FATIGUE STRENGTH OF STEEL BRIDGE GIRDERS WITH DISTORTION INDUCED STRESSES

by

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ABSTRACT

A fairly large number of welded steel bridge girders have developed fatigue cracking due to out-of-plane distortion. These occurrences of cracking are often found at unstiffened gaps at the ends of connection plates of transverse and longitudinal members. This includes diaphragm connection plates in multigirder bridges, floorbeam connection plates and gusset connection plates in floorbeam-girder bridges, and diaphragm connection plates in box girder bridges.

In this dissertation both multigirder and floorbeam-girder bridges are studied. The field studies include field inspections and stress measurements. The stresses at the gap region were further analyzed by a finite element analysis model which shows good agreement with the field test results. These high out-of-plane bending stresses are highly localized in the vicinity of the gap regions. The stress redistribution at the gap region after the development of a through-thickness crack is studied analytically. Parametric study on multigirder bridge systems using the calibrated analytical model is also carried out to examine the effects of the global structural parameters on the stress distributions in these gap regions.

A finite element program with singularity elements at the crack tip is utilized to determine the stress intensity factors. A parametric study on loading and geometric conditions is also conducted. Subsequently, the S-N curves corresponding to different loading and geometric conditions are established utilizing the Paris rule and integrating through the half thickness of the web plate. The results show that the S-N curves for out-of-plane bending are very
close to the Category C fatigue strength line of AASHTO for in-plane stresses, and provide a lower bound for experimental data of details with out-of-plane distortion.

Several by-products of the fatigue strength investigation are quite important. The sensitivity test shows that the relative weldment size (ie. h/t) does not render a significantly different fatigue strength. Results of study also show that stress gradient in the web along the gap direction is insignificant with respect to the fatigue strength of details subjected to out-of-plane distortion.

The application of the results for existing and new bridge girders is discussed; areas for further studies are highlighted.
Chapter 1
INTRODUCTION

1.1 Problem Statement

In recent years, there has been an increase in the number of steel bridges which have developed fatigue cracking due to out-of-plane distortion. Welding often leads to joints or connections with higher local stresses than those seen in bolted or riveted connections because smaller gaps exist in welded details. While more refined analytical procedures are being used for bridge design, the procedures used for analyzing connections have not changed greatly. Many of the welded steel structures developing cracking from distortion-induced stresses have been in service for only a few years. In an extreme case, cracks developed 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 [18].

In the design process, only the in-plane stresses of bridge members are generally considered [2]. The interaction of the primary and secondary members is often only given a cursory examination. However, the primary cause of fatigue cracks in bridge structures is the high magnitude of the so-called "secondary bending stresses" which occur from out-of-plane distortions. These out-of-plane distortions develop because of the three-dimensional behavior of the bridge structure. Many details which are susceptible to distortion-induced fatigue cracking have been identified [19]. In general, any detail which leaves small segments of the web plate unstiffened and is subjected to out-of-plane distortion is a candidate for early fatigue cracking.
Some of these unstiffened gaps at web plates have resulted from the past practice of detailing and fabrication to avoid making transverse welds on the tension flanges of bridge girders [1]. Weldments on the tension flange present a potential location for fatigue cracking or fracture to develop in the flange. As a result, unstiffened gaps exist at many connections between transverse and longitudinal members. It is at these gaps that fatigue cracking has developed.

1.2 Summary of Previous Work

1.2.1 Field Studies

A. Diaphragm Connection Plates in Multigirder Bridges

In multigirder bridges, diaphragms are used between girders to assist in erection and to distribute loads laterally. Usually the diaphragm members are connected to the steel girders through vertical connection plates as illustrated in Fig. 1. The connection plates are welded to the web, sometimes welded to the compression flange, and often cut-short of the tension flange creating a small unstiffened gap as shown in Fig. 1 for a diaphragm near a pier.

A survey of gap cracking in the webs of some 50 multigirder bridges in Iowa was reported by Brakke [7]. The observations that were drawn from these cases of cracking include:

1. The cracks occur at the upper end of diaphragm connection plates whether they are tight fit or cut-short of the top flange (which is rigidly held by the deck slab). Most cracks are horizontal and are located at the toe of a fillet weld joining the web to the top flange. Vertical or diagonal cracks can occur at the ends of the vertical fillet welds attaching the connection plate to the girder web.

2. The cracks can occur at the bottom of the connection plate which is cut short from the bottom flange when the bridge is skewed.
3. The cracks can develop in both exterior and interior girders.

4. Web cracks can occur at connection plates for both rolled-section type and truss type diaphragms. Some indications suggest that the potential for web cracks at "K" type truss diaphragms is considerably less than at "X" (cross) type truss or rolled-section diaphragms.

5. The potential for these cracks is greater on skewed bridges, although many cracks have been found in the negative moment regions of non-skewed bridges.

6. The minimum time for the cracks to develop in bridges carrying less than 3,000 trucks per day is about 10 years and in most cases considerably longer.

Cracks were also discovered and examined in two multispan continuous bridges on I-79 near Charleston, West Virginia [21]. In these two bridges, none of the diaphragm connection plates were welded to either flange at a cross-section. The connection plates were fitted to the top flange throughout the structures. Measurements of the web stresses were made at several of the gaps adjacent to the top flanges. The diaphragms were X-type and made of structural angles. Stress ranges between 10 to 20 ksi were observed at the weld toes.

Most multigirder structures that have experienced cracking at diaphragms have been retrofitted by drilling holes at the tips of all cracks in the web [7].

B. Floorbeam Connection Plate of Floorbeam-girder Bridges

Small unstiffened portions of girder webs in floorbeam-girder bridges are bounded by the longitudinal girder flanges and the floorbeam connection plates. A typical detail is shown in Fig. 2. Unlike the diaphragms in multigirder bridges, the floorbeams are designed as load-carrying members. Usually the
concrete slab is supported by stringers which in turn are supported by the floorbeams. Under normal traffic loading the floorbeam will deflect and develop end moments as well as create out-of-plane bending moments in the webs of the longitudinal girders. Since the top flanges of the longitudinal girders are restrained by the concrete slab, much of the deformation has to be accommodated by the more flexible small gaps that exist at the ends of the floorbeam connection plates. Extensive cracking has been observed in these small gap regions.

A comprehensive field study of the web cracking at floorbeam connections of the Poplar Street Bridge Approaches, East St. Louis, Illinois conducted by Koob et al. resulted in the following conclusions [40]:

1. Cracks were found in the negative moment regions of girders, at the connection plates of the first interior floorbeam and also at the heavy bearing stiffeners to which the floorbeam was connected at piers. The cracking occurred in the web at the top gap of the connection plates which were not attached to the top flange.

2. The web stresses in the gap at the flange-web weld toe extrapolated from measured stresses when the bridge was under typical truck traffic, can be as high as 10 to 15 ksi. The relative out-of-plane distortions between the top flange and the end of connection plate are in the order of thousandth of an inch for gaps with short crack or without crack. For longer cracks, (2 to 7 inches in length,) the out-of-plane distortions were one order of magnitude larger being 0.02 to 0.04 inch.

3. Core samples from gap regions of bearing stiffeners showed cleavage facets which were an indication of brittle fracture. No evidence of fatigue crack extension was detected.

4. Retrofitting by drilling four holes at the ends of cracks at the gap regions of first interior floorbeam connection plates did not satisfactorily contain the cracks. In less than ten years, branching cracks have reinitiated through or bypassed the holes. Fractographic examination of the surface of these cracks showed striation-like features which were indications of fatigue crack growth.
5. The majority of the floorbeam connections located in the positive moment regions were functioning satisfactorily except two locations. At these two locations, signs of cracking were found at the top gaps where the connection plates were not welded to the top flange as specified on the shop drawings.

C. Lateral Gusset Connection Plates in Floorbeam-girder Bridges

The lateral bracing system in bridge structures is used primarily to resist lateral forces and lateral movement due to wind or live loading. Often these lateral members are attached to the girder web at the cross sections where the floorbeams connect to the girder. The horizontal gusset plates which connect the lateral members to the girder web are often welded or bolted to the girder web and not directly attached to the floorbeam transverse connection plate. In these cases, a small gap exists on each side of the transverse connection plate between the vertical connection plate and the horizontal gusset plate. Often, the gap is less than 1 inch between the weld toes. This condition has resulted in vertical cracks forming in the gap or on the exterior surface of the girder web along the vertical weld toe of the outside stiffener opposite to the gap.

A example of this detail is shown in Fig. 3 which is taken from a three-span continuous bridge on I-79 near Charleston, West Virginia [21]. The strain measurement at this detail revealed that double curvature out-of-plane bending developed in the web gap. The combination of out-of-plane bending and primary in-plane bending in the girder web resulted in stress ranges of 9 to 15 ksi at the vertical weld toe. The measured strains in the lateral bracing members on each side of floorbeam showed a time lag, see Fig. 4. This condition implied differential forces which tended to rotate the lateral connection plate. This rotation increased the out-of-plane distortion of the web in the gap.
Examination of core samples taken from the bridge verified the existence of fatigue cracks along the vertical weld toe.

D. Diaphragm Connection Plate at Tie and Box Girder Bridges

Cracks have been detected in the web plates of tie-girders in gaps at internal diaphragms where floor beams frame into the tie girder. As shown in Fig. 5, the diaphragm is often bolted to one web plate and welded to the other. Since the diaphragm plate is not connected to the top or bottom flange, four unstiffened web portions, or gaps, are formed bounded by the internal diaphragm and flanges. Under traffic loading the floorbeam tends to deform the cross-section of the tie girder and develop out-of-plane deformations at the gap regions. This type of distortion-induced cracking has been found and examined at two bridges, on I-79 at Neville Island, Pennsylvania and on I-470 at Wheeling, West Virginia [18, 20].

Significant stress gradients were found in the web within each of the four gaps. Stress ranges of 5 to 10 ksi were extrapolated from strain gage locations to the point of diaphragm weld termination. Similarly ranges of 10 to 20 ksi were extrapolated to the roots of the backup bars.

Examination of cores taken from the bridges indicated that lack of fusion existed behind the backup bar and provided a severe notch condition at the root of the weld. The notch was perpendicular to the forces transferred at the diaphragm welds and enhanced crack growth in the direction of the tie girder.

Cracks were also found in webs of curved box girders, at the gaps where
the internal diaphragm connection plate was cut short of the tension flange. A typical crosssection is shown in Fig. 6. This crosssection is taken from the curved steel box girder of the Ramp C Viaduct at the intersection of I-695 and I-83 near Baltimore where some twenty-seven of these cracks were found after 8 years of service [16, 47]. The truck traffic volume was very high with an estimated average daily truck traffic of 5100 vehicles.

1.2.2 Analytical and Experimental Studies

Since the early 70's numerous cracks from distortion-induced fatigue have been found in highway and railway bridges [13, 19]. Large numbers of these cracks usually form at similar locations on a bridge before corrective action is taken. Despite the large number of cracks found in the web at gap regions, relatively little analytical and experimental study has been conducted to examine the causes and effects. In the only reported laboratory test program concerning out-of-plane distortion-induced fatigue cracking [14], the girders were laid horizontally on two pedestals and the web at each of the gaps at the ends of cut short stiffeners was caused to crack by applying out-of-plane cyclic loads. After cracking was observed, circular holes were drilled in the web at the crack tips and then the girders were placed upright and subjected to cyclic in-plane flexural loading. These tests simulated one means of retrofitting fatigue cracks, which could occur due to handling or shipping, at the ends of transverse stiffeners to which no diaphragm members are connected. Since no out-of-plane distortion was imposed after the holes were drilled, the tests did not simulate the retrofitted conditions at diaphragm connection plates with the diaphragm in place.
Tests are currently being carried out under NCHRP Project 12-15(5), Fatigue Behavior of Variable Loaded Bridge Details near the Fatigue Limit. These tests are on welded plate girders under simulated random loading. For the first pair of guiders tested, the out-of-plane distortion of the web at the gap of diaphragm connection plate continued after retrofit holes were installed at crack tips. Cracks reinitiated from the holes because the out-of-plane distortion was large and it was necessary to remove the diaphragm in order to arrest the cracks to permit continued testing of the girders. Tests are also underway in NCHRP Project 12-28(6), Distortion-Induced Fatigue Cracking in Steel Bridges. These tests simulate the out-of-plane distortion at transverse connection plates and at lateral bracing connection plates.

