NATIONAL COOPERATIVE
HIGHWAY RESEARCH PROGRAM REPORT **227**

FATIGUE BEHAVIOR OF FULL-SCALE WELDED BRIDGE ATTACHMENTS

TRANSPORTATION RESEARCH BOARD
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FATIGUE BEHAVIOR OF FULL-SCALE WELDED BRIDGE ATTACHMENTS

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TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. NOVEMBER 1980

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Staff
Transportation Research Board

This report contains the findings of an extensive laboratory evaluation of the fatigue behavior of welded steel bridge members. The recommendations are immediately applicable and will be of interest to engineers, researchers, members of specification writing bodies, and others concerned with the design, construction, and maintenance of steel structures.

Relatively large reductions in fatigue strength of many welded details occur when cracks initiate and grow from the micro-sized defects that exist at the weld periphery. This behavior has been demonstrated by studies on cover-plated beams and other structural details, and has been reported in *NCHRP Report 102,* "Effect of Weldments on the Fatigue Strength of Steel Beams"; *NCHRP Report 147,* "Fatigue Strength of Steel Beams with Welded Stiffeners and Attachments"; *NCHRP Report 188,* "Welded Steel Bridge Members Under Variable-Cycle Fatigue Loadings"; and *NCHRP Report 206,* "Detection and Repair of Fatigue Damage in Welded Highway Bridges."

Recently, fatigue problems have developed in a number of bridges with gusset plates welded to webs or flanges. Cracks have grown in the web gap between the end of the gusset weld and the transverse stiffener. This condition is complicated by the high residual stresses developed in these severely restrained configurations and also by out-of-plane movement caused by the lateral bracing. Information is needed on the fatigue strength of these details and on the efficacy of applicable retrofit measures.

This report contains the findings of NCHRP Project 12-15(3), "Fatigue Behavior of Full-Scale Welded Bridge Attachments." The objective of this study was to examine the fatigue strength of beams with web and flange lateral attachment plates. In addition to providing a more comprehensive data base for this type of detail, the program was intended to examine the influence of lateral bracing members on the out-of-plane distortion of the lateral plates. Further work also was undertaken during the experimental studies on the effectiveness of peening and gas tungsten arc remelting the fatigue-damaged connections and on the ability of drilled holes to arrest crack growth.

A total of 18 beams, each with three welded gusset plate details, were tested in fatigue with stress ranges of 6 to 15 ksi. Several other details were welded to the girder web in order to simulate beam flanges framing into a web plate. The results of these tests were used to assess the adequacy of the applicable provisions of the AASHTO specifications. In addition, the influence of lateral bracing on the fatigue performance of the attachments was evaluated. Recommendations for modifications to current practice are included in the report.

Additional NCHRP research is in progress at Lehigh University under Project 12-15(4), "Steel Bridge Members Under Variable-Amplitude, Long-Life Fatigue Loading." The results of this study will provide additional information on fatigue crack growth behavior of steel bridge members under randomly applied variable-amplitude loadings in the fatigue limit, extreme life region. The findings of NCHRP Project 12-15(4) are scheduled to become available late in 1983.

CONTENTS

ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 12-15(3) by the Fritz Engineering Laboratory, Lehigh University, with John W. Fisher, Professor of Civil Engineering, as principal investigator. The other authors of this report are Bernard M. Barthelemy, former Research Assistant (engineer with the French Technical Center for Steel Construction, Puteaux, France); Dennis R. Mertz, Research Assistant; and John A. Edinger, Research Assistant.

Special thanks are extended to Fritz Laboratory staff members Richard Sopko, who was responsible for photographic coverage, and John M. Gera, who was responsible for drafting the illustrations. Robert Dales and Charles Hittinger, laboratory foremen, and their staff provided valuable assistance during the test setup and actual testing. Sincere thanks are also due Hans Hausammann, Senior Research Assistant, who provided helpful advice, and Professor Alan W. Pense, who provided assistance on several occasions.

FATIGUE BEHAVIOR OF FULL-SCALE WELDED BRIDGE ATTACHMENTS

SUMMARY

The research described in this report is the result of studies performed under NCHRP Project 12-15(3). It is intended to provide information on further evaluating the fatigue resistance of welded attachments (particularly lateral connection plates) and on improving the fatigue resistance of details susceptible to damage from crack growth at weld toes. The research consisted of laboratory studies of welded attachments under various constant amplitude loads. Each test girder contained a number of welded attachments. For this test program, the girders were cycled until fatigue crack growth was detected at a detail. Having defined the fatigue life of the detail, an appropriate retrofitting technique was employed to attempt to increase the life of the detail. By using this procedure, the remaining welded details on a beam could be evaluated for fatigue life while the retrofitting technique was studied. On further cracking of a detail, the procedure was repeated. Eighteen full-size beams were tested during the program with various web gusset plates, flange gusset plates, flanges framing into or piercing through the girder web, and gusset plates attached to the flange surface. The retrofitting techniques studied included peening of the weld toe, gas tungsten arc (GTA) remelting of the weld toe, and drilling of holes to eliminate the crack tips.

A pilot study was also carried out on longitudinal stiffener welds containing lack of fusion regions in order to study the crack propagation mechanism.

Four types of lateral connection plates welded to the girder web were investigated. All performed similarly, and satisfied their current fatigue classification of Category E. The effect of secondary stresses or displacements in the girder web due to lateral bracing connected to the gusset plates was insignificant as was the relative stiffness of the lateral bracing system.

Gusset plates with radiused transitions ($r = 2$ in. or 6 in. (51 mm or 152 mm)) attached to the flange tip by means of a groove weld provided a fatigue resistance comparable to Category C. Such gusset plates with no end transition, but with a ground end, resulted in behavior equivalent to Category D. This indicates that a Category E groove-welded attachment can be improved significantly by simply grinding the ends of groove-welded flange gusset details.

The groove-welded details experienced better fatigue resistance than fillet-welded flange gussets. This further verifies the higher probability of discontinuities at fillet weld roots than in groove welds.

Gusset plates attached to the flange surface did not provide adequate fatigue resistance. The plates were in most cases severed from the flange through the weld root at stress range levels well below Category E.

Girder flanges framed into or piercing through the web provided fatigue resistance defined by the lower confidence limit of Category E or E′. The fatigue life decreased with increasing girder size and with coped end holes at the flange tip.

Retrofitting procedures of peening and gas tungsten arc remelting provided

good increases in life when applied when the cracks were shallow. Retrofitting with holes and tightened bolts provided good fatigue life increases under the proper circumstances.

The pilot study on longitudinal stiffener welds containing lack of fusion regions indicated that the fatigue crack that developed at the lack of fusion regions propagated through the web in a semicircular shape with crack radius near the tip of the longitudinal stiffener. Because of the inability to quantify the varying degrees of lack of fusion and the small number of fatigue tests, no direct comparison of fatigue resistance to other details was made.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

BACKGROUND

Experience with several highway bridges during the past decade has demonstrated the need for further studies to define the strength and behavior of a variety of attachments to beams and girders. The welded lateral connection plate of the Lafayette Street Bridge (*1*) has given evidence that serious damage can result when fatigue crack growth developes from a lack of fusion region in the welded connection of the gusset plate and the transverse stiffener. Numerous bridges have used comparable lateral connection plates welded to either the web or flange to connect the lateral bracing system to the longitudinal girders. Both groove welds and fillet welds have been used to connect the lateral connection plates to girder webs and transverse stiffeners. When attached to the girder flange, groove welds are most frequently used when the connection plate is welded to the flange tip. Fillet-welded connections have been used on a number of occasions when the lateral connection plate was attached to the top surface of the bottom flange.

The studies on beams with cover plates and various welded attachments (*2, 3*) have indicated that attachment length is a major factor governing fatigue strength. Most of these attachments are classified as Category E details in accordance with the AASHTO specifications (*4*). More recent studies performed under NCHRP Project 12-15(2) have shown that full-size cover plate details on beams with flange thickness greater than 0.8 in. (20 mm) have less fatigue resistance than Category E. As a result, a new lower bound fatigue resistance (stress Category E′) was developed from tests reported in Fisher et al. (*5*) and adopted by AASHTO in 1978 (see *Interim Specifications Bridges* (*4*)).

Hence, it has been desirable to evaluate the fatigue strength of several of these welded attachments and also to determine whether or not they can be retrofitted by peening or gas tungsten arc remelting the weld toe, as was examined in *NCHRP Report 206* (*5*) for small-scale and full-size cover-plated beams.

Also of concern has been the possibility of secondary out-of-plane displacement-induced stresses in the web gaps of lateral connection plates that frame around transverse stiffeners and the possible distortion of the lateral connection plate as a result of the relative movement of adjacent girders in bridge structures. These distortions have the potential of causing out-of-plane displacements in the lateral connection plates at the small gaps that exist between the ends of the lateral bracing members and the girder web. Fisher et al. (*5*) reported that out-of-plane displacements in web gaps with lengths between 10 and 20 times the web thickness resulted in fatigue cracks if the cyclic movement exceeded 1/1000 times the gap length. Such displacements appeared possible in the lateral connection plates.

The available studies on lateral connection plates for bracing members are not very extensive. The basic studies reported in Fisher et al. (*3*) only simulated gusset plates and no members framed into the attachment. A few members were tested in Switzerland by Hirt (*6*), and several beams were tested by Comeau and Kulak (*7*). Both of these studies considered lateral attachments on small beams comparable to those used in the studies carried out under NCHRP Project 12-7 (*2, 3*). The studies by Kulak were undertaken partially as a result of high stresses that were observed in the gusset plates of the Conestoga River Bridge in Ontario (*8*). A few other tests are available on simulated specimens (*9*). These tests were used when classifying welded details (*3*).

The pilot studies carried out on web details that simulated penetration of a girder flange through the web of an intersecting member documented in *NCHRP Report 206* (*5*) have shown the severity of this type of connection. Fatigue cracking developed at stress range levels well below design Category E′. This indicated a need for further studies of comparable details to ascertain the variability and significance of slight variations in geometry such as end copes and members welded to each side of the girder web at the penetration. In addition, it was desirable to evaluate the resistance of large flanges that frame into the girder web—whether attached by fillet or groove welds. A large number of bridge structures in service are known to have such conditions.

OBJECTIVES AND SCOPE

On the basis of the need for additional studies of welded attachments to the web or flange, this project was initiated with the following objectives:

1. To evaluate the fatigue strength of gusset connections welded to the web or flange and connected to the lateral bracing system and to determine whether or not they can be retrofitted by peening, gas tungsten arc remelting of the weld toe, or drilling holes.
2. To determine whether or not flange thickness and out-of-plane stiffness of the flange influence the fatigue behavior of lateral connection gusset plates.
3. To investigate the nature of the weld and near-fusion-line flaws that are characteristic of groove welds that will be used to connect the lateral gussets to the flange in contrast to the fillet welds used in NCHRP Project 12-15.
4. To evaluate the fatigue strength of several welded web attachments, such as the connection of a beam flange to a girder web. One such attachment consists of a flange plate passing through the web of a girder and welded on one or both sides. An alternate detail has the flange plates welded against both sides of the girder web with no penetration of the girder web by the flange.
5. To evaluate the fatigue behavior and crack growth characteristics of a lack-of-fusion condition in a longitudinal stiffener transverse groove weld as observed in the field on the Quinnipiac River Bridge.
6. To review and evaluate methods for improving fatigue life and arresting the progress of fatigue damage that occurs at the weld toes of severe notch producing details where the probability of failure is highest—the experimental variables to include crack length at the time of repair or improvement, stress range, and maximum stress.

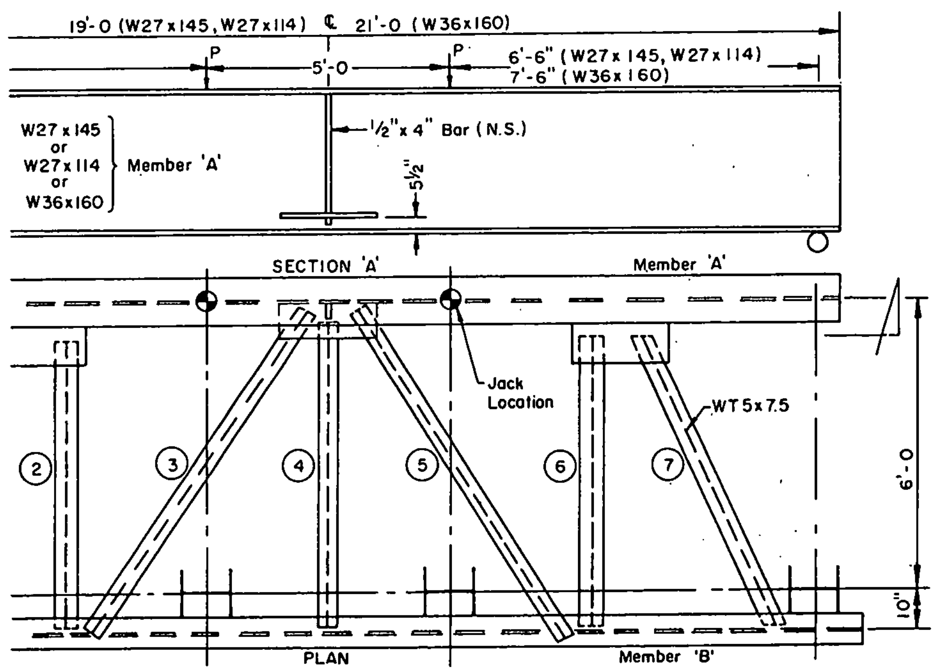
During the experimental research program, 18 full-size beams fabricated from ASTM A36 steel were fatigue tested to determine the fatigue strength and behavior of the aforementioned details and to further evaluate techniques intended to extend their fatigue life. Three such retrofitting techniques were evaluated during this test program. They consisted of: (1) remelting the weld toe using the gas tungsten arc process (GTA; also commonly referred to as the tungsten inert gas process, TIG); (2) airhammer peening the weld toe to introduce compression residual stresses, and (3) drilling holes (1 in.) at the crack tips to remove the crack tip (in some cases tightened high strength bolts were installed).

The beams were tested under various constant amplitudes of cyclical loading. After cracks developed at a particular detail defining its fatigue life, this detail was retrofitted and the test proceeded to further determine the fatigue life of other details on the beam and to evaluate the effectiveness of the retrofitting technique used.

RESEARCH APPROACH

Laboratory Tests

The 18 full-size beams tested during the course of this project were rolled W27 x 145, W27 x 114, and W36 x 160 beams of A36 steel. The W27 x 145 and W27 x 114 beams were 19 ft (5791 mm) long and tested on 18-ft (5486-mm) span under 4-point loading as shown in Figure 1. The W36 x 160 beams were 21 ft (6401 mm) long on



Note: Dimensions are in English units

Figure 1. Schematic of test setup.

20-ft (6096-mm) spans. Each beam had one gusset plate welded on the web at midspan and two flange gussets welded to the tension flange in each shear span. All of these details were either fillet welded or groove welded onto the beams. Lateral bracing members connected these details to a stationary beam approximately 6 ft away (Fig. 2). The web gusset details were grouped into four types, as shown in Figure 3. The flange gusset details were grouped according to the radius at the end of the connection with radii of approximately 0 in., 2 in. (5 cm), and 6 in. (15 cm), as shown in Figure 4. Most of the tested beams had additional details in the form of attachments to the web and/or tension flange. These details are shown in Figure 5.

The first two such details consist of either two 15¾ × 4 × 2-in. (40 × 10 × 5-cm) plates welded to both sides of the web (see supplementary detail 1 in Fig. 5) or one 15¾ × 8 × 2-in. (40 × 20 × 5-cm) plate inserted through the web and fillet welded in place on both sides (see supplementary detail 4 in Fig. 5). The two plates welded to both sides of the web were attached using groove or fillet welds. These details were positioned in the shear spans to achieve a given stress range value. Another detail consisted of two 23½ × 3 × ⅜-in. (60 × 7.5 × 0.95-cm) plates welded together with varying degree of incomplete penetration welds and then fillet welded to the web (see supplementary detail 3 in Fig. 5). The final supplementary detail 2 consisted of two (23½ × 8 × ½-in.) (60 × 20 × 1.25-cm) plates welded on the lower flange opposite the gusset plates welded to the tension flange (radius of 0 in.). Fillet welds with ⅜ in. (0.95-cm) legs were placed transversely at each end of the plate terminating ½ in. (1.25 cm) from the flange tip. No longitudinal welds were used.

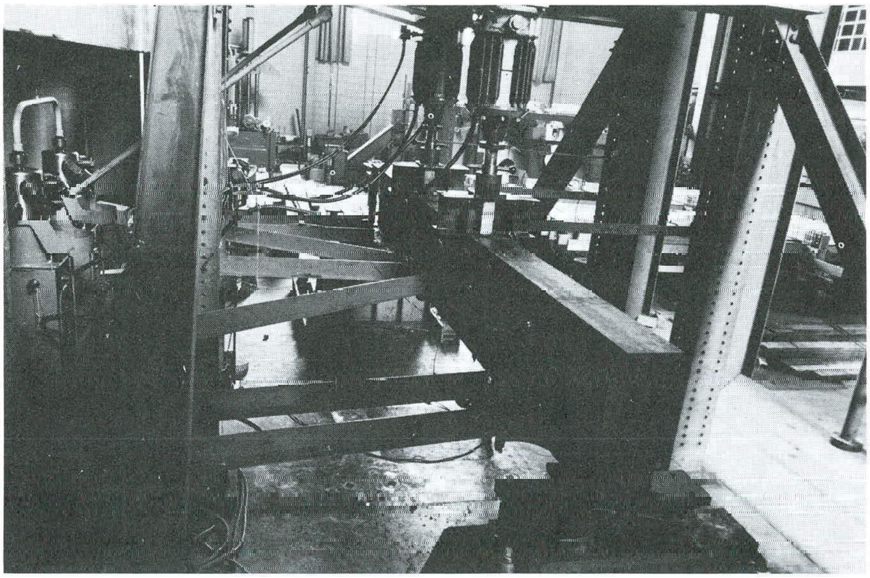


Figure 2. Amsler setup for W27x114 and W27x145 beams.

