracing can cause lateral deflection in steel girder webs, resulting in fatigue cracking. Cracks have also developed in the webs of girders due to differential deflection of parallel girders and end rotations of transverse floor beams. These out-of-plane deflections and rotations generate high-magnitude secondary plate-bending stresses. Furthermore, when secondary stresses are directly superimposed over the primary stresses in steel members, fatigue cracking occurs quite early.

These secondary stresses are not calculated in normal design and rating procedures; thus fatigue provisions have not been applied to this type of cracking. The effect of out-of-plane distortion on a few structural details is presented with the review of case study results.

FLOOR BEAM CONNECTION PLATES

Extensive cracking has developed in girder web plates at the ends of floor beam connection plates. Results of studies indicate the cracking to be the direct result of cyclic, secondary bending stresses that are generated through relative out-of-plane movement. These displacements, though often less than 0.005 in. (0.130 mm), concentrate in gaps between the floor beam connection plate and the girder tension flange. Figure 1 shows the crack developed in a web gap at a floor beam-to-girder connection plate in the girder's negative moment region. As is common practice, the connection plate is cut short of the tension flange. The gap containing the crack is bounded by the web-to-flange and connection plate-to-web fillet welds.

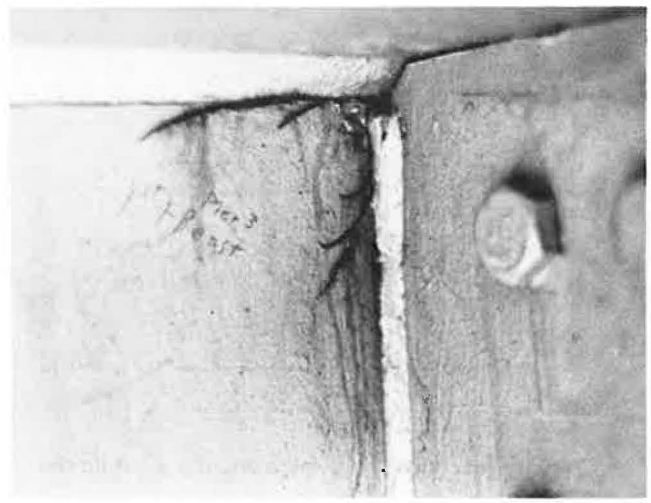

FIGURE 1 Typical cracks in web at floor beam-to-girder connection.

This mode of fatigue cracking was found at numerous gaps on the web of main girders on the Woodrow Wilson Bridge *(1)*. The structure consists of 19 continuous multiple-girder spans ranging from 62 ft (18.9 m) to 212 ft (64.6 m) in length. Floor beams are 25.7 ft (7.8 m) and 36 in. (914 mm) deep. No positive attachment was made between either flange and the connection plate. The deck slab was cast directly on the outside main girders and supported by stringers and floorbeams in the interior. Thus, the deck restrained movement of the top flange of the main girders and, as the transverse floor beam deformed, caused out-of-plane distortion of the web within the web gap. The stress gradients from measured strains next to the gap are shown in Figure 2. The gradient shows that double-curvature web plate deflection exists in the short gap length. When extrapolated, a stress of 10.6 ksi (68.9 MPa) is expected at the longitudinal weld toe at the top. Within the gap, the surface stress at the weld toe can be two to three times greater than those recorded just outside the local region, as calculated by finite element analysis.

The stress range spectrum for the first gauge at the gap is shown in Figure 3. Also shown are the maximum and effective stress ranges at the weld toe, by adjusted extrapolation. Results of case studies indicate that the maximum stress range often

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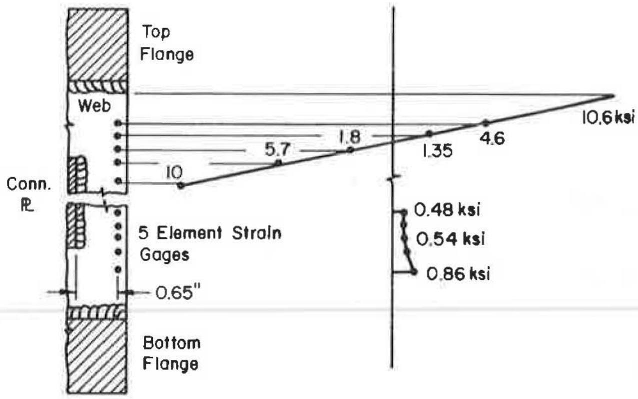

FIGURE 2 Measured and extrapolated stress gradient on web surface.