An analytical study on the out-of-plane distortion of a simple span four-girder highway bridge was reported by T.A. Fisher and Kostem [15]. In this investigation a "typical" bridge from a highway structures design handbook was used [70], and examined under HS20-44 loading. The web at the gap of the diaphragm connection plate in the positive moment region of a girder was analyzed by finite element modeling. A parametric study encompassing variations in the girder dimensions and the type of diaphragm member connection details was conducted. Since the bridge was not an actual structure, no verification of the analytical results through field measurement could be made.

An analytical study including comparison with several field measurements of web stress distribution at gap regions was presented by Mertz [47]. Finite
element modeling was conducted using a “zooming method”. A structure was first modeled by a coarse mesh of finite elements and then the area of interest was further discretized into a finer mesh model. Through this multi-level modeling, the stresses at a local area could be calculated. It was shown that this technique provided satisfactory results when compared with the data from field measurements.

A related phenomenon of fatigue cracking due to out-of-plane deflection of the webs of plate and box girders has been studied in this country and in Japan [45, 49, 50, 69, 74]. Since the initial out-of-flatness and tension field action in bridge girders could cause the web to deflect out-of-plane and result in high out-of-plane bending stresses under in-plane loading, fatigue cracking could develop along the web boundary. However, the cause of these out-of-plane deflections is not the interaction of transverse and longitudinal members. Consideration has been given to this phenomenon when formulating design provisions for web plates [71]. There is no known case of web cracking in bridge structures as a result of this type of out-of-plane deflection.

1.3 Objectives and Scope of This Study

Although the experimental studies and a few analytical case studies confirmed that steel bridge girders can develop fatigue cracks due to distortion in the small gaps at the girder webs, there exists no information for correlating the local web stresses and the global structural parameters of a bridge systems. In other words, information is not available on how to control bridge design parameters so as to alleviate the problem of out-of-plane distortion and possible fatigue cracking.
Furthermore, current design codes only deal with the fatigue resistance behavior of steel bridge structure members under in-plane loading situations. The lack of fatigue strength curves for out-of-plane distortion-induced stresses prevents accurate quantitative estimation of the fatigue life of bridge details subject to such stresses.

The objectives of this study are to examine the influence of global geometry on local out-of-plane stresses in webs at diaphragm connection gaps of girder bridges and to develop appropriate fatigue strength curves. The study consists of five phases; each is presented in a separate chapter subsequently.

The first phase is a field study of two types of girder bridge structures, multigirder and floorbeam-girder systems. This includes field inspection and field measurement of primary and local stresses of the girders at the gap region of connection details.

The second phase compares the calculated stresses at the gap regions with the field measurement data of the multigirder system. The calibrated analytical finite element model is then used to make parametric studies on structural factors influencing stress distribution in these gap regions.

The third phase deals with the estimate of stress intensity factors of the cracks at the weld toe. A correction factor to the stress intensity factor is obtained by utilizing a finite element program with singularity elements. The effects of joint geometry and loading conditions upon stress intensity factors are
studied for different crack lengths. The results are compared with those from an approach employing Green’s function.

The fourth phase estimates fatigue life of connection details under distortion-induced stresses by a linear elastic fracture mechanics approach and utilizing the stress intensity factors from the third phase. The purpose is to estimate the fatigue strength quantitatively and to examine the influence of geometric and loading conditions upon the fatigue strength of these details. The sensitivity of initial crack size on the fatigue strength of the connections is also studied. The analytical results are compared with previous and current experimental results of several studies on fatigue strength due to out-of-plane bending of girder webs.

The fifth phase examines the application of the quantified fatigue strength for the design and evaluation of fatigue strength of girder details with distortion-induced stresses. The conservativeness involved in establishing the fatigue strength line is also discussed.
Chapter 2  
FIELD STUDIES

2.1 Multigirder Bridges

A. Beaver Creek Bridge

a. Brief Description of the Structure

This multigirder bridge is one of a twin structure carrying eastbound traffic on I-80, in Clarion County, Pennsylvania. The 392-foot long bridge has four spans of 78', 118', 118' and 78' as illustrated in Figure 7. The diaphragms are X-type, and consist of angle members, spaced 19 ft to 24 ft apart. The arrangement is shown in Fig. 8. All intermediate stiffeners and diaphragm connection plates on the girder webs are cut 1 in. short of the tension flanges (Fig. 9). The connection plates are fitted to the compression flange. A typical cross section with diaphragms in the negative moment region of the bridge is shown in Fig. 10. The 3/8 in. web has a constant depth of 58 inches. The steel is ASTM A36 grade. The girder top flanges are embedded in the concrete deck. No shear connection was provided between the concrete slab and the steel girders.

The design stress range was 8.6 ksi in the bottom flanges of the girders at diaphragm D5 where most strain gages were mounted for the field study. The design live load stress varied from 5.4 ksi in tension to 3.2 ksi in compression.

b. Results of Field Inspection

Visual inspection revealed indications of cracks in spans A and B in the gaps at the ends of the diaphragm connection plates. No other mechanical or electrical device was used to verify the existence of the cracks.
The cracks found were by and large similar and were arbitrarily grouped into the four types, shown schematically in Fig. 11. Following is a description of these crack conditions.

A -- Horizontal cracks along the web weld toe between the tension flange and web.

B -- Horizontal or inclined cracks at the end of the web weld toe at the cut short connection plate.

C -- Vertical cracks at the weld between the connection plate and web. These cracks tend to peel the connection plate away from the web surface.

D -- Cracks at the end of the weld toe of the tight fitted connection plate.

All of the cracks were small and appeared to have a very shallow depth. Fig. 12 shows two of these cracks; one at the end of a connection plate weld and one along the web-flange weld.

c. Instrumentation and Test Procedures

Electrical resistance strain gages were utilized for strain measurement. (Details of the strain gage locations and the measured stresses are given in Ref. [44].) Strain measurements were made when the bridge was subjected to a "test truck" load as well as to the loads of random truck traffic. The test truck was a semi-trailer type (a 3S2 "low boy") with gross weight of 94.6 kips (see Fig. 13).
Four groups of test truck runs were made: two with the truck in the driving (curb) lane, and two in the passing lane. Each group of measurements consisted of one slow run (about 10 mph) and one fast run (about 55 - 60 mph).

d. Summary and Discussion of the Test Results

Typical strain responses on the web surface in the gap at the top of the connection plate and in the bottom flanges at diaphragm D5 are shown in Fig. 14 for two fast runs of the test truck. Essentially the strain-time records are influence lines with superimposed dynamic effect. The fluctuation of strains in the gap at the connection plate is more prominent than in the girder flanges, resulting in a higher number of cycles.

The gradient of vertical stresses on the web surface in the top gap at diaphragm D5 on girder G5 is shown in Fig. 15. The web stresses at exterior girder G5 were slightly greater than those recorded in adjacent girder G4. The stresses in the web at the gap of the exterior girder were higher when the test truck was in the driving lane. The maximum stress range at the flange-to-web weld toes was estimated by extrapolating the measured strain data. This suggested that 16 ksi occurred in girder G5 and 12 ksi in girder G4 at diaphragm D5. Stresses at diaphragm D3 were small.

The magnitude of the stress range for fast runs of the test truck was slightly higher than that observed for slow runs of the test truck, indicating the existence of a dynamic effect.
B. Mill Creek Bridge

a. Brief Description of the Structures

The Mill Creek Bridges are dual 4-span continuous welded plate girder bridges on I-81 in Schuylkill County, Pennsylvania. The bridges carry south and northbound traffic on separate roadways over Conrail tracks and Mill Creek. The south and northbound bridges are 413′ (91' - 124' - 124' - 74′) and 509′ (118' - 136.5' - 136.5' - 118') in length respectively, and the piers are skewed 15 degrees.

The deck system of each bridge is supported by five girders which are connected by transverse diaphragm members. The plan view and diaphragm locations are shown in Fig. 16. Three types of diaphragms were used, two of which are shown in Fig. 17. Type A diaphragms were only used at the abutments of the northbound bridge and were identical to Type B except for a thicker concrete haunch in contact with the top cross member. The diaphragm connection plates and intermediate stiffeners were all welded to the compression flanges and fitted to the tension flanges. The connection plates and stiffeners are provided on both sides of the web at interior girders, but only on the inside web surfaces of the exterior (fascia) girders.

The girder webs were 7/16 in. thick plates for both bridges and the girder depth was 60 in. and 66 in. for south and northbound bridges, respectively. The structural steel was all ASTM A36 material. The girder top flanges were embedded in the concrete deck with galvanized bridge form attached to the flange edges. No shear connection was used to connect the 8 in. deck to the steel girders.
The girders were newly painted at the time of field measurement and no evidence of cracks was detected. None of the fitted ends of the connection plates indicated that movement was occurring between the connection plate and the tension flange.

b. Instrumentation and Test Procedures

All strain gages were installed in the first span next to the west abutments. Most of the gages were on the southbound bridge. The details of the strain gages and recording system are provided in Ref. [44].

Strain measurements were made during the HS20 test truck (72.9 k) runs and when a number of random trucks passed over the southbound bridge. Regular traffic was stopped when the test truck was on the southbound bridge. Only measurements under random truck traffic were made on the northbound bridge.

c. Summary and Discussion of the Test Results

The magnitudes of all measured stresses were low. The highest value of stress recorded during passage of the test truck was 2.6 ksi on the web at the gap of a diaphragm connection plate.

The vertical stresses at the top gap of a stiffener on the exterior face of girder G1 at diaphragm D2 (Type C) are shown in Fig. 18. The gradient of vertical stresses on the web surface is similar to that observed in the Beaver Creek Bridge (see Fig. 15). The connection plate at diaphragm D2 was welded to the top (compression) flange. Hence, it was expected, that the web gap stresses would be relatively low.
Gages placed in the bottom gap of a stiffener on the exterior face of girder G1, where the diaphragm connection plates were fitted to the bottom tension flange, had negligible strains.

Strain gages mounted on the web at the edges of weld toes at the top and bottom gap at diaphragm D3 (Type B) on the exterior girder G5 of the southbound bridge gave somewhat smaller stresses than those measured in the top gap at diaphragm D2 (Type C). The concrete slab was haunched at the Type B diaphragms and was in contact with the channel spanning between the girders. The higher diaphragm stiffness reduced the out-of-plane distortion and the web plate bending stresses were minimized.

2.2 Canoe Creek Floorbeam-girder Bridge

A. Brief Description of the Structure

The Canoe Creek Bridge is located on I-80 in Clarion County, Pennsylvania. The structure consists of two separate bridges, one supporting eastbound traffic and the other, westbound traffic.

Built in the 1960's, each bridge is a twin floorbeam-girder type structure consisting of five continuous spans and a simply-supported multigirder end span. The continuous portion of the structure consists of two side spans of 135 ft. each and three center spans of 162 ft. each. The continuous girders are haunched over the piers and vary in depth from 8 ft. in the constant depth region to 14 ft. over the piers. The haunch varies as a 250 ft radius circular arc over a 50 ft. horizontal length on either side of the piers. Figure 19 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. The web plates vary in thickness along the bridge's length. For a distance of 20 ft. to either side of an interior pier, the web is 0.5 in. thick. The remainder of the webs are 0.375 in. thick.