Coupon tension specimens taken from the girder flanges (ASTM-A370) gave yield points of 37.0 ksi (255.1 MPa), 35.8 ksi (246.5 MPa), and 38.7 ksi (266.8 MPa) for W27 x 145, W27 x 114, and W36 x 160 cross-sections, respectively. The tensile strength was 60.4 ksi (416.4 MPa), 63.8 ksi (440.0 MPa), and 63.1 ksi (435.0 MPa) for the W27 x 145, W27 x 114, and W36 x 160 flanges, respectively.

For this test program, stress range was selected as the controlled stress variable. The normal flexural stresses in the beam web at the bottom of the web gusset at midspan and at one end of the flange gussets were equal and used as the controlled stress variable. Most of the tests were made at a stress range of 6, 9, 12, or 15 ksi (41.3, 62.0, 82.7, or 103.4 MPa). A few tests were conducted at slightly higher levels of stress range. The experiment design is shown in Figure 6.

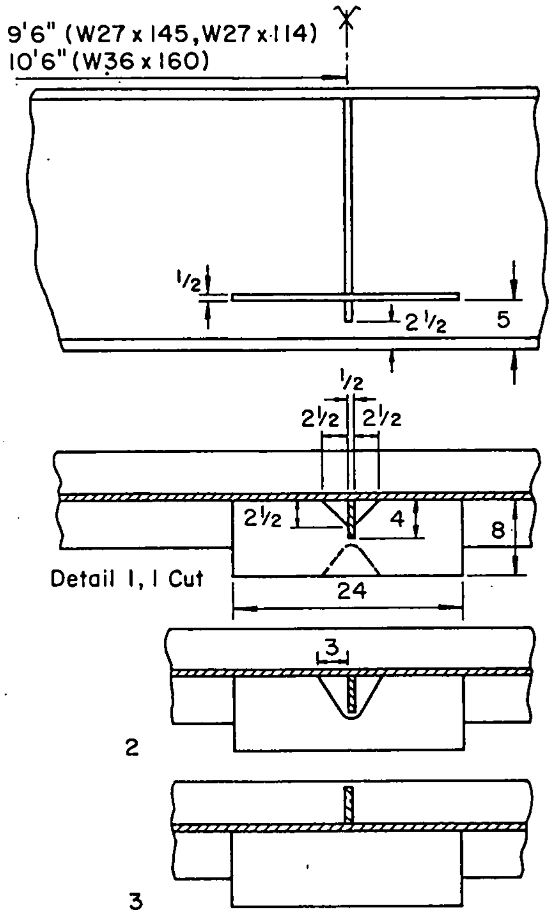
The tests proceeded until visible crack growth was detected—usually with 10X magnification. At this stage, the crack was retrofitted and the test continued, thereby providing fatigue life information on the other tested details as well as on the retrofitting procedures.

All beams were fabricated by Bethlehem Fabricators, a local fabrication shop. The fabricator was instructed to use normal fabrication and inspection procedures. Each rolled section was produced from the same heat. The supplementary details were welded by Fritz Engineering Laboratory technicians to the same level of quality as the fabrication.

Figure 7 shows the ends of the flange attachments. The "0" radius end (Fig. 7(a)) shows the small ground radius that was furnished for this attachment. Figures 7(b) and (c) show the ends of typical 2-in. (50-mm) and 6-in. (150-mm) radiused flange attachments.

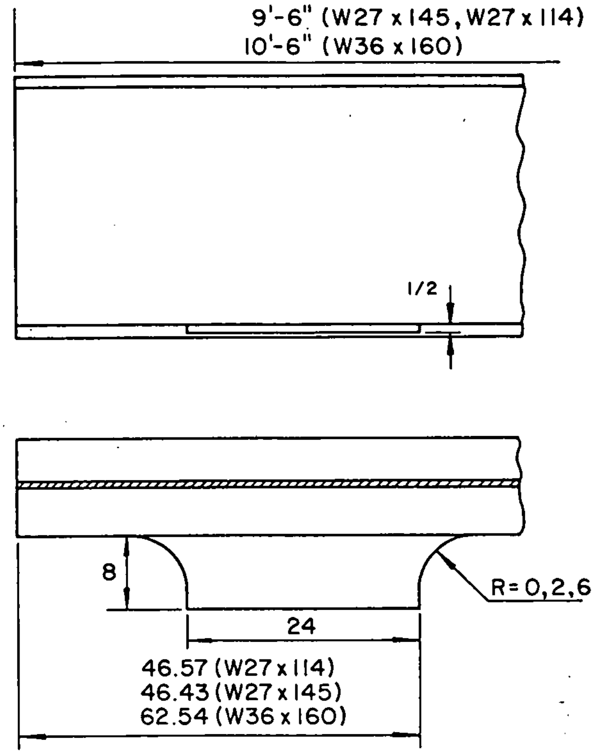
Test Procedures

All beams were tested on the dynamic test bed in the Fritz Engineering Laboratory at Lehigh University. In



All Dimension in Inches (1" = 25.4 mm)

Figure 3. Schematic of web gusset details.



Note: Dimensions are in English units

Figure 4. Schematic of flange gusset details.

the case of the W27 x 114 and W27 x 145 beams, the cyclical load was applied with an Amsler variable stroke hydraulic pulsator with two 110-kip (500-kN) jacks (see Fig. 2) operated at a constant frequency of 260 or 520 cycles per minute (cpm), depending on the stress range. The W36 x 160 beams were tested using an MTS system consisting of two hydraulic jacks each with a capacity of 195 kip (889.6 kN). The loading cycle was sinusoidal; the minimum applied stress was always tensile. All testing was carried out at room temperature between 60 F and 80 F (15 C and 27 C).

Prior to subjecting the test beam to cyclic loading, lateral bracing members were bolted or clamped to the lateral connection plates, as shown in Figure 2. Figures 8 and 9 show typical connections to the web gusset plate at the midspan of each girder. The flange details had two transverse members attached as shown in Figure 10. The lateral bracing members were welded tee sections that provided bending stiffness comparable to those used in a typical bridge structure.

Each beam was cycled until a crack was detected at a detail. The examination was made visually with 10X magnification. Having defined the fatigue life of the detail, an appropriate retrofitting procedure was used, increasing the life of the cracked detail. The test was continued in order to determine the fatigue life of the other details and the effectiveness of the retrofitting details. This procedure was repeated as cracking developed at other details, and this permitted the fatigue strengths for all details on a beam to be determined.

Retrofitting Procedures

The gas tungsten arc process (GTA) involved remelting existing metal at the weld toe (*5*). The tungsten electrode was moved manually along the weld toe melting base metal and fillet weld, thus removing the crack. Nonmetallic intrusions present along the toe are removed by this process and stress concentrations reduced. In the presence of a small crack, a small, yet sufficient, volume of surrounding metal was melted to incorporate the crack. On solidification, the crack ceased to exist when the procedure was successful. GTA remelting of the weld toe was examined in Fisher et al. (*5*) for cover-plated beams.

The welding apparatus used in this study was a 300-amp, DC power source with drooping V-I characteristics. High frequency was used to start the arc. A Linde HW-18 water-cooled torch with a 0.156 in. (4 mm) diameter, 2 percent thoriated tungsten electrode was used.

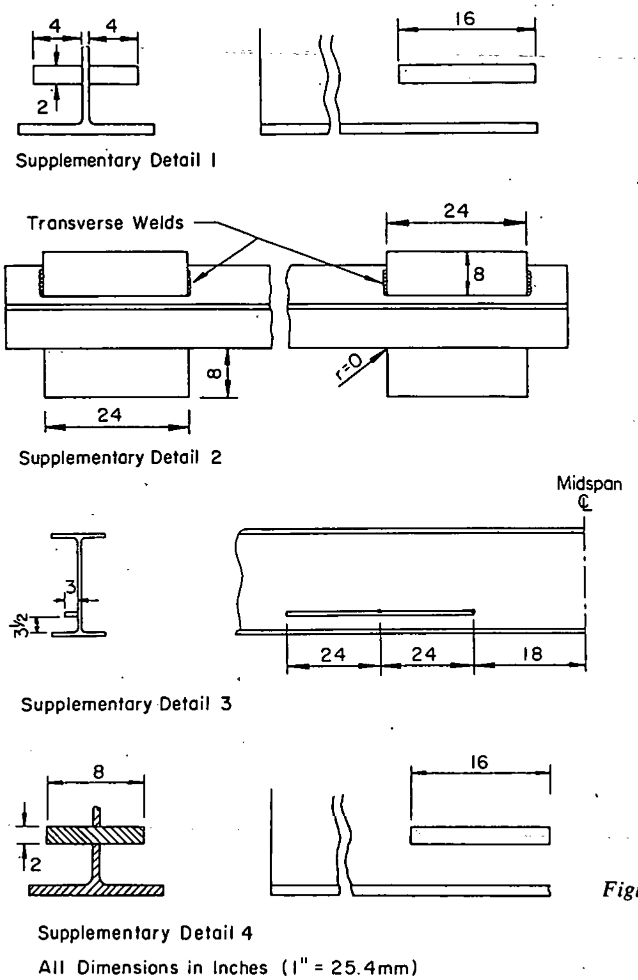


Figure 5. Supplementary details 1, 2, 3, and 4.

Cross Section	No.	Web Gusset Types			Flange Gusset R(mm, in)	Fillet Welded	Groove Welded	Insert	Flange Plate
W27x145	5	X			0				
	2	X			0	X			X
	3		X		5 (2)	X			
	4		X		5 (2)	X			
	1			X	15 (6)				
	9			X	15 (6)		X	X	
W27x114	14	X			0				X
	13	X			0			X	X
	16		X		5 (2)		X	X	
	18		X		5 (2)	X		X	
	10			X	15 (6)		X		
	11			X	15 (6)			X	X
W36x160	15	X			0			X	
	8	X			0		X	X	
	7		X		5 (2)		X		
	6		X		5 (2)	X			
	12			X	15 (6)			X	X
	17			X	15 (6)			X	

Figure 6. Experiment design.

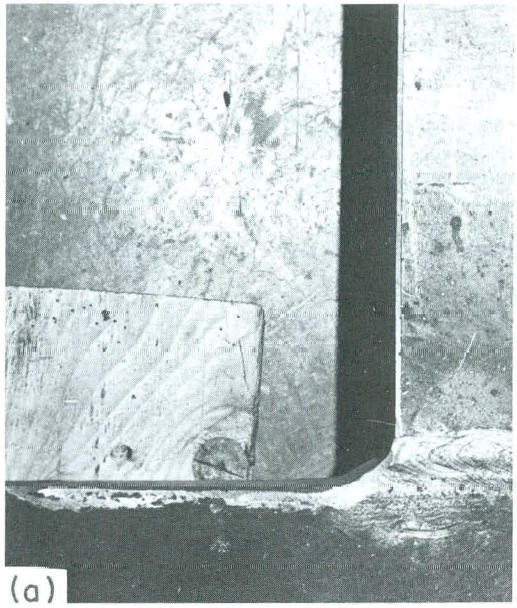
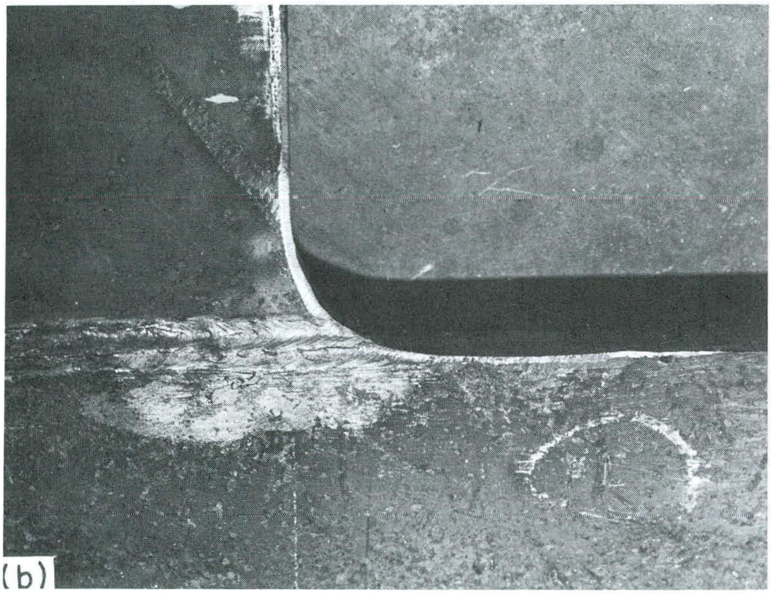
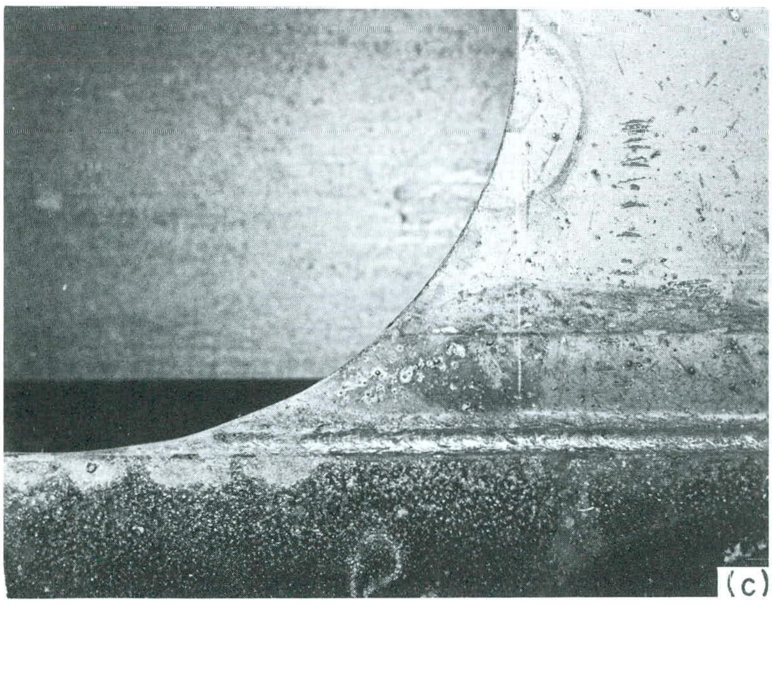


Figure 7. Flange gusset: (a) 0 radius; (b) 2-in. (50-mm) radius; (c) 6-in. (150-mm) radius.





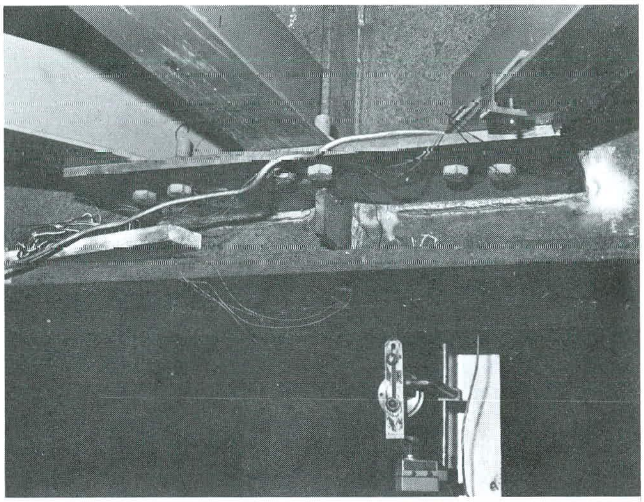


Figure 8. Lateral bracing connected to web gusset (bottom view).

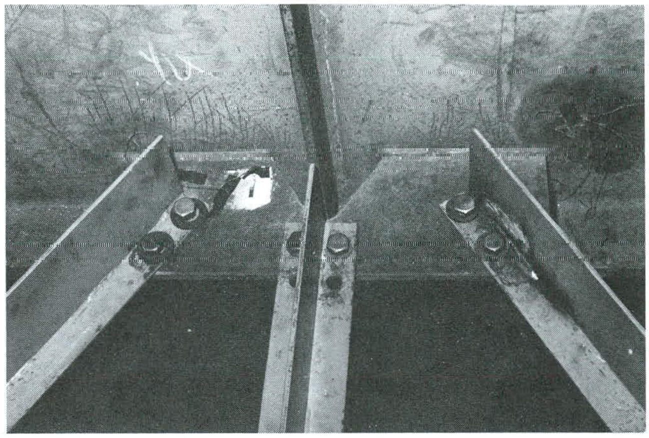


Figure 9. Lateral bracing connected to web gusset (top view).

The depth of the remelting zone was critical to the success of this method. Insufficient depth of penetration will leave a crack buried below the surface, resulting in premature failure. Such failures, along with cracks initially deemed unsuitable because of a relatively large depth, were treated usually by drilling holes. Figure 11 shows a GTA-remelted weld toe.

Peening, an established method to improve performance, has been shown to be applicable for improving both small specimens and beams (*NCHRP Report 206*). In this study, the weld toe was plastically deformed by mechanical air-hammer peening. Compressive residual stresses were introduced, preventing the full tensile stress range from being effective at the peened area. Thus, both crack initiation and growth were influenced.

Peening was performed with an Ingersoll-Rand model 1940 pneumatic air hammer operated at 25 psi (0.17 N/mm^2). The peening tool is shown in Figure 12. When a crack was visually apparent (10X magnification) peening was continued until the crack was no longer visible and the weld toe became smooth. An example of a weld toe treated by peening is shown in Figure 13.

Larger fatigue cracks, for which GTA-remelt and peening were deemed inappropriate, or which had recracked, were retrofitted by drilling holes at the crack tips. For the details studied herein, 0.5- or 1.0-in. (13-mm or 25-mm) holes were drilled into the web (Fig. 57). All holes were drilled by using a magnetic base drill.

The success of this method was dependent on the crack length and magnitude of applied stress range. In many cases the crack reinitiated from the hole and grew further. For larger cracks, where drilling the hole was not considered adequate to ensure no further initiation, preloaded high strength bolts were installed in the drilled holes. Earlier studies had demonstrated that this retarded crack initiation.

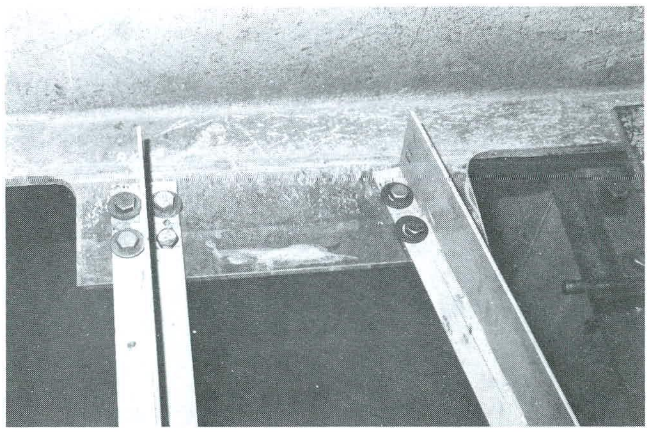


Figure 10. Lateral bracing connected to flange gusset.

