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DISTORTION INDUCED STRESSES IN A FLOORBEAM-GIRDER BRIDGE: CANOE CREEK

by

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Two details in a girder-floorbeam bridge are investigated for web distortion in a gap. One is the cut-short floorbeam connection plate, the other is the lateral wind bracing gusset plate connection to girder web and floorbeam. The conditions and stress distribution at these small gap regions were examined through field inspection, stress measurements and evaluation by finite element method. Metallographic and fractographic examination of the cores removed from the cracked details was also carried out.

Distortion cracking in webs was found to occur at the top end of connection plates for floorbeams adjacent to the piers, at the horizontal gaps between the lateral bracing gusset plates and floorbeam connection plates, and less definitive at the vertical gap between the floorbeam connection plate and the bottom flange. Computed stresses at these locations were in good agreement with measured values. Stresses in the gaps were high enough to cause fatigue cracks. Recommended retrofitting schemes include attaching of floorbeam connection plates to the top flange of girders, increasing of gap length between lateral gusset plate and floorbeam connection plates, and drilling of holes at ends of vertical cracks along transverse stiffeners.
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Department of Transportation

Office of Research and Special Studies

Project 83-21: The Causes of Deformation Induced Cracking in Steel Bridges and Methods to Retrofit the Damage

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Lehigh University

Office of Research

Bethlehem, Pennsylvania

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1. INTRODUCTION

1.1 Background

In recent years, there has been an increase in the number of steel bridges which have developed cracking due to displacement-induced fatigue. Welding generally leads to a joint with higher restraint stresses than those seen in bolted or riveted connections because smaller gaps result from welded details. While more refined analytical procedures are being used on bridge systems, those used to analyze connections have not changed greatly. Many of the steel structures developing cracking from displacement-induced stresses have been in service only for short periods of time. In an extreme case, cracking was seen to develop even before the bridge was open to normal traffic. A combination of construction traffic and the aerodynamic response of the structure was enough to initiate cracking\(^1\).

In the design process, only the in-plane behavior of the bridge members is generally considered\(^2\). As a result, the interaction of the primary and secondary members is often not adequately examined. The primary cause of the fatigue cracks are the high secondary bending stresses which result from the out-of-plane displacements. These out-of-plane displacements occur because of the three-dimensional behavior of the bridge structure.

Many details which are susceptible to displacement-induced fatigue cracking have been identified\(^3\). In general, any detail which leaves small segments of the web plate unstiffened, is a candidate for early fatigue cracking. These unstiffened gaps in web plates have resulted from the past practices of detailing and fabrication and from acceptance of a rule of
thumb that it was bad practice to make transverse welds on the tension flange of bridge girders. Weldments on the tension flange represents a potential location for cracking to develop in the flange.

It seems probable that the adoption of this rule evolved from early experience with welded bridges in Europe during the 1930's. Fractures developed at transverse welded details on a number of these early structures. It is likely that low toughness material and difficulties with weldability resulted in large initial defects and brittle fracture. To avoid this problem, weldments were not placed on tension flanges.

During the last twenty years, studies on fatigue and fracture have demonstrated that weldments on the tension flange are not any more severe than welded attachments to the web. Proper design and recognition of the detail severity accounts for the strength of the welded detail placed on flange or web. In addition, mandatory notch toughness requirements have eliminated steels which are susceptible to brittle fracture originating from small defects and weld toe stress concentrations.

In general, connections of riveted systems are more flexible and can more easily accommodate the out-of-plane distortions that develop in complex structures.

This report provides an evaluation of the Canoe Creek Bridge located on Interstate Highway I-80 in Clarion County, Pennsylvania. These structures are dual two lane bridges in the eastbound and westbound directions. Figure 1 shows a view of part of the two girder - floor beam structure. The studies were carried out on the westbound bridge. Two details are investigated for web distortion in a gap. One is the cut-short floor beam connection plate, as illustrated in Fig. 2. As vehicles traveling on the bridge cross over a floor beam, end rotation occurs. This rotation tends to
pull or push the small unstiffened portions of web at the ends of the connection plate out-of-plane with respect to the rest of the girder. In addition to the end rotation of the floor beam, the differential pulling of the laterals also influences the out-of-plane movement of the web gap. As a result, high secondary bending stresses are introduced into these small gaps, and fatigue cracking will occur in a relatively low number of stress cycles.

Another detail that was investigated is the lateral wind bracing gusset plate and its intersection with the floor beam. On the Canoe Creek Bridge, the gusset plates are framed around the transverse floor beam connection plates, as illustrated in Fig. 3. During the field studies and inspection of this structure in October 1984, small cracks were unexpectedly discovered along the weld toes of the transverse stiffeners and the gusset plate connection plate intersections. As a result, strain gages were installed on the web in the gaps of these connections and a detailed analysis was undertaken on the complex connection details.
2. DESCRIPTION OF THE STRUCTURE AND FIELD STUDY

2.1 Description of the Structure

The Canoe Creek Bridge is located on Interstate 80 in Clarion County, Pennsylvania. The structure consists of two separate bridges, one supporting eastbound traffic and the other, westbound traffic. Both structures are identical in geometry and are heavily traveled by trucks, as Interstate 80 provides a major link between the eastern and western halves of Pennsylvania.

Built in the 1960's, each bridge is a twin girder - floor beam type structure consisting of five continuous spans and a simply supported multi-girder end span. The continuous portion of the structure consists of two side spans of 135 ft. (41.45 m) each and three center spans of 162 ft. (49.38 m) each. The continuous girders are haunched over the piers and vary in depth from 8 ft. (2.44 m) in the constant depth region to 14 ft. (4.27 m) over the piers. The haunch varies as a circular arc over a 50 ft. (15.24 m) horizontal length on either side of the piers. Each haunch has a centerline radius of 250 ft. (76.2 m). Figure 4 shows the plan and elevation of one of the girders.