Floorbeams between the girders are welded built-up flexural members, (see Fig. 20). The two end spans of the bridge have a floorbeam spacing of 23.5 ft., whereas in the center spans this spacing is either 23.33 ft. or 23 ft.. The laterals (ST7WF39) are connected to the floorbeam flanges and the girder web through two gusset plates 0.375 in. thick. Their arrangement is depicted in Fig. 21.

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. No shear connection was provided between the concrete slab and the girders although both top flanges of the longitudinal girders and the stringers are cast into the concrete deck.

The design stress ranges in the bottom flanges of the girders at FB19 and FB30, where strain gages were mounted, were 9.3 ksi and 11.2 ksi, respectively [22].

B. Summary of Field Inspection, Instrumentation and Test Procedure

The examination of the westbound bridge was carried out in early October, 1984. Substantial evidence of fatigue cracking was discovered at three locations in the girder webs: in the horizontal gap between the lateral bracing
connection plates tabs and floorbeam connection plates, in the vertical gap at the top end of floorbeam connection plates in the negative moment region, and at the far ends of the lateral bracing connection tabs which are welded to the girder web.

The vertical floorbeam 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. 22. A number of these gaps exhibited crack-like indications in the paint, as illustrated in Fig. 22. These indications generally formed in the direction of the girder flange, parallel to the primary bending stresses. Although no cores of web plate were removed for examination, the indications appeared to be primarily cracks in the paint film and not fatigue cracks in the girder web.

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 floorbeam connection plates in the negative moment region. Figure 23 shows typical cracking at a floorbeam in the negative moment regions adjacent to the piers. Retrofit holes had been drilled in the web plate in 1983 shortly after cracks were detected. A number of these cracks had reinitiated, and additional holes were drilled to arrest or retard further crack growth, as illustrated in Fig. 23.

No cracks were detected in the vertical gaps at the piers where double floorbeams existed and three large bearing stiffeners were welded to the web and the bottom flange.
Fatigue cracks occurred in the small horizontal gaps between the vertical floorbeam connection plate and the lateral gusset plate tabs. Figure 24a shows one of these gaps. 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. 24b. On the outside surface of the girder web, opposite to the horizontal gap, cracks were detected along the vertical stiffener, as shown in Fig. 25. These cracks were observed on each side of the vertical stiffener at the level of the lateral gusset plates. The cracks were observed at nearly every floorbeam location examined in both negative and positive moment regions.

Fatigue crack indications were also discovered at the far ends of the lateral bracing gusset plate tabs as shown in Fig. 26. These cracks formed at Category E details.

The existence of cracks on the inside and outside surfaces of the girder web at the horizontal gaps was unexpected. The lateral gusset plates are bolted to the bottom flange of the floorbeam as well as to the two horizontal connection plate tabs which are welded to the girder web, as illustrated in Fig. 21. 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. No adverse experience has been reported when a positive attachment was provided between the gussets and the transverse connection plates. The present case of connecting the lateral gussets to the bottom flange of the floorbeam did not seem to provide sufficient restraint for local distortion.
Strain gages were mounted at five cross-sections on the north girder and are identified in Fig. 19.

The test truck configuration and weight, and the test procedures were identical to those used for the Beaver Creek Bridge described in Sec. 3.1.1.

C. Summary and Discussion of the Test Results

The most comprehensive instrumentation and measurements were made at FB19. This floorbeam is located in the center span adjacent to the dead load inflection point. The location was selected for the focus of a computer analysis.

Figures 27, 28, and 29 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 indicated that passage of a single truck in either lane resulted in very high magnitude stress cycles in the horizontal gaps of the lateral gusset plate tab (see gage 33 in Fig. 27). 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. 27).

The largest stress range occurred in the web at gusset plate gap at floorbeam 30. Floorbeam 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. This verified that no cracking
should develop, and there was no evidence of cracks or movement. The extensive cracking observed in the web at the top gap of the transverse connection plate of floorbeam 22 (Fig. 23) had demonstrated that the fatigue cracks were reinitiating at the drilled holes along the connection plate and web-flange weld toes. The response from a gage installed near these holes is shown in Fig. 28. It can be seen that large cyclic stresses are introduced adjacent to these retrofit holes thus crack extension will continue.

Figure 29 shows the response of the laterals, the bottom flange of the main girder and the inside and outside surfaces of the web at the end of the welded lateral gusset plate tab at floorbeam 22. The test results show that the laterals were subjected to tension or compression and therefore introduced an out-of-plane distortion of the web plate as the vehicle crossed the span. This can also be seen in Fig. 30 where the stress gradients at the end of the gusset plate about 2 ft. west of floorbeam 19 are plotted for the instant when maximum strain response occurred in the bottom flange. The in-plane bending stress gradient is nearly linear when the average stress from gages inside and outside of web at the end of lateral connection plate 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 Fig. 29 that the stresses in the laterals were first of the same sign, indicating tension in both members, then of opposite signs and are out-of-phase during passage of the vehicle. This latter condition is compatible with the observed response at the end of the lateral connection plate. 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 at the measured section.
The stress range at the weld toe was 4.9 ksi for the response shown in Fig. 29. The stress range is lower than the design stress range which was 9.3 ksi. Nevertheless, it was above the fatigue limit for Category E and crack growth would be expected.

The stress range near the weld toe 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 as can be seen in Fig. 29.

The stress gradients at the maximum and minimum response of the strain gages on the outside surface of the web at the horizontal and vertical gaps at floorbeams 19 and 30 are plotted in Figs. 31, 32 and 33. 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 highest measured stress range developed at FB30 in span 5 where the measured stresses extrapolated to the stiffener weld toe is about 27 ksi. The extrapolated stress range at the stiffener weld toe of floorbeams 19 was about 8 ksi.

Figure 32 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 FB19. The out-of-plane stress was not significant.
Chapter 3
STUDY OF MULTIGIRDER STRUCTURES

3.1 The Local Web Stresses in Gaps and the Design Parameters

The field study described in Chapter 2 shows that the local web stresses are high. The imminent questions are "How long will it take to develop cracks at these details under high local web stresses?" and "What design parameters should be changed or added to alleviate possible problems?" The first question will be examined in Chapters 4 and 5. The effects of global structural parameters on the local web stresses in gaps in multigirder structures are examined in this chapter.

Bridge engineers at present do not consider as design parameters the out-of-plane web stresses at gaps of structural details. There are no guidelines for evaluation, and it is extremely impractical to calculate the local stress by elaborate computer programs. In order to assure that no problem of fatigue cracking will develop at the gaps, either alternate detail can be used (eg. lateral connection plate details suggested by Kuzmanovic [41]) or the components at the details can be directly attached to reduce the stresses [22, 44]. For both procedures as well as for examination of local stresses in existing structures, a parametric study on the girder geometry is essential.
3.2 Analytical Modeling of Multigirder Beaver Creek Bridge

In order to determine the forces and displacements in the diaphragm or lateral members and the associated stresses and displacements that develop in the gaps at floorbeam or diaphragm connection plates, finite element models were developed. Beaver Creek Bridge was analyzed using the SAP IV program [5]. Three levels of model were made for analyzing each detail. They are (1) A Gross discretization of the Superstructure, (2) Substructure No. 1 and (3) Substructure No. 2. The gross model discretized the entire bridge superstructure using a system of coarse mesh. Its goal was to obtain an accurate displacement field some distance away from the area of interest. The subsequent substructures containing the area of interest were “loaded” along the model boundaries by the displacements (or forces) which resulted from the previous solution.

A detailed description of these analytical procedures is given in Refs. [44]. The analytical models were first calibrated with the results of field measurements.

3.2.1 Stresses in Diaphragm Members

The predicted stresses in the diaphragm members at diaphragms D5 are compared with the measured stresses in Fig. 34. The measured and predicted values followed the same trends and agreed fairly well.

The computed displacement and rotation fields of girder G5 are summarized in Fig. 35 for diaphragm D5. The load is placed at D5. The results indicate that large relative out-of-plane displacements and rotations are
confined to the vicinity of the diaphragm connection plates. Similar results were observed at other diaphragms [44].

3.2.2 Web Stresses in Gaps

The predicted and measured vertical stresses in the web adjacent to the top gap of girder G5 at diaphragm D5 are compared in Fig. 36. The computed values and the measured data extrapolated to the weld toe show fairly good agreement when the load is in the driving lane. Extrapolation of the measured stresses to the weld toe indicates a maximum stress of 10 to 12 ksi. When the load is in the passing lane, the measured and computed stresses are low and the discrepancy is slightly larger by percent.

Examination of the computed stresses at the diaphragm connection plate top gap regions shows that the out-of-plane bending stresses in the web plate are highly localized. The magnitude of the web bending stresses decreases rapidly away from the vertical centerline of the connection plate. A example is given in Fig. 37 which shows the predicted web surface stress gradients at three sections 0.5, 1.5 and 3 inches away from the centerline of the connection plate, respectively. This condition of localized out-of-plane bending stresses is in good agreement with the localized relative out-of-plane displacements and rotations that were summarized in Fig. 35.

The out-of-plane web plate bending stresses were found to be small at the bottom gap of the connection plates. Figure 38 compares the predicted and measured stresses at the bottom gap of girder G5 at diaphragm D6 in the positive moment region of the bridge. The measured stresses due to loads on
the passing lane were nearly zero. The web plate bending stresses decrease rapidly away from the connection plate.

3.2.3 Nominal Web Stresses at Gaps After Cracks Form

When cracks develop in webs at gap regions, the stress distribution at the vicinity changes. A finite element model was used to model the cracked details at diaphragm connection plates.

In the case of a gap at the end of a multigirder diaphragm connection plate, the crack was simulated by replacement of elements. The "shear lock" phenomenon, that prevents the surfaces on the opposite sides of the crack from moving laterally with respect to each other, was simulated through release of the end moment for the beam elements [5]. The results of the analysis are presented in Figs. 39 to 42. Figure 39 shows the magnitude of transverse (vertical) plate bending stresses on the web surface along the web-to-flange weld toe at a diaphragm connection plate. The existence of a crack reduces the magnitude of the highest stresses, and the reduction is more pronounced as the crack grows longer along the horizontal weld toe. On the other hand, the magnitude of vertical stresses at the end of the diaphragm connection plate increases with the crack length. This is depicted in Fig. 40 which shows the stress gradient along a vertical line at the connection plate-to-web weld toe. This suggests that after the development of a fatigue crack along the flange-web weld toe, additional cracks can develop at the end of the connection plate. This condition has been observed in the field.

When cracks have developed at both ends of a gap at the diaphragm
connection, the gradient of vertical stresses along the flange-to-web weld toe remain about the same. This can be seen by comparing Figs. 39 and 41. The stresses at the end of the connection plate, however, are reduced, as can be detected by comparing Figs. 40 and 42.

The magnitude of stress plotted in Figs. 39 to 42 result from the 94.6 kip test truck discussed in Section 3.1. The maximum predicted stress is between 15 and 20 ksi, and decays rapidly away from the gap region. These stress magnitudes are consistent with measured stresses. The conditions indicate that drilled retrofit holes are likely to be successful in arresting crack growth in multigirder diaphragm connection plates.
3.3 Parametric Study of Multigirder Bridges

3.3.1 Computer Model and Parameters

A finite element model similar to the one used for the Beaver Creek Bridge formed the basis for a parametric study of the stresses in the web at diaphragm connection plate gaps adjacent to the top flanges of girders. In order to reduce the computation time, a two span bridge was chosen. This still allowed a satisfactory simulation of the gaps in negative moment regions of bridge girders.