Figure 11 GTA remelted web gusset weld.

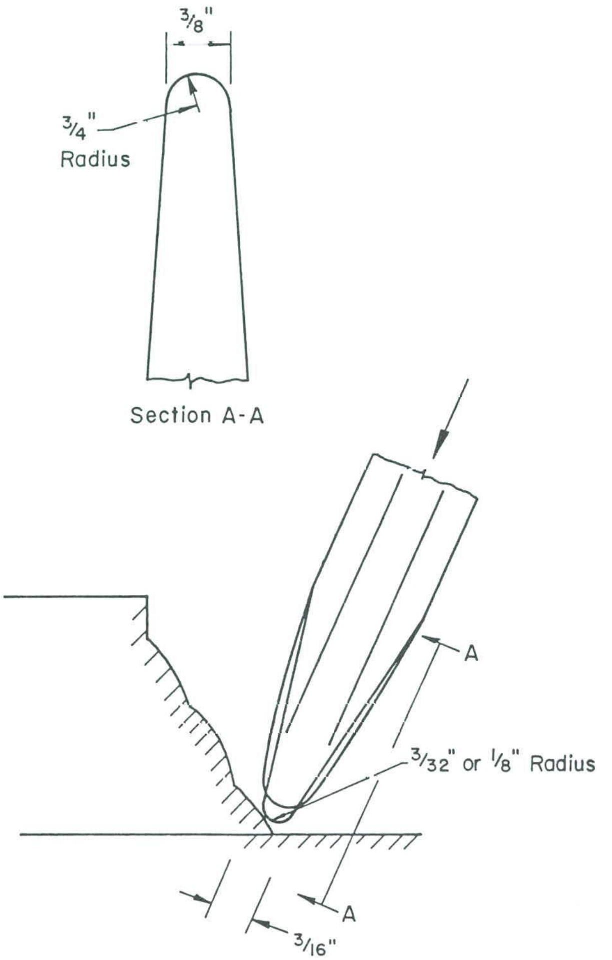


Figure 12. Schematic of peening tool (from British Naval Construction Research Establishment (15)).

Figure 13. Peened weld.

CHAPTER TWO

FINDINGS

The findings of NCHRP Project 12-15(3) are summarized in this chapter. A detailed evaluation of the experimental data is given in Chapter Three. Detailed documentation of the test results is provided in Appendix A.

DETECTION OF CRACKS AT WELD TOES

1. Visual inspection with 10X magnification and dye penetrant revealed the presence of cracks as small as 0.25 in. (6 mm) in length.

2. Visual and dye penetrant inspection did not provide a means of ascertaining the crack depth; however, employing an empirical relationship between crack width and depth provided reasonable crack depth estimates.

3. Although it was possible to retrofit shallow depth cracks after initial detection during this project, field detection of weld toe cracks to enable retrofitting will be more difficult.

FATIGUE BEHAVIOR OF WEB GUSSET PLATES

1. The toe of the transverse fillet weld joining the web gusset plates to the web provided fatigue crack initiation sites. Fatigue cracking initiated at this site because of the stress concentration that developed as a result of the geometric conditions and the greater probability of microscopic discontinuities at the fillet weld toe.

2. All four types of gusset plate configurations tested

either equaled or exceeded the fatigue resistance provided by Category E of the AASHTO specifications.

3. Relatively slight differences in fatigue resistance of the four details were found. However, a large variation in fatigue resistance as a function of applied level of stress range was observed.

4. All the cracks that were detected developed at the lateral gusset plate ends. None were detected in the girder web gap adjacent to the transverse stiffener.

5. Variation of the flexural rigidity of the lateral bracing member did not affect the fatigue resistance of the web gusset plate details.

6. Finite element models at the web gusset details indicated that a reduction of the stresses in the web at the transverse stiffener resulted from the presence of the lateral connection plate. Therefore, cracking at this location would not be expected and did not develop.

7. During the experimental testing, no cracks were detected in the transverse welded connections between the gusset plate and transverse stiffener. On completion of the test, destructive inspection revealed cracks in two of the four details investigated.

FATIGUE BEHAVIOR OF FLANGE GUSSETS

1. Detectable fatigue cracking only occurred at the ground "0" radius details. Out of a possible 24 crack sites, three locations were visually detected as cracked, while four more cracked locations were found using ultrasonic procedures.

2. Destructive testing and subsequent studies of the fractured surfaces revealed that subsurface defects were the origin of crack growth.

3. All of the "0" radius details with ground ends exceeded the lower confidence level provided by Category E.

4. No detectable fatigue crack growth occurred at the 2-in. (50-mm) and 6-in. (150-mm) radius details.

5. Use of fillet welds instead of groove welds to attach gusset plates to the flange increases the probability of crack initiation sites in the form of weld root flaws. Tests on 4-in. (100-mm) radius transition details attached to the girder web by fillet welds provided fatigue resistance comparable to Category D, which was in accord with the AASHTO standard specification provisions. None of the 2-in. (50-mm) and 6-in. (150-mm) radius transition details attached to the girder flange by groove welds developed fatigue cracks within the same cyclic loading.

FATIGUE BEHAVIOR OF GUSSET PLATES ATTACHED TO FLANGE SURFACE

1. Cracking at such a detail, a plate attached to the beam flange by transverse welds alone, developed from the weld root, propagating through the entire width of the transverse weld. Because the maximum weld size is the plate thickness, it is not possible to avoid the problem by increasing the weld size.

2. In most cases crack development resulted in complete severing of the gusset plate from the girder flange and provided a relatively low fatigue resistance comparable with design Category E′ or less.

3. In one test, cracking also developed in the beam flange from the transverse weldment. The fatigue resistance of this detail was between Category E and E′.

4. The limited testing that was carried out suggests that with greater flange thickness, lower fatigue resistance is encountered.

5. Use of transverse welds alone in attaching gusset plates to the flange surface does not provide any improvement in fatigue resistance. This detail was examined to establish whether or not longer details could be improved by removing the longitudinal welds.

FATIGUE BEHAVIOR OF WEBS WITH GIRDER FLANGES FRAMING INTO WEB OR PIERCING THROUGH WEB

1. Tests conducted on beams with thin webs ($t_w = 0.25$ in. (6.4 mm)) yielded approximately the same fatigue resistance, whether the plates framed into the web or pierced through.

2. Test results for these smaller beams were in agreement with design Category E, and the results for larger size girders tend to fall at or below the lower confidence limit provided by Category E. Category E′ better represents the fatigue resistance of thick flanges welded to the surface of girder webs.

3. Semicircular end copes at the tips of the thicker (2-in. (50-mm)) flange plates that pierced the web provided a fatigue resistance level less than Category E′ (see Fig. 47).

4. Welding on both of the web sides of details which penetrated the web reduced the unfused region of the detail thereby improving fatigue resistance to Category E′.

FATIGUE BEHAVIOR OF LONGITUDINAL STIFFENER WELDS CONTAINING LACK OF FUSION REGIONS

1. The fatigue crack propagation that developed at the lack of fusion regions indicated that the crack propagated through the web in a semicircular shape with the crack radius near the tip of the longitudinal stiffener.

2. Because of the inability to quantify the varying degrees of lack of fusion and the small number of fatigue tests, no direct comparison of fatigue resistance to other details was made.

RETROFITTNG PROCEDURES

1. When the entire depth of a relatively shallow crack could be successfully remelted, GTA remelting procedures greatly improved the fatigue resistance of cracked details (up to a 75 percent improvement).

2. When the entire depth could not be treated because of limitations in depth of penetration, the embedded cracks quickly grew out of the remelted area and the retrofit was not effective.

3. Greater retrofitting success was encountered at the thicker details because by the nature of the geometry of the crack it was easier to detect smaller cracks. As a result, peening or GTA remelting could be attempted with a greater hope of success.

4. When a crack was detected in its very initial state of

development, and it was still relatively shallow, peening showed a great increase in fatigue life (up to 240 percent improvement). However, because the crack must be very shallow in order to succeed when retrofitting by peening, most cracks when initially detected were deemed too deep to peen (greater than 0.125 in. (3 mm)) and were therefore treated by GTA remelting in an attempt to ascertain the limits of the method.

5. Placement of the holes at crack tips is paramount to the success of retrofitting by drilling holes. If the actual crack tip, which can be difficult to locate, lies beyond the hole, crack propagation will be accelerated.

6. Improved resistance was obtained by placing tightened high strength bolts in the drilled holes producing a zone of compressive stress. This procedure limits the ability of the inspector to detect reinitiation of cracks until after the crack reaches a very severe state.

7. A threshold level of the ratio of the stress-intensity-factor fluctuation to the square root of the notch-tip radius ($\Delta K/\sqrt{\rho}$) less than four times the square root of the yield strength ($4\sqrt{\sigma_y}$, for σ_y in ksi) was required to ensure that cracks did not reinitiate at the welded details investigated and retrofitted by the drilling of holes.

APPLICATION

1. With the exception noted in item 5 below, web gusset plates can continue to be designed in accord with the AASHTO specification provisions for fatigue.

2. Existing structures which have gussets attached to the transverse stiffener and have intersecting welds are susceptible to cracks developing from the transverse fillet welds and web intersection as was demonstrated by the cracking that developed in the transverse weld connections of this study.

3. Gusset plates attached to the tips of flanges by groove welds were shown to develop the fatigue resistance provided by the AASHTO specification.

4. Small ground radii (0.2 to 0.4 in. (5 to 10 mm)) at the ends of rectangular gusset plates provide fatigue resistance equal to Category D.

5. Results of other research indicate that fillet-welded gussets with radius transitions cannot achieve the improvement suggested by the AASHTO specification for transitions greater than 6 in. (150 mm). Subsurface discontinuities in the transition region are much more likely at the fillet weld roots and limit the fatigue resistance. Therefore, all fillet-welded gussets with radius transitions of greater than 6 in. should be classified as AASHTO Category D. No improvement for greater radii should be considered.

6. Gusset plates should not be welded to the flange surface with transverse fillet welds alone. At least one longitudinal weldment is necessary to prevent the gusset plate from cracking from the flange.

7. Flanges that frame into girder webs—whether fillet or groove welded to the web surface—should be designed as Category E′ attachments when the flange thickness exceeds 1 in. (25 mm). Flanges less than 1 in. (25 mm) thick can be designed as Category E details.

8. When flanges pierce through girder webs, but are connected to each web surface by fillet welds, they should be designed for Category E′ fatigue resistance.

9. Drilled holes at the ends of fatigue cracks that develop at structural details can prevent further crack growth when the following relationship is satisfied:

$$\frac{\Delta K}{\sqrt{\rho}} < 4\sqrt{\sigma_y} \text{ (for } \sigma_y \text{ in ksi)}$$

$$\left[\frac{\Delta K}{\sqrt{\rho}} < 10.5\sqrt{\sigma_y} \text{ (for } \sigma_y \text{ in MPa)}\right]$$

where ΔK = stress-intensity-factor range, ρ = hole radius, and σ_y = yield strength.

10. Holes drilled at the ends of fatigue cracks should have the perimeter of the hole placed at the apparent crack tip (Fig. 57).

CHAPTER THREE

RESULTS AND EVALUATION OF EXPERIMENTAL DATA

The results of the experimental and theoretical work are summarized in this chapter. The fatigue behavior of several types of gusset plates attached to the web or flange and used to connect lateral bracing members is evaluated and discussed. Both the effect of the welded attachment to the main longitudinal girders and the influence of relative distortion of the gusset plate as the members deflected are examined. The fatigue behavior of several cases where large beam flanges pierce or frame into girder webs is examined, and their fatigue resistance is determined from the test results reported herein and other studies. A pilot study on the behavior and modes of crack growth caused by incomplete fusion in a transverse weld of a longitudinal stiffener similar to that detected on the Quinnipac River Bridge is discussed and summarized. Efforts to improve fatigue life through gas tungsten arc remelting, peening, and drilling holes are also reported. A hypothesized value for the crack reinitiation stress intensity threshold developed from simple tests (*10*) is compared to the calculated values provided by the fatigue-cracked web details considered in this study.

DETECTION OF CRACKS AT WELD TOES

During the course of this experimental study, considerable effort was made to detect the small semielliptical surface cracks that formed at the termination of the groove and fillet welds. The primary crack detection method employed was visual inspection with a 10X magnification and dye penetrant. Previous experience had shown that this method was more reliable than ultrasonic examination and other inspection methods.

This detection procedure revealed the presence of cracks as small as 0.25 in. (6 mm) in length as shown in Figure 14. It did not provide a reliable means of ascertaining the depth of shallow cracks. A reasonable estimate of the crack depth could be made in most cases by employing an empirical relationship (*11*) between crack width and depth at the time of observation. (The width is between four and eight times the depth.) Often two or more cracks coalesced and formed a longer shallow crack.

Because these tests were undertaken under controlled laboratory conditions, it is probable that comparable detection capability will not be possible under field conditions unless very skilled personnel are available and trained to look at the sites of crack initiation.

FATIGUE BEHAVIOR OF WEB GUSSET PLATES

A total of 18 A36 steel beams were tested, including six beams each of W27 x 114, W36 x 160, and W27 x 145 cross sections. The selected beams provided varying lateral stiffnesses of the tension flange in the ratio of 1:2:3, respectively. Their web thicknesses were typical of the webs used in plate girder bridges. All girders had lateral bracing attached to each lateral bracket as shown schematically in Figure 1. WT5 x 7.5 lateral bracing members were used.

The length of the lateral bracing member was varied between 5 ft (1.5 m) and 7 ft (2.1 m) providing a relatively stiff bracing member. The longer length provided about the same stiffness (L/EI) as the members used in Dorton (*8*). The support beam at the other end of the bracing (see Fig. 1) had more than six times the stiffness of the test beam. This was accomplished by providing intermediate support points and a larger girder (W36 x 230). In addition, the gap between the lateral gusset-web weld and the nearest bolt in the lateral connection plate was varied. Figure 15 shows the test setup, and a view of the lateral bracing can be seen from Figure 2. The web details used for this study are shown in Figure 3. Detail 1 is a typical detail used in the past at transverse stiffeners. Detail 2 has been suggested as a possible alternative that avoids placing a weld perpendicular to the stress field (*12*). Detail 3 shows a detail often used when no transverse stiffener is present. An additional web gusset type, detail 1-cut, was later added to the study. In an attempt to simulate the type of detail fabricated by welding two separate plates into the corners formed by the girder web and the transverse stiffener, detail 1-cut was conceived. An existing detail 1 gusset plate was altered by cutting a vee notch (as shown schematically in Figs. 3 and 18). Since the weld was not disturbed, two distinct plates were nearly simulated. This pilot study yielded valuable results with a minimum amount of alteration. All web details were attached to one side only using fillet welds. The gusset plate to lateral stiffener connection is shown in Figures 8 and 9. The beams were tested so that the stress range at the end of the lateral connection plate was the control variable. Stress ranges of 6, 9, 12, 15, and 21 ksi (41, 62, 83, 103, and 145 MPa) were used for the study (see Fig. 6).

Figure 14. 0.25-in. (6-mm) crack at flange framed into web, groove welded.

The fatigue cracks initiated at the toe of the transverse fillet weld joining the gusset to the web. This was the point of greatest intensity due to the applied stress, the geometry of the fillet-welded connection, and the microscopic discontinuities at the weld toe. Cracks normally formed in the weld toe region, as shown in Figure 16, and propagated into the web.

Figure 17 shows the formation of typical fatigue cracks into the girder web. These cracks propagated through the web thickness as semielliptical shaped cracks. Thereafter they became through cracks and were arrested by the placement of drilled holes as illustrated in Figure 17(b).

The experimental data acquired from the web gusset plates are summarized in Figure 18. All four types of gusset plate connections that were tested equaled or exceeded the fatigue resistance provided by Category E of the AASHTO specification. Destructive testing of these welds afterwards revealed that in all cases cracks were present but had not propagated to the surface, as shown in Figure 19. Detail 1 demonstrated slightly better fatigue resistance than web details 2 and 3. However, the slight differences were not significant. A relatively large variation in fatigue resistance was observed at each level of stress range that was examined. Because detail 1-cut was made by cutting out a notch that did not extend to the transverse weld of the stiffener, stress was transferred not only through the weld-stiffener connection but also through a small area of the web gusset plate that was not removed. When this type of detail is fabricated by welding two separate plates into the corners formed by the girder web and the transverse stiffener, a more severe condition exists and all of the stress will be transmitted across the weld.

As can be seen in Figure 18, the removal of a portion of the web gusset plate did not alter the fatigue resistance of the web gusset plate. The details identified as 1-cut provided the same fatigue resistance as the other web gusset connections. None of the Type 1 and Type 2 details developed cracks in the girder web adjacent to the transverse stiffeners. All cracks that were found developed at the ends of the lateral gusset plates.

The test data also demonstrated that changes in the bending stiffness of the lateral bracing members did not influence the fatigue resistance. The L/EI of the lateral bracing was varied from 5/EI to 7/EI without any discernible influence.

In order to evaluate the lateral bracing and the stresses in the girder web at the web gusset connection, the test girders and lateral system were modeled using the finite element method. Three two-dimensional discretizations using the substructuring technique were used to assess the stress fields in the girders and minimize the computation time.

First, an analysis of the entire half-span was performed, as shown in Figure 20. Plate bending elements were used in the web, the lower flange, and the gusset plates. Plane stress elements were used in the stiffener and the upper flange. The lateral bracing members discretized by beam elements were connected to external corners of the gusset plates and fixed at the other end. The effects of the type of connection between the gusset plate and the lateral bracing, as well as those of an elastic support of the other end, were studied separately.