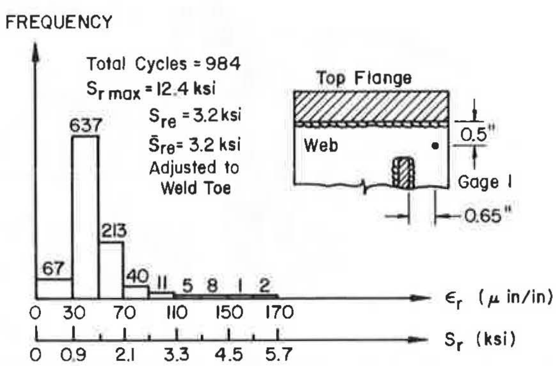

FIGURE 3 Stress range spectrum of a point on the web.

FIGURE 4 Cracking at base of bearing stiffener and adjacent transverse stiffener.

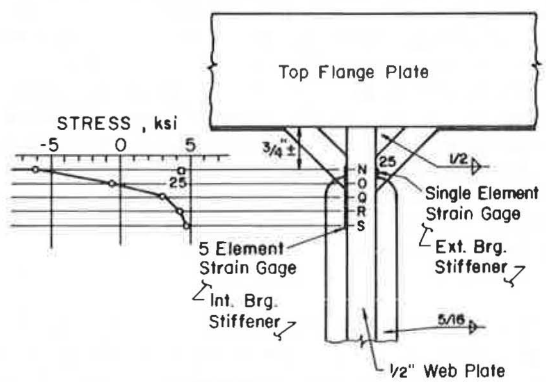

FIGURE 5 Typical stress gradient of web at bearing stiffener gap.

exceeds 12 ksi (82.7 MPa) and a Miner's effective stress range of 3 ksi (20.7 MPa) is quite common. These out-of-plane deflection-induced stress ranges are high enough for fatigue cracking in the web plate.

Similar cracking prevailed in main girder webs at the floor beam connection plates of the Poplar Street Bridge *(2)*. The multiple girder structure has six straight or curved continuous spans of 75 to 100 ft (22.9 to 30.5 m) length. In the negative-moment region, the floor beam connection plates are attached to the compression flange. The corresponding stiffener on the exterior face of the girder is only fitted to the compression flange and cut 5/8 in. (16.0 mm) short of the tension flange. The floor beam attachment in the positive moment region of the girder is similar except that the bottom flange is in tension. At piers, floor beams are connected directly to bearing stiffeners, which are tightly fitted to both top and bottom flanges and welded only to the girder web. Cracks developed in the web at each of these three types of connection plates.

Typical cracking at the base of a bearing stiffener and within the gap of an adjacent transverse stiffener is shown in Figure 4. Nine bearing stiffener-to-floor beam connections were monitored. Cracking prevailed at the toe of the longitudinal flange-to-web weld and through the throats of the transverse connection plate-to-web welds. Representative stresses at the top flange of one connection are plotted in Figure 5. The back-to-back gauges "N" and "25" had stresses of –6.0 ksi (–41.4 MPa) and 4.2 ksi (28.9 MPa), respectively. These magnitudes

TABLE 1 RESULTS OF MEASUREMENTS AT BEARING STIFFENER, POPLAR STREET BRIDGE

Maximum Out-of-Plane Displacement		Web Stresses			
		Random Trucks		Maximum	
(in.)	(mm)	(ksi)	(MPa)	(ksi)	(MPa)
>0.001	>0.025	9.8–13.0	67.6–89.6	14.0	96.5
0.001	0.025	5.6–8.2	38.6–56.5	10.5	72.4
0.0005	0.013	4.2–5.1	28.9–35.2	6.50	44.8
0.001	0.025	2.6–4.2	17.9–28.9	4.90	33.8
>0.0005	>0.013	1.0	6.90	1.40	9.70

and the stress gradient indicate that the web was undergoing out-of-plane bending. The maximum measured out-of-plane displacement was approximately 0.001 in. (0.0254 mm) within this region.

For the case of Figure 5, the projected stress was –14.0 ksi (–96.5 MPa) at the longitudinal weld toe. A list of selected results from field measurements at some bearing stiffeners is presented in Table 1. The web stresses have been adjusted to the flange-to-web weld toe by linear extrapolation.