The two longitudinal girders are welded plate girders with flanges that vary in cross-sectional area over the length of the spans. The largest flange plate has a cross-sectional area of 38.5 in.$^2$ (248.4 cm$^2$), while the smallest is 22.0 in.$^2$ (141.9 cm$^2$). The web plates vary in thickness along the bridge's length. For a distance of 20 ft. (6.1 m) to either side of an interior pier, the web is 0.5 in. (12.7 mm) thick. The remainder of the webs are 0.375 in. (9.5 mm) thick.
Floor beams between the girders are welded built-up flexural members, as illustrated in Fig. 5. The two end spans of the bridge have a floor beam spacing of 23.5 ft. (6.858 m), whereas in the center spans this spacing is either 23.33 ft. (7.087 m) or 23 ft. (7.01 m). The laterals (ST7WF39) are connected to the floor beam as well as the girder through two gusset plates 0.375 in. (9.5 mm) thick. Their arrangement is depicted in Fig. 3.

As shown in Fig. 4, the bearings at two piers supporting the center span are fixed against expansion. The rocker-type bearings at other supports allow longitudinal expansion.

All steel in the structure is ASTM A36 mild carbon steel. The deck is of reinforced concrete and is supported by stringers (W21X55) and the two longitudinal girders. The bridges are not designed for composite construction. No shear studs were used, but the flanges of both longitudinal girders and stringers are cast into the concrete deck. Composite action under live loads takes place by friction in the longitudinal direction and by positive restraint in the transverse direction between the deck and the girders and stringers.

The design stress ranges at FB19 and FB30 where strain gages were mounted on bottom flange are 9.3 ksi and 11.2 ksi (64.12 MPa and 77.22 MPa), respectively. These will be compared with the field measurements later.

2.2 **Summary of Field Examination**

The examination of the westbound bridge was carried out in early October, 1984. Evidence of fatigue cracking was discovered in four locations in the web: in the vertical gap at the bottom end of the floor beam connection plates, in the horizontal gap between the lateral bracing...
connection plates and floor beam connection plates, in the vertical gap at
the top end of floor beam connection plates in the negative moment region,
and at the ends of the lateral bracing connection plate tabs welded to the
girder web.

The vertical floor beam connection plates in the positive moment region
are not attached to the bottom flange. Hence, a vertical gap exists between
the web-flange junction and the end of the connection plate, as illustrated
in Fig. 6. As a result of floor beam rotation and differential pulling of
the laterals, the small gap is pushed and pulled out-of-plane with respect
to the remaining portion of the girder web. These distortions may result in
secondary bending stresses at the weld toes in the gap. A large number of
these gaps exhibited crack indications in the paint, as illustrated in Fig.
6. These cracks generally form in the direction of the girders, parallel to
the primary bending stresses. Therefore, these cracks are not initially
critical to the performance of the structure at the time of their
development. However, as cracks grow out of the gap region, they may be
influenced by the shear forces in the web. The cracks will turn, following
a path perpendicular to the principal stresses.

The Canoe Creek Bridge was selected for this study, in part, because it
was known that extensive cracks had developed at the top of the floor beam
connection plates in the negative moment region. Figure 7 shows the
cracking that was typical at each floor beam in the negative moment regions
except those at the piers. Retrofit holes had been installed in 1983
shortly after the cracks were detected. However, a number of these cracks
had reinitiated, and additional holes were drilled to arrest or retard
crack growth, as illustrated in Figs. 7 and 8.
No cracks were detected at the piers where double floor beams existed and three large bearing stiffeners were welded to the web and the bottom flange.

Inspection of the lateral bracing gusset plates revealed three types of fatigue crack indications. The first occurred at the end of the gusset plate tabs at the weld toe of the tab, as shown in Fig. 9. The second occurred in the small horizontal gaps between the vertical floor beam connection plate and the lateral gusset plate tabs. Figure 10a shows a view of a web gap between the transverse connection plate and the welded gusset plate tab used to attach the gussets to the girder web. A large number of these gaps were observed to have crack-like indications in the paint film at the weld toe at the end of the plate tab, as illustrated in Fig. 10b. The third type of crack indication was related to the horizontal web gap, but occurred on the outside surface of the girder web along the vertical stiffener, as shown in Fig. 11. These crack indications were observed on each side of the vertical stiffener at the level of the lateral gusset plates. They were observed at every floor beam location examined in both negative and positive moment regions.

The existence of the second and third types of cracks was unexpected. The lateral gusset plates are bolted to the bottom flange of the floor beam as well as the two horizontal connection plate tabs which are welded to the girder web, as illustrated in Fig. 3. This type of joint has a high degree of restraint, and no evidence of slip was detected in the bolted joints. The out-of-plane movement necessary to cause the cracks was not thought likely to develop at this type of connection. Cracking has been observed in bridges where lateral bracing gussets were attached to the girder web but not to the transverse connection plates\(^4\). No adverse experience has been
reported when a positive attachment was provided, between the gussets and the transverse connection plates.

2.3 Instrumentation and Experimental Procedures

Thirty-nine electrical resistance strain gages were mounted at five cross-sections on the north girder. These locations are identified as floor beams 19, 21, 22, 23 and 30 in Fig. 4. Details of the gage locations are summarized in Figs. 12, 13 and 14 and Table 1.

At FB19 (see Fig. 12) gages were installed on a vertical section of the girder in order to obtain the stress gradient due to in-plane bending. These are identified as gages 32, 40, 41 and 42. Gages were also installed on the two lateral members framing into the lateral gusset plates at this cross-section. Strip gages were installed in the vertical web gap at the bottom of the transverse connection plate (gages 27 to 31) and on the outside web surface, next to the stiffener and opposite the horizontal gap between the transverse connection plate and lateral connection plate (gages 33 to 38). Additional single element gages were installed at the end of the lateral connection plate (gage 24) and on the web plate in the horizontal gap region (gage 39).

