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CANOE CREEK BRIDGE - A CASE STUDY OF
SOME WELDED DETAILS IN A STEEL BRIDGE

by
Craig C. Menzemer

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A THESIS
Presented to the Graduate Committee
of Lehigh University
in Candidacy for the Degree of
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in
Civil Engineering

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This thesis is accepted and approved in partial fulfillment of
the requirements for the degree of Master of Science.

May 21, 1985

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ABSTRACT

The design process does not account for the three-dimensional behavior of bridge structures. As a result of primary and secondary member interactions, fatigue cracking at some components or details can quickly occur. Field inspection revealed displacement-induced fatigue cracking had occurred at several locations in the Canoe Creek Bridge. An analytical investigation of cut-short floor beam connection plates and the lateral gusset to web connection was carried out. The primary purpose of the investigation was to determine the stress distribution in the small web gaps which are a part of these details. Then, determine if there was a way to retrofit the details so that the stress fields were reduced to acceptable magnitudes.
1. INTRODUCTION

1.1 Description of Problem

During the past two decades, knowledge of fatigue and fracture behavior of welded steel bridges has been growing. Despite this growth and the newly sparked interest, the condition of welded steel highway and railroad bridges in the United States continues to deteriorate due to corrosion or fatigue. Government authorities estimate that 223,000 of our bridges are "structurally deficient" or "functionally obsolete" (1). A more recent illustration of the deterioration of bridges in the United States was the collapse of the Mianus River Bridge (2).

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 than seen in bolted or riveted connections. While more refined analytical procedures are being used on bridge systems, those used to analyze connections have not changed greatly. In any case, the majority of steel structures developing cracking from displacement-induced stresses have been in service for short periods of time. In an extreme case, cracking was seen to develop before the bridge was open to traffic. A combination of construction traffic and the aerodynamic response of the structure was enough to initiate cracking (3).
In the design process, only the in-plane behavior of the bridge members is considered\(^{(4)}\). 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 result from the three-dimensional behavior of the bridge structure.

Many details which are susceptible to displacement-induced fatigue cracking have been identified\(^{(5)}\). In general, any detail which leaves small, unstiffened segments of web plate is a probable candidate for early fatigue cracking. These gaps at web plates have resulted from the following past practices of detailing and fabricating, and from the old rule of thumb that it was bad practice to make a transverse weld on the tension flange. An additional weld on the tension flange represents a potential for cracking into the flange. A crack in the tension flange represents a serious condition which could lead to catastrophic failure. During the days when riveted construction was the accepted practice, displacement-induced fatigue was virtually nonexistent. Two basic reasons which have been cited are\(^{(6)}\):

1. Riveted joints provide conditions of restraint different from that of welded joints.

2. Truck traffic has been increasingly heavier and of a higher volume today.
In general, connections of riveted systems are more flexible and can more easily accommodate the distortion.

This study will concentrate on the Canoe Creek Bridge located on Interstate Highway I-80 in Clarion County, Pennsylvania. More specifically, two details will be investigated. First is the cut-short floor beam connection plates (Fig. 1). As a vehicle traveling on the bridge crosses over the floor beam, its 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 rotation action of the floor beam, it seems that when lateral bracing members are located close to the bottom flange, the differential pulling of the laterals also influences the out of plane movement of the gap. As a result, high secondary bending stresses are introduced into these small gaps. With high stresses, fatigue cracking will occur in a relatively low number of stress cycles.

Another detail to be investigated is the lateral wind bracing gusset plate. On the Canoe Creek Bridge, the gusset plates are framed around the transverse stiffeners (Fig. 2). Out-of-plane displacements will be accommodated in the horizontal gap. This particular joint utilizes a large number of mechanical fasteners and should be quite stiff. However, even with a high degree of restraint, significant stresses develop in the gap.
1.2 State of the Art

Several attempts have been made at quantifying the phenomenon of displacement-induced fatigue. Instead, general "rules of thumb" have evolved. Most recently, Mertz has suggested positive attachment of connection plates to the tension flange as a means of retrofit(6). In order to retrofit the wind lateral gussets, J. W. Fisher has suggested increasing the gap length as a means of accommodating the distortion(7). This, however, has not been proven as an effective means of reducing the high bending stresses in the gap. T. Fisher investigated a multiple girder bridge with small "x" bracing type diaphragms(8). As a means of relieving the out-of-plane bending stresses at the ends of the diaphragm connection plates, a distance of eight to ten web thicknesses between the flange and connection plate has been recommended(8). Figure 3 illustrates this condition.