The second 2D mesh only considered the part of the beam between the cross-section under the load and the midspan. The lateral bracing members connected to the central gusset were suppressed, and the displacements and

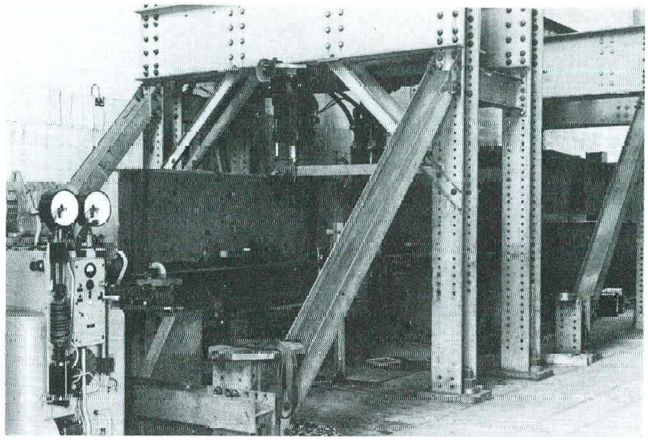


Figure 15. MTS setup for W36x160 beams.

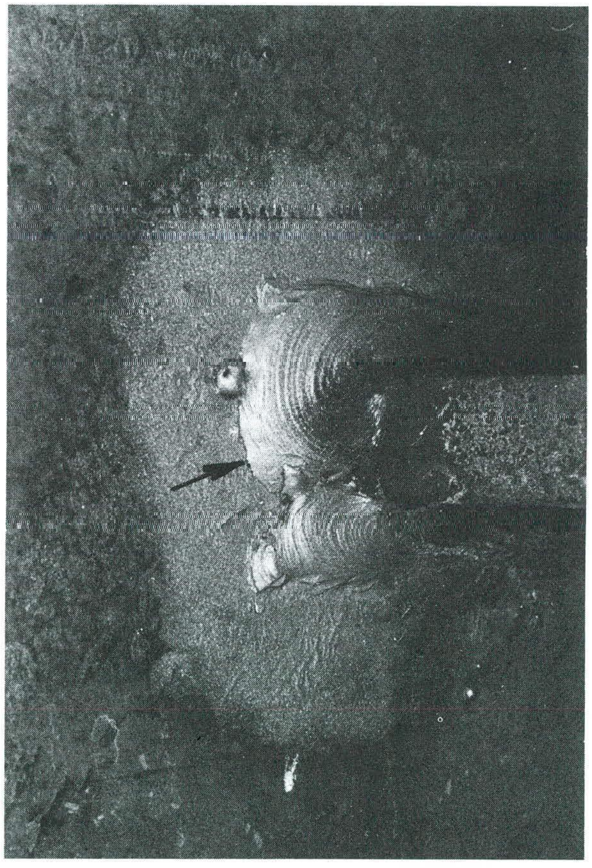


Figure 16. Weld toe crack at end of web gusset.

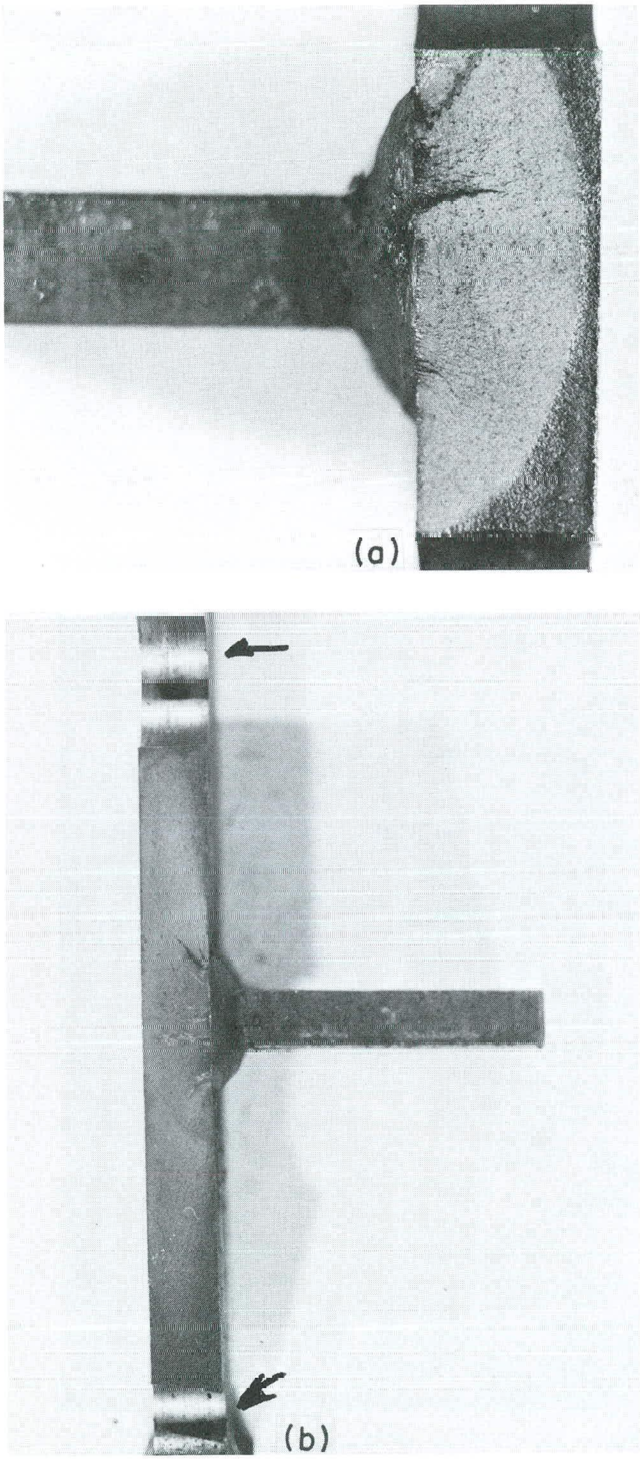


Figure 17. Fatigue cracks in girder web from web gusset (arrows point to drilled hole).

rotations computed in the first analysis were induced through boundary elements at external corners of the gusset plate. The displacements and rotations were applied to the nodal points of the cut-off sections. This mesh for the Type 1 detail is shown in Figure 21.

The third step was a 2D analysis of each detail in the areas judged most critical by experience. These areas are shown in Figure 22. For detail 1, three critical locations were possible at the web-to-gusset and gusset-to-stiffener welds. In case of detail 2, only the locations along the web-to-gusset remained because the stiffener is not welded to the gusset plate. Figure 23(a) shows the mesh used for locations b and c for the Type 1 detail. Figure 23(b) shows the mesh used for location b for the Type 2 detail.

The stresses in web elements 1 to 4 (see Fig. 23) for a nominal stress range of 12 ksi (82.7 MPa) at the end of the gusset-to-web connection are summarized in Table 1.

TABLE 1

STRESSES AT WEB GAP MP_a

Web Detail	Element 1	Element 2	Element 3	Element 4
1	40.3	48.0	67.9	53.9
2	41.9	57.1	79.8	63.8

The element size in the final analysis shown in Figure 23 was 65 mm. This indicates that the stress range in the web gap (b) at the transverse stiffener was always less than the stress range at the end of the gusset plate (a).

The results indicate that the lateral connection plate reduces the stresses in the web at the transverse stiffener. Hence, the fact that no cracking was detected in the beam web at the ends of the weldments at the web gap would be expected.

Because the out-of-plane movement of the gusset plate is induced by the bending rigidity of the bracing members, the type of connection between the bracing members and the gusset plate is of major importance. The following factors appear to be the main parameters influencing this connection:

1. The length of the connection and the gap between the end of the bracing member and the web plane.
2. The type and the size of the fasteners (bolts or welds).
3. The angle between the bracing members and the girder axis.

Figure 24 shows these parameters.

A 2D mesh was used to examine the effect of the type of connection by changing the attachment points of the bracing members that were discretized using beam elements. These attachment points are numbered 1 to 10 in Figure 25 which shows the gusset plate discretization. The conditions considered were the gusset plate and stiffener welded together on the same side of the web (details 1, 1-cut, and 2).

Four different connection lengths were considered. They were denoted cases 1 to 4, as follows:

Case	Connecting Points	Gap (cm)
1	1, 6	5.84
2	3, 8	10.92
3	4, 9	13.46
4	5, 10	21.08

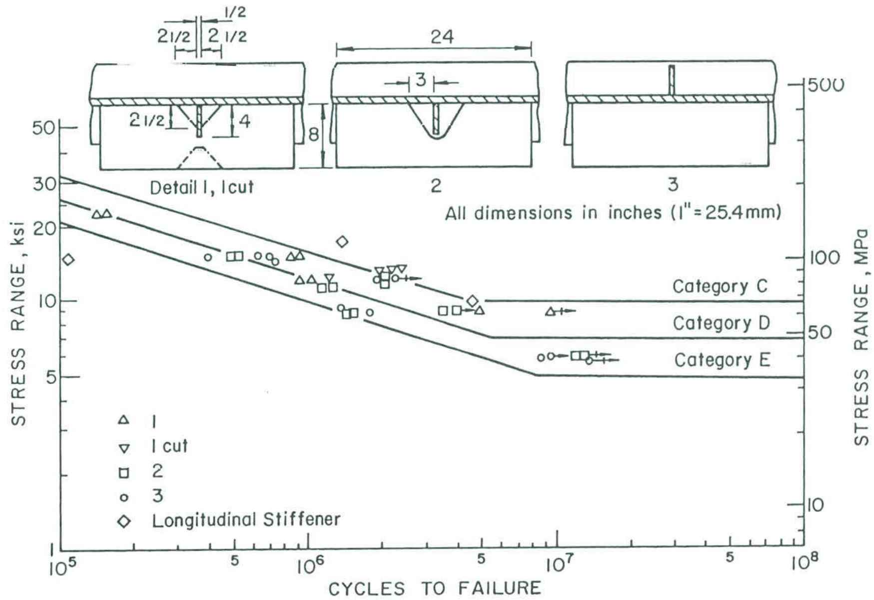


Figure 18. Fatigue life of web gusset plates.

In addition, the behavior of the connection was studied with a zero gap. The largest displacements were always obtained at the web-to-gusset weld toe. Examination of these results showed that the displacements were not sensitive to the position of the stiffener or to the gap between the web and the end of the bracing member.

A comparison was also made of the out-of-plane bending moments along the web-to-gusset weld. These moments are summarized in Figure 26. Elements E1 to E6 are the six plate bending elements along the web-to-gusset weld toe (see Fig. 25). They indicate that the smaller gaps result in smaller bending moment, but the stresses induced by the out-of-plane movement stayed at a very low level in all cases. Thus, there is no major advantage of a reduction of the gap.

The predicted behavior is in agreement with experimental data. None of the beam tests gave any indication of fatigue crack growth along the web-to-gusset weld connection as a result of distortion.

In order to evaluate the behavior of more flexible (or weaker) connections such as provided by detail 1-cut, analyses were conducted by removing part C alone and parts A, B, and C, respectively (see Fig. 25). The results of these studies were used to examine the out-of-plane bending stresses and stresses transverse to the welds. Both web-to-gusset and the stiffener-to-gusset welds were considered. When analyzing the web-to-gusset connection, bending stresses and axial stresses perpendicular to the web are of primary concern. When considering the behavior of the stiffener-to-gusset connection, stresses perpendicular to the stiffener are of interest. The bending stresses along the web-to-gusset weld were not significantly different from those shown in Figure 26. However, the transverse stresses along the stiffener-to-gusset weld increased substantially when part C or parts A, B, and C were removed.

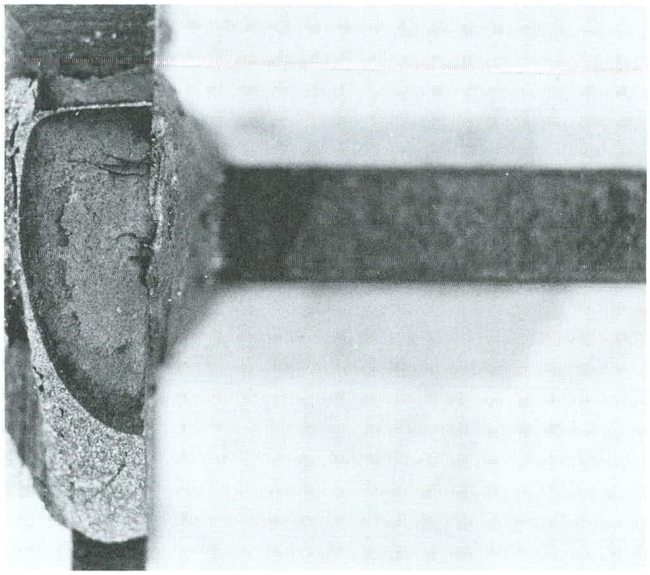
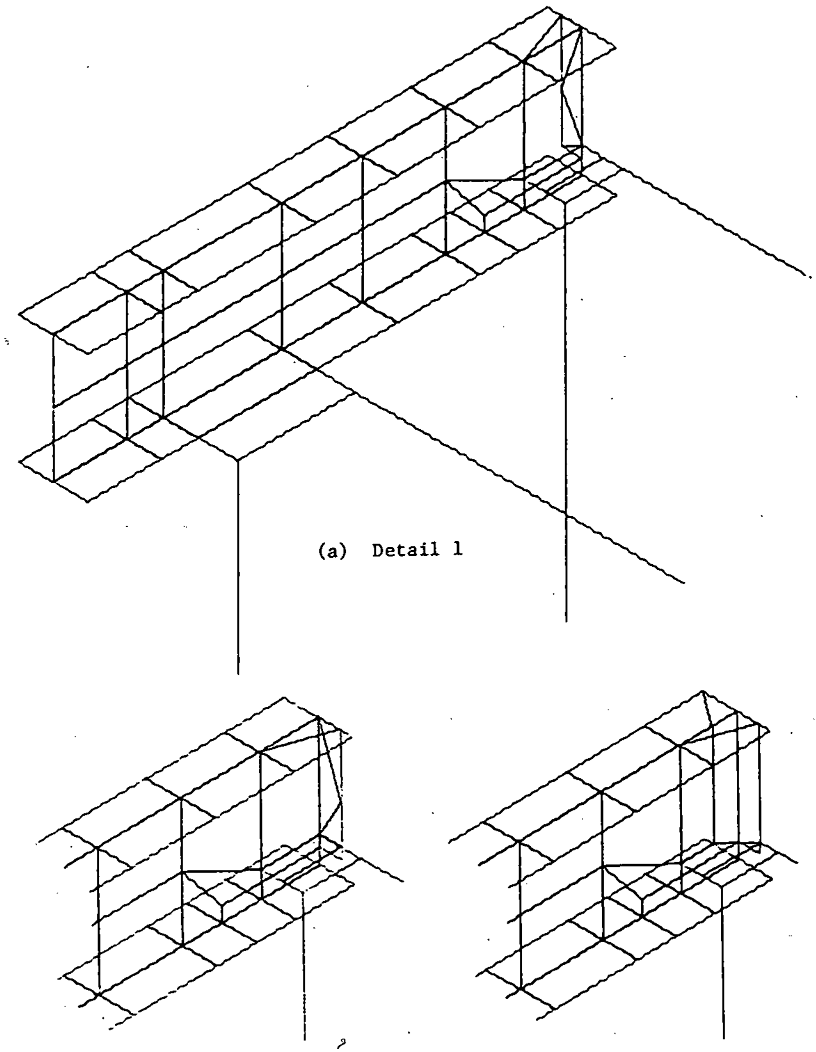


Figure 19. Fatigue crack in girder web from web gusset.

Table 2 summarizes the stresses in gusset plate elements E1, E7, E8, E9, and E10 for the three cases examined. The stress ranges correspond to a nominal stress range of 12 ksi (82.7 MPa) in the beam web at the end of the gusset plate. The stress range for the case with parts A, B, and C removed suggested that fatigue crack growth was highly probable in the gusset-stiffener welded connection.

Although no cracks were detected in the transverse welded connections from the external weld surface, saw cuts were made through the gusset plate-stiffener connection, as shown schematically in Figure 27. Each saw cut edge was ground and etched to determine whether or not crack growth had developed in the transverse weldments.



(b) Modification for Detail 2 (c) Modification for Detail 3

Figure 20. Two-dimensional analysis of the whole half-beam.

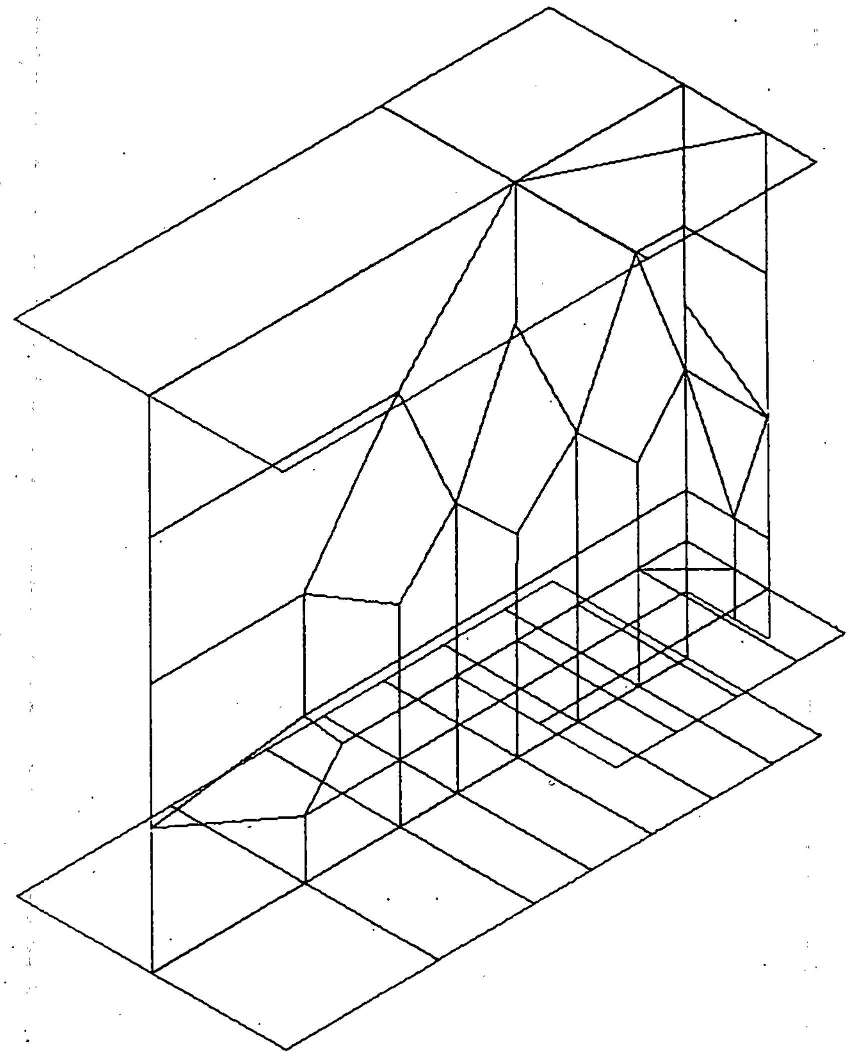


Figure 21. Two-dimensional analysis of the girder central part in case of detail 1.