At pier 3 (FB21), a linear gage was mounted near the vertical web gap between the end of the bearing stiffener and the top flange-to-web weld (gage 21), as illustrated in Fig. 13. No other gages were installed as there was no evidence of movement or distress at any pier.

Three gages were located next to drilled holes at FB22 where extensive cracking existed.
Figure 14 shows the gage placement at FB23 and FB30. Strip gages were installed on the outside web surface at the transverse stiffeners and opposite the horizontal gap at the lateral gusset plate tab on the east side of the both floor beam locations. In addition, three linear gages were installed at FB30, one on the top surface of the bottom flange and the other two on the outside web surface at each end of the west lateral gusset plate tab.

The strain gages were connected to oscillograph recorders which were obtained from FHWA and provided analog traces of the strain variation on light-sensitive paper. In addition, several measurements were obtained on a few gages using a magnetic tape recorder.

2.4 Loading

Strain readings were acquired under random truck traffic and under a "test vehicle" which was supplied by District 10-0 of the Pennsylvania Department of Transportation. The test truck had five axles and a gross weight of 94.6 kips (420.8 kN). Figure 15 shows the axle spacing and weight of the test truck and a view of the truck crossing the bridge in the outside lane.

The test truck runs consisted of both crawl and speed runs in both westbound lanes. The crawl runs were at 5 mph (8 km/h) and minimized the dynamic response of the structure. The speed runs were carried out at about 55 mph (90 km/h).
2.5 Summary and Discussion of the Test Results

The most comprehensive instrumentation and measurements were made at FB19 (see Figs. 4 and 12). This floor beam is located in the center span adjacent to the dead load inflection point and was selected for the focus of the computer analysis.

Figures 16 to 21 show typical strain-time responses of strain gages in the various gaps (or unstiffened web plate segments), on the lateral bracing members and at the end of the lateral gusset plate tabs. The responses indicate that passage of a single truck in either lane results in very high magnitude stress cycles in the horizontal gaps of the lateral gusset plate tab (see gage 33 in Fig. 16). The cyclic stress in the vertical gap between the end of the transverse connection plate and the bottom flange was much less than in the horizontal gusset plate gap (see gage 27 in Fig. 16).

Tables 2 and 3 summarize the measured maximum test truck load stress and stress range at each gage location for the slow speed crawl run (static response) and the fast speed run (dynamic response). The results are shown for the test vehicle in the driving lane and in the passing lane. The stress range values are listed in the parenthesis. The largest stress range occurred in the web at gusset plate gap at floor beam 30 (see Table 2). Floor beam 30 is also located adjacent to a dead load inflection point.

At the pier, no significant stress due to distortion was measured at the vertical gap at the top of the connection plate (see Fig. 17). This verified that no cracking should develop, and there was no evidence of cracks or movement by field examination.
The extensive cracking observed in the web at the top gap of the transverse connection plate of floor beam 22 (Figs. 7 and 8) had demonstrated that the fatigue cracks were reinitiating at the drilled holes along the connection plate and web-flange weld toes. The response from gages installed near these holes are plotted in Figs. 18 and 19. It can be seen that large cyclic stresses are introduced at these retrofit holes and that crack growth will occur at the additional holes. Large secondary cycles can be seen resulting from vibrations.

Figures 20 and 21 show the response of the laterals, the bottom flange of the main girder girder and the inside and outside surfaces of the web at the end of the welded lateral gusset plate tab at floor beam 19. The test results show that the laterals introduce an out-of-plane distortion of the web plate as a vehicle crosses the span. This can also be seen in Fig. 22 where the stress gradients about 2 ft. (0.61 m) west of floor beam 19 are plotted for the instant when maximum strain response occurred in the bottom flange with the load in the driving and passing lanes. The in-plane bending gradient is nearly linear when the average stress from gages 24 and 40 is used. The individual gage readings show that the out-of-plane bending is large at the level of the lateral connection plate. This occurs for loads in either lane. It can be seen in Figs. 20 and 21 that the stresses in the laterals were either of the same sign, indicating tension in both members, or of opposite signs and are out-of-phase during passage of the vehicle. This latter condition implies that the lateral connection plate was forced to rotate out-of-plane. At the gaged west end of the plate tab, this increased the stress range at the weld toe on the inside surface of the web and decreased the stress range on the outside surface.
The measured stress ranges on the bottom of flange at FB19 and FB30 are shown in Tables 2 and 3. The averaged stress range for load in driving lane and passing lane is 2.9 ksi (19.99 MPa) for FB19 and is 3.6 ksi (24.82 MPa) for FB30. These stress ranges can be compared with the design stress ranges which are 9.3 ksi (64.12 MPa) and 11.2 ksi (77.22 MPa) for FB19 and FB30, respectively.

The magnitude of stress range near the weld toe on the inside surface of web was about 5 ksi (34.47 MPa) for the test vehicle in either lane. This magnitude resulted from the superposition of the in-plane bending stress and the out-of-plane web bending stress due to distortion and rotation of the lateral connection plate. As a result, the stress range at the weld toe of the lateral gusset plate tab was nearly twice as great as the stress range in the bottom flange. Gages 32 and 24 provide the strain-time response for these two locations in Figs. 20 and 21.

The stress gradients at the maximum and minimum response of the strain gages on the outside surface of the web at the gaps at floor beams 19, 23 and 30 are plotted in Figs. 23 to 27. The solid symbols show the stresses at maximum response and the open symbols show the stresses at minimum response for both the slow crawl runs and the fast runs. Hence, the distance between two corresponding solid and open symbols represent the stress range experienced at that gage.