Cracking also developed in the web at the connection plates of floor beams. Typical web behavior is best represented by the

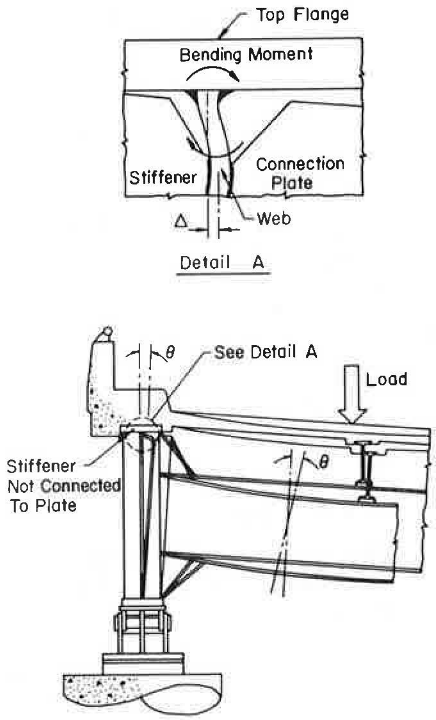

FIGURE 6 Representation of floor beam end rotation and web behavior.

TABLE 2 RESULTS OF MEASUREMENTS AT FLOOR BEAM CONNECTIONS, POPLAR STREET BRIDGE

Crack Length		Maximum Out-of-Plane Displacement	
(in.)	(mm)	(in.)	(mm)
7.0	177.8	0.043	1.09
6.0	152.4	0.030	0.76
4.5	114.3	0.025	0.64
3.5	88.9	0.020	0.51
3.25	82.6	0.021	0.53
3.0	76.2	0.021	0.53
2.75	69.9	0.025	0.64
2.5	63.5	0.008	0.20
1.875	47.6	0.024	0.61
1.75	44.5	0.022	0.56
1.5	38.1	0.028	0.71
Uncracked		0.001	0.03
Uncracked		>0.001	>0.03
Uncracked		0.000	0.00

schematic in Figure 6 for the negative moment region of the girders. The connecting plates are tightly fit to, or cut short of, the top (tension) flange. Floor beam end rotations introduce out-of-plane displacement in the web of girders and cause cracking of the web plate. A listing of crack length and the maximum out-of-plane displacement measured under normal traffic loads is presented in Table 2. There appears to be a direct relationship between the displacement and the length of the crack. The displacements at the 1/2-in. (12.7-mm) gaps with cracks were 10 to 20 times larger than those at the bearing stiffeners.

Review of field data indicates that a displacement of about 0.025 in. (0.64 mm) and a surface stress on the order of 20 ksi (138 MPa) at the flange-to-web weld toe are typical of floor beam connection plate gaps 1/4 to 3/4 in. (6.4 to 19.1 mm) long. Connection plates tightly fit to the tension flange have smaller out-of-plane displacements but the plate-bending stresses could still be relatively high. Stresses of 22 ksi (151.7 MPa) with a displacement of only 0.008 in. (0.20 mm) have been recorded at such sites.

MULTIPLE GIRDER DIAPHRAGM CONNECTION PLATES

A similar condition of cracking exists in the gap at the junction of longitudinal girder flanges and the transverse connection plates of diaphragms or cross-bracing in multigirder bridges. As the girders deflect unevenly under load, the diaphragms transfer lateral forces that cause the connection plate to displace and rotate. Such movement results in out-of-plane distortion of the web plate and the development of fatigue cracks in the gap.

Cracks in the web plate at the gap develop along the toe of the flange-to-web weld and at the weld across the ends of the transverse connection plate in both the positive- and negative-moment regions of girders. Figure 7 shows a crack in the positive-moment region of a simple-span structure. Figure 8 shows cracks at the stiffener weld and at the web-to-flange weld at a diaphragm connection plate in the negative-moment region of the continuous-span Beaver Creek Bridge *(3)*.

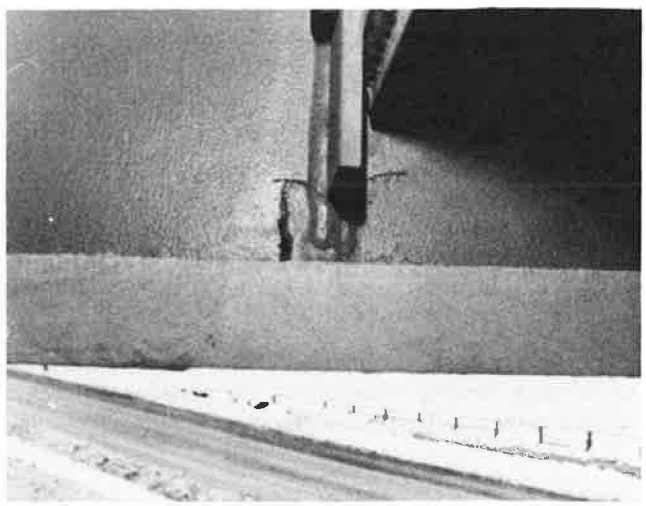

FIGURE 7 Cracking across transverse welds at diaphragm connection plate in the positive-moment region.