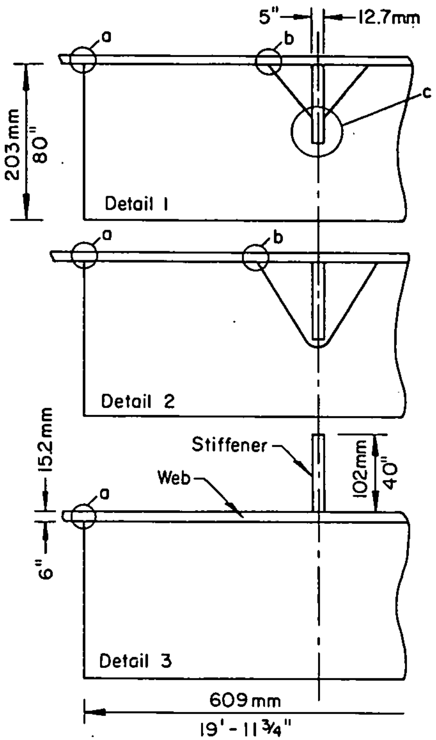


Figure 22. Selected details and critical locations.

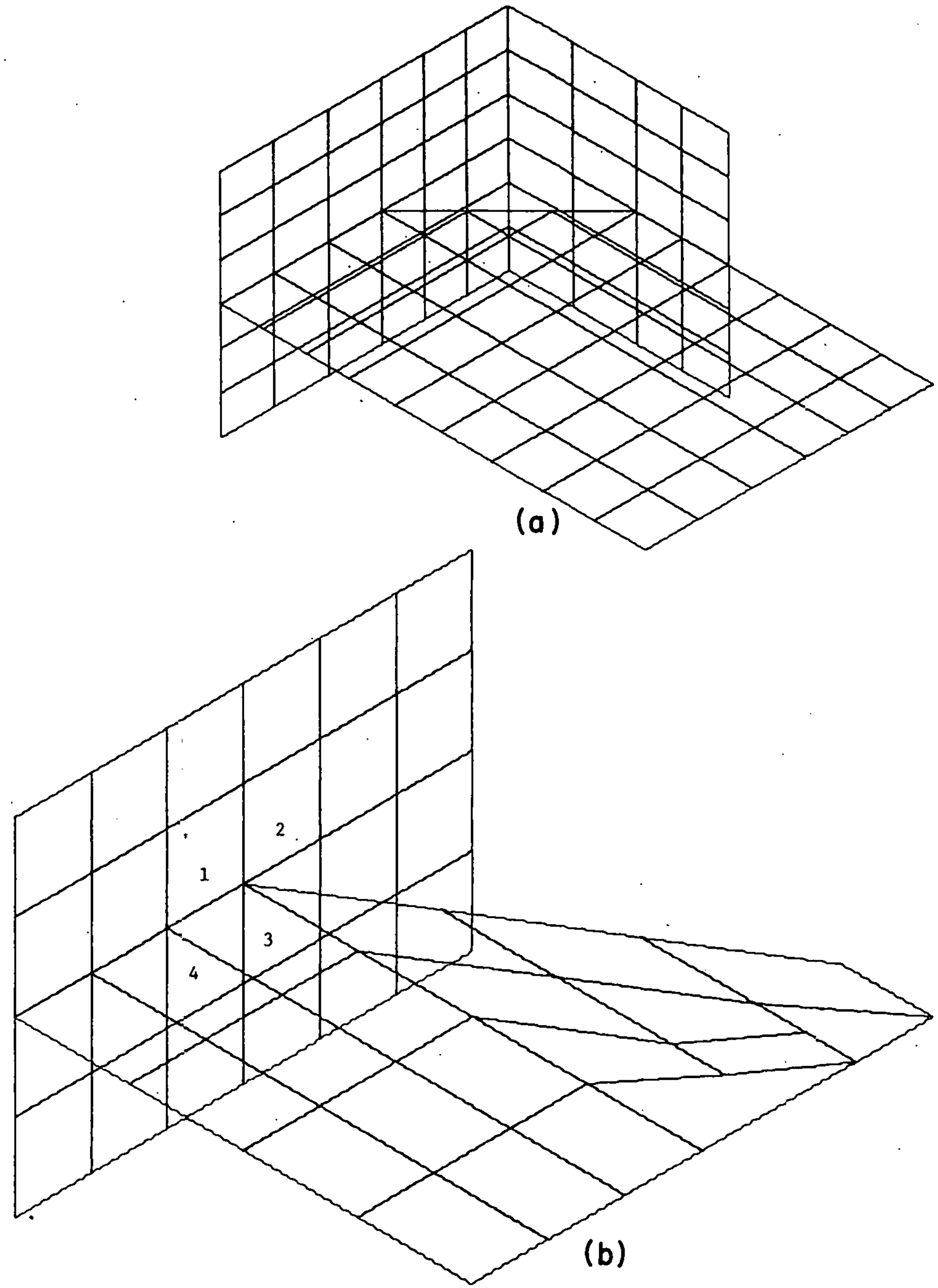


Figure 23. (a) Two-dimensional analysis of critical locations b and c in detail 1; (b) two-dimensional analysis of critical location b in detail 2.

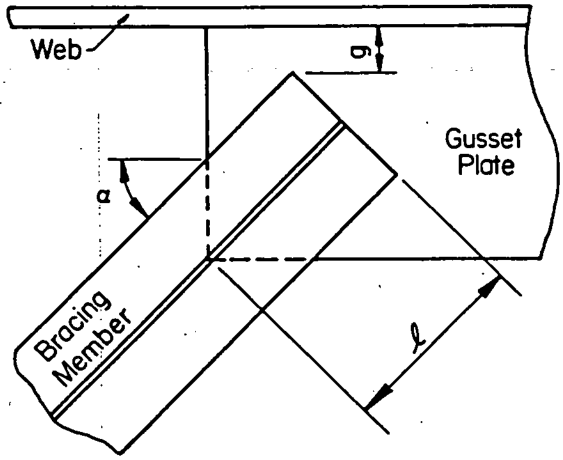


Figure 24. *Parameters of the gusset-to-bracing members connection.*

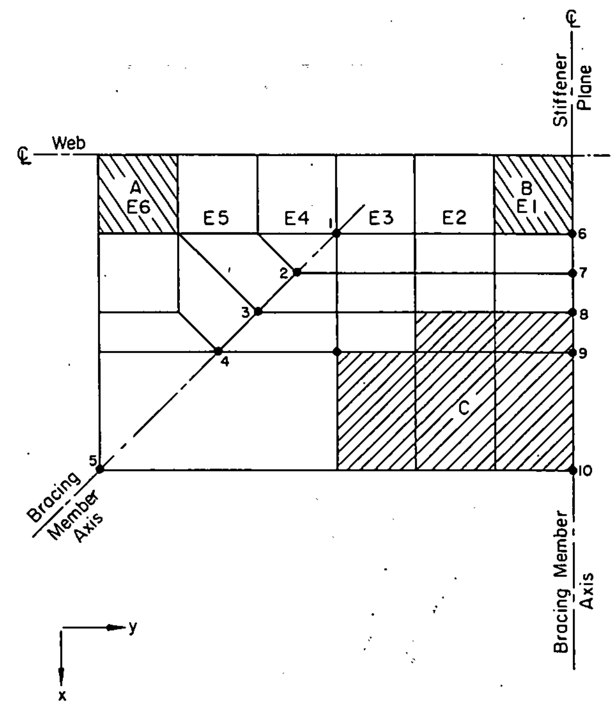


Figure 25. *Discretization used in the study of the effect of the bracing-to-gusset connection length.*

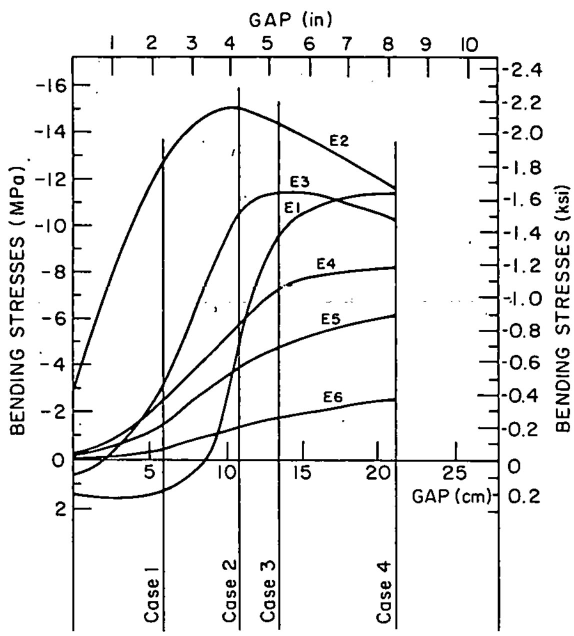


Figure 26. *Out-of-plane bending stresses along web-to-gusset weld.*

TABLE 2

STRESSES ALONG GUSSET PLATE-TO-STIFFENER WELD (SEE FIG. 25)

Element	No Cut-Outs		Part C Removed		Parts A, B & C Removed	
	ksi	(MPa)	ksi	(MPa)	ksi	(MPa)
E1	6.6	(45.5)	7.4	(51.0)		
E7	4.7	(32.4)	6.4	(44.2)	8.5	(58.7)
E8	3.8	(26.2)	6.5	(44.8)	6.8	(46.9)
E9	2.9	(20.0)				
E10	1.7	(11.7)				

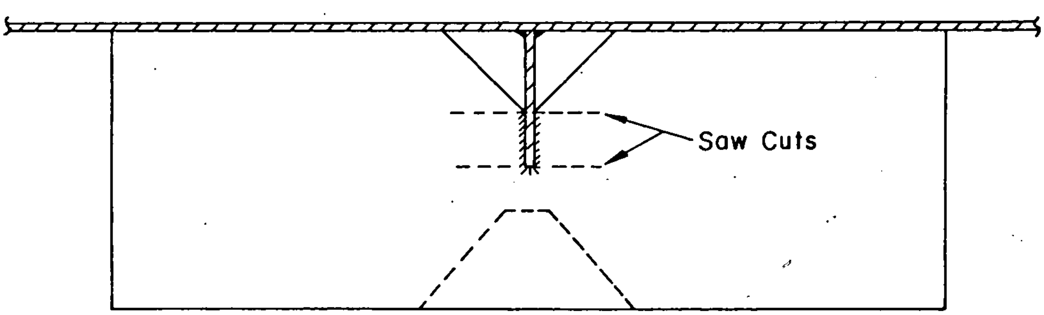


Figure 27. *Schematic of saw cuts.*

Figures 28 through 31 show the polished and etched edges. Fatigue cracks were detected in two Type 1 details adjacent to the girder web. An examination of the weldments shows that the transverse weldments tended to be smaller adjacent to the girder web (see Fig. 31). The region of incomplete fusion in these load-carrying fillet-welded connections provided the mechanism for crack growth (*13*). The decrease in fillet weld size adjacent to the girder web and the

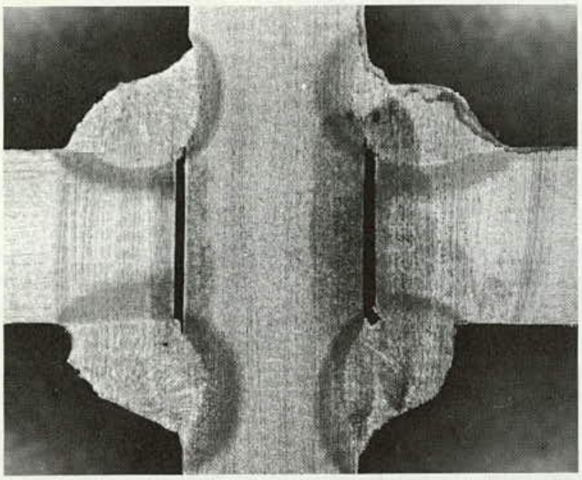


Figure 28. Gusset-stiffener connection adjacent to cope—beam 8 ($S_r = 12$ *ksi), detail 1.*

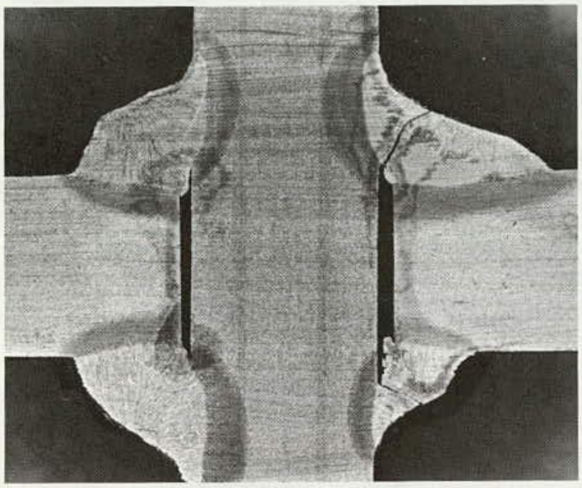


Figure 29. Gusset-stiffener connection adjacent to cope—beam 15 ($S_r = 21$ *ksi), detail 1 showing cracks in weld.*

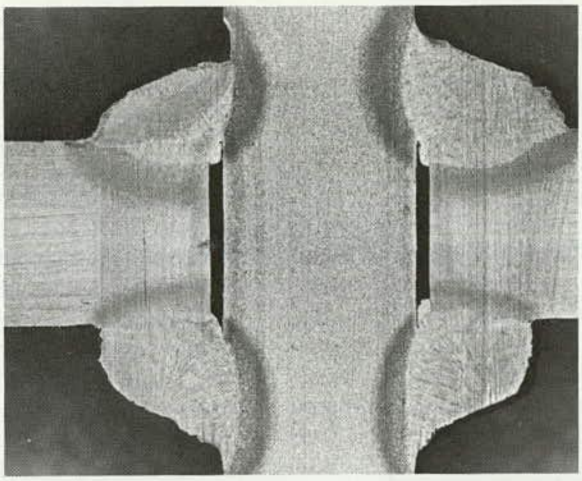


Figure 30. Gusset-stiffener connection adjacent to cope—beam 13 ($S_r = 13$ *ksi), detail 1-cut.*

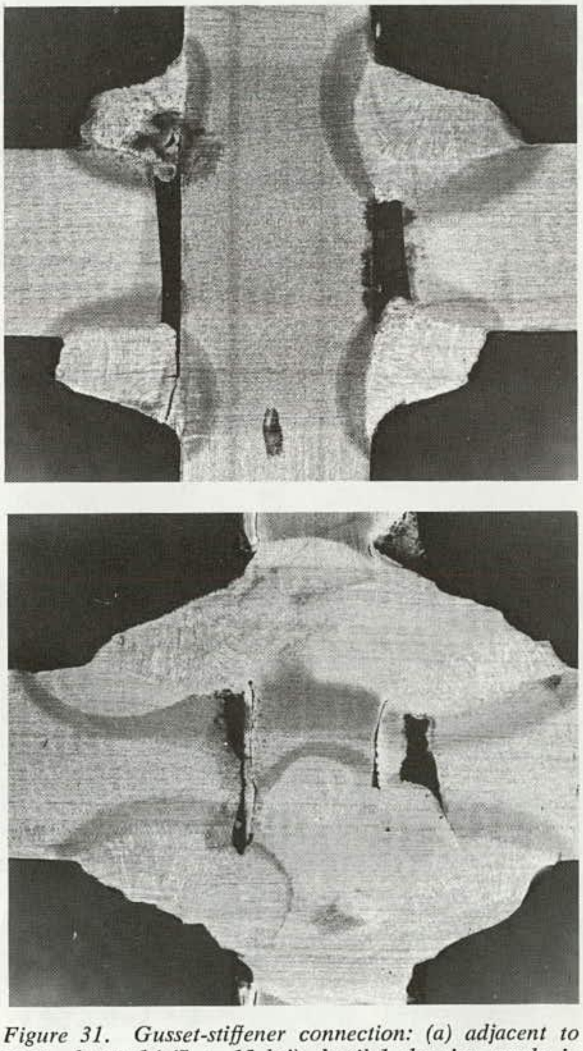


Figure 31. Gusset-stiffener connection: (a) adjacent to cope—beam 14 ($S_r = 15$ *ksi), detail 1 showing cracks in weld; (b) at stiffener end—beam 14* ($S_r = 15$ *ksi), detail 1.*

increased stress range across the welded connection provided the conditions needed for crack propagation.

The degree of fatigue crack growth shown in Figure 30 at detail 1-cut was not as extensive as that shown in Figures 29 and 31 at Type 1 details. Crack growth is most apparent in Figure 30 at the root of the smallest fillet weld. The stress in the Type 1-cut gusset plate was estimated from the finite element studies. This indicated that the shear stress acting on the transverse fillet weld was less in beam 13 than the values obtained in beams 14 and 15 which developed the largest cracks. Although the geometric configuration of detail 1-cut represents a more severe condition than provided by detail 1, the controlling factor of this test was weld size. Relatively small, random differences in weld size influenced the stress condition in the details much more than the notch cut out of the gusset plate to produce detail 1-cut. Although all details were fabricated with the same nominal size welds, the small sampling of beams showed that the weld leg size varied from 0.25 to 0.40 in. (6.4 mm to 10.2 mm). One of the detail 1-cut gusset plates had a larger transverse weld than one of the detail 1 gussets.