The results plotted in Figs. 23, 25, 26 and 27 show the stress gradient on the surface of the web at one of the horizontal gaps between the floor beam connection plate and the ends of the lateral gusset plate tabs. The geometry of the gap is also shown at the top of each figure, as well as the location of the individual strain gages. It is readily apparent that the highest stress range developed at floor beam 30 in span 5. If the measured
stresses are extrapolated to the stiffener weld toe, the stress range is about 26 ksi (179 MPa) when the vehicle is in either lane.

The extrapolated stress range at the stiffener weld toe of floor beams 19 and 23 are about 8 ksi (55.16 MPa). The largest difference in stress range is related to the differences in the lateral bracing forces and will be examined in Chapter 4.

Figure 24 shows the measured stress gradients on the surface of the web at the vertical gap between the end of the transverse connection plate and the bottom flange at floor beam 19. It can be seen that an out-of-plane stress gradient developed but was not significant. The stress range at the weld toe was less than 2 ksi (13.79 MPa).
3. EXAMINATION OF CORES FROM EASTBOUND STRUCTURE

Two cores were removed from the south girder of the eastbound structure. The structure was examined on October 7, 1985 and the third and fourth floor beams in Span 2 were identified as having small "fatigue-like" cracks in the paint along the weld toe of the transverse stiffeners on the outside girder web. Figures 28 and 29 show the west weld toe of the transverse stiffener at the level of the lateral gusset plate. Small hairline cracks can be seen in the paint film at both locations.

A core was removed from each of these locations, as shown schematically in Fig. 30. The inside and outside surfaces of these cores are shown in Figs. 31 and 32.

A horizontal saw cut was made through each of the two cores near the centerline of the lateral gusset plate tab. These saw cut sections were polished and etched. Figure 33 shows the polished and etched section of the core from floor beam 3. A crack was found at the inside web surface at the weld termination of the lateral gusset plate tab of the large (west) horizontal gap. This can be seen in Fig. 34 which shows the crack at 100X.

The polished and etched saw cut section of the core from floor beam 4 can be seen in Fig. 35. A crack was found in this core at the west weld toe of the transverse stiffener where the crack and oxide was seen in Fig. 29. The crack can be seen in Fig. 36. The crack appeared to be at least 0.25 in. long. A saw cut was made into the web and the crack surface was exposed after the specimen was cooled in liquid nitrogen and pulled apart along the cut. Figure 37 shows the SEM fractograph of the exposed fatigue crack and low temperature fracture surface. The fatigue crack area is
circled in Fig. 37.

Higher magnification views of the fatigue crack surface and the adjacent fractured web can be seen in Figs. 38 and 39. The fatigue crack is outlined in Fig. 38. The crack can be seen to have extended about 0.005 in. into the web and created a long edge crack condition at the weld toe. The cleavage facets from the fractured surface are readily visible in Figs. 38 and 39 and mark the boundary between the fatigue crack and the area exposed by fracture.

The examination of the two cores removed from the eastbound bridge has demonstrated that small fatigue cracks have developed at both locations. Neither of these cores were removed from sites that have the pronounced crack appearance shown in Fig. 11. The floor beam locations in the westbound structure appear to have experienced more crack extension than observed in the eastbound structure.
4. **FINITE ELEMENT ANALYSIS**

4.1 **Global Discretization Model**

The global discretization of the Canoe Creek Bridge for finite element analysis by SAP IV had 1588 nodes and 7500 resulted in equations. In general, the "ideal" model of coarse mesh is one which minimizes the utilization of computer resources while yielding accurate displacement fields. Accurate displacements are necessary for any subsequent analyses of regions of the bridge structure. Any inaccuracy at this level will be carried throughout the modeling process. The global model size was determined due to the existence of transverse floor beams, the high number of cross-section changes, the haunched profile of the girders, and the spans and length of the bridge.

The transverse diaphragm members are often ignored in the global analysis of box girders and multi-girder bridges. It has been shown that accurate vertical displacements can be obtained by ignoring these relatively flexible structural members. However, in two girder, floor-beam bridges, the floor beams are primary bending members and contribute significantly to the bridge's overall stiffness. In order to obtain accurate displacement fields, the floor beams must be included in the global finite element model. The existence of floor beams at uneven spacings made node numbering and mesh generation difficult. One plate bending element with membrane stiffness through the depth with five divisions along the length made up a typical floor beam. The top and bottom flanges of the floor beams were modeled as beam elements.
The two longitudinal girders were modeled with a combination of plate bending and beam elements. Three plate elements were used for the depth of the girder web with 78 divisions along the bridge's length. Nodal points were placed at points of intersection between the longitudinal girders and the floor beams. Primary bending stress and shear stress gradients were additional considerations. With three elements through the depth of the girder web, a check on the model's accuracy as compared to field measurements could be ascertained. The girder flanges were modeled as beam elements.

There was no significant slippage or movement detected between the deck and the steel superstructure which indicated composite action. The composite action was modeled by embedding the top flanges of the longitudinal girders and stringers in the plate bending elements of the deck. This is accomplished by the sharing of nodes.

In the global modeling of such a large structure, inclusion of small unstiffened segments of web plate is virtually impossible. This would increase drastically both the number of nodal points and number of elements due to element aspect ratio considerations. The extent to which a small gap in the floor beam connection plate or lateral gusset plate has influence on global deformations is difficult to ascertain. Equally difficult to determine is the magnitude of the error that results from ignoring these gaps in the global model. The vertical connection plate gap in the global model was simulated using the beam release codes available in SAP IV(5). At the end of the connection plate beam element, all moments and shears were released. As a result, only axial force is transferred. This technique has been successfully used in modeling connection plates on the global level(6).
Boundary elements were used to simulate both the fixed and expansion condition at the piers.

Equivalent concentrated nodal loads were applied to the global discretization model. Wheel loads from the test truck were distributed as nodal loads using simple beam reactions. In the majority of cases, these wheel loads did not coincide with existing nodal points. A simple beam spanning the width of the deck plate element was assumed. This procedure was repeated until all loads were distributed to the node points.