The values of the parameters, that is, the dimension of the components of the model bridge, are tabulated in Table 1. The primary factors studied were the type of diaphragm (K or X), depth of girder (58" or 96"), number of girders (3, 4, or 5), spacing of the girder, and the lateral position of loads. The load is shown in Fig. 13, and the fourth axle was placed directly over the cross section of interest. The distance between the top flange of the girder and diaphragm attachment to connection plate is 6 inches.

The results from the parametric study show that all gap regions at diaphragm connections are subjected to double curvature bending as illustrated in Fig. 43. By and large, the out-of-plane bending stresses at both ends of the gap are about the same magnitude. Therefore, only the stresses at the flange-to-web weld toe will be reviewed.
3.3.2 Effects of Diaphragm Type

The forces in the members of X- and K-type diaphragms are summarized in Figs. 44 to 47. For all cases except the 3-girder bridge with a 96 in. depth, the K-type diaphragms exert less force on the webs than the X-type diaphragm. This result partially confirms the suggestion derived from field observation that a K-type truss should be used for intermediate diaphragms whenever possible instead of X-type truss or rolled-section diaphragm [7].

The stress distributions along the web-to-flange weld toe at the diaphragm are shown in Figs. 48 to 53. The average reduction of stress from an X-type to a K-type diaphragm is about 20% for the outside girder.

3.3.3 Effects of Girder Depth

Two girder depth, 58" and 96", were investigated. The fifty-eight inch depth web with 3/8 inch thickness was comparable to the Beaver Creek Bridge. The web thickness for the 96" deep girder was increased to 1/2 inch. The corresponding web slenderness ratio of $D/t = 192$ is the maximum proposed for girder webs without transverse stiffener when $F_y = 36$ ksi. The cross sectional moment of inertia was kept the same for the two depths.

The computed forces in the diaphragm members, (see Figs. 44 to 47), indicate that the values are 15 to 20% lower for 96" girders. The beneficial effect for deeper girder was further enhanced by the thicker girder web which provides higher out-of-plane bending stiffness. The web gap stresses for 96" girder were 50% - 60% of those predicted in for 58" girder, as depicted in Figs. 48 to 53.
3.3.4 Effects of Girder Spacing or Number of Girders

In order to examine the effects of girder spacing on the web stress in the gap, the width of the bridge was kept constant, while the number of girders varies. Girder spacings of 7' 6", 10' and 12' 6" were used for the 5-girder, 4-girder and 3-girder structures. The overhanging portion of the concrete deck was 3' 3" for the 4- and 5-girder bridges and 5' 9" for the 3-girder bridge.

Examination of Figs. 44 to 47 indicates that the diaphragm member forces are about the same for the 5- and 4-girder system, with the 4-girder system being slightly higher. The corresponding forces in the diaphragm members with the 3-girder arrangement are much lower. The out-of-plane bending stresses, shown in Figs. 48 to 53, follow these same trends as would be expected.

3.3.5 Effects of Lateral Position of Loads

From the field measurement, it was known that the maximum stress in the web at gaps occurs when the load is directly over the diaphragm. Figure 54 shows the influence lines for vertical web bending stresses at the top gap of the fascia girder for 3-, 4- and 5-girder bridges. The corresponding horizontal resultants of the diaphragm member forces near the gap are shown in Fig. 55. The compatibility between the web gap stress and the out-of-plane forces exerted by the diaphragm members, permits a qualitative evaluation of web behavior at gaps through an examination of the horizontal resultants of the diaphragm member forces from a global analysis.

Since the influence line magnitude for the loads in the driving lane is lower for a 3-girder bridge than for the 4- and 5-girder bridges (Fig. 55), the
web stresses at diaphragm connection plate gaps in 4- and 5-girder bridges will be higher than those in a 3-girder bridge.

The influence lines for the horizontal resultant for the first interior girder of 4- or 5-girder bridges are shown in Fig. 56. Higher web stresses at the gap will develop when trucks are in the middle of the bridge or in the opposite lane for the 4-girder bridge. The central girder response of 3- and 5-girder bridges is given in Fig. 57. The gap stresses are negligible for the 3-girder system for all loading conditions. The gap stresses have opposite signs when the trucks are in the driving and the passing lane.

The results of parametric study from the computer analysis revealed the importance of considering the global geometry and dimensions in designing multigirder bridges. For the traditional width and thickness of the bridge deck, 4-girders appear to have slightly higher diaphragm forces and web stresses at the gaps of diaphragm connection plates. Qualitative and quantitative evaluation of the effects of the stresses on cracking will be made next.
Chapter 4
EVALUATION OF STRESS INTENSITY FACTOR BY SINGULAR FINITE ELEMENT

4.1 Review of Approaches

The question, "How long will it take to develop cracks in the web at diaphragm regions under high local web stresses?" is the main concern in this and the following chapter. A linear elastic fracture mechanics approach will be applied to predict the fatigue strength of the connection details with out-of-plane distortion of the web plate.

Closed-form analytical solutions exist for stress intensity factors of cracks with idealized geometries [52, 62, 66, 67], but for all practical problems numerical solutions must be obtained. Three numerical methods are currently available for evaluating the stress intensity factor of cracks at the weld toe propagating through the plate thickness.

One method is to refine the finite element mesh size at the crack tip to account for the high stress gradient caused by the crack tip. The stress intensity is then related to the change in detail compliance with varying crack depth [24]. This method is rather tedious and cumbersome especially for a crack growth investigation.

Another method is a superposition approach utilizing the Green's function.
The stress gradient correction factor (or geometry correction factor) is calculated and combined with other correction factors for the front free surface and for a finite width to determine the stress intensity. Because only one stress analysis needs to be made for the uncracked geometry, this method enjoyed most popularity in the past decades for crack growth investigations [4, 8, 9, 32, 37, 39, 42]. Formulas have been derived for the stiffener-to-flange weld toe by utilizing Green's function and have been employed to predict the stress intensity factor of the details at diaphragm connection plates [19]. However, some degree of uncertainty is introduced when the correction factors are combined. Furthermore, usually only the open mode (mode I) of cracking is considered in this method with the effects of shear stresses disregarded [4, 52, 76].

The third method is the use of finite elements with an inverse square root singularity to simulate the region adjacent to the crack tip. The stress intensity factor can be inferred from the results of computation [33, 57], or computed directly through the introduction of a stress intensity factor in the assumed displacement function [26].

In the first method, and sometimes the second method, attempts to account for the singularity either at the crack tip or at the weld toe are made through changing the finite element mesh size, and thus they are mesh-size sensitive. In this chapter the stress intensity factors of the cracks at the weld toe under different geometrical and loading conditions will be investigated by the singularity method mentioned above.
The results will be compared with those from the approach utilizing the Green's function.
4.2 Enriched QUAD-12 Elements -- Element with $r^{1/2}$

Singularity at the Crack Tip

The finite element computer program APES is utilized to conduct the parametric study of cracks in this study [26]. The enriched QUAD-12 element used in program APES was derived by Gifford and Hilton [25] based on the work of Benzley and Beisinger [6]. (See Appendix for details.) Instead of using a finite element mesh size to account for the singularity, the crack tip element has a built-in singularity. The displacement assumption taken for the enriched QUAD-12 element contains the usual displacement for a normal QUAD-12 element [77] plus the leading terms of the singular displacement expansion. The singular displacement components, $u_s$ and $v_s$, can be given in terms of the unknown stress intensity factors by,

$$u_s = K_f f_1 + K_{H1} g_1$$
$$v_s = K_f f_2 + K_{H2} g_2$$

(Eq. 4.1)

where $f_1(r,\theta)$, $g_1(r,\theta)$ and $f_2(r,\theta)$, $g_2(r,\theta)$ are given by [59]

$$f_1(r,\theta) = \sqrt{\frac{r}{2\pi 4G}} \{ \cos \alpha [(2\kappa - 1) \cos \frac{\theta}{2} - \cos \frac{3\theta}{2}]$$
$$- \sin \alpha [(2\kappa + 1) \sin \frac{\theta}{2} - \sin \frac{3\theta}{2}] \}$$

$$g_1(r,\theta) = \sqrt{\frac{r}{2\pi 4G}} \{ \cos \alpha [(2\kappa + 3) \sin \frac{\theta}{2} + \sin \frac{3\theta}{2}]$$
$$+ \sin \alpha [(2\kappa - 3) \cos \frac{\theta}{2} + \cos \frac{3\theta}{2}] \}$$

$$f_2(r,\theta) = \sqrt{\frac{r}{2\pi 4G}} \{ \sin \alpha [(2\kappa - 1) \cos \frac{\theta}{2} - \cos \frac{3\theta}{2}]$$
$$+ \cos \alpha [(2\kappa + 1) \sin \frac{\theta}{2} - \sin \frac{3\theta}{2}] \}$$
\[ g_2(r, \theta) = \sqrt{\frac{r}{2\pi 4G}} \left( \sin \alpha(2\kappa + 3) \sin^2 \frac{\theta}{2} + \sin \frac{3\theta}{2} \right) \\
- \cos \alpha(2\kappa - 3) \cos^2 \frac{\theta}{2} + \cos \frac{3\theta}{2} \right) \]

\( K_1 \) and \( K_{II} \) are the stress intensity factors for Mode I and Mode II crackings, \( G \) is the shear modulus, \( \alpha \) is the angle of the crack being positive counterclockwise from the X-axis, and \( r \) and \( \theta \) are polar coordinates centered at the crack tip. The angle \( \theta \) is positive counterclockwise from the extension of the crack direction. The quantity \( \kappa \) is dependent on the problem type and is given by

\[
\kappa = \begin{cases} 
3 - 4\nu \ (plane \ strain \ or \ axisymmetric) \\
(3 - \nu)/(1 + \nu) \ (plane \ stress) 
\end{cases}
\]

where \( \nu \) is Poisson's ratio. When properly differentiated to yield strains and stresses, this displacement assumption yields the correct singularity, \( r^{-1/2} \), for stresses at the crack tip.

Following the routine procedure of the finite element method, constructing the element stiffness matrix and assembling the global stiffness matrix, the unknown stress intensity factors can then be solved together with the unknown displacements at the node points.
4.3 Modeling of the Connection Details with 2-D Mesh

From reported cases, all the cracks found in the bridge details at diaphragm and lateral connection plates are semi-elliptical surface cracks. The loading at the crack appears to be of combined plate bending and membrane tension with different magnitudes along the weld toe. To date, the investigations for surface cracks under plate bending are all under the condition of uniform and pure bending [46]. There has been no information in the literature about the stress intensity factor of a surface crack under non-uniform bending stresses of the type shown in Figs. 48 to 53.

From the results of the approximate estimation in Section 3.2.1., Figs. 39 and 41, it is found that the magnitude of nominal maximum surface stress at the crack tip decreases as the crack length increases along the weld toe. This nature of stress redistribution in the girder web due to forming of the crack further complicates the conditions for analysis of cracks.

Previous studies have indicated that a uniform plate bending stress field tends to increase the crack shape ratio [46], a/b, shown in Fig. 58. On the other hand, the bell-shaped plate bending stress field seems to increase the crack depth into the web. Therefore, for cracks in the web at weld toes, it seems reasonable to assume a through-the-thickness crack growth under a uniform stress field within a finite width centered on the crack. By doing so, the three-dimensional crack growth problem is simplified to two two-dimensional components. One component is the growth of a crack in the depth direction of the web, and the other in the longitudinal direction along the flange-to-web weld.
In this study only the crack growth in the direction of the web thickness is considered. In the through-the-thickness direction, the weld profile and the web plate are sketched in Fig. 58. The flank angle, $\phi$, is assumed to be 45 degrees.