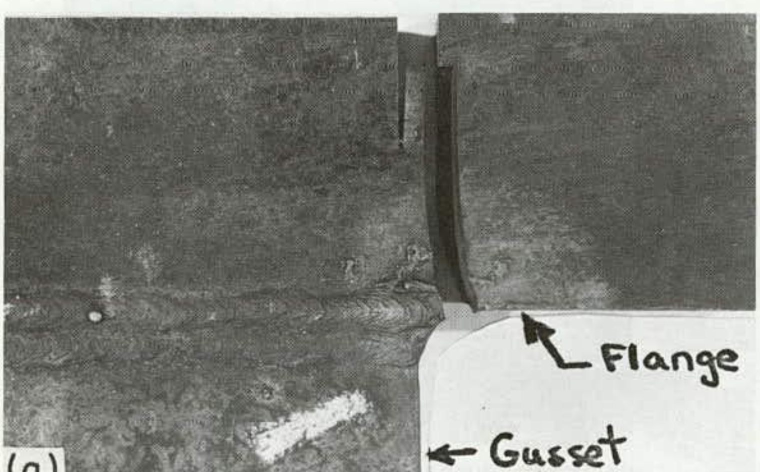
This examination and behavior have demonstrated that a critical condition is likely to exist in bridges at the transverse welded connection of gusset-to-stiffener welds adjacent to the web where the likelihood of an undersized weld is great due to geometrical considerations and accessibility for the welder. This condition will be particularly critical if the transverse weldment intersects the longitudinal and vertical welds in the corner. Such a condition will promote fatigue crack propagation into the girder web similar to the condition that developed in the Lafayette Street Bridge (*1*).

FATIGUE BEHAVIOR OF FLANGE GUSSETS

Each of the 18 girders tested during this study had two flange details attached to the tension flange as shown schematically in Figure 1. The details were located along the beam span so that one end of the gusset plate had the flange stress range identical to the magnitude experienced at the ends of the web gusset plate. Three types of connection plates were attached to the flange tips, as shown in Figure 4. The intent was to examine flange attachments with end radii of 0, 2, and 6 in. (0, 51, and 152 mm). Figures 7(a), (b), and (c) are photographs of the actual radii provided. Although the 0-in. attachment was intended to be directly welded to the flange tip without further treatment, the fabricator ground the end of the longitudinal weldment so that the actual radius varied between 0.2 to 0.4 in. (5 and 10 mm). This is apparent in Figure 7(a).

The only fatigue cracking that was detected in this study occurred at the 0-in. radius details. Since two gusset plates were attached to each girder and each had two weld ends on the flange tip, there were 24 possible crack locations at the 0-in. radius details. Fatigue cracking was detected at 8 of these 24 locations. Only 2 of these 7 cracks were detectable without ultrasonic testing, and only 1 crack was detected at the end with the lowest stress range.

Figures 32, 33, and 34 show the detail and crack



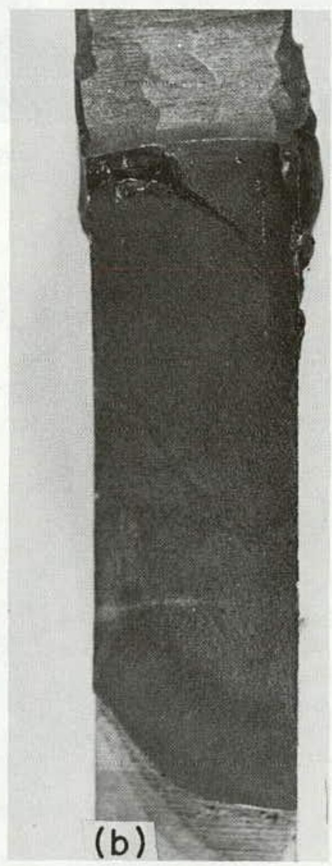


Figure 32. Flange gusset detail and exposed surface (beam 8, r = 0 in.).

surfaces at the reentrant corner of the 0-in. radius ground details that had visible cracks. An examination of the crack surfaces indicates that subsurface defects were the origin of crack growth. Figures 35 and 36 show two of the other four cracks that were detected after the tests were completed and the details destructively examined. The amount of fatigue crack growth is small at each of these details. It is again apparent that small subsurface discontinuities were the crack initiation sites. All of the "0" details with the ground end exceeded the lower confidence limit provided by Category E, as can be seen by the test data plotted in Figure 37(a).

Also plotted in Figure 37(b) are the test data for the 2-in. and 6-in. radius details. None of the details experienced any detectable crack growth. All test data shown as circles or squares are indicated as having no cracking by the arrows attached to each data point. These beam tests of groove-welded flange attachments with a ground-radiused end demonstrated the adequacy of the existing AASHTO specifications which provide for such details (*4*). Transitions between 2 in. (51 mm) and 6 in. (152 mm) are assigned design Category D. These tests suggest that this is a conservative design limitation for groove-welded details fabricated in this manner and undergoing stress ranges within the limits of this experiment.

Radiused gussets attached to the beam web by fillet welds were tested by Comeau and Kulak (*7*). A 4-in. (100-mm) radius transition that was ground smooth provided a fatigue resistance compatible with Category D. Their test data are plotted as diamonds in Figure 37(b) and fall at or above the lower confidence limit provided by Category D. Nearly all of these details cracked, and the comparison of the two sets of test data reveals that the 2-in. radius groove-welded flange attachment was superior to the 4-in. fillet-welded web detail. It seems reasonable for this to occur on the basis of the probable defects that are likely to be more prevalent at the roots of the fillet welds. A similar behavior was observed in fatigue tests of curved plate girder assemblies that had several details develop early cracking from large weld root flaws that were near the radius surface near the points of frequency (*15*).

The results of the studies on details with radiused transitions attached to the web or flange by fillet welds suggest that large increases in fatigue resistance will not be able to be achieved as suggested by the specification. The probability of weld root flaws in the transition region is greatly increased with such connections. Hence, it is doubtful that Categories C and B can be developed unless these details are attached by groove welds that have been carefully inspected to ensure that significant flaws do not exist near the points of tangency. Therefore, fillet-welded details with radius transitions of 6 in. or greater should be classified as Category D.

FATIGUE BEHAVIOR OF GUSSET PLATES ATTACHED TO FLANGE SURFACE

Flange gusset plates have often been fillet welded to the flange surface, as shown in Figure 38. The fatigue data on this type of connection from previous studies had indicated that they were appropriately classified as Category E (*3*, *14*). Inasmuch as length of the longitudinal weld appeared to be a primary reason for the decreased fatigue strength, several tests were carried out on attachment plates attached to the beam flange by transverse welds alone (see Fig. 5). These details were attached to the flange on the side directly opposite the radiused transition details. One such detail is shown in Figure 38. The detail was a 8 × 24 × 0.5-in. (203 × 610 × 113-mm) plate attached to the girder flange by 0.37-in. (9.5-mm) transverse fillet welds that terminated 0.5 in. (13 mm) from the edge of the flange.

The cracks at this detail were found to develop from the weld root and quickly propagated throughout the entire transverse weld (see Fig. 39). Most of these details did not result in the crack propagating into the beam flange. Rather, the crack growth from the weld root severed the connection plate from the flange.

The test data are plotted in Figure 40 and compared with design Category E and E′ curves. These curves represent the lower confidence limit of severe details (*3*, *5*). It is apparent that the detail has low fatigue resistance as represented by Category E′. The stress range is the condition in the flange and does not represent the stress on the weld throat. An accurate estimate of the stress in the weldment is difficult to make because the force in the attachment plate is not known.

Therefore, the force in the attachment plate was crudely estimated through finite element modeling. This force was determined to be approximately 20 percent of the force in the girder flange away from the attachment. From this estimate, it was calculated that the lowest shear stress on the throat of the weld that was tested and cracked was 10 ksi (69 MPa).

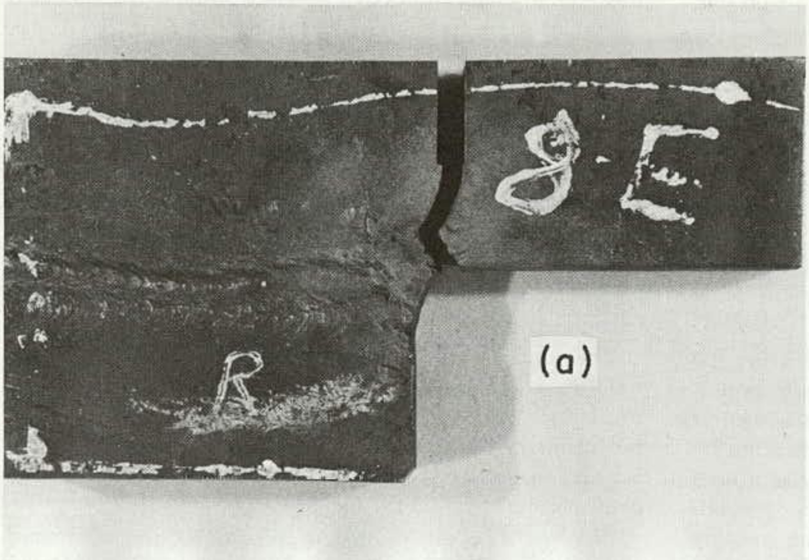
The tests suggest that the greater the flange thickness the lower the fatigue resistance. However, only one test was carried out on the W36 x 160 beam. One of the details resulted in the fatigue crack propagating into the girder flange. The cycle life at which this occurred was greater than the life necessary to develop a crack in the transverse weld and sever the detail from the flange. The test is shown as a solid (triangle) symbol in Figure 40.

These results reveal that the use of transverse fillet welds previously described will not provide an adequate level of fatigue resistance. Crack growth will initiate at the weld root and propagate through the throat severing the plate from the beam flange.

The results of the full-scale details reported in Roberts et al. (*14*) are also plotted in Figure 40. All of these details equaled or exceeded the fatigue resistance provided by Category E.

FATIGUE BEHAVIOR OF WEBS WITH GIRDER FLANGES FRAMING INTO WEB OR PIERCING THROUGH WEB

A pilot study was carried out and reported in Ref. 5 on web details that simulated penetration of a beam flange through a girder web. These tests simulated the passage of a girder flange through a box pier cap. Because of the narrow width of the box pier cap, the flanges were



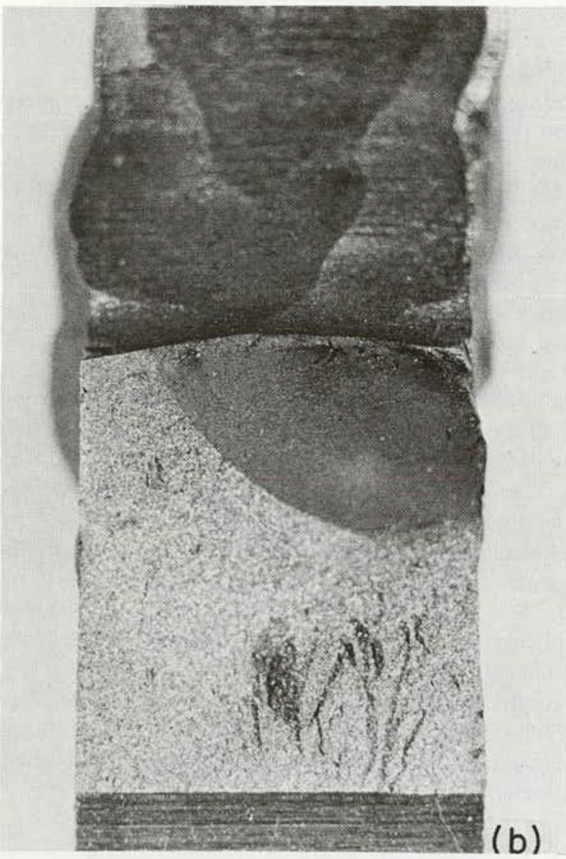


Figure 33. Flange gusset detail and exposed surface (beam 8, r = 0 in.).

connected to the box web with 0.375-in. (10-mm) seal welds only around the edges of the girder flange on the outside of the box.

In order to supplement the earlier tests and to ascertain the significance of attaching larger flange plates to girder webs, additional tests were carried out and are summarized in this section. Tests were carried out on details that framed into the web without passing through, as well as further tests on plates that pierced the web. These details were attached to the girder webs in the shear span near the flange attachments (see Fig. 38). Several types of the details were used as shown in Figure 5. When the plates were welded to each side of the web without penetration, $2 \times 16 \times 4$-in. ($51 \times 406 \times 102$-mm) plates were used. The attachment plates were either fillet or groove welded against the sides of the web of the test girder. Figure 41 shows typical fatigue cracks that formed at the weld toes of the fillet-welded and groove-welded details. When the crack surfaces were exposed they showed the typical semi-elliptical surface cracks that generally propagate under these conditions. Figure 42 shows the crack surfaces of two such details.

Several smaller beams with 0.75-in. (20-mm) thick web attachments that pierced or were welded against the web were tested at the University of Alberta in Edmonton, Alberta, Canada (*7*). The web thickness of these beams was approximately 0.25 in. (6.35 mm). Each beam contained two details within a constant moment region. The 0.75-in. (20-mm) thick plates were welded to each side of the 0.25-in. (6.35-mm) beam web with 0.25-in. (6.35-mm) fillet welds for both types of attachments. The results of these tests are plotted in Figure 43 as open circles and squares. The data from the larger beams with 2-in. (51-mm) thick plates that framed into each side of the beam web tested in this program are plotted as closed circles and triangles to differentiate between groove-welded and fillet-welded connections. Also plotted in Figure 43 are the lower confidence limits that correspond to design Categories D, E, and E′ (*4*).

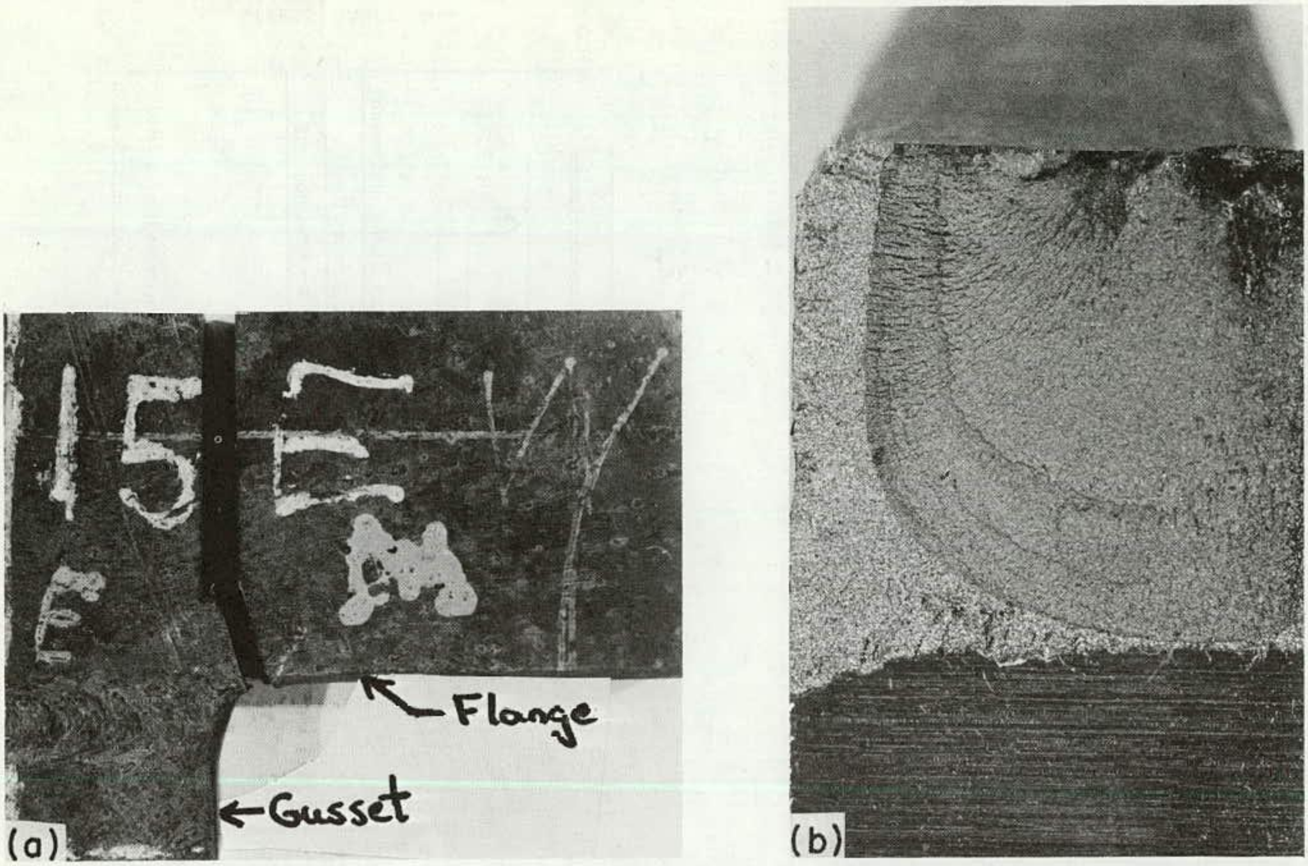


Figure 34. Flange gusset detail and exposed surface (beam 15, r = 0 in.).

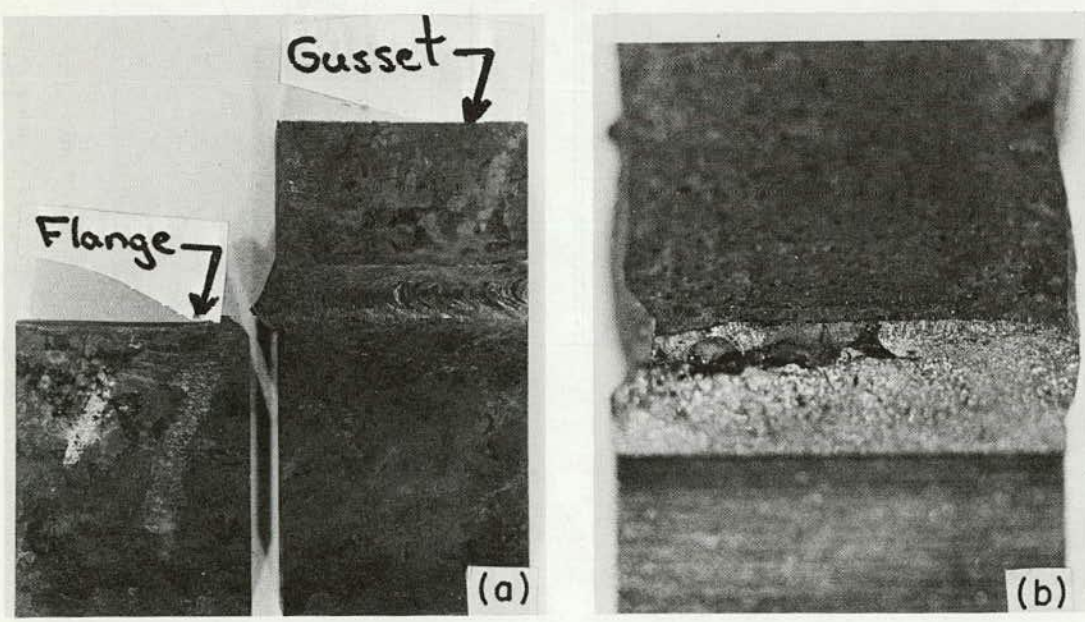


Figure 35. Flange gusset detail and exposed surface (beam 15, r = 0 in.).