This type of fatigue cracking in welded multigirder bridges was detected in the I–79 Bridge 2680 over Big Sandy Creek *(4)*. The bridge has two 108-ft (32.9-m) side spans and two 130-ft (39.6-m) interior spans, with six continuous girders of 5 ft (1.5 m) depth. The diaphragms are equally spaced and the configurations are shown in Figure 9. Small cracks were found in the web along both the flange-to-web weld and the connection plate weld in many diaphragm connection plate gaps. The gap lengths ranged from 1/2 to 5/8 in. (12.8 to 15.9 mm). The cracks were parallel to the primary stresses in the web plate. The maximum measured vertical stress on the web surface was 20 ksi (138 MPa). At all locations, measured strains revealed

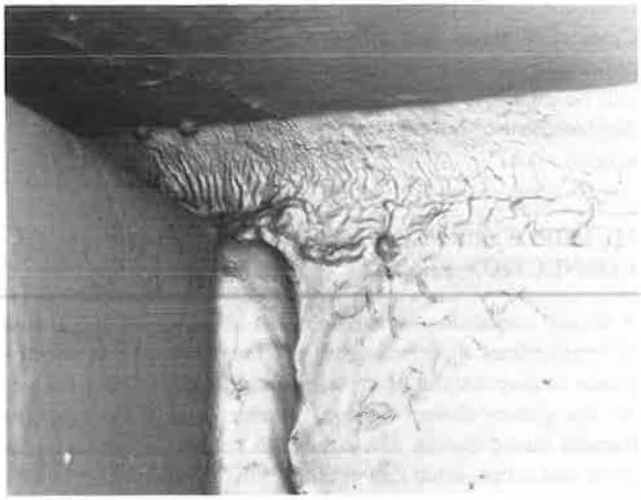

FIGURE 8 Cracking at the diaphragm connection plate gap in the negative-moment region.

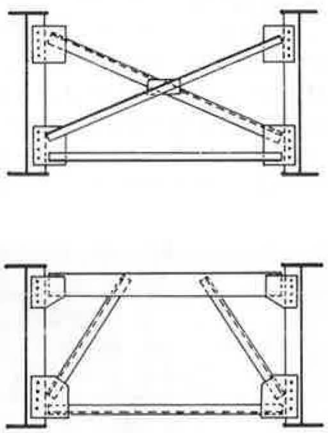

FIGURE 9 Diaphragm connection details for Bridge 2680.

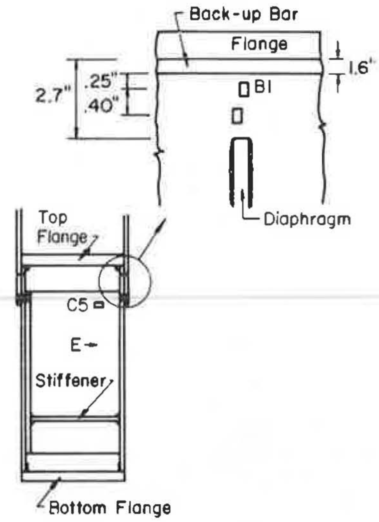

FIGURE 10 Typical box section and expanded detail, I–79 box girder.

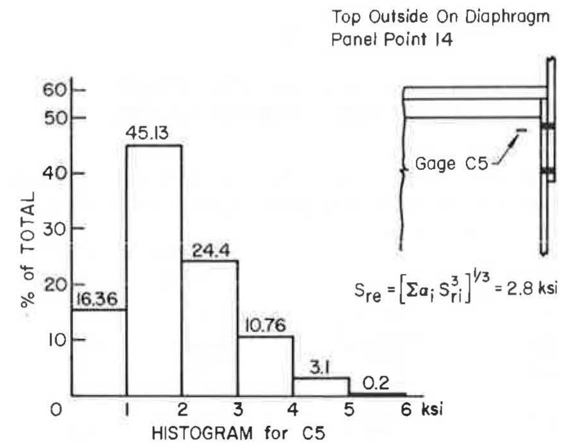

FIGURE 11 Stress range histogram of box diaphragm.

that the diaphragm was distorting the web and inducing cyclic, out-of-plane bending stresses with the passage of live load, leading to the development of cracks.