One method of retrofitting cut-short floor beam connection plates evolved from the classical slope deflection solutions for a prismatic member. Assuming small or negligible end rotations, the end moment is given by:

\[ M = \frac{6 \, EI \Delta}{L^2} \]

Where:  
- \( L \) = gap length
- \( \Delta \) = out-of-plane displacement
- \( I \) = moment of inertia of a unit width
Hence, the stresses are proportional to inverse of the gap length squared. As such, an easy solution to the problem consists of cutting out a portion of the connection plate. As the gap length increases, the stresses should decrease. However, this is not always a reliable solution. Some details are not "displacement-limited"\(^8\). Hence, increasing the gap may increase the stresses and make the situation worse. While this out-of-plane behavior is generally recognized only for the top gap of vertical connection plates in the floor beam-girder system, several conditions should be recognized. First, that the bottom flange retains some degree of out-of-plane rigidity. This being the case, it is quite possible to have fatigue crack growth in the bottom web gap region. Secondly, with a connection plate which is much wider than the flange's half width, the transfer of the wind laterals' push or pull action down into this gap is quite possible.

Another form of retrofit which applies to cracks in general, consists of drilling holes at the crack tips (Fig. 4). The only completed laboratory study was reported by Fisher in 1979\(^9\). Welded built-up girders were initially subjected to out-of-plane cyclic forces. Once damaged, the cracks were retrofitted by drilling holes at the ends of the crack tips. The girders were subsequently put under cyclic in-plane forces with no apparent crack reinitiation. Despite the results, field observations indicate that the cracks will eventually reinitiate, as in the case of the bridge under study.
1.3 Objectives

There is one primary objective of this reported study on the Canoe Creek Bridge: a study of the gap region at the cut-short vertical connection plate. In this manner, the determination of whether or not the detail is displacement-limited, and what factors influence the stress and displacement fields in the gap area can be determined. The effects of the gap length on the web stresses and gap displacement fields will be investigated.

A secondary objective is to determine whether or not it is possible to model the gusset plate gap region of the web in the Canoe Creek Bridge. This joint utilizes a large number of mechanical fasteners. Since field observations revealed no evidence of slippage in the joint, finite element modeling of this region should be possible.

To accomplish the outlined objectives, a three step finite element modeling process was employed using the program, SAP IV - A Structural Analysis Program for Static and Dynamic Response of Linear Systems (10). A global analysis of the bridge followed by three regional analyses were conducted. The global analysis uses a relatively coarse mesh, yet fine enough to yield accurate displacement fields. The substructure models use finer meshes in the local region that is under investigation. The input to the substructure analyses are the nodal point displacements of the previous analysis.
The global model's validity will be verified by comparisons to field data. Once verified, the influence of the gap length on the stress and displacement fields is investigated. This investigation consisted of changing the variables in the finite element model and noting the effect on the web stresses in the gap.
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 represents a major link between the eastern and western halves of Pennsylvania. Additionally, it represents an alternative to the Pennsylvania Turnpike.

Built in the early 1960's, the Canoe Creek Bridge is a twin girder - floor beam type structure consisting of five continuous spans and a simply supported multigirder end span. The continuous portion of the structure consists of two end spans of 41.45 m (135 ft.) each and three center spans of 49.38 m (162 ft.) each. The continuous girders are haunched over the piers and vary in depth from 2.44 m (8 ft.) in the constant depth region to 4.27 m (14 ft.) over the piers. The haunch varies as a circular arc over a 15.24 m (50 ft.) horizontal length on either side of the piers. Each haunch has a centerline radius of 76.2 m (250 ft.). Figure 5 shows the elevation of one of the girders in the center span.