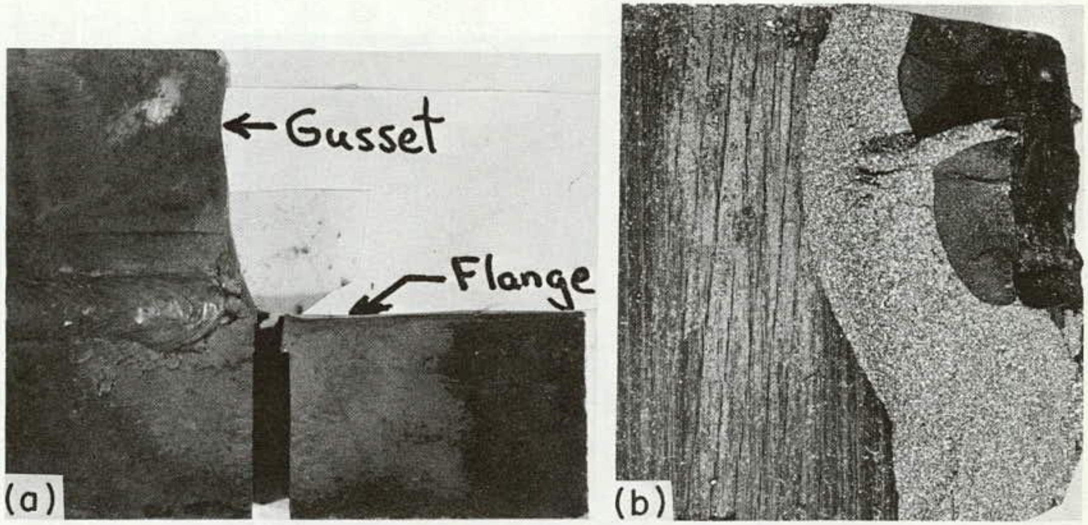


Figure 36. Flange gusset detail and exposed surface (beam 15, r = 0 in.).

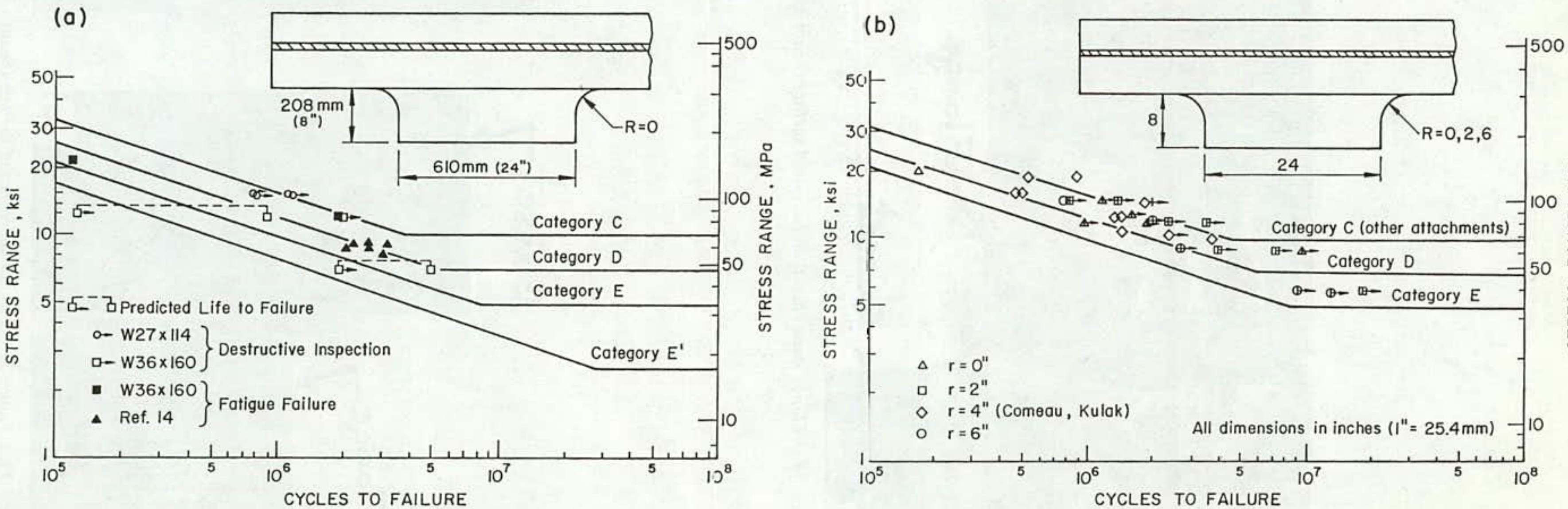


Figure 37. Fatigue life of flange gussets: (a) 0-in. radius; (b) 2-in. and 6-in. radius.

Figure 38. Flange surface gusset plate.

Figure 39. Weld crack through throat of fillet weld at root.

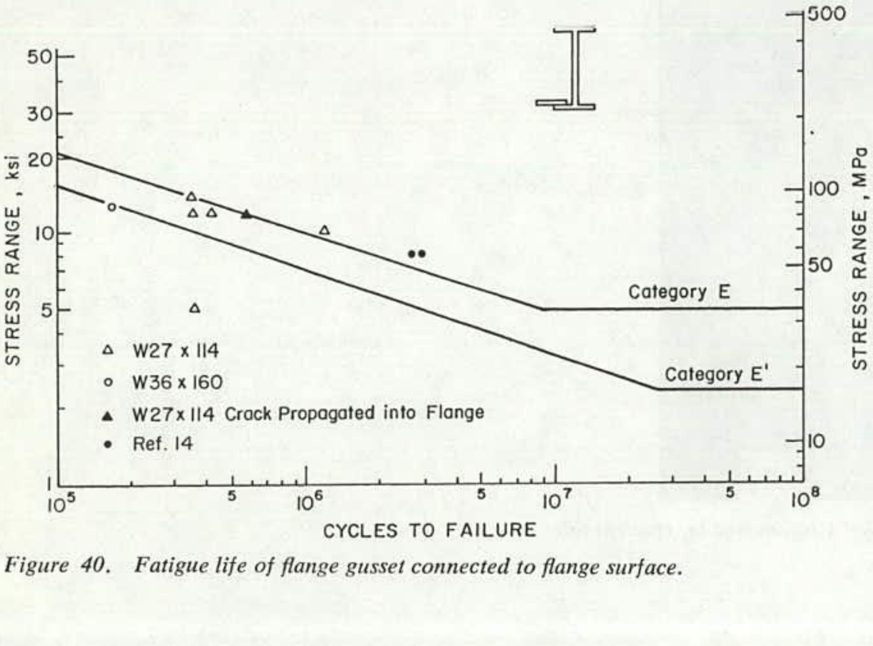


Figure 40. Fatigue life of flange gusset connected to flange surface.

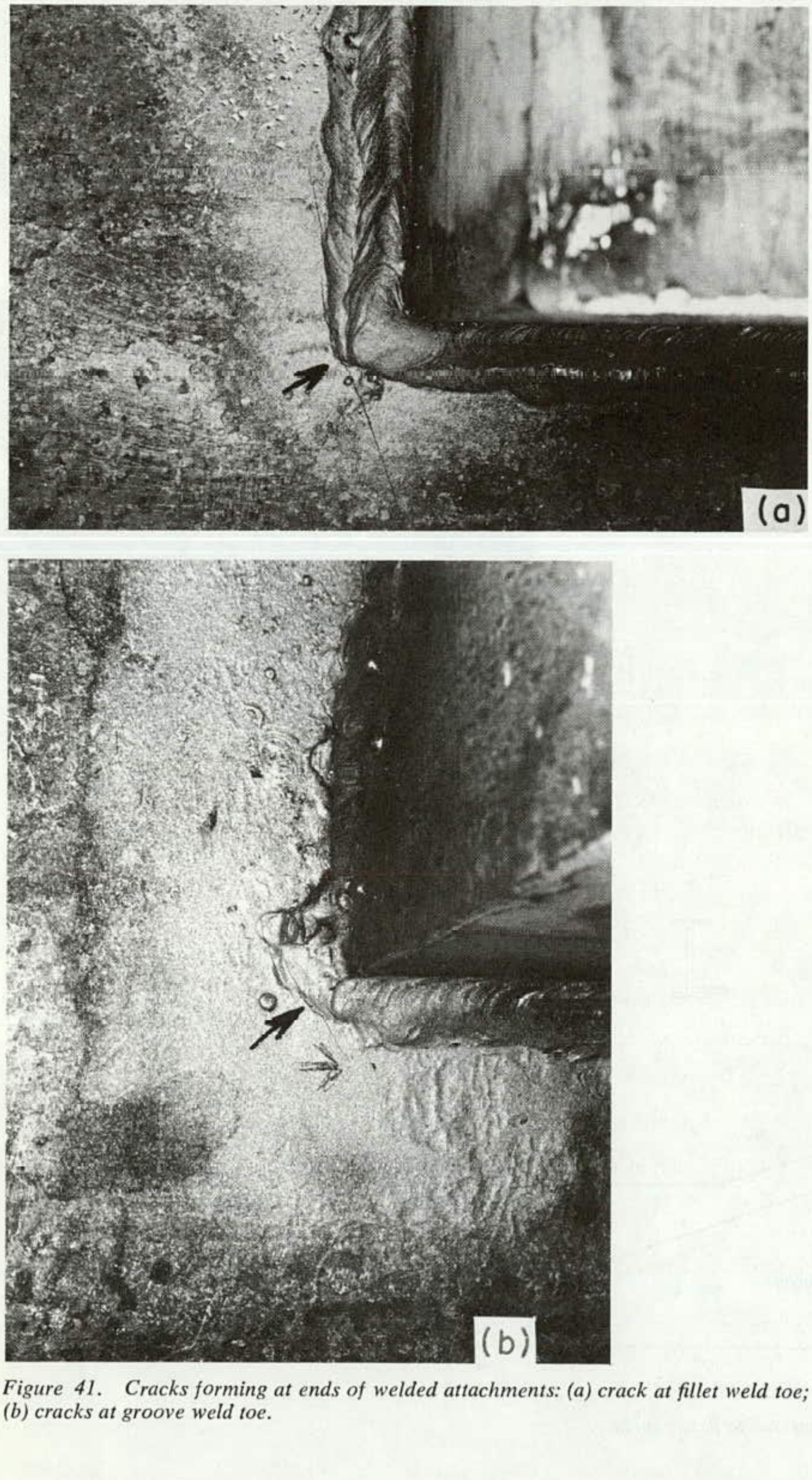


Figure 41. Cracks forming at ends of welded attachments: (a) crack at fillet weld toe; (b) cracks at groove weld toe.

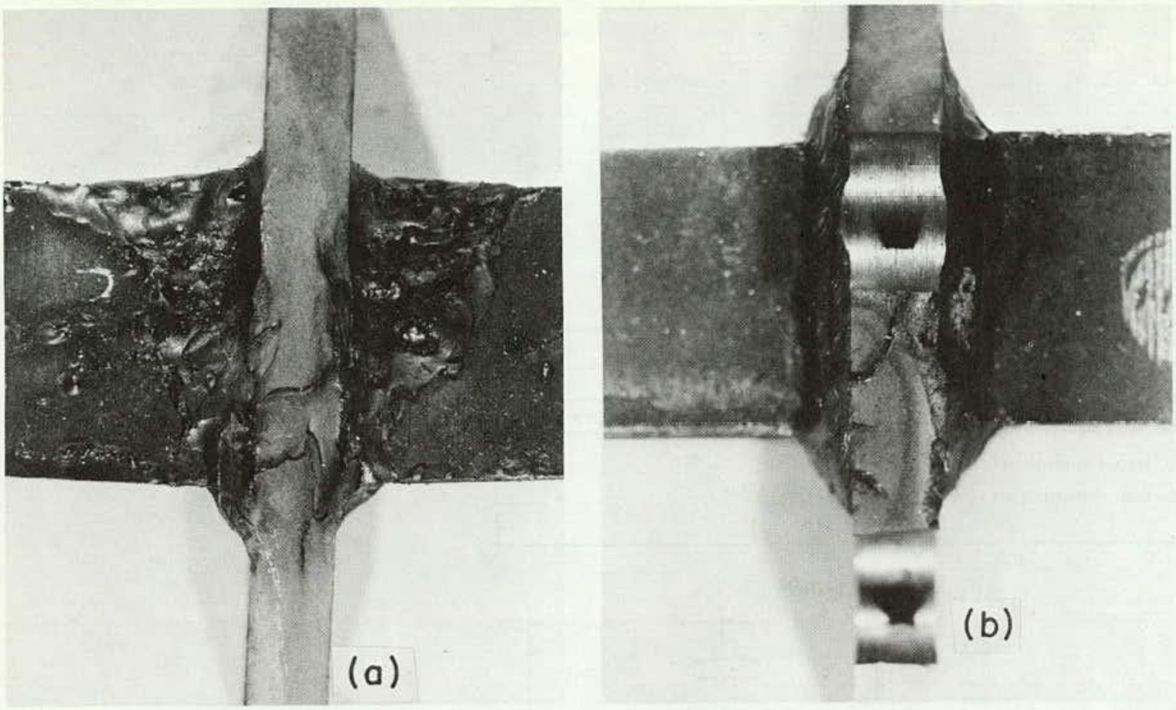


Figure 42. (a) Crack growth from ends of groove-welded attachments to web surface; (b) crack growth from ends of fillet-welded attachments to web surface.

All test data on beams with thinner webs ($t_w = 0.25$ in. (6.4 mm)) yielded about the same fatigue resistance. This included plates that framed into the web or that penetrated through the web and fillet welded on each side. The test data for the smaller beams were in agreement with Category E.

The test data from the larger size girders ($t_w \cong 0.60$ in. (15 mm)) tend to fall at or below the lower confidence limit for cover-plated beams and other attachments that are classified in accord with Category E. It is apparent that Category E′ best represents the fatigue resistance of heavy flange plates fillet welded or groove welded to a thicker web.

A finite element analysis was carried out on the web-flange connection to determine stress gradient in the beam web at the end of the welded attachment (*17*). A regression analysis was performed on the stresses determined by the finite element method, resulting in the stress gradient correction factor:

$$F_G = \frac{SCF}{1 + 0.88\ a^{0.576}} \tag{1}$$

where:

SCF = stress concentration factor = 7.0 and 8.0 (again determined from F.E.M. results); and
a = crack size, minor semidiameter of elliptical crack.

The stress intensity range was taken as:

$$\Delta K = F_E\, F_S\, F_W\, F_G\, S_r\ \sqrt{\pi a} \tag{2}$$

where:

F_E = elliptical crack front correction $= \dfrac{1}{E(k)}$;
F_S = front free surface correction factor $= 1.211 - 0.186\sqrt{a/b}$;
F_W = finite width correction factor = 1.0;
F_G = stress gradient correction factor (Eq. 1);
S_r = applied stress range;
a = minor semidiameter of elliptical crack; and
b = major semidiameter of elliptical crack.

The crack shape was taken as

$$b = 3.55 + 1.29\ a \tag{3a}$$

when $a < 4$ mm and

$$b = 1.506\ a^{1.241} \tag{3b}$$

when $a > 4$ mm. A few measurements of the crack shape indicated that these were reasonable estimates of the crack shape, as can be seen in Figure 44.

The fatigue resistance was estimated by integrating between the limits of the assumed initial crack size and the penetration of the crack through the girder web:

$$N = \frac{1}{C}\int_{a_i}^{a_f} \frac{da}{\Delta K^3} \tag{4}$$

where:

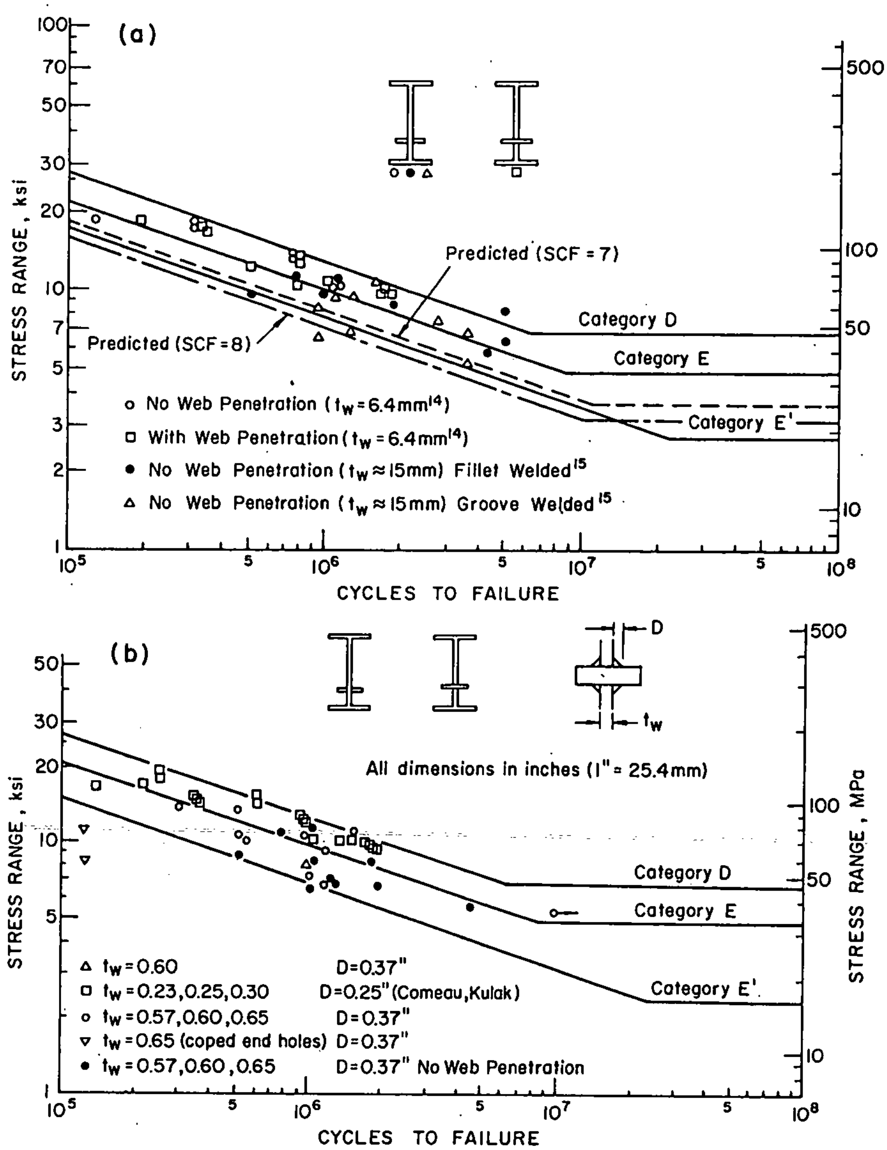


Figure 43. Fatigue strength of web attachments: (a) previous work; (b) current work.