Inspection of the strain versus time oscillographs taken during test truck runs (see Figs. 16 to 21), indicated that the maximum response of a region of the structure occurred when the truck was adjacent to this region. To obtain the maximum response at floor beam 19, three loading cases were adopted. Figures 40 to 42 show the positions of the vehicle relative to floor beam 19. Each successive case had the truck shifted a small distance in the longitudinal or transverse direction. Results from each case were reviewed to determine which truck position corresponded to maximum response at floor beam 19.

Figures 40 to 42 also show the predicted stresses in the lateral system for the 94.6 kips (421.3 kN) test truck. Each floor beam to girder connection is subjected to lateral forces which are slightly different in magnitude and opposite in sign. Figure 43 shows the out-of-plane displacement of the bottom flange in span 3 when the test vehicle is placed near the centerline of the structure at floor beam 19, as shown in Fig. 41. None of the lateral movements of the bottom flange exceed 0.04 in. (1 mm), and this is partly the reason for the small stresses developed in the bottom laterals (see Fig. 41).
An examination of the stresses in the bottom laterals shown in Figs. 40 to 42 indicates that larger unbalanced forces develop at floor beams 23 and 30 than at floor beam 19 when the truck is near floor beam 19. This suggests that the local stresses are high in the lateral gusset plate gaps at floor beams 23 and 30. That this is the situation is confirmed by the summary of measured stresses in Table 2, and by the stress gradient plots in Figs. 26 and 27 for floor beam 30.

4.2 Substructure Model No. 1

The first substructure model of the region at floor beam to girder connection (Fig. 44) consisted of 11.6 ft. (0.35 m) of girder each side of floor beam 19, the corresponding length of deck and two stringers. The model contained 1610 nodes and resulted in 7400 equations.

It has been shown that at floor beam to girder web connections, the transverse dimension of the structure model should be taken at least 20 to 25 times the length of the vertical gap or 1 to 1.5 times the girder depth away from the connection plate. Since the web plates in the gap regions of the floor beam connection plates and gusset plates were of primary concern, the substructure model boundaries were selected to be at one and one-half times the girder depth to each side of the floor beam connection plates, and at two stringers away from the girder.

A combination of truss elements, beam elements, plate elements and boundary elements were used in substructure model 1. One hundred eighty-four truss elements were used to simulate the stringer flanges. Two hundred fifteen beam elements were used to simulate the girder flanges, the floor beam flanges, the connection plates and the wind laterals. A total of 1208
plate elements were used to simulate the girder web, the floor beam web, the stringer webs and the reinforced concrete deck. A combination of boundary elements and torsionally rigid linear springs were used to impose nodal point displacements, computed from the global model, onto the substructure model.

The one inch gap at the bottom of the vertical connection plate was modeled with one plate element. This element spans between the end of the connection plate and the bottom flange. The beam elements simulating the girder flanges were placed along their centroidal axes. The results of the computed in-plane stresses in the longitudinal girder at the gaged section of floor beam 19 are compared with the test results in Fig. 45. The average of measured stresses at the end of the lateral gusset plate is used in order to obtain the in-plane stress. It can be seen that the measured and predicted results are in good agreement. This indicates that the substructure model 1 is quite adequate. The assumption of composite behavior between the steel structure and the concrete slab was reasonable even though it was designed and built noncompositely.

4.3 Substructure Model No. 2a - Vertical Gap Between Floor Beam Connection Plate and Tension Flange

The substructure for the vertical gap between the end of the floor beam connection plate and the bottom flange consisted of plate and beam elements. Figure 46 shows the element mesh which was centered on the transverse floor beam connection plate. This provided a 24 in. x 38.67 (610 mm x 982 mm) structure and incorporated a substantial amount of the web plate. The position of the lateral gusset plate was at the top of the substructure.
This model consisted of 1101 nodal points and 5698 equations. Altogether, 972 plate bending elements were used for the web plate. Three rows of elements were used to span the 1 in. web between the end of the floor beam connection plate and the bottom flange. In addition, 150 beam elements were used to model the transverse connection plate, gusset and bottom flange. Boundary elements and torsionally rigid linear springs were used to impose the nodal point displacements.

The computed out-of-plane vertical bending stresses in the vicinity of the vertical gap are summarized in Figs. 47 to 49 for the loading case 1 with the test vehicle in the driving lane at FB19 (see Fig. 40). Figure 47 shows the out-of-plane web bending stress along a line parallel to the flange and at the end of the transverse connection plate. Predicted maximum live load stresses are less than 2 ksi (13.79 MPa) except for the local region at the gap.

The computed stresses along the top of the flange at the bottom of the vertical gap are shown in Fig. 48.

The computed web stresses were also compared with the measured stresses at floor beam 19 where strip gages 27 to 31 were installed on the outside web surface. Figure 49 shows the measured and predicted stress gradient in the gap. The computed stresses exceed the measured values by a significant amount and the computed values are large enough to cause fatigue crack growth. Although the measured stresses were relatively low, it is possible that some of the crack-like indications (see Fig. 6) are indeed fatigue cracks.
4.4 Substructure Model 2b - Horizontal Gaps at the Lateral Gusset Plate

The substructure model for the horizontal gaps between the lateral gusset plate tabs and the transverse floor beam connection plate also consisted of a combination of plate and beam elements. Figure 50 shows the element mesh for the web of the main girder, the floor beam, and the lateral gusset plate. Since the horizontal gaps on both sides of the transverse connection plate and floor beam were object of study, the substructure model was quite large. The model included the entire gusset plate, a portion of (each of the two) laterals, the bottom flange, part of the floor beam and 27 in. (68.6 m) of the web plate above the lateral gusset plate. The length of the substructure was 66 in. (167.6 m).

A total of 3376 nodal points and the resulted 16,286 equations were used to simulate the gusset plate and floor beam to girder web connection. Three thousand thirty-eight plate elements were used to model the girder web, the floor beam web, and the gusset plate.