The two longitudinal girders are welded plate girders with flanges that vary in area over the length of the spans. The largest flange plate has a cross-section area of 248.4 cm² (38.5 in²), while
the smallest is 141.9 cm² (22.0 in.²). The web plates vary in thickness along the bridge's length. For a distance of 6.1 m (20 ft.) to either side of an interior pier, the web is 12.7 mm (0.5 in.) thick. The remainder of the webs are 9.525 mm (0.375 in.) thick.

Figures 6 to 8 show the three typical cross-sections which make up the structure. Between the girders are floor beams which are welded built-up flexural members. The two end spans have a distance of 6.858 m (23.5 ft.) between transverse floor beams, while in the center spans, this spacing is either 7.087 m (23.33 ft.) or 7.01 m (23 ft.). As shown in Fig. 8, the interior piers have a double transverse floor beam running between the girders. This arrangement is presumably used for raising the bridge during construction.

As shown in Fig. 9, the two piers supporting the center span are fixed against expansion. The other piers allow longitudinal expansion via a rocker-type bearing.

All steel in the structure is a mild carbon steel, ASTM A36. The deck is of reinforced concrete and is supported by stringers (W21X55 rolled sections) and the two longitudinal girders. The bridges are essentially composite construction with the flanges of both girders and stringers cast into the deck. No shear studs were used. The composite action is obtained by friction in the longitudinal direction and by positive restraint in the transverse direction (Fig. 10).
2.2 Field Examination of Cracks

The examination of the westbound bridge was carried out in early October 1984. Fatigue cracking was discovered in three locations in the web: in the gap at the bottom end of the floor beam connection plates, at the wind lateral connection plates, and in the gap at the top end of the floor beam connection plate. Only the first two types will be described, as they are of primary importance to this study.

As earlier stated, the vertical connection plates of floor beams in the positive moment region are not attached to the bottom flange. Hence, a gap exists between the web flange junction and the end of the connection plate (Fig. 11). As a result of floor beam rotation and differential pulling of the laterals, the small gap is pushed/pulled out-of-plane with respect to the remaining portion of girder web. High secondary bending stresses in the web in the gap are the result. This type of suspected behavior is consistent with the results of observations made during the inspection. A large number of these gaps developed small fatigue cracks. These cracks generally form in the direction of the girders, parallel to the primary bending stresses. Consequently, these cracks are not serious at the time of development. However, as the cracks grow out of the gap region, the influence of the shearing forces will take effect. The cracks will turn, following a path perpendicular to the principal stresses.
Inspection of the wind lateral gusset plates revealed three types of fatigue cracks. The first occurred at the end of the gusset plate at the weld toe. The second occurred in the small gap between the vertical connection plate of floor beam and the wind lateral gusset plate. The third type occurred on the outside surface of the girder web along the vertical stiffener. The existence of these cracks was somewhat unexpected. The wind lateral gusset plate is bolted to the bottom flange of the floor beam as well as to two horizontal connection plates which are welded to the girder web, one on each side of the floor beam connection plate. In other words, this joint has a high degree of restraint. The out-of-plane movement necessary to cause the cracks was not thought likely to develop. Two dangers exist with these types of cracking. First, these cracks formed perpendicular to the primary stresses in the girder and will continue to grow. Secondly, with a high degree of restraint, the possibility of sudden fracture is high\textsuperscript{(11)}.  

2.3 Instrumentation and Recording

A total of 42 electrical resistance strain gages were mounted between five cross-sections from span 3 to span 5. This study will focus attention on the details adjacent to floor beam 19 in span 3 (Fig. 12). This location was chosen since it is close to the dead load inflection point and should yield the highest stress ranges. The particular details under investigation include the web in the gap at the end of the floor beam connection plate and the wind lateral gusset connection.
As a truck crosses the bridge at a floor beam location, end rotation of the beam occurs. This rotation is the sum of two effects. First, the eccentric nature of the loading with respect to bridge centerline results in a differential vertical displacement between the two longitudinal girders. The second, the rotation introduced as the result of loading on the floor beam (Fig. 13). Additionally, the differential pulling of the wind laterals introduces forces perpendicular to the web plate. This rotation coupled with the lateral action, pushes the unstiffened segment of web at the end of the connection plate out-of-plane with respect to the rest of the girder. A strip of strain gages to measure strains was placed in this gap area.