BOX MEMBER DIAPHRAGMS

Cracks have been detected in web plates of box girders at gaps of internal diaphragms opposite floor beams attached to the box girder. When the diaphragm plate is not connected to the top or bottom flanges of the box, unstiffened web portions or gaps are formed. Furthermore, backup bars are often used at the junction of a web plate and the flange. Lack of fusion at the backup bar is a common phenomenon and constitutes a discontinuity at the weld. The combination of out-of-plane displacement and rotation within the gap, and the existence of relatively large initial flaws can cause fatigue cracking in the gap.

This type of distortion-induced cracking has developed in the 750-ft (228.6-m) box girder of the tie arch structure at Neville Island *(5)*. Cracks were found to exist in the welded connections between diaphragm plates and the outside web of the 42- × 150-in. (1,067- × 3,810-mm) box. A typical box section and expanded detail are shown in Figure 10. The strain-time data from the strain gauges show that major stress cycles occur when trucks pass over the 8-ft (2.44-m) floor beam. The diaphragm and web plate in all four gaps are subjected to high-magnitude stress cycles that include stress reversal.

A stress range histogram from a gauge on the diaphragm is shown in Figure 11. The maximum range of stress is 5.6 ksi (38.6 MPa) and the equivalent constant-amplitude stress range is 2.8 ksi (19.3 MPa). The stress range histogram from a gauge on the web is given in Figure 12. The maximum stress range within such gaps varies between 8 and 11 ksi (55.2 to 75.8 MPa), with effective stress ranges of 3 to 4 ksi (20.7 to 27.6 MPa). These stresses cause cracking in the web on both inner and outer surfaces of the web plates, similar to the condition at floor beam connection plates with out-of-plane displacements.

Out-of-plane displacement generates a stress gradient within the gap. Stress gradients at the instant of maximum and minimum stress in gauge B1 during a high-magnitude stress cycle are constructed in Figure 13. The difference between the maximum and minimum is the stress range for this cycle of

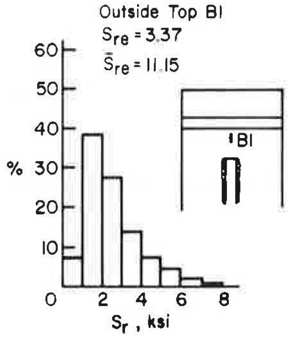

FIGURE 12 Stress range histogram of box girder web.

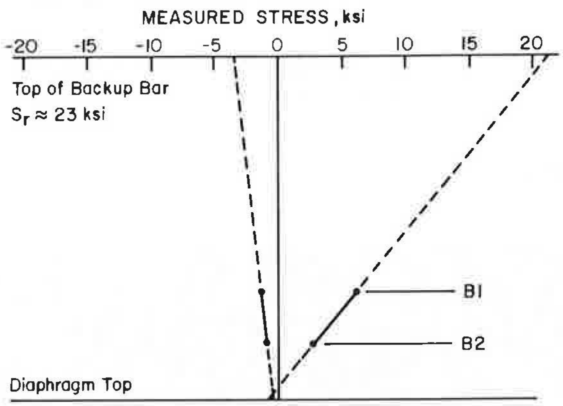

FIGURE 13 Stress gradients at top outside gap of box.

instantaneous stress excursion. Extrapolation (dashed lines) to the backup bar root shows that stress ranges could be on the order of 23 ksi (159 MPa). Such a magnitude of live-load stress can cause cracking at the root of the backup bar.

Throughout the review of the cases involving box distortion and subsequent fatigue cracking, double curvature has always been detected in the web plate at the gaps. This curvature and the corresponding cyclic bending stresses were induced by the restraint between floor beams and the tie-girder with its diaphragms. At each floor beam connection, the distortion of the box girder rendered all four gaps at the corners susceptible to fatigue cracking. When lack of fusion areas existed behind the backup bar and served as notches perpendicular to the direction of stresses, development of cracks occurred with a low number of stress cycles. Furthermore, when the crack propagation changes direction in accordance with the stress field around the diaphragm-to-web connection, a crack perpendicular to the primary tension of the tie box could result. Early detection of these cracks is therefore of paramount importance.