Beam elements were used for the bottom flange, the connection plate, and the wind laterals. Torsionally rigid springs were used to apply nodal point displacements. A total of 319 beam elements were used.

At least three plate elements were used in a gap region of the web. Seven elements were used to span the gusset plate gap, while three elements were used to represent the ends of the horizontal gusset plates. The stresses in these regions are influenced by the end rotation of the floor beam and the forces in the laterals.

The horizontal gaps was subjected to out-of-plane bending stresses. Figure 51 summarizes the results of the substructure model in the two
horizontal gaps at gusset plate. The predicted values are shown as solid lines for the outside web surface and dashed lines at the inside web surface. The highest predicted stress was at the larger gap. Also plotted in Fig. 51 (and Fig. 23) are the measured stresses when the test vehicle was at floor beam 19. The measured values are nearly the same on each side of the stiffener. The measured stresses at floor beam 30, however, are closer to the predicted magnitudes in this horizontal gap.

Figure 52 shows the variations of the out-of-plane horizontal web bending stress as along the weld toe of the transverse stiffener on the outside surface of the web. It can be seen that the distortion-induced out-of-plane web bending stresses reduce from relatively high values at the gaps to acceptable levels about 6 in. (15.2 m) above and below the line of the lateral gusset plate. This suggests that cracking along the vertical weld toes probably will be limited to this region.

The measured and predicted stresses shown in Figs. 51 and 52 indicate that the web is being pushed out-of-plane in a bulging fashion. This has resulted in large tensile stresses at the transverse stiffener weld toes as well as at the ends of the gusset plate attachments to the girder web.

The predicted stresses at the level of the gusset plate on the inside surface of the web plate are plotted in Fig. 53. The solid dots and squares are computed values at 3 in. (76 mm) from the stiffener and at 6 in. (152 mm) intervals thereon extending beyond the ends of the gusset. Both membrane stresses and plate bending stresses on web surface are shown. The distribution of stresses indicates that high out-of-plane bending stresses are developed at the ends of the gusset plates. The predicted level of stress is higher than the measured values shown in Figs. 20 to 22.
5. RETROFITTING THE GUSSET PLATE CONNECTIONS

The eastbound structure has had several of the lateral gusset connections retrofitted by removing the welded bars on each side of the floor beam and replacing the bars with a bolted angle, as shown in Fig. 54. Removal of the two welded bars from the girder webs and repair of any cracks at the ends of these bars eliminates the undesirable condition at these longitudinal weld terminations. The resulting terminations of the bolted connections are no longer Category E details, as bolted connections provide a Category B resistance. None of the measured or predicted stresses would result in cracking at the bolted connection(7).

However, replacing the welded bars with bolted angles does not provide a retrofit of the vertical cracks that are forming at the weld toes of the transverse connection plates. The geometrical conditions at the gaps have not been appreciably altered, and high out-of-plane web bending stresses will still develop along the stiffener and connection plate weld toes. The stresses will continue to induce growth of these vertical web cracks.

As was demonstrated in Fig. 52, large out-of-plane bending stresses only develop adjacent to the lateral gusset connection bars. The replacement of the bars by connection angles will likely extend this distance vertically somewhat. In order to arrest the cracks along the vertical weld toe, it is necessary to install holes on each side of the stiffener and the lateral gusset plates, as illustrated in Fig. 55. This shows a schematic of the holes and a photograph of a similar retrofit(4).

All lateral gusset connection plates on the east and westbound bridge need to be retrofitted. As was shown in Section 3, cracks have developed at joints with little evidence of growth.
In addition, it is recommended that a larger horizontal gap be provided between the floor beam connection plate and the gusset connection angles bolted to the girder web. A minimum distance of 4 in. (100 mm) is desired. Figure 56 shows the recommended geometric details for the lateral gusset connections at the Canoe Creek Bridges.
6. SUMMARY AND CONCLUSIONS

Measurements of the stress conditions developed in the main girders of the Canoe Creek westbound bridge under a 94.6 kips (420.78 kN) test truck, the results of the field examination, and the analysis of the structure and test results have resulted in the following observations, conclusions and recommendations.

1. Web gap distortion cracking was found to occur in three locations. The most prominent was the top end of the transverse floor beam connection plates adjacent to the piers. Other locations included the horizontal gaps between the lateral bracing gusset plates and the transverse floor beam and exterior stiffeners, and less definitive the vertical gap between the transverse floor beam connection plate and the bottom flange.

2. The strain measurements and finite element analysis of the structure demonstrated that these cracks resulted from the out-of-plane web bending stresses that resulted when vehicles crossed the structure. The end rotation of the floor beam and the out-of-phase forces developed in the bottom laterals twisted the gusset and deformed the web gaps. At the lateral gusset plate web gaps the bolted connections between the gusset plates and web and the bottom flange of the floor beam did not prevent web gap distortion. The out-of-phase loading of the laterals also twisted the gusset and web out of plane. This increased the stress in the web plate at the end of the welded attachment, as both in-plane and out-of-plane cyclic stresses resulted.
3. In the negative moment region, no significant out-of-plane bending stress was observed at the upper ends of the bearing stiffeners at the piers. This was in part due to the deeper floor beams and the multiple fitted bearing stiffeners.

4. Floor beams on each side of the piers were provided with transverse connection plates that were not directly connected to the top flange. Extensive out-of-plane web cracking existed, and attempts to arrest the crack growth were not effective using drilled holes alone. Strain measurements in these cracked regions demonstrated that the out-of-plane bending stress at the retrofit holes was excessive. The out-of-plane deformation and lack of straight through thickness cracks caused cracks to reinitiate and extend beyond the drilled retrofit holes. Only a positive attachment between the transverse connection plate and the top flange can be expected to prevent cracks from reinitiation.