A second set of gages was placed on the lateral wind bracing and on the web in the gap between the lateral gusset plate and the floor beam connection plate. This small segment of web is subjected to high secondary bending stresses due to the action of the laterals and rotation of the floor beam.

The final set of gages were placed through the depth of the cross-section. Measurements from these gages yield the primary bending stress gradient at the cross-section. The results from these measurements will be used to check the validity of both the gross finite element model and the first substructure model.

All of the strain gages were hooked up to the analog trace recorders (of Federal Highway Administration). The recorded traces depict strain variations with respect to time on light sensitive
recording paper (Fig. 14). From these graphs, stress gradients, stress ranges, and frequencies could be ascertained.

2.4 Loading

Strain readings were acquired under both random truck traffic and under a "test vehicle" of known axle spacing and weight. The test truck, supplied by the Pennsylvania Department of Transportation, had five axles and was loaded to a weight of 421,250 N (94,600 lbs.). Figure 15 shows the test truck axle spacing and the individual wheel loads. The test truck runs are important since these will allow calibration of the finite element models.

The test truck runs consisted of crawl and speed runs in both lanes. Table 1 gives a summary of the runs. Crawl runs generate static response of the structure while the speed runs produce the dynamic response. It is the static response or crawl runs which will be used to calibrate the finite element models. Any differences between the crawl and speed runs can be used to determine the relative magnification in stresses due to the dynamic response.
3. FINITE ELEMENT ANALYSIS

3.1 Gross Discretization Model

The gross discretization of the Canoe Creek Bridge by SAP IV had 1588 nodes and 7500 degrees of freedom. 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 a must, as they are used as input for the subsequent regional analyses. Any inaccuracy at this level will be carried throughout the modeling process. The existence of a transverse diaphragm, the high number of cross-section changes, the haunched profile of the girders, and the spans and length of the bridge led to the gross model's immense size. Such complexity also made automatic generation of the mesh virtually impossible.

In the analyses of many box girders and multigirder "I" beam bridges, the transverse diaphragm members are often ignored in the global analysis. It has been shown that accurate vertical displacements can be obtained by ignoring these small structural members (12). However, in two girder floor beam bridges, the floor beams are primary bending members and as such, contribute significantly to the bridge's overall stiffness. In order to obtain accurate displacement fields, the floor beams must be included in the finite element model. The existence of such members at uneven spacings made node numbering and mesh generation difficult. One plate bending element 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 are modeled with a combination of plate bending and beam elements. A total of three plate elements make up the depth of the girder web with 78 divisions along the bridge's length. The minimum number of divisions through the girder depth was determined by cross-section geometry. 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 not any observed slippage or movement between the deck and the steel superstructure, indicating 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 the small web gaps is virtually impossible. An attempt to model such a gap in the global structure would increase both the number of nodal points and number of elements drastically due to aspect ratio considerations. The extent to which a small gap has influence on global deformations is difficult to ascertain. Equally difficult to determine is the magnitude of the error in subsequent substructure models.

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resulting from ignoring such a gap in the global model. As such, the vertical connection plate gap in the global model was simulated using the beam release codes available in SAP IV\(^{(10)}\). 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 conditions at the piers.

Equivalent concentrated nodal loads were used to load the gross discretization model. Wheel loads from the test truck were broken into nodal loads by finding reactions from a simple beam analysis. In the majority of cases, wheel loads did not coincide with existing nodal points. A simple beam spanning the width of the deck plate element was assumed, and the reactions calculated (Fig. 16). This procedure was repeated until all loads were resolved into node points. This approximation has been shown to yield accurate results\(^{(13)}\).