For this condition of through-the-thickness crack growth, the nominal stresses must be computed through analysis as described in Chapter 3. The resulting stresses from the out-of-plane distortion include the effects of stress raising due to the attachment of the connection plates and diaphragms.

From results of analyzing uncracked structures, Figs. 59 and 60, it is apparent that at a distance only about 1/5 of the thickness of the web plate from the weld toe, the stress concentration effect due to the weld toe is completely diminished. This posts a restriction on the size of the finite element model. It also indicates that a strain gage mounted at 0.3t away from the weld toe is not able to register the stress concentration due to the weld toe. A similar observation has been shown in Ref. [43].

The loading of bending, shear or tensile stresses will be applied at the far edge of the web in the model as shown in Figs. 59 and 60. Since linear elastic behavior was assumed, any loading condition can be obtained by simple superposition. For direct comparison, the nominal stresses at the weld toe are set to be the same for bending, shear and tension in the study. The shear loading is applied at about one web-thickness away from the weld toe.
For mild steel weldments with low ability to strain harden, the fatigue properties of the heat-affected zone, HAZ, and weld metal are often similar to those of the base metal [43]. Therefore, in all subsequent finite element analyses of this study, the weld metal, base metal and HAZ are assumed to be linear, isotropic, and homogeneous. The Young’s modulus, $E$, is taken as 30,000 ksi. The Poisson’s ratio, $\nu$, is set at 0.3. Plane strain condition (ie. $\kappa=3-4\nu$) is used in this 2-D modeling of crack growth [60].
4.4 Crack Path

The direction of crack propagation under combined loading has long been the topic for extensive research [11, 28, 61, 65, 73], but no generally accepted analysis has been developed yet.

A pilot study on the crack path in plates under out-of-plane distortion was carried out by utilizing the maximum principal stress criterion [11]. Different initial flaw directions were assumed. The results summarized in Table 2 show that after the initial crack growth, the crack tends to grow perpendicular to the surface away from the weld toe.

All the above analyses were based on the assumption that the crack path is determined by the state of stress at the crack tip alone. However, it must be borne in mind that the fatigue crack growth direction is also sensitive to other factors in a complex structural detail in addition to the stress intensity factor at the crack tip. These factors include material anisotropy, environment, residual stress, etc. For example, Frank found that cracks at the weld toe frequently follow the HAZ boundary; therefore, he took the crack path according to the experimental observation at $\beta = \pi/14$ [24]. Skorupa assumed a 15 degree inclined crack path from the normal line [63].

Sumi found that as long as $K_{II}/K_{I}$ was less than 20%, the predicted crack path generally agreed with the measured path very well [65].

By assuming the crack growing straight, perpendicular to plate surface and
away from the weld toe, the $K_{II}/K_I$ values are all within 20% as shown in Tables 3 to 5. Therefore, a straight crack path is assumed in this dissertation. Although examination of actual cases shows there is usually a “cusp” or shear-lock forming near the mid-thickness of the web plate and influencing the stress distribution (see Sect. 3.2.1), the curving of the crack occurs when the through-the-thickness fatigue life is almost exhausted. Consequently, the assumption of a straight crack growth direction and the evaluation of stress intensity accordingly should not be of critical significance to the prediction of fatigue life of through-the-thickness cracks.
4.5 Regression Analysis for Stress Intensity Correction Factor

Five crack lengths were analyzed in this study for each loading case of uniform tension, bending or shear and each geometrical condition of \( h/t = 0.50, 0.833 \) or 1.00. \( h \) is the weld size of the weldment and \( t \) is the thickness of the web plate. The stress intensity factors, \( K_1 \) and \( K_{II} \), for each case were solved as described in Sect. 4.2 and then combined according to the following equation [11]:

\[
K_{leq} = K_I \cos^3 \frac{\theta_m}{2} - 3K_{II} \cos^2 \frac{\theta_m}{2} \sin \frac{\theta_m}{2} \geq 0 \quad (Eq.4.2)
\]

where

\[
\left( \tan \frac{\theta_m}{2} \right)_{1,2} = \frac{1}{4K_{II}} \pm \frac{1}{4} \sqrt{\left( \frac{K_I}{K_{II}} \right)^2 + 8}
\]

All the results are tabulated in Tables 3 to 5.

The non-dimensional correction factor, \( F \), is obtained by dividing the equivalent stress intensity factor with \( \sigma \sqrt{\pi a} \),

\[
F = \frac{K_{leq}}{\sigma \sqrt{\pi a}} \quad (Eq.4.3)
\]

For each geometrical condition and loading case \( F \) as a function of crack length, \( a/t \), is fitted by a third order polynomial. This yields

**For pure bending**

\( h/t = 0.500, F = 1.4551 - 5.4906(a/t) + 19.713(a/t)^2 - 17.012(a/t)^3 \)
h/t=0.833, \( F = 1.5727 - 6.4844(a/t) + 21.882(a/t)^2 - 18.696(a/t)^3 \)

h/t=1.000, \( F = 1.5821 - 6.4207(a/t) + 21.135(a/t)^2 - 17.655(a/t)^3 \)

**For shear**

h/t=0.500, \( F = 1.6892 - 7.9261(a/t) + 26.933(a/t)^2 - 24.263(a/t)^3 \)

h/t=0.833, \( F = 1.7987 - 8.2970(a/t) + 26.953(a/t)^2 - 23.723(a/t)^3 \)

h/t=1.000, \( F = 1.6540 - 7.5472(a/t) + 24.310(a/t)^2 - 21.248(a/t)^3 \)

**For uniform tension**

h/t=0.500, \( F = 1.9575 - 6.8497(a/t) + 25.725(a/t)^2 - 16.564(a/t)^3 \)

h/t=0.833, \( F = 1.9092 - 6.6848(a/t) + 25.789(a/t)^2 - 16.772(a/t)^3 \)

h/t=1.000, \( F = 1.7089 - 5.6947(a/t) + 24.809(a/t)^2 - 16.562(a/t)^3 \)

To summarize, the stress intensity correction factor can be expressed as

\[ F = C_0 + C_1(a/t) + C_2(a/t)^2 + C_3(a/t)^3 \]

(Eq. 4.4)

The coefficients \( C_0, C_1, C_2, \) and \( C_3 \) for different loading cases and geometrical conditions are tabulated in Table 6. The interpolating accuracy is within 3%.

The results from the analyzed cases and the corresponding interpolating functions of Eq. 4.4 are also plotted in Figs. 61 to 66. From these plots, it is apparent that pure tension cases generate higher values of the correction factor and are more severe than bending and shear cases for the same magnitude of
nominal stresses at the weld toe. The differences between pure bending and shear loading cases are small.
4.6 Comparison with Results from Approach of Green's Function

Besides the results of this study, the only other available results of stress intensity correction factor are by Zettlemoyer [75]. These latter results, however, were derived for estimating the fatigue strength of beam and girder flanges with stiffener to flange weld subjected to in-plane loading. For the derivation, the approach of Green's function was used. The stress gradient correction factor, $F_g$, has been defined through the stress concentration factor at the weld toe. Because the stress concentration factor is theoretically infinite for the model used and is extremely mesh sensitive, analysis was made for an average flange thickness assuming that the results are applicable for slightly different geometrical conditions.

The results of the analysis by Zettlemoyer are also shown in Figs. 61 to 66 for comparison with those of the current study. The expressions for $F$ by Zettlemoyer are as follows [75]:

$$ F = F_e \times F_w \times F_g \times F_s \quad (Eq. 4.5) $$

where

$F_e$, crack shape correction factor, $= 1.0$

$F_w$, back surface correction factor, $= \sqrt{\frac{2t}{\pi a} \tan \frac{\pi a}{2t}}$

$F_g$, stress gradient correction factor, $= \frac{1.621 \ln(h/t) + 3.963}{1 + 2.776(a/t)^{0.2487}}$

and $F_s$, front surface correction factor.
For pure bending

\[ F_s = \frac{0.923 + 0.199(1 - \sin \frac{\pi a}{2t})^4}{\cos \frac{\pi a}{2t}} \]

For uniform tension

\[ F_s = \frac{0.752 + 2.02(a/t) + 0.37(1 - \sin \frac{\pi a}{2t})^3}{\cos \frac{\pi a}{2t}} \]

For a weld size equal to half the web plate thickness, \( h/t = 0.5 \). Figures 61 and 64, show that the current results are in good agreement with results interpolated from two cases analyzed by Zettlemoyer for \( h/t = 0.3205 \) and 0.6410. For higher \( h/t \) values of 0.833 and 1.0, (that is, for relatively thinner web plates), the in-plane loading cases of Zettlemoyer provide correction factors higher than those from the current results. Current results show that the correction factors are by and large unchanged for different \( h/t \) ratios.
Chapter 5
FATIGUE STRENGTH FOR DETAILS WITH OUT-OF-PLANE DISTORTION

5.1 Crack Propagation Life

For convenience of analysis, the fatigue life, $N_{f}$, of connection details can be considered as consisting of two parts: the crack initiation part, $N_{i}$, and the crack propagation life, $N_{p}$.

$$N_{f} = N_{i} + N_{p} \quad (Eq.5.1)$$

For welded structures, the crack initiation stage is usually non-existent due to the fact that small, sharp discontinuities always exist at the toes of the weld by conventional welding processes [12]. Therefore,

$$N_{f} \approx N_{p} \quad (Eq.5.2)$$

The crack propagation life $N_{p}$ is estimated by integration utilizing the Paris semi-empirical rule [51], resulting in the following expression

$$N_{p} = \int_{a_{i}}^{a_{f}} \frac{1}{C(\Delta K)^{m}} da = \int_{a_{i}}^{a_{f}} \frac{1}{C(F\Delta \sigma \sqrt{\pi a})^{m}} da \quad (Eq.5.3)$$

where C and m are material constants, F is the correction factor discussed in Chapter 4, $\Delta \sigma$ the nominal stress range, $\alpha$ the crack size, $a_{i}$ the initial flaw size, and $a_{f}$ the final crack size.

Under the condition of constant stress range, $\Delta \sigma = S_{r}$ and
Equation 5.4 can be transformed into the familiar logarithmic forms of $S_r$-N curve,

$$\log N_p = Q - m \log S_r$$  \hspace{1cm} (Eq.5.5)

where

$$Q = \log \int_{a_i}^{a_f} \frac{1}{C(F\sqrt{\pi}a)^m} da$$

The material constants, $C$ and $m$ for plain materials and weld metal have been investigated by many investigators [30, 34, 35, 55]. The values of $3.6 \times 10^{-10}$ and 3.0 for $C$ and $m$, respectively, have been used for the derivation of $S$-N curves for design [38], and are also used in this study.

It has long been recognized that the initial flaw size has the dominant effect on the estimation of the fatigue strength by the approach of linear elastic fracture mechanics. Numerous studies have been conducted to evaluate the initial flaw sizes [3, 29, 54, 58, 64]. However, because of the random nature of this parameter, it remains to be one of the least certain factors. For this study, the effects of three different flaw sizes were examined, with the initial flaw size of 0.01 in. chosen for comparison with results from other studies and from test results. The assumed initial flaw size of 0.01 in. is consistent with the one used by Lawrence [43] and is comparable with those used by Fisher in
Ref. [19], where he suggests the initial flaw size being 0.03 in. for manual welding process and 0.015 in. for automatic welding process. The final crack size, \( a_n \), should be the plate thickness for through-the-thickness cracks. In actual cases of fatigue cracks in webs of bridges, the cracks often propagate from both surfaces of the plate at slightly different positions and join at the middle [19, 47]. For this reason and for the convenience of computation, the final crack size, \( a_f \), is assumed equal to half of the web plate thickness.