LATERAL BRACING GUSSET PLATES

Lateral bracing is often placed between adjacent girders at a level above the bottom flanges. The attachment is made through gusset plates connected to the web plates. Thus, any force or displacement of the bracing members causes out-of-plane movements and corresponding stresses in the web.

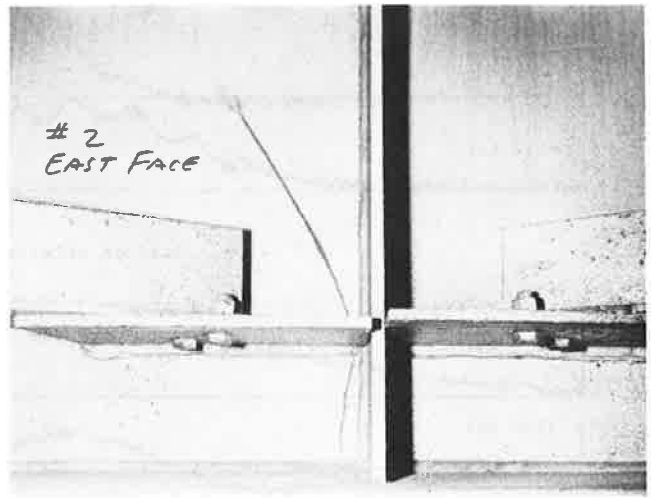

FIGURE 14 Typical crack growth from gusset plate gap.

The gusset plate may be welded, bolted, or cut free of the transverse stiffener in addition to being connected to the web. Figure 14 shows one case of two separate gusset plates welded to the web only. The gap bounded by the weld toe of the transverse stiffener and the near end of the gusset plate is less than 1/4 in. (6.4 mm) in length. The out-of-plane displacement and cyclic bending stresses have caused the crack development and growth.

Cracking in web plates at the lateral gusset plate gaps has developed in I–79 Bridge 2682 over Big Sandy Creek *(4)*. The twin bridge is supported by two main steel girders with four stringers resting on transverse floor beams. The continuous girders have three spans of 120, 130, and 120 ft (36.6, 39.6, and 36.6 m). The lateral bracing members at a floor beam location are bolted to a gusset plate welded to the girder web, but not to the floor beam connection plate, forming gaps such as that shown in Figure 15. The gaps vary from 1/4 to 1 in. (6.4 to 25.4 mm) in length. The prevailing condition has resulted in the formation of vertical cracks on the outside surface of the web plate along weld toes of the transverse stiffener.

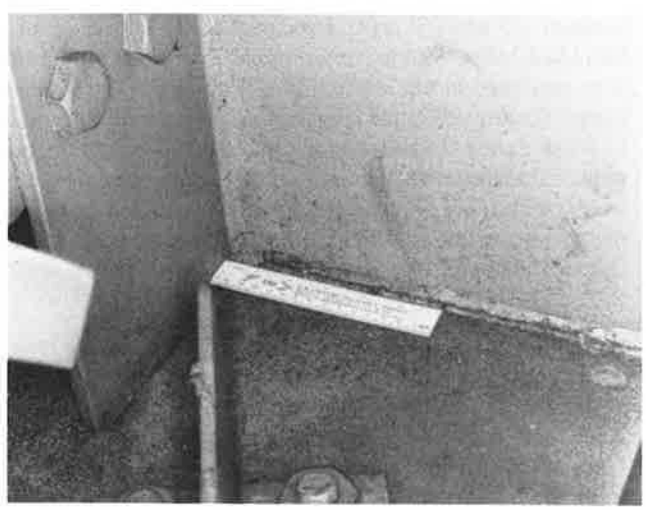

FIGURE 15 I–79 Bridge 2682 lateral gusset plate connection detail.