Inspection of the strain versus time oscillographs taken during test truck runs revealed that the maximum structural response occurred while the truck was adjacent to and directly over the gaged cross-sections. To obtain the maximum response around floor beam 19, three loading cases were adopted for the model. Each successive case had the truck shifted a small distance in the longitudinal direction. Results from each case were reviewed to determine which truck position corresponded to maximum structural response.
3.2 Substructure Model No. 1

The first substructure model of the floor beam to girder region of the Canoe Creek Bridge (Fig. 17) consisted of 0.35 m (11.6 ft.) of web to either side of floor beam 19 and the deck with two stringers. It contained 1610 nodes and 7400 degrees of freedom. Any regional or substructure analyses poses three immediate problems. First, where should the boundaries be chosen so as to eliminate errors arising from St. Venant's effect? Secondly, the substructure modeling of a global structural model generally adopts a mesh of square elements. Large band widths result and large amounts of computer resources are then needed. The third problem is the question how the substructure is to be loaded. A choice between displacements, forces, or a combination of displacements and forces must be made.

It has been shown that for the region of floor beam to girder web connections, the transverse boundaries of the structure model should be taken at least 20 - 25 gap lengths away from the area of interest (6). Since the regions of floor beam connection plates and gusset plates were of primary concern, the substructure model boundaries were chosen as one and one-half times of girder depth to either side of the floor beam connection plates, and two stringers away from the girder.

A combination of truss elements, beam elements, plate elements and boundary elements were used in the substructure model. 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 rein-
forced concrete deck. A combination of boundary elements and
torsionally rigid linear springs were used to impose nodal point
deflections on the substructure model.

The one inch gap at the bottom of the vertical connection plate
is modeled with one plate element. This element spans between the
end of the connection plate and the bottom flange. Care must be
taken in modeling such gaps. If the girder depth is taken as the
web plate depth plus the thickness of one flange, the gap length will
be increased by half of the flange thickness. This increased gap
length will lead to erroneous results. To accurately model the gap,
the girder depth was decreased by half of the bottom flange thickness.
In other words, the bottom flange centroid was moved "up". Figure 18
better illustrates the technique involved in modeling the web gap (6).

3.3 Substructure Model No. 2a

Substructure model No. 2a is basically a model of plate and beam
elements centered around the vertical connection plate and its bottom
gap (Fig. 19). The model extends 0.61 m (24 in.) to either side of
the connection plate centerline and 0.61 m (24 in.) from the bottom
flange to the top boundary. The transverse boundaries were chosen
based on the gap size. Other studies have shown that the distance
influenced by out-of-plane displacements in the web to be of 20-25 web gaps in length(6).

A total of 1101 nodal points and 5698 degrees of freedom are used to simulate the connection plate gap and its influenced region. Nine hundred seventy-two plate elements were used in modeling the web. Three elements were used to span the initial gap region of the 0.0254 m (1.0 in.). Aspect ratios varied between 2.25:1, 2.0:1 and 0.75:1.

Beam elements were used to model the connection plate and the bottom flange of the girder. A total of 150 beam elements were used. Boundary elements and torsionally rigid linear springs were used to impose nodal point displacements on the substructure model.

Establishment of the stress and displacement fields around this gap region was of primary importance. Once established, factors which influence the behavior of this detail could be examined. Once the nature of the joint is known, retrofit procedures can be established.

3.4 Substructure Model No. 2b

Substructure model No. 2b is a model of plate and beam elements simulating the gusset plate and surrounding web region. As previously mentioned, the model size is a function of gap size. Since the gaps to be modeled included the two horizontal ones resulting from the slotted gusset plate and that at the end of the floor beam connection
plate, the substructure model was quite large. Extending a longitudinal distance of 1676 mm (66 in.), the substructure model included the entire gusset plate, a portion of both wind laterals, the bottom flange segment, part of the floor beam and 685.8 mm (27 in.) of web above the gusset plate level. Figure 19 shows the finite element mesh as generated by the computer, while Fig. 2 shows the joint details.