The numerical integration for \( N_p \) was performed by Subroutine DCADRE in the IMSL library [10, 36]. The required accuracy was arbitrarily set by,

\[
\frac{(N_p)_i - (N_p)_{i-1}}{(N_p)_{i-1}} \leq 10^{-5} \quad (Eq. 5.6)
\]

The resulting \( S_r-N \) curves for different geometrical and loading conditions and different initial flaw sizes are shown in Figs. 67 to 72. Figures 67 to 69 present the curves for the different geometrical and loading conditions with \( a_t \) equal to 0.01 inch. Figures 70 to 72 compare the effects of flaw size.

A number of conclusions and discussions can be made from these results.

1. The fatigue strength of joints or connections due to pure bending and simple shear are quite comparable for the same magnitude of nominal flexural stress at the weld toe. This is anticipated because of the low value of the \( K_{II}/K_1 \) ratio.

2. Joints under uniform tension in the web have lower fatigue strength than those under shear and bending loads. This indicates the effects of stress gradient through the thickness of the plate.

3. The influence of \( h/t \) is negligible. The \( S_r-N \) curves are essentially
identical for each loading condition and initial flaw size. This implies that, within the range of weld size to web plate thickness of this study, the weld size has very little effect on the fatigue strength.

4. The fatigue strength of out-of-plane bending is at least equal to or higher than that of in-plane bending of flange plates as estimated by Zettlemoyer.

5. As expected, larger initial flaws (cracks) produce lower fatigue strength of the connections.

6. The S_r-N curves for bending and shear loading cases with initial flaw size of 0.01 in. are all in the vicinity of the Category C fatigue strength curve of AASHTO. The corresponding curves for tension are comparable to that of Category D. (See Fig. 73; the dash lines with slope of -3 are the results of this study.)
5.2 Comparison with Test Results

The earliest tests of concern in the literature are those by Roberts [53] and by Thum [68]. Both used welded mild steel T sections with the flange bolted to the test bed and horizontal load applied at the far end of the outstanding stem. In the early 1960s Goerg used similar test specimens and setup to study the fatigue strength of ST37 and ST52 steels [27].

In late 1960s Mueller and Yen conducted a series of tests on full-scale plate girders under in-plane loading causing out-of-plane web panel deflections and inducing plate bending along the flange-web junction [49, 74]. The nominal stresses on the web surface along the web panel boundaries were estimated by the finite different method and correlated with results from strain gage measurements.

In 1971 Haibach presented an S-N curve for a cruciform joint under out-of-plane bending [31]. The test setup was not given in his discussion. Fisher, Mertz, and Zhong also ran out-of-plane bending tests on cruciform joints recently [17].

All these tests were conducted with different definitions of failure for \( N_f \) and the initial flaw sizes were not known. Direct comparison of these test results with the prediction curves of Figs. 67 to 72 is not possible.

From the analytical results of Sect. 5.1, it is clear that the fatigue strength for loading cases of shear and bending are almost identical for the
same magnitude of flexural stress at the weld toe, and the Category C fatigue strength of AASHTO is a reasonable approximation. Therefore, the test results from bending of T-sections or cruciform specimens and from out-of-plane bending due to out-of-flatness and tension-field-action in plate girder webs may all be compared with the $S_r-N$ curve of Category C as an indirect comparison with those for out-of-plane distortion in a small gap at connection details. All the above-mentioned test results were plotted on an $S_r-N$ diagram with a log-log scale, Fig. 74. Fatigue strength curves of AASHTO categories and the results from the current study (i.e. the dash lines with a slope of -3) are also shown. The Category C curve obviously provides a lower bound for the test data.

Test results are being generated at Lehigh University to examine the fatigue strength due to out-of-plane distortion at gaps of transverse diaphragm connection plates and at lateral connection plate gaps. Results reported by Wagner [72] and additional results to date are plotted in Fig. 75. The test results are all above the Category C strength curve.
5.3 Fatigue Strength for Combined Loading

The loading condition of out-of-plane deformation at connection details is a combination of plate bending, shear and membrane tension. Analytically the fatigue strength of a connection under combined loading of bending, shear and tension can be examined if the appropriate stress intensity factor is derived through the procedure of Chapter 4 and integration of fatigue life is conducted by the procedure of Sect. 5.1. In actual cases, the magnitudes of the loading components are seldom known for the connection details of bridge girders. A simplification procedure needs to be developed for fatigue strength evaluation for design.

Since linear elastic properties of materials are assumed, the stress intensity correction factor, $F$, for the combined stress can be expressed by a linear proportion. For example,

$$F_c = \left(\frac{S_t}{S_b+S_t}\right)F_t + \left(\frac{S_b}{S_b+S_t}\right)F_b$$  \hspace{1cm} (Eq.5.7)

where, $F_t$, $F_b$ and $F_c$ are stress intensity correction factors for tension, bending and combined loading respectively, $S_t$ is the range of membrane tensile stress, and $S_b$ is the range of bending stress. The fatigue strength of the combined loading case can then be integrated according to Sect. 5.1.

Examples of the results of integration are shown in Fig. 76 for different proportions of bending and tension stresses on the web surface at connection details. Because the fatigue strength curve for pure tension is at the fatigue strength Category D of AASHTO, and that for pure plate bending is
corresponding to that of Category C, the fatigue strength for any combination of tension and bending can be estimated by linear combination of Categories C and D according to the proportion of bending and tension.

Field measurements and analysis indicate that, for transverse diaphragm connection plates or floorbeam connection plates the vertical membrane tensile stress in the web at the connection plate gap is usually one or two orders of magnitude lower than the out-of-plane plate bending stress and is often negligible [23]. The membrane stress in the web at the horizontal gap of lateral bracing connection plates could be high especially where the connection plate is near the bottom flange in the positive moment region of the girder. The magnitude from measurements, however, is still only a fraction of the out-of-plane stress. Consequently, plate bending loads dominate in most cases, rendering the combination less important or even unnecessary.
6.1 Out-of-Plane Distortion and In-Plane Stresses

The fatigue strength curves of Figs. 67 to 72 have been derived analytically for the web surface stress at connection details under out-of-plane distortion. As is indicated in Chapter 5, the fatigue strength of these connection details can be represented by AASHTO fatigue strength Categories C and D, respectively, for pure out-of-plane plate bending and pure tension. The orientation of the out-of-plane distortional stresses, however, may be perpendicular or parallel to that of the primary stresses in the flanges of the bridge girders, from which the AASHTO fatigue strength Categories have been derived. Furthermore, the magnitudes of the out-of-plane distortional stresses are not directly controlled by or proportional to the primary stress in the flange at the connection detail. Consequently, estimation of the out-of-plane distortional stresses is necessary.

For existing bridge girders with short unstiffened portions of web at diaphragm or floorbeam connection plates and at lateral bracing gusset plates, the estimations of the nominal out-of-plane distortional stresses can only be made through actual measurement or by analytical procedures such as the finite element method. Because both approaches require great effort, the evaluation of fatigue life expectancy of connection plate joints remains a difficult task even with the establishment of the fatigue strength curves for out-of-plane distortional stresses.
For in-plane loading conditions, the nominal stresses do not include the effect of stress concentration due to different attachment lengths. It is taken into account by different fatigue strength categories [12]. In contrast to in-plane nominal stresses, the nominal out-of-plane distortional stresses at the weld toe include the stress concentration due to the attachment (e.g., the connection plates).

It must also be pointed out that both the in-plane, primary stresses in plate girder components and the out-of-plane distortional stresses in webs at connection gaps redistribute at the vicinity of a crack. Whereas the stress magnitude often increases in flanges when cracks develop therein, the stresses in webs due to out-of-plane distortion at connection details frequently decrease when cracks propagate, (See Fig. 39). This phenomenon has been conservatively ignored in the derivation of the $S_r$-$N$ curves for the out-of-plane loading conditions.

6.2 Evaluation of Fatigue Cracking at Connection Details

The evaluation of fatigue cracking at connection details requires the quantitative determination of a number of items [12]: the stress range magnitudes at the detail, the corresponding cycles of occurrence, and the fatigue strength curve applicable to the detail. For the bridges described in Chapter 2, the results of stress analysis in Chapter 3 confirmed the data from field measurement that local stresses due to out-of-plane distortion are fairly high at some connection details. The fatigue strength curves for details under out-of-plane distortion have been estimated in Chapter 5 for different loading conditions at the connection detail. There is unfortunately not sufficient
information on the stress cycles for direct evaluation of the fatigue cracking of the details. However, an indirect estimation can be made through the procedure using a "fatigue truck" currently proposed [48, 56].

The measured stress range on the web of Girder 5 at Diaphragm 6 of the Beaver Creek Bridge is 16 ksi due to the 94.6 k test truck. The proposed "fatigue truck" weight of 50 k would generate a corresponding stress range of $16 \times 50 / 94.6 = 8.5$ ksi on the web surface in the gap. From the $S_r-N$ curve for out-of-plane bending (AASHTO Category C), the estimated fatigue life is $9 \times 10^6$ cycles when fatigue cracking could be expected to develop. For an average daily truck traffic (ADTT) volume of 2500, the very conservatively estimated lower bound fatigue life is about 10 years. Cracks have been detected at many diaphragm connection plate gaps in the bridge.

6.3 Considerations for Design

One of the questions raised from the field studies is which design parameters should be changed or added to alleviate the possible problems of fatigue cracking due to out-of-plane distortion. Results of the parametric study in Chapter 3 indicated the importance of global geometry and dimensions on the forces in the diaphragm members and, specifically, the governing factor of web stresses in the gap at the diaphragm connection plates. For designing new bridge girders, it is therefore essential to control these stresses in the gap.

Examination of retrofitting schemes for diaphragm connection plates with fatigue cracks in the Beaver Creek Bridge revealed that direct positive attachment of the diaphragm plates to the girder flanges would reduce
significantly the out-of-plane distortional stresses [44]. For the case studied, the magnitude of stress would reduce by one order of magnitude, from 16 ksi to about 2 ksi at the weld toe. This magnitude of web stresses should not cause fatigue cracking. Positive attachment of diaphragm connection plates to the flanges is therefore an effective method to control the stresses in the gaps at the ends of these diaphragms. This condition is now specified by AASHTO for all new bridge girders.

For other connection details, such as lateral bracing connection gusset plates, where a small gap may have to exist for construction, the procedure of “fatigue truck” in Sect. 6.2 can be used to ensure the gap geometry produces web plate bending stresses lower than required for the design life of the bridge girder. Another procedure is to use the “design stress range” and “equivalent cycles” concept described in Ref. [12] but with modification. The nominal stress range can be calculated by placing the standard “design truck” along the bridge [47]. The correction factor $\alpha$ to account for the difference between real stress range and design stress range is not needed. Since not all truck traffic are the same weight as the design truck, the stress cycles should be converted to an equivalent lower number. This is done as follows:

$$ N = (ADTT) \times (D_L) \times (FF) $$

where $ADTT = $ Average Daily Truck Traffic, $D_L = $ Design Life in Days, $FF=\Sigma \gamma_i \phi_i^3$, $\gamma_i$ = fraction of $(ADTT)$ for $(GVW)_i$, and $\phi_i$ = ratio of actual vehicle weight to design vehicle weight, $(GVW)_i/(GVW)_D$.

Both these procedures require computation of local stresses at the
connection details, a time-consuming and costly step. Additional parametric studies on the influence of global and local geometry at connection plate details can give valuable information on the local stresses. Such studies should be carried out.
Chapter 7
SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary and Conclusions

Field study

Based on the field observations and measurements, and on the analytical studies, the following observations can be made on the behavior of small gaps at the ends of diaphragm connections of multigirder bridges and floorbeam and lateral connections of floorbeam-girder bridges. Specifically, the summary is drawn from the studies on Beaver Creek, Mill Creek and Canoe Creek Bridges, and on observations of other similar structures.