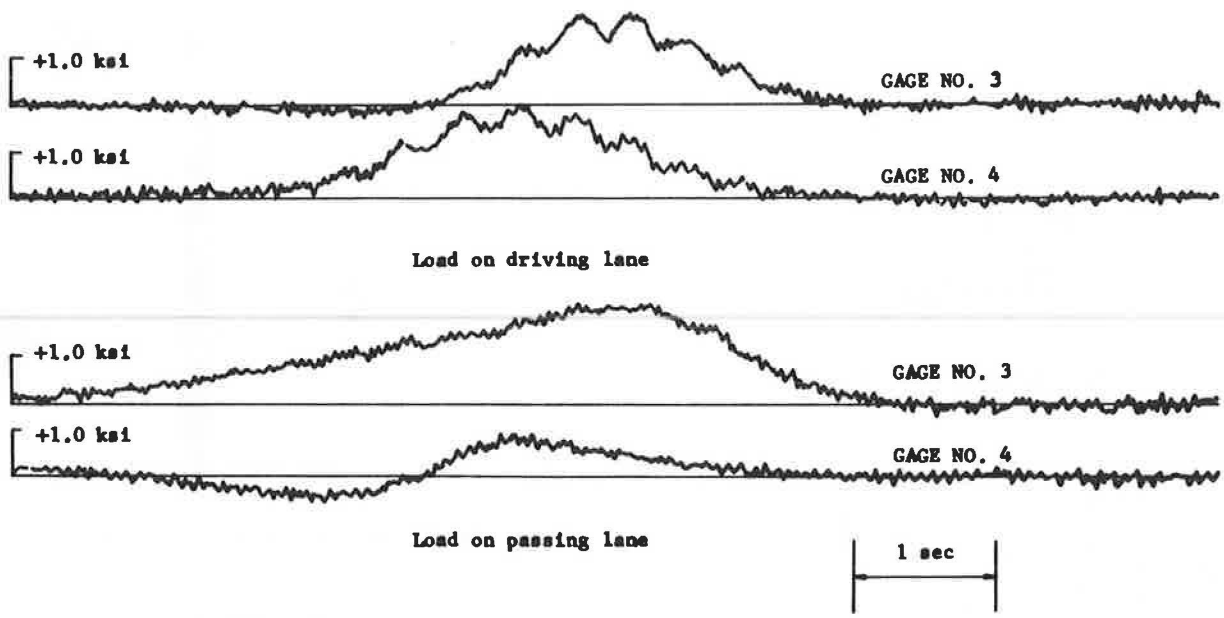

FIGURE 16 Time-dependent strain response of the lateral bracing.

Strain measurements indicated a maximum flexural bending stress range of 3 to 5 ksi (20.7 to 34.5 MPa) in girder flanges and a maximum stress range of 2 to 3 ksi (13.8 to 20.7 MPa) in the lateral bracing members framing into a gusset plate. Figure 16 shows the time response of the two laterals as a truck crossed the bridge. The time lag in the development of tensile stresses and the difference in magnitudes imply rotation of the lateral connection plate in addition to transverse displacement.

Strains were measured at the weld toe of a transverse connection plate in the gusset plate gap and at the weld toes of the outside (facia) stiffener. Stress ranges of 8 to 12 ksi (55.2 to 82.7 MPa) were obtained at the weld toe on the outer surface of the web directly opposite the gusset plate gap. The stress range at the transverse weld in the gap was revealed to be closer to 7 ksi (48.3 MPa). Maximum stresses of 9 to 15 ksi (62.1 to 103.4 MPa) were induced at the transverse weld in the gap.

Similar cracking was investigated on the Canoe Creek Bridge *(6)*. This six-span structure has five spans with lengths between 135 and 162 ft (41.1 and 49.4 m). Stress ranges of 18 ksi (124.2 MPa), with maximum stresses of 12 ksi (42.8 MPa), were measured in the 1½-in. (38.1-mm) gaps at lateral gusset plates. The stresses in the laterals were of opposite sign and out of phase during the passage of vehicles. This condition again implies that the lateral connection plate was forced to rotate as well as deflect out of plane. Such rotations reduced the stress at one weld toe, but elevated the stress at another and induced cracking thereon.

INTERMEDIATE TRANSVERSE STIFFENERS ADJACENT TO FIELD SPLICES

Numerous cases of cracking at intermediate transverse stiffeners of girder webs have been reported with the causes traced to the transportation of the girders. One such crack that was investigated is shown in Figure 17. The crack initiated at the termination of the transverse stiffener welds and extended along the web-to-flange weld parallel to the longitudinal axis of the girder. Because of the comparatively slender and flexible web, there may often be a relative movement between the top and bottom flanges during transportation. The support points of girder segments during transportation are usually directly under intermediate transverse stiffeners. At these locations, the lateral movement is accommodated by the flexible gap between the stiffener and flange. Hence, the out-of-plane lateral deformation and corresponding stresses are again introduced in the gap. This condition has led to cracking at the end of transverse stiffener welds and at the web-to-flange fillet weld. These cracks are often unnoticed until an in-service inspection is performed.