A total of 3376 nodal points and 16,286 degrees of freedom are used to simulate the gusset plate and floor beam to web connection. Three thousand thirty-eight plate elements were used to model the girder web, the floor beam web, the gusset plate, and the longitudinal stiffeners. Aspect ratios ranged from 6.0:1 to 1.2:1.

Beam elements were used to simulate the bottom flange, the connection plate, the wind laterals and torsionally rigid springs which aided in applying nodal point displacements. A total of 319 beam elements were used.

No fewer than 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 connection plates. The stresses in these regions are influenced by the end rotation of the floor beam and the action of the laterals. Although it is a mechanically fastened joint, the finite element model was undertaken as field inspection revealed no slippage in the joint.

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4. **RESULTS OF THE COMPUTER ANALYSES**

4.1 **Verification of the Finite Element Models**

Verification of the gross discretization model involved three separate tests. First, a load case was devised which combined the dead weight gravity load with a small uniform pressure load. The results from such an analysis should be and were symmetric within a tolerance of approximately five percent. If the reactions are not symmetric, an error exists. Either the system stiffness matrix is ill conditioned or the data input contains an error. Ill conditioning may be the result of a highly graded mesh or a poor choice of aspect ratios. Second, a load case was devised which combined the dead weight gravity loads, a small uniform pressure load and two concentrated loads placed symmetrically on the bridge. The resulting reactions from this load case are shown in Table 2. As can be seen, all reactions are symmetric within a tolerance of three percent.

Final verification of the gross discretization model resulted from a comparison to field data. As earlier stated, one area of the bridge under investigation was floor beam 19 in span 3. A large number of strain gages were placed in this area. Four of these gages were placed throughout the depth of the cross-section so as to obtain the primary bending stress gradient. The bending stress gradient from the finite element model is compared to the measured values in Fig. 20. All values from the model were within a few percent of the measured values. As such, the global model was accepted as an
accurate representation of the structure. The differences can be attributed to simplifications in modeling the haunched sections of the girders, simplifications in structural geometry allowing for mesh generation, the inherent inaccuracies in the finite element method and of course the inaccuracies of field measurements.

Verification of subsequent structure models consisted of comparisons of model stress fields to measured stress fields. Substructure model No. 1 consisted of 0.35 m (11.6 ft.) of web to either side of floor beam 19 and was cut two stringers deep (Fig. 17). The primary bending stress gradient was again compared to the measured values. Results can be seen in Fig. 21. Again, agreement was within a few percent, verifying both the accuracy of the substructure model as well as the gross discretization model.

Finally, a comparison was made between the computed stresses from substructure model 2A and the measured stresses obtained from a strip of strain gages placed in the bottom gap of the floor beam connection plate. Due to the small gap size and the general location of the gap in the bridge structure, it was very difficult to place gages in the gap and was impossible to cover the gap's entire depth. This condition notwithstanding, stresses in the gap were measured and are compared with computed stresses in Fig. 22. The agreement is very good. As a result, the models are considered as accurate representations of the bridge details and the computed stresses and displacements should provide indications of conditions of the gaps.
4.2 Results of the Study on the Connection Plate Gap

4.2.1 Response of the Web Plate Gap

The stress variation along the gap length, as shown in Fig. 23, indicates that the web plate surfaces at the top and bottom of the gap are subjected to opposite signs of stresses. The condition corresponds to double-curvature bending of the web plate. To confirm the double curvature, computed web displacements perpendicular to the plane of the web are examined along the gap. The results are shown in Fig. 24 for two vertical locations. Not only is there double-curvature bending of the web plate, also revealed is the difference in magnitudes of displacement on the two sides of the floor beam connection plate.

The difference in out-of-plane displacements on either side of the floor beam connection plate is, in part, induced by the forces in the laterals. Figure 25 shows the out-of-plane displacements at the gusset plate level and Fig. 26 at the level of the top of the gap. There is double-curvature bending of the web plate in the horizontal direction at both levels. These displacement shapes are consistent with the forces in the laterals.