1. The stresses in the web at the gaps on the top and bottom of diaphragm connection plates are induced by truck loads on these multigirder highway bridges. There is direct correlation of time between stresses in the girder flanges and in the web at the gaps.

2. The out-of-plane displacement induced web gap stresses at the gaps are highly localized in the vicinity of the gap regions. The vertical (transverse) stresses reduce rapidly in the horizontal direction away from the centerline of the connection plate, and in the vertical direction away from the end of the connection plate.

3. The web gap stresses in a diaphragm connection plate are dependent upon the lateral position of a truck load. The web stresses in gaps at exterior girders are higher when the load is over or near the top of the girder. At interior girders, loads in the traffic lane away from the girder cause higher web stresses in the connection plate gap.

4. The web gap stresses at diaphragm connection plates in the positive moment regions (i.e. at gap adjacent to the bottom flange) of continuous girders were small, being much lower than those in negative moment regions of girders. Cracks were only detected at the ends of connection plates adjacent to the top flange in highway bridges. Cracks have been detected in gaps adjacent to the bottom flange in simply-supported railroad and mass transit bridges.
Measurements on these types of structures have indicated that the web gap stresses are about the same magnitude as those observed in highway bridges adjacent to the top flange.

5. The highest measured and computed live load stresses in the web at the gap of diaphragm connection plate was between 10 and 15 ksi for the Beaver Creek Bridge.

6. Under normal traffic loading, K-type truss diaphragms were found to exert less force on the girder webs than the X-type truss diaphragms. Other conditions being the same, the web stresses in the gaps of connection plates for K-type diaphragms are lower and not as quick to exhibit cracking.

7. The concrete haunch placed over the top of the horizontal diaphragm channel of the Mill Creek Bridge incorporated the channel into the deck system and significantly stiffened the region in the vicinity of the diaphragm connection plate. The transverse stresses in the web were found to be negligible.

8. The stresses in the web at the vertical gaps at the ends of floorbeam connection plates and at the horizontal gaps at lateral gusset connections are caused by unaccounted for secondary stresses from normal traffic. The stress-time responses for the web gaps are directly related to the response of the girder flanges, floorbeams and laterals. However, current design simplifications do not provide a means of evaluating these stress conditions.

9. Displacement caused cracks in girder webs of girder-floorbeam bridges were found in three general locations. The most prominent was the top end of the vertical connection plate of floorbeams adjacent to bridge piers. Next were the horizontal gaps between the lateral bracing gusset plates and the floorbeam connection plates, (and outside the fascia girder along the exterior stiffeners opposite the gaps). A small number, of cracklike indications were detected in the gaps between the floorbeam connection plates and the bottom flange. These indications could not be verified as actual fatigue cracks.

10. At the upper ends of bearing stiffeners over the piers, no significant web plate bending stresses were measured. The bearing stiffeners are fitted to and bearing against the top flange. The vertical (transverse) connection plates for floorbeams on each side of the piers developed more movement between the top flange and the web plate gaps developed cracks.

11. The out-of-plane displacement induced web stresses at the gaps are highly localized in the vicinity of the gap regions. This condition is
similar to the conditions that develop at diaphragm connection plates. The magnitude of the deformation is greater at floorbeams.

12. The web plate of the main girders at the ends of floorbeam connection plates is subject to out-of-plane bending because of the end rotation of the floorbeam. The web plate at the gusset plate gap is subject to out-of-plane plate bending primarily because of the out-of-phase forces developed in the laterals. These forces introduce twisting into the lateral gusset plate region.

**Parametric study of multigirder bridges**

A finite element model calibrated by the field measurement formed the basis for a parametric study of the stresses in the web gap adjacent to the top flange of girders. The results from the parametric study show:

1. With all other parameters remain the same, bridge with K-type bracing is more favorable for the web gap response than with X-type bracing.

2. For the same distance between the flange and the connection of diaphragm components, deeper girders have lower forces in the components of the diaphragm and this results in lower web stresses at the diaphragm connection plate gap.

3. For two-lane, multigirder highway bridges under normal traffic loads, the out-of-plane plate bending stresses in the web gaps at diaphragm connection plates are substantially lower for 3-girder bridges than predicted for 4- or 5-girder bridges.

**Fatigue strength study**

1. For the h/t ratio investigated, from 0.5 to 1.0, it is found that the fatigue strength of details with out-of-plane distortion is not sensitive to h/t ratio.

2. The fatigue strength for nominal out-of-plane bending stress at the weld toe due to pure bending is about the same as that due to shear applied at some distance causing equal bending stress at the weld toe. In other words, the stress gradient in the web direction along the gap is insignificant for fatigue strength.

3. Based on the nominal out-of-plane bending stress at the weld toe in the gap as the major fatigue strength prediction, it is shown both experimentally and analytically that fatigue strength Category C of
AASHTO is a reasonable lower bound for details with out-of-plane distortion at a small web gaps.

7.2 Recommendations for Further Research

Local Stress Range vs. Global Structural Design Parameters

As indicated in Chapter 3, the local web gap stress range can be systematically investigated for each specific structural type (e.g., girder-floorbeam system, box girder system, or tie girder system). Reference [23] shows another example of this effort for a girder-floorbeam systems. But there is much more to be done before a comprehensive guideline for design can be given for different structure systems.

Redistribution of Stresses at Cracked Details or Members

Cracks cause the stress to redistribute in the vicinity of the cracked details of a member. This is not considered in current in-plane and out-of-plane fatigue strength estimation. The reserve strength can be large enough to warrant the further consideration of this remaining strength in fatigue design and evaluation. This information is especially useful for the bridge maintenance engineer, when the cracked member or detail is discovered by inspection and the decision with regard to whether an imminent retrofitting measure is necessary.

Secondary Vibration

High frequency vibration has been observed at the bottom gap of floorbeam connection plates in the positive moment regions at Canoe Creek Bridge and at similar locations on a railroad bridge [47]. The frequency of the vibration appears to be close to the natural frequency of the plate panel. The stress range caused by this vibration is about as large as the stress range caused by loads for the distorted web gaps. This high frequency vibration was
also observed at the edges of the drilled holes at the top gap of the floorbeam connection plate, and may account for the reinitiation and development of cracks in such short time intervals as was the case for the Canoe Creek bridge. Studies are needed to determine the geometric conditions and characteristics of lateral systems and bridges that are susceptible to this type of response.
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<td></td>
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<td>2&quot; × 14&quot;</td>
<td>0.875&quot; × 10&quot;</td>
</tr>
<tr>
<td>4 girder</td>
<td>2&quot; × 20&quot;</td>
<td>1.25&quot; × 12&quot;</td>
</tr>
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<td>3 girder</td>
<td>2.25&quot; × 22&quot;</td>
<td>1.5&quot; × 14&quot;</td>
</tr>
<tr>
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<td></td>
</tr>
<tr>
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<td>7.5’</td>
<td>7.5’</td>
</tr>
<tr>
<td>4 girder</td>
<td>10’</td>
<td>10’</td>
</tr>
<tr>
<td>3 girder</td>
<td>12.5’</td>
<td>12.5’</td>
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<tr>
<td><strong>Span Length</strong></td>
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<td>118’ - 118’</td>
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<td><strong>Width (Out-to-Out)</strong></td>
<td>36.5’</td>
<td>36.5’</td>
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<tr>
<td><strong>Concrete Deck Thickness</strong></td>
<td>8”</td>
<td>8”</td>
</tr>
<tr>
<td><strong>Diaphragm Members</strong></td>
<td>L3-1/2×3-1/2×5/16</td>
<td>L3-1/2×3-1/2×5/16</td>
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<tr>
<td><strong>Diaphragm Types</strong></td>
<td>K- and X-types</td>
<td>K- and X-types</td>
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Table 2: Summary of Crack Growth Direction Study

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<td>4.34</td>
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<td>1.80</td>
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<td>$K_{leq}$ 6.63</td>
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<td>$\theta_m$ 9.1</td>
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$t = 0.375^\circ$

nominal stress at weld toe = 15 ksi

unit for $K$'s: ksi$\sqrt{\text{in}}$

$\beta$ = assumed initial crack direction measured positive counterclockwise from the line which is normal to the web plate surface and away from the weld toe

$\theta_m$ = predicted crack growth direction measured positive counterclockwise from the line which is the extension of the original crack direction
Table 3: Summary of the Stress Intensity Factors, h/t=0.5

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$t = 0.625"$
nominal stress at weld toe = 15 ksi
unit for K's: ksi√in
Table 4: Summary of the Stress Intensity Factors, $h/t=0.800$

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$t = 0.375^\circ$  
nominal stress at weld toe = 15 ksi  
unit for K's: ksi√in
Table 5: Summary of the Stress Intensity Factors, h/t=1.0

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t = 0.3125°
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unit for K's: ksi√in

73
Table 6: Coefficients of Third Order Polynomials for F from Regression Analysis

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<td>24.310</td>
<td>-21.248</td>
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Figure 1: Schematic of Diaphragm Connection Plate Detail  
- Multigirder Bridge

Figure 2: Schematic of Floorbeam Connection Plate Detail  
- Floorbeam-girder Bridge
Figure 3: Schematic of Lateral Gusset Connection Plate Detail - Floorbeam-girder Bridge
Figure 4: Time-dependent Strain Response of the Lateral Bracing
Figure 5: Schematic of Diaphragm Connection Plate Detail
- Tie Girder Bridge [18]
Figure 6: Schematic of Diaphragm Connection Plate Detail
- Box Girder Bridge [16]
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Figure 8: Girder-Diaphragm System of Beaver Creek Bridge
(Eastbound Lane)
Figure 9: Connection plate, Cut-short at Tension Flange, Beaver Creek Bridge
Figure 10: Typical Diaphragm Cross-section, Beaver Creek Bridge
Figure 11: Typical Crackings at Beaver Creek Bridge
Figure 12: Two Typical Cracks at Top Gap, Beaver Creek Bridge
(a) Photo of Test Truck

(b) Axle Spacing and Weight

Figure 13: Test Truck
Figure 14: Typical Strain Records at Diaphragm D5, Beaver Creek Bridge
Figure 15: Gradient of Vertical Stresses at Top Gap of G5 at D5, Beaver Creek Bridge
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Figure 17: Two Types of Diaphragm Used in Mill Creek Bridge
Figure 18: Gradient of Vertical Stresses at Top Gap of G1 at D2 (Southbound Lane) Mill Creek Bridge
Figure 19: Steel Framing Plan and Girder Elevation of Canoe Creek Bridge
36'-6" Out to Out Superstructure
31'-0" Curb to Curb

Passing Lane
Driving Lane

Half Section at Constant Depth
Girder Region

Half Section at Piers 1, 2, 3, 4

Figure 20: Cross-section of Canoe Creek Bridge
Figure 21: Sketch and Photo of Web Gap Detail
Figure 22: Vertical Gap at End of Transverse Connection Plate and Bottom Flange

Figure 23: Cracks and Retrofit Holes at Ends of Transverse Connection Plate Near Top Flange at FB22
View of intersection of transverse connection plate and horizontal gusset plate and tab

Crack indications in web in horizontal gap

**Figure 24:** Horizontal Gap Between Transverse Connection Plate and Welded Lateral Gusset Plate Tabs
Figure 25: Vertical Crack Along Weld Toe of Transverse Stiffener on Outside Web Surface at the Gusset Plate Gap