5. Cores removed from two floor beam - lateral gusset plate web gap regions of the eastbound structure were found to have small fatigue cracks developing at the stiffener or lateral gusset plate welded connections. These cracks had caused the paint film to crack along the weld toe of the transverse stiffener. No significant evidence of oxide from the crack was apparent. The core examination demonstrated that small cracks were likely to exist at all lateral gusset plate web gaps. Hence, retrofits need to be carried out on all lateral gusset locations in the east and westbound structures.
6. Good agreement was obtained between the measured web gap stresses at the lateral gusset plates and the finite element substructure results. Both yielded levels of stress that would cause crack growth at the weld toes of the transverse welded plates and the lateral gusset plates.

7. The predicted results and measurements both indicated that removal of the welded longitudinal gusset plates from the girder web and replacing them with a bolted web connection (without increasing the gap length) would not prevent continued crack growth along the vertical weld toes of the transverse stiffener and floor beam connection plate.

8. Use of bolted web angle attachments for the lateral gusset plates which provides a web gap of at least 4 in. (101.6 mm) between the connection plate and angle (see Fig. 56) will provide more tolerance for the web gap distortion and will minimize or arrest crack growth. In addition, holes need to be installed in the girder web above and below the lateral gusset, so that the existing transverse weld toe cracks can be arrested.

9. It is recommended that a more rigid bolted connection be provided between the end of the transverse floor beam connection plate and the top flange at floor beams on each side of the pier for the westbound structure. The flange connection should have as many high strength bolts as the transverse connection plate. A more rigid angle (i.e. 8 x 8 x 3/4 in.) (203 mm x 203 mm x 19 mm)
rather than a C12x31 will provide a more rigid bolted joint. Measurements on several structures have indicated that more than two bolts are needed for a shear plane\(^8\).

10. The stress gradients measured in the bottom vertical web gaps of several floor beam transverse connection plates in the Canoe Creek Bridge were smaller than predicted by the analysis. Although, these smaller distortion stresses and the lack of well defined cracking suggest that no retrofits are needed at these locations, there are, however, many areas with indication of cracks in the paint film. Examination of four cores removed from the lateral connection gaps with much more pronounced cracks in the paint film suggests that the indications at the ends of the connection plate are not significant. Nevertheless, it will be prudent to continue to inspect these locations during the normal inspection interval.
ACKNOWLEDGMENTS

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The metallographic and fractographic examination of the cores removed from eastbound structure was carried out by Dr. Eric Kaufmann.

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Tables
### Table 1  Gage Types and Locations

<table>
<thead>
<tr>
<th>Gage No.</th>
<th>Gage Location</th>
<th>Gage Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-6</td>
<td>NG, FB30, Gusset Gap</td>
<td>Strip</td>
</tr>
<tr>
<td>7-8</td>
<td>NG, FB30, Bottom Flange, Web</td>
<td>Linear</td>
</tr>
<tr>
<td>11-16</td>
<td>NG, FB23, Gusset Gap</td>
<td>Strip</td>
</tr>
<tr>
<td>17-19</td>
<td>NG, FB22, Beside Holes</td>
<td>Linear</td>
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<td>21</td>
<td>NG, FB21, Web Beneath Deck</td>
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<td>23, 25, 26</td>
<td>Lateral Bracing, FB19</td>
<td>Linear</td>
</tr>
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<td>24</td>
<td>NG, FB19, End of Gusset Plate</td>
<td>Linear</td>
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<td>NG, FB19, Bottom Gap</td>
<td>Strip</td>
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<td>NG, FB19, Bottom Flange and Web</td>
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<td>33-38</td>
<td>NG, FB19, Gusset Gap</td>
<td>Strip</td>
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<tr>
<td>39</td>
<td>NG, FB19, Gusset Gap</td>
<td>1/16 Inch</td>
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</tbody>
</table>

**NG:** North Girder

**FB:** Floorbeam

Linear Gage: 1/4" Gage Length

Strip Gage: 1/32" Gages in a Line With 0.08" Intervals
Table 2 Measured Stresses at FB 30 and 23, Test Truck Runs

<table>
<thead>
<tr>
<th>Gage No.</th>
<th>Max. Live Load Stress (Stress Range)</th>
<th>Driving Lane Slow Run</th>
<th>Driving Lane Fast Run</th>
<th>Passing Lane Slow Run</th>
<th>Passing Lane Fast Run</th>
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<td></td>
<td>Maximum Live Load Stress (Stress Range)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Driving Lane Slow Run</td>
<td>Driving Lane Fast Run</td>
<td>Passing Lane Slow Run</td>
<td>Passing Lane Fast Run</td>
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<tr>
<td>2</td>
<td></td>
<td>8.2 (12.6)</td>
<td>9.5 (15.2)</td>
<td>11.4 (17.1)</td>
<td>12.0 (18.3)</td>
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<tr>
<td>3</td>
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<td>0.7 (4.0)</td>
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<td>4</td>
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<td>-6.0 (6.0)</td>
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<td>14</td>
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<td>15</td>
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<td>16</td>
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### Table 3 Response at FB 19, Test Truck Runs

<table>
<thead>
<tr>
<th>Gage No.</th>
<th>Maximum Live Load Stress (Stress Range)</th>
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<tbody>
<tr>
<td></td>
<td>Driving Lane Slow Run</td>
<td>Driving Lane Fast Run</td>
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<tr>
<td>Bottom Laterals</td>
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<tr>
<td>23</td>
<td>1.1 (1.4)</td>
<td>1.4 (1.7)</td>
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<tr>
<td>25</td>
<td>-0.8 (1.1)</td>
<td>-0.7 (1.1)</td>
</tr>
<tr>
<td>26</td>
<td>0.5 (0.7)</td>
<td>0.6 (0.9)</td>
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<td>Girder Web at Gap at Bottom of Transverse Connection Plate</td>
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<tr>
<td>27</td>
<td>1.0 (0.5)</td>
<td>1.2 (0.7)</td>
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<tr>
<td>29</td>
<td>-0.4 (0.7)</td>
<td>-0.4 (0.7)</td>
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<td>-1.3 (1.3)</td>
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<td>Girder Web at Gusset Plate Gap</td>
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<td>2.5 (4.5)</td>
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<tr>
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<tr>
<td>32</td>
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<td>2.0 (3.2)</td>
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Fig. 1 View of Westbound Canoe Creek Bridge