With double-curvature bending of the web plate in both vertical and horizontal directions, the plate bending stresses on the surfaces of the web plate at the gap region are different from point to point. Figure 23 indicates that the vertical bending stresses are highest at the top of the gap, with an extrapolated magnitude of 75 MPa (10.8 ksi).
Examination of Figs. 27 and 28 reveals that this condition of higher stress at the top is true for either side of the floor beam connection plate. Figure 27 shows the variation of vertical plate bending stresses along the horizontal line at the top of the gap. Figure 28 shows the corresponding stresses at the bottom of the gap. It is obvious that the double-curvature bending in the vertical direction is only confined to a short distance on either side of the floor beam connection plate. Away from the connection plate, the stresses reduce to much lower values and are of the same sign at the top and bottom level of the connection plate gap.

4.2.2 Effects of the Gap Length

In order to examine the relationship between gap length and vertical plate bending stresses at the gap. Substructure model No. 1 was modified. The connection plate gap length was changed in this model from the as-built 25.4 mm (1 in.) to 50.8 mm (2 in.) and 101.6 mm (4 in.), as well as to (zero), simulating the condition of positive attachment between the floor beam connection plate and the bottom flange.

Figures 23, 29, 30 and 31 show the stress gradients at the gap on the outside surface of the web plate for the four different gap lengths. By comparing the maximum plate bending stresses at the ends of the gap, it can be seen that increasing the gap length does not necessarily reduce the magnitude of the stresses in the gap. This condition is better shown in Figs. 32 and 33. The curves in Figs. 32
and 33 depict the vertical plate bending stress at the top and bottom of the gap, respectively, as a function of gap length. At the top of the gap, the plate bending stress increases with the gap length. Only a positive attachment of the connection plate to the bottom flange will reduce the stresses at both ends of the gap.

Another phenomenon which can be deduced from this study of gap length is that the region of higher plate bending stresses increases with the gap length. Figures 27, 28; 34, 35; 36, 37 and 38, 39 show the variation of plate bending stress to either side of the connection plate for the four values of gap length. As the gap length is increased, the region of high bending stresses spread out from the centerline of the connection plate. Again, only when a positive attachment is made between the connection plate and the bottom flange is the region affected by the floor beam connection reduced.

The conclusion from the examination is that, for this case of bridge geometry and loading conditions, attachment of the floor beam connection plate to the tension flange will be an effective method of reducing out-of-plane bending stresses.

4.3 Response of the Gusset Plate Connection

The regions of the girder web modeled in substructure model No. 2 includes the horizontal gaps between the floor beam connection plate and the weld toes of the gusset plate connection and the areas at the ends of the longitudinal gusset connection welds. The gaps at the gusset plate are not of equal length. No measurement of the
actual gap length at floor beam 19 was made, and estimates had to be
made from drawings and photographs for the substructure model. As a
result, only quantitative examination on the nature and gradients of
the stresses could be made.

Figure 40 shows the computed gradients of horizontal plate bend­
ing stresses on the outside surface of the web in the horizontal gaps.
There was no change of sign of stress. All along the horizontal gap
the stresses were tensile in nature. However, the average magnitude
of these stresses was a few times more than the primary plate girder
bending stress at this elevation (see Fig. 22). This condition
implies that the web plate was subjected to horizontal out-of-plane
displacement toward the outside surface of the web. That this was
ture has been shown in Fig. 25.

The steep stress gradient in the large gap of the gusset plate
also implies that there was non-uniform web plate bending in this
gap. The stress gradient in the smaller gap was more gentle, cor­
responding to minor plate bending in this gap. This phenomenon of
less plate bending in a shorter gap is in total agreement with the
results of study on the gap lengths at floor beam connection plates.
Consequently, similar conclusion can be drawn, that positive attach­
ment of the gusset plate to the floor beam connection plate will be
an effective method of reducing out-of-plane bending stresses in this
area.