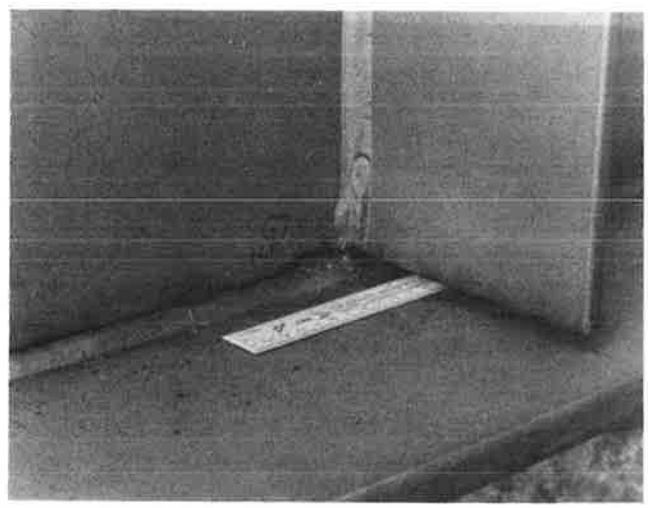

FIGURE 17 Cracking at a transverse stiffener web gap adjacent to a field splice.

This type of cracking has been found in I–79 Bridges 2680 and 2682 over Big Sandy Creek *(4)*. The intermediate stiffeners with cracks are not serving as floor beam or diaphragm

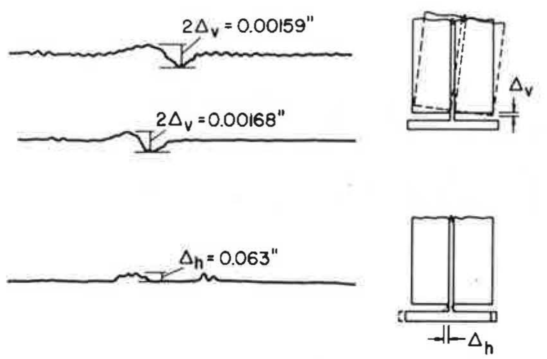

FIGURE 18 Displacement-time response of gaps at stiffener adjacent to a field splice.

connection plates but are adjacent to bolted field splices. Careful inspection of the crack surfaces showed the presence of both the red primer paint and the final finish coat, indicating that the cracks existed at the time of application of the primer coat. Continued propagation of the crack along the web-to-flange weld is considered to be the result of lateral movement during transportation, with little in-service extension.

Two cracked locations were monitored during normal random truck traffic. Figure 18 shows the displacement-time responses. At one location, the presence of a long horizontal crack allowed a relative horizontal displacement to occur between the flange and stiffener. The rotation of the stiffener with respect to the flange below the gap is minimal. The other location had only a short crack. Truck loads induced relative rotation between the stiffener and the flange with little relative horizontal movement. The distortions were compatible with the small out-of-plane bending stresses measured on the web at the gap. At the ends of these cracks, the in-service, out-of-plane cyclic stresses were always less than 1 ksi (6.9 MPa). Such stress levels should not produce a threat to the life of the structure.

SUMMARY AND CONCLUSIONS

Numerous common bridge details have been noted as susceptible to out-of-plane distortion-induced fatigue cracking. Though the details serve different functions in the overall behavior of the structure, a single feature is common to all. This feature is a short length of unstiffened web plate at a gap between flanges, connection plates, stiffeners, or gusset plates. When lateral forces or displacements are induced by bridge components at or nearby the gap, this length of web plate absorbs the majority of out-of-plane displacements. The local distortion in a gap is often in the form of double curvature and results in high secondary bending stresses that are not accounted for in present design procedures. Fatigue cracks could develop.

Cracking in the gap usually extends along the weld toes and perpendicular to maximum stresses. In the cases where the in-plane flexural bending stresses of the web plate are in the same direction as the out-of-plane secondary stresses, the growth of the crack could be relatively rapid. Thus, cracks in horizontal gaps at lateral gusset plate connections could pose a serious threat to the serviceability of the bridge, as vertical cracks could propagate toward the tension flange.

It has been shown that distortion in gaps can be reduced by positive attachment of component parts. The review of available measurements indicates that a positive connection should be made between the girder flanges and the transverse connection plates for diaphragms and floor beams. This requirement has been incorporated into design specifications. Furthermore, a study is in progress to develop schemes for connections of lateral bracing members and transverse stiffeners.

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