Fig. 2 Schematic of Floorbeam-Girder Connection Detail
Fig. 3 Sketch and Photo of Web Gap Detail
Fig. 4 Steel Framing Plan and Girder Elevation of Canoe Creek Bridge
Fig. 5 Cross-section of Canoe Creek Bridge
Fig. 6 Vertical Gap at End of Transverse Connection Plate and Bottom Flange

Fig. 7 Cracks and Retrofit Holes at Ends of Transverse Connection Plate Near Top Flange at FB22
Fig. 8 Close-up of Cracks and Additional Retrofit Holes

Fig. 9 Typical Crack Indications Observed at Ends of Welded Gusset Tabs
(a) View of intersection of transverse connection plate and horizontal gusset plate and tab

(b) Crack indications in web in horizontal gap

Fig. 10 Horizontal Gap Between Transverse Connection Plate and Welded Lateral Gusset Plate Tabs
Fig. 11 Vertical Crack Along Weld Toe of Transverse Stiffener on Outside Web Surface at the Gusset Plate Gap
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Section EE

(a) Gages Placed at FB 22

Elevation at Pier 3 (North Face) FB 21

Fig. 13 Strain Gages at Pier 3 and FB 22
Fig. 14 Strain Gages at FB23 and FB30
(a) Crossing Bridge in Outside Lane

(b) Axle Spacing and Weight

Fig. 15 Test Truck
Bottom Web Gap
Gage No. 27

Outside Web in Gusset Plate Web Gap
Gage No. 33

Outside Web-End of Gusset
Gage No. 40

Load in Driving Lane

Load in Passing Lane

100 μ in/in (3 ksi)
Load in Driving Lane

Gage No. 21

Load in Passing Lane

Gage No. 21

100 μ in/in (3 ksi)

1 sec

Fig. 17 Strain Response of Web at Top Vertical Gap at Pier 3 (FB21)
Fig. 18  Typical Response of Horizontal Strain at FB22 Below Drilled Holes Along Weld Toe of Transverse Connection Plate
Fig. 19 Typical Response of Vertical Strain at FB22 Beyond Drilled Hole Along Toe of Web-flange Weld
Load in Driving Lane

100 μ in/in (3 ksi)

Gage No. 32

Gage No. 23

Gage No. 25

Gage No. 24

Gage No. 40

Fig. 20 Comparison of Strain Response of Laterals and Girder Web at End of Lateral Connection Plate.
Fig. 21 Comparison of Strain Response of Laterals with Girder Web at End of Lateral Connection Plate Tab, FB19
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Fig. 24 Stress Gradients in Web at the Bottom Gap at FB19
Fig. 25 Stress Gradients in Web at the Gusset Plate Gap at FB23
Fig. 26 Stress Gradients in the Web at Gusset Plate Gap at FB30

Stress Range = 27 ksi
Fig. 27 Stress Gradients in the Web at Gusset Plate
Gap at FB30

Stress Range = 26 ksi
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Viewing From South

Elevation
(South Girder)
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(a) Outside Surface of Core

(b) Inside Surface of Core
(a) Outside Surface of Core

(b) Inside Surface of Core

Fig. 32 Core Removed From South Girder
   Eastbound at Floorbeam 4

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Fig. 33 Polished and Etched Section Showing Portion of the Floorbeam Connection Plate Stiffener and Lateral Gusset Plate Tab at Floorbeam 3

Fig. 34 Fatigue Crack Propagating into Web at Weld End of Larger Gap @ 100X
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Fig. 36 Fatigue Crack at Weld Toe on Outside Surface
See Fig. 38 for circled area

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Fig. 41 Test Vehicle with Rear Axles at FB19 in Driving Lane Near Bridge Centerline, (Load Case 2)
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in Driving Lane Near Curb, (Load Case 3)
Fig. 43 Out-of-plane Movement of Bottom Flange in Span 3 with Load at FB19
• Boundary Points For Substructure Model 2a

East Lateral

Floor Beam

Stringer

Concrete Slab

West Lateral

North Girder

Gusset Plate

Fig. 44 Finite Element Mesh for Substructure No. 1
Fig. 45 Comparison of Computed Stress Gradient Using Substructure No. 1 in Main Girder with Test Results
Transverse Connection Plate

Girder Web

Girder Bottom Flange

Fig. 46 Finite Element Mesh for Substructure 2a
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Fig. 48 Out-of-plane Bending Stress Variation Along a Horizontal Line at the Bottom of the Vertical Gap
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Fig. 50 Finite Element Mesh for Substructure 2b
Fig. 51 Comparison of the Measured Stresses Gradient in Gusset Plate Gap at FB19 with the Computed Stress Gradient
Fig. 52 Variation in the Out-of-plane Web Bending Stress Above and Below the Gusset Plate Along the Transverse Stiffener.
Fig. 53 Computed Stresses on the Inside Web at Gusset Plate Level
**In some locations the lateral bracing connection plates are sloped. At these locations the new angle will have to have part of one leg bent or be in two pieces.**

Fig. 54 Sketch and Photo of Retrofit Gusset
Fig. 55 Sketch and Photo of Revised Retrofit Detail
In some locations the lateral bracing connection plates are sloped. At these locations the new angle will have to have part of one leg bent or be in two pieces.

**Fig. 56 Proposed Retrofitting at Gusset Level**
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