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Figure 32: Gradients of Vertical Stresses in Web at the Bottom Gap at FB19
Figure 33: Gradients of Horizontal Stresses in the Web at Gusset Plate Gap at FB30
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Figure 36: Comparison of Gap Stresses at Top Gap of Girder G5, Diaphragm D5, Beaver Creek Bridge
Figure 37: Out-of-plane Stress Field at the Vicinity of Top Gap, D5, G5, Beaver Creek Bridge
Figure 38: Comparison of Gap Stresses at Bottom Gap, G5, D6, Beaver Creek Bridge
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Figure 40: Out-of-plane Bending Stress Along the Vertical Line
(Crack at Top of Gap Only)
Figure 41: Out-of-plane Bending Stress Along the Web-flange Weld Toe (Cracks at Both Ends of the Gap)
Figure 42: Out-of-plane Bending Stress Along the Vertical Line
(Crack at Both Ends of the Gap)
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nominal stress at web toe = 15 ksi

CONTOUR VALUES Y STRESSES (unit: ksi)

1 -- -.25E+02
2 -- -.23E+02
3 -- -.20E+02
4 -- -.18E+02
5 -- -.15E+02
6 -- -.13E+02
7 -- -.10E+02
8 -- -.75E+01
9 -- -.50E+01
10 -- -.25E+01
11 -- .32E-10
12 -- .25E+01
13 -- .50E+01
14 -- .75E+01
15 -- .10E+02
16 -- .13E+02
17 -- .15E+02
18 -- .18E+02
19 -- .20E+02
20 -- .23E+02
21 -- .25E+02
CONTOUR VALUES Y STRESSES

1 -- -.26E+02  
2 -- -.24E+02  
3 -- -.21E+02  
4 -- -.18E+02  
5 -- -.16E+02  
6 -- -.13E+02  
7 -- -.10E+02  
8 -- -.79E+01  
9 -- -.52E+01  
10 -- -.26E+01  
11 -- -.17E-08  
12 -- .26E+01  
13 -- .52E+01  
14 -- .79E+01  
15 -- .10E+02  
16 -- .13E+02  
17 -- .16E+02  
18 -- .18E+02  
19 -- .21E+02  
20 -- .24E+02  
21 -- .26E+02

(unit: ksi)

nominal stress at web toe = 15 ksi

Figure 60: Stress Contour under Shear
Figure 61: Correction Factor, $F$, for $h/t = 0.50$
Figure 62: Correction Factor, F, for h/t = 0.833
Figure 63: Correction Factor, $F$, for $h/t = 1.00$
Figure 64: Correction Factor, $F$, for $h/t = 0.50$

$$F = \frac{1.9575 - 6.8497(a/t) + 25.725(a/t)^2 - 16.564(a/t)^3}{1.621 \ln(h/t) + 3.963 \left(1 - \frac{\pi a}{2t} \right) \times \left\{ 0.752 + 2.02(a/t) + 0.37 \left(1 - \frac{\pi a}{2t} \right)^3 \cos \frac{\pi a}{2t} \right\}}$$
Figure 65: Correction Factor, $F$, for $h/t = 0.833$

\[ F = (1.0) \times \left( \frac{\pi a}{2t} \right) \times \frac{1.621 \ln (h/t) + 3.963}{1 + 2.776(a/t)^{2/4}} \times \frac{0.752 + 2.02(a/t) + 0.37(1 - \sin \frac{\pi a}{2t})^3}{1 + 2.776(a/t)^{2/4}} \times \cos \frac{\pi a}{2t} \]

$F = 1.9092 - 6.6848(a/t) + 25.789(a/t)^2 - 16.772(a/t)^3$

$F = \{1.0\} \times \{\sqrt{\frac{2t}{\pi a \tan \frac{\pi a}{2t}}}\} \times \frac{1.621 \ln (h/t) + 3.963}{1 + 2.776(a/t)^{2/4}} \times \frac{0.752 + 2.02(a/t) + 0.37(1 - \sin \frac{\pi a}{2t})^3}{1 + 2.776(a/t)^{2/4}} \times \cos \frac{\pi a}{2t}$
Figure 66: Correction Factor, F, for h/t = 1.00

\[ F = \{1.0\} \times \left\{ \sqrt{\frac{2t}{x_0}} \tan \frac{x_0}{2t} \right\} \times \left\{ \frac{1.621 \ln (h/t) + 3.963}{1 + 2.776(a/t)^{0.2487}} \right\} \times \left\{ \frac{x_0 \sin \frac{x_0}{2t}}{\cos \frac{x_0}{2t}} \right\} \]

\[ F = 1.7089 - 5.6947(a/t) + 24.809(a/t)^2 - 16.582(a/t)^3 \]
Figure 67: $S_r$-$N$ Curve for $h/t = 0.50$, $a_i = 0.01^\circ$
Figure 68: $S_r$-$N$ Curve for $h/t = 0.833, a_i = 0.01^\circ$
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Figure 76: $S_r$-$N$ Curves for Combined Primary Tension and Secondary Bending Stresses
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APPENDIX

NOMENCLATURE

a  Crack Depth

a_i Initial Crack Size

C  Material Constant for Paris Law

C_0, C_1, C_2, C_3  Coefficients for Interpolating Polynomial Function, F

D  Girder Web Depth

F  Non-dimensional Stress Intensity Correction Factor

F_b  Stress Intensity Correction Factor for Bending

F_c  Stress Intensity Correction Factor for Combined Loading

F_e  Crack Shape Correction Factor

F_g  Stress Gradient Correction Factor

F_s  Front Surface Correction Factor

F_t  Stress Intensity Correction Factor for Pure Tension

F_w  Back Surface Correction Factor

F_y  Yield Stress

f_1, f_2  Functions for Displacement Due to Singularity at Crack Tip Associated with Mode I Stress Intensity Factor

g_1, g_2  Functions for Displacement Due to Singularity at Crack Tip Associated with Mode II Stress Intensity Factor
G       Shear Modulus
h       Weld Size
Ki      Stress Intensity for Mode I Cracking
KII     Stress Intensity for Mode II Cracking
Keq     Equivalent Stress Intensity Factor
m       Material Constant for Paris Law
Ni      Fatigue Life Due to Crack Initiation
Np      Fatigue Life Due to Crack Propagation
Sr, Sr  Stress Range
Sb      Applied Bending Stress Range
St      Applied Tensile Stress Range
t       Thickness of Steel Girder Web Plate
us      Singular Displacement Component in X-direction
vs      Singular Displacement Component in Y-direction

α1, α2... Coefficients for Assumed Displacement
β       Assumed Initial Crack Direction
ϕ       Frank Angle of Weld
\( \kappa \) Coefficient for Plane Strain, Plane Stress or Axisymmetric Case

\( \nu \) Poisson’s Ratio

\( \theta_m \) Predicted Crack Growth Direction
CONVENTIONAL AND ENRICHED QUAD-12 ELEMENTS [25]

Conventional QUAD-12 element

The displacement assumption for QUAD-12 element is given by [77]

\[
\begin{align*}
u &= \sum_{i=1}^{12} N_i(s,t)u_i \\
v &= \sum_{i=1}^{12} N_i(s,t)v_i
\end{align*}
\]  
\(\text{(Eq.A.1)}\)

where \(u\) and \(v\) are displacement components in the \(x\) and \(y\)-coordinate directions, \(N_i\) are interpolating polynomials, and \(u_i\) and \(v_i\) are the unknown displacement components at node \(i\). The \(N_i\) correspond to a complete interpolation of displacement over the element in its local coordinates, \(s\) and \(t\), and are given by

for nodes at \(s=\pm 1, t=\pm 1\)

\[
N_i(s,t) = \frac{1}{32}(1+s_{i})(1+t_{i})[-10+9(s^2+t^2)]
\]  
\(\text{(Eq.A.2)}\)

for nodes at \(s=\pm, t=\pm 1/3\)

\[
N_i(s,t) = \frac{9}{32}(1+s_{i})(1-t^2)(1+9t_{i})
\]

and similarly for the other side nodes.

The element is made to be isoparametric (same parameters) by letting the geometry vary as generally as the displacement field,
where $x_i$ and $y_i$ correspond to the cartesian coordinates of node $i$. Thus the element edges may take on curved shapes without needing to introduce numerous elements.

Equations A.1 to A.3 are sufficient to express the differential strain-displacement relationships for planar or axisymmetric cases in terms of unknown nodal displacement in matrix form. This relation, along with the appropriate stress-strain relation allows the 24 by 24 element stiffness matrix to be formed by numerical integration in the usual way as described by Zienkiewicz [77].

**Enriched QUAD-12 Element**

The displacement assumption taken for the enriched QUAD-12 element is a combination of the usual bicubic displacement assumption and singular displacement field given by Eq. 4.1. That is

$$u(s,t) = \alpha_1 s^3 + \alpha_2 s^2 + \alpha_3 s + \alpha_4 t^3 + \alpha_5 t^2 + \alpha_6 st^2 + \alpha_7 s^2 t + \alpha_8 st^2 + \alpha_9 st + \alpha_{10} t^3 + \alpha_{11} s^3 + \alpha_{12} s^2 t + K_I f_I(s,t) + K_{II} g_I(s,t)$$

(Eq.A.4)

where the $\alpha$'s, $K_I$ and $K_{II}$ are undetermined constants. Similar assumption is taken for the $v$ component of displacement.

Equation A.4 can be written in a form similar to the displacement assumption taken for the conventional QUAD-12 element. In matrix form, Eq.A.4 may be written
By evaluating Eq.A.5 at each of the nodes, the matrix equation for unknown nodal displacements may be written

\[ \{u\} = \{C\}\{\alpha\} + K_1\{f_1\} + K_{II}\{g_1\} \]  

(Eq.A.6)

in which all matrices are known constants except \(\{u\}\) and \(\{\alpha\}\). Solving Eq.A.6 for \(\{\alpha\}\) then gives

\[ \{\alpha\} = [C]^{-1}\{u\} - K_1[C]^{-1}\{f_1\} - K_{II}[C]^{-1}\{g_1\} \]  

(Eq.A.7)

Substituting Eq.A.7 into Eq.A.5 for \(\{\alpha\}\) then gives

\[
\begin{align*}
\{u\} &= \{P(s,t)\}[C]^{-1}\{u\} - K_1\{P(s,t)\}[C]^{-1}\{f_1\} \\
&
\quad - K_{II}\{P(s,t)\}[C]^{-1}\{g_1\} + K_1\{f_1\} + K_{II}\{g_1\}
\end{align*}
\]

(Eq.A.8)

The matrix \([P(s,t)]/[C]^{-1}\) is simply the matrix of standard interpolation function \(N_i\) [77]. i.e.

\[ [P(s,t)]/[C]^{-1} = [N_1, N_2, N_3, \ldots N_{12}] \]

Therefore, Eq.A.8 may be written

\[
\begin{align*}
u(s,t) &= \sum_{i=1}^{12} N_i u_i + \sum_{i=1}^{12} N_i f_{1i} + K_{II} \left( \sum_{i=1}^{12} N_i g_{1i} \right) \\
&= \sum_{i=1}^{12} N_i u_i + \sum_{i=1}^{12} N_i f_{2i} + K_{II} \left( \sum_{i=1}^{12} N_i g_{2i} \right)
\end{align*}
\]

(Eq.A.9)

where the second subscript on \(f_1\) and \(g_1\) indicates "evaluated at node \(i\)". The analogous expression for the \(v\) component of displacement is

\[
\begin{align*}
v(s,t) &= \sum_{i=1}^{12} N_i v_i + \sum_{i=1}^{12} N_i f_{2i} + K_{II} \left( \sum_{i=1}^{12} N_i g_{2i} \right)
\end{align*}
\]

(Eq.A.10)
VITA

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