The computed stress gradient at the end of the gusset connection
plate is plotted in Fig. 41 for the inside face of girder web. The
stress drops off fairly rapidly away from the end of the plate. At the end of the plate, the magnitude of stress was quite high, being about 76 MPa (11 ksi). This magnitude of stress was much higher than that corresponding to girder primary bending stress of Fig. 22.
5. **FATIGUE STRENGTH AND RETROFIT SCHEMES**

5.1 **Fatigue Strength**

The maximum computed static live load stresses due to the test truck was 75 MPa (10.8 ksi) at the top of the floor beam connection plate gap just above the bottom flange, 46 MPa (6.9 ksi) at the gusset plate gaps, and 76 MPa (11 ksi) at the end of gusset plate connections. The corresponding dynamic stresses were higher when this and other trucks traveled over the bridge at high speeds. In all cases, when the maximum stress range at any of these details exceeds the fatigue limit, fatigue cracks will develop after large number of stress cycles (14).

The fatigue strength due to out-of-plane displacement at transverse stiffener gaps has been defined as that of Category C for in-plane stresses of steel bridge details (9). The fatigue limit is 76 MPa (11 ksi). For the gusset plate gaps and connections, the fatigue strength is of Category E, with a fatigue limit of 34.5 MPa (5 ksi). Therefore, that maximum stress range at the floor beam connection plate gaps and gusset plate gaps and connections were all above the fatigue limit.

The bridge, being on I-80, is subjected to high volume of truck traffic. Furthermore, field measurements revealed relatively high frequency vibrations, multiplying the number of stress cycles at the details. Table 3 summarizes the observed vibrational frequencies.
With large number of cycles of stresses, some of which are above the fatigue limit, cracks developed in these details.

5.2 Schemes for Retrofitting

Based on the results of finite element model analyses, the most effective scheme to reduce out-of-plane bending stresses at gaps of the floor beam connection plate is to introduce positive attachment of connection plate to the bottom flange. This scheme is recommended. Figure 42 shows two possible ways of attachment.

The nature of out-of-plane plate bending in the horizontal gap between the gusset plate and floor beam connection plate, has been shown to be analogous to that of the gap at the end of the floor beam connection plate. The retrofitting scheme of Fig. 42 can also be employed here. Double angles are needed, one on each side of the floor beam connection plate.

Retrofitting scheme for the ends of gusset plate connections could not be finalized based on the results of model analyses. It appears at this time that a combination of items, such as reduction of vibration of the laterals and positive attachment of the gusset plate to the floor beam connection plate, could reduce the magnitude of stress ranges at the detail and hence retard crack development. More analysis is necessary for the retrofitting. Parametric studies to examine the lateral system in two girder steel bridges is also essential.

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From the analytical investigation, the following conclusions can be made.

1. Modeling of the two girder-floor beam bridge system and its structural details can be achieved accurately through global and substructure models.

2. Substructure model boundaries chosen on the basis of 20-25 times the length of gap at floor beam connection plate, gives accurate results. Distribution of nodal point displacements through the use of torsional rigid linear springs is an acceptable procedure.

3. The web plate at the gap of floor beam connection plate is subjected to double-curvature out-of-plane bending.

4. The web plate at the gaps between a floor beam connection plate and a gusset plate is also subjected to high plate bending stresses.

5. These plate bending stresses are higher than the fatigue limits of the respective details.

6. Increasing the gap lengths at the bottom of the floor beam connection plate does not decrease the magnitude of stresses in the gap. A positive connection of the
connection plate to the bottom flange, on the other hand, reduces the stresses.

(7) Retrofitting of floor beam connection plate gaps by attachment to the bottom flange is recommended.

(8) Study on the behavior of laterals is suggested.
7. TABLES
TABLE 1: SUMMARY OF TEST TRUCK RUNS

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C = CRAWL
D = DRIVING
S = SPEED
P = PASSING
TABLE 2: REACTIONS FROM GORSS DISCRETIZATION VERIFICATION RUN NO. 2

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<td>408.8</td>
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TABLE 3: VIBRATIONAL FREQUENCIES

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<tr>
<td>LATER.</td>
<td>15 - 20</td>
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8. FIGURES
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\[ M = \frac{6EI\Delta}{L} \]
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HAUNCH

GIRDER

F - FIXED

E - EXPANSION
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