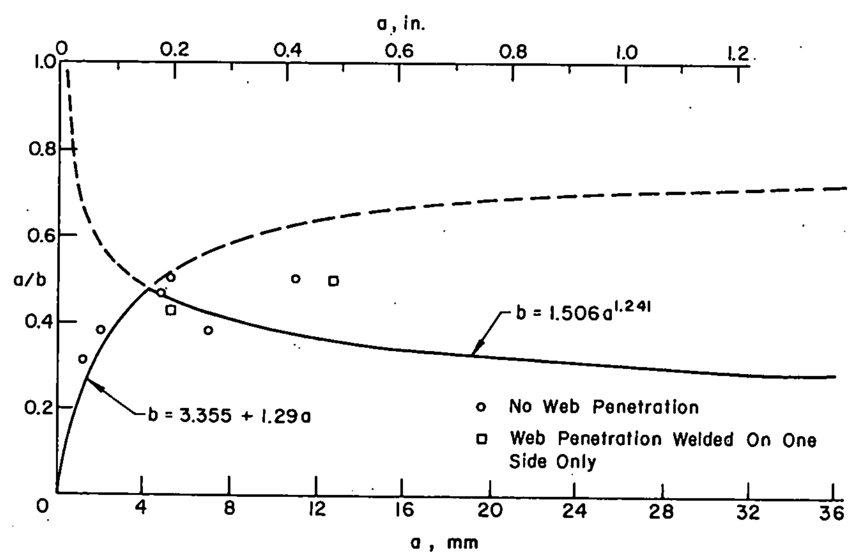


Figure 44. Crack shape variation with crack depth.

N = fatigue resistance, cycles;
a_i = assumed initial crack size = 0.02 in. (0.5 mm);
a_f = final crack size = web thickness;
ΔK = stress-intensity-factor fluctuation; and
C = crack growth rate constant = 3.6×10^{-10} [(in.$^{11/2}$)/(kip^3 cycles)].

A lower bound to crack propagation was estimated by equating the stress intensity range to the crack growth threshold:

$$\Delta K = \Delta K_{th} = 2.75 \text{ ksi } \sqrt{\text{in.}} \ (3.0 \text{ MPa } \sqrt{\text{m}}) \quad (5)$$

The predicted fatigue resistance is plotted in Figure 43(a) as the dashed lines. The mathematical model tends to overestimate the fatigue resistance and predicts a strength close to Category E.

Tests were also carried out on large girders with 2-in. (51-mm) by 16-in. (406-mm) insert plates that were fillet welded to each side of the girder web with 0.375-in. (10-mm) welds. The girders generally cracked through the weld at the ends of the inserted flange plate (see Fig. 45). When the crack surfaces were exposed, they showed that the primary mode of crack propagation was from the interior lack of fusion zone. Figure 46 shows one such crack surface. Altogether, nine details were tested with these geometric conditions.

Two beams were tested with an end cope at the tips of the flange plates that pierced the web. Figure 47 is a photograph of the half-circle end cope. Longitudinal fillet welds connected the flange plate to each side of the girder web. As expected, cracking developed at the ends of the weldment in the beam web. Figure 48 shows the crack surface in the web and the edge of the flange.

The test results of the details that penetrated the beam web are summarized in Figure 49. The small scale beams with similar details are also plotted in Figure 49 as closed circles. The smaller beams equaled or exceeded the fatigue resistance provided by Category E as was demonstrated in Figure 43. The full-size beams with thicker webs and flange inserts that were completely welded around the web opening provided a fatigue resistance that was in agreement with Category E′. The details that contained the half circle and cope provided a fatigue resistance that was less than Category E′.

Welding to each side of the beam web improved the fatigue resistance of the details that penetrated the web when the weldments were placed around the flange. This tended to decrease the unfused portion of the flange tip so that the detail was not as severe as the condition that existed when the flange was welded to one side of the web alone.

When the web cutout was provided with end copes and longitudinal welds were used to connect the flange plate to the beam web, the longitudinal fillet weld end terminated at the tangent of the circular end cutout. This provided a more severe condition of defect and stress concentration that decreased the fatigue resistance below Category E′. Decrease was not as severe as encountered with the inserts that were welded to one side alone (*5*). These results from Fisher et al. (*5*) are plotted in Figure 49 as open circles and fall well below the fatigue resistance of the other details.

The analytical model (Eqs. 1 to 5) that was used to predict the fatigue resistance of the web attachments was based on the assumption that there was full fusion through the thickness of the web. The model simulated the case where two plates are welded up against the sides of a girder web or the case where there is nearly full penetration by a plate in a relatively thin web. To account for larger regions of lack of fusion in girders with thick webs, it is necessary to modify the solution.

There are two basic configurations that can be used with web penetrations. The first case involves a flange plate piercing the web of a girder and welded in place on one side only, as reported in Fisher et al. (*5*). The second case incorporates welding on both sides of the web. The area of fusion can be determined as a percentage of the overlapping area of the two intercepting plates. The overlapping area is defined as the product of the thickness of the web times the thickness of the penetrating flange plate, as shown in Figure 50. An examination of the details welded to one side indicated that the fused area was equal to about 40 percent of the overlapping area (Fig. 50). This causes the effective stress in the fused area to be about 2½ times greater than it would be if there was full fusion. If the flange plate was welded to both sides of the web, the fused area would be about 80 percent of the overlapping area. The effective stress in the fused area would be about 1¼ times greater than it would be if there was full fusion.

The analytical model was used with these adjustments to the stress intensity factor. The effect of the stress amplification would be to reduce the fatigue resistance at a given stress range level. The predictions for both cases are shown in Figure 49 as dashed lines. The detail with one welded side is designated as Case I. Case II refers to the detail welded on both sides. The predicted fatigue behavior for Case II falls just below Category E′. The predicted behavior for Case I falls well below this level and is in general agreement with test results shown in Figure 49.

FATIGUE BEHAVIOR OF LONGITUDINAL STIFFENER WELDS CONTAINING LACK OF FUSION REGIONS

In October 1973 a large crack was discovered in a fascia girder of the suspended span of the Quinnipiac River Bridge near New Haven, Conn. (*16*). The crack developed in the girder web of the suspended span. When discovered, the crack had propagated to the middepth of the girder and had penetrated the bottom flange surface. The bridge structure had experienced approximately nine years of service.

From the subsequent investigation, it was concluded that the crack originated in a transverse weld across the width of a longitudinal stiffener. During fabrication, a very crude partial penetration weld was placed across the width of the stiffener, thus creating the possible origin for fatigue cracking. This portion of the project was designed to investigate the mechanism behind this crack propagation. In order to accomplish this, two (24 × 3 × 0.375-in.) (600 × 75 × 9.5-mm) plates were welded together with varying degrees

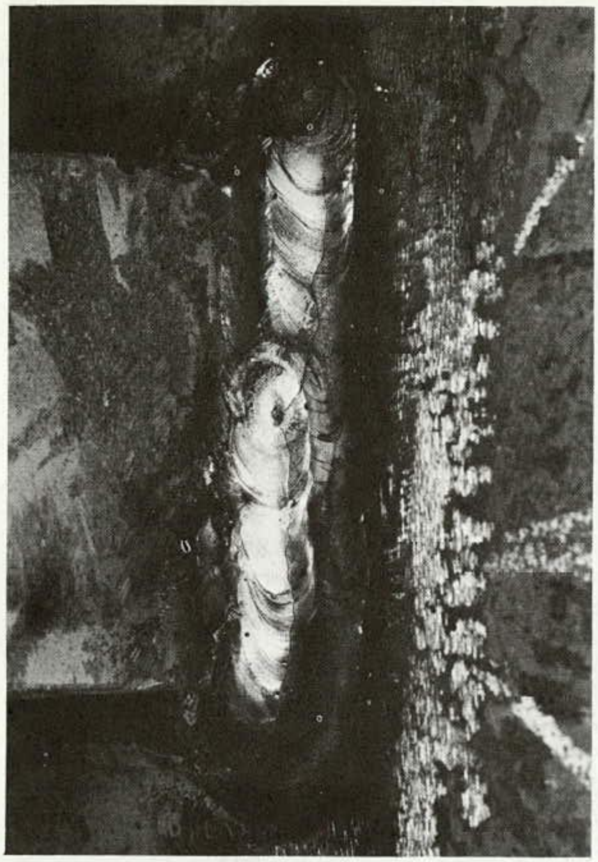


Figure 45. Crack through throat of fillet weld connecting insert plate to girder web.

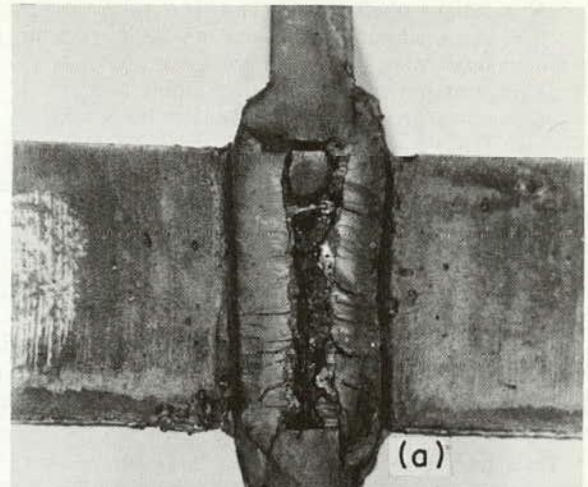


Figure 46. Cracked surface of flange: (a) framing into web detail; (b) penetrating web detail.

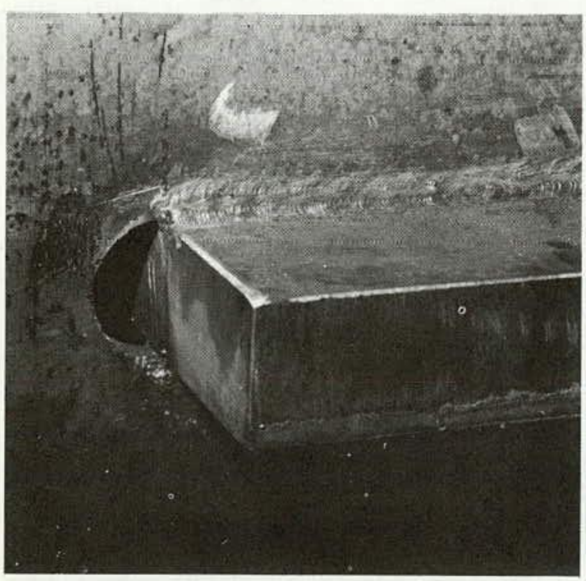


Figure 47. Coped end of flange insert.

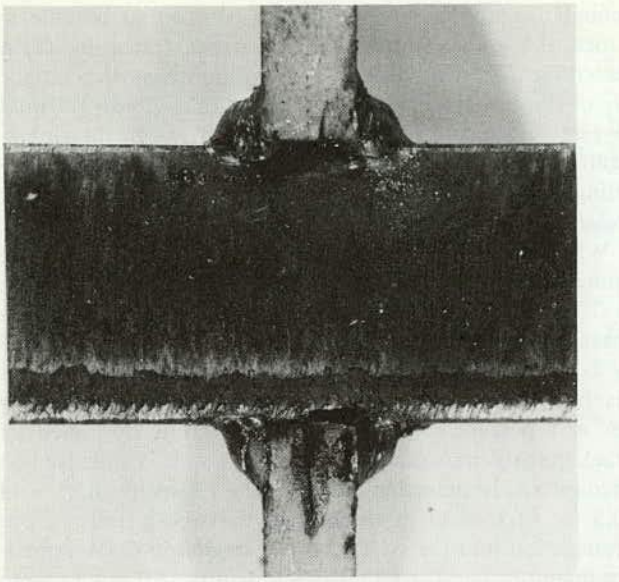


Figure 48. Crack surface in web with end cope.

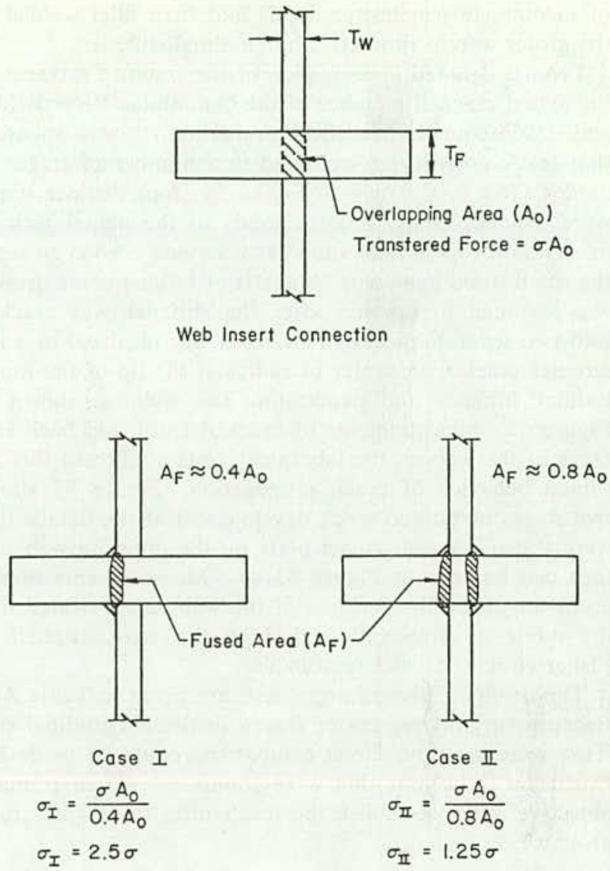


Figure 50. Estimation of fused areas.

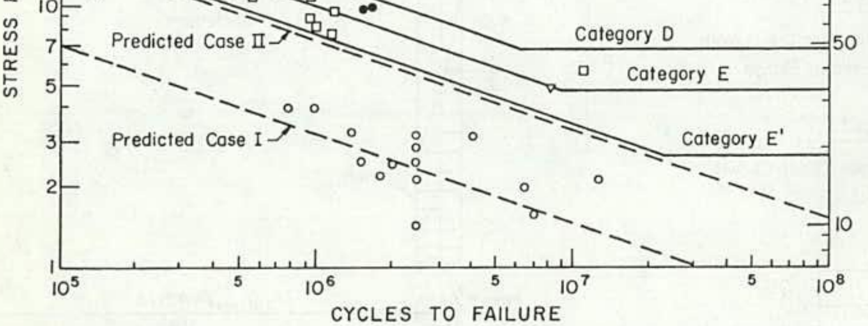


Figure 49. Comparison of test results with fatigue life prediction of simulated web penetration with unfused areas in the web.

of incomplete penetration welds and then fillet welded to the girder web to simulate a longitudinal stiffener.

From a detailed investigation of the fracture surfaces of the actual cracked member of the Quinnipiac River Bridge and a subsequent theoretical evaluation, it was apparent that crack growth had occurred in a number of stages or modes (*16*). As shown in Figure 51, four distinct stages were defined. Stage I corresponds to the actual lack of penetration region and some early service needed to sever the small fused ligaments. Stage II of fatigue crack growth was assumed to develop after the stiffener was cracked into two separate pieces. This crack was idealized as a flat circular crack with center of radius at the tip of the longitudinal stiffener and penetrating the web, as shown in Figure 52. Measurements of cracked front and back surfaces of the web on the laboratory tests confirmed this assumed behavior of crack propagation. Figure 53 shows two stages of fatigue crack development at the details that were tested. A web gusset plate on the opposite web surface can be seen in Figure 53(a). Measurements of the crack length on the surfaces of the web demonstrated that the mode of propagation through the web assumed in Fisher et al. (*16*) was reasonable.

The results of these fatigue tests are given in Table A-9. Because varying degrees of fusion in the longitudinal stiffener were used, no direct comparison could be made between the three tests that were conducted. Their primary objective was to establish the mechanism of fatigue crack propagation.

RETROFITTING PROCEDURES

Gas Tungsten Arc (GTA) Remelting

Visually detected fatigue cracks thought to be shallow enough (approximately 0.125 in. (3 mm)) to be able to remelt the entire depth of crack were treated by GTA remelting. This procedure greatly improved the fatigue life of the cracked detail when the entire crack depth could be remelted successfully. The results of the further investigation into the effectiveness of GTA remelting as a retrofitting procedure are in agreement with those reported in *NCHRP Report 206* (*5*).

When the entire crack could be remelted, this procedure produced increases in fatigue life at web gusset plates up to 75 percent (see Fig. 54). Several cases of weld toe cracks at web gusset plates were not successfully retrofitted by GTA remelting because the entire crack depth was not reached during remelting. In these cases, life increases as low as 8 percent were experienced as the newly imbedded crack quickly grew out of the remelted area. Difficulty was encountered in detecting cracks in the relatively short weld (0.5 in. (2.5 cm)) at the end of the web gussets before propagation into the web occurred rendering GTA remelting inappropriate for larger crack depths. All cracks were at least 0.5 in. (2.5 cm) long before they were retrofitted by this method. Figure 55 shows a typical cracked detail, before and after retrofitting by GTA remelting.

Greater success was achieved in retrofitting the 2-in. (5-cm) flanges welded to or penetrating the beam web by the GTA remelting process than was achieved for the web gusset plates. Although several of the tests were discontinued because of failure of another detail before new fatigue cracks could initiate in the retrofitting area, the tests had proceeded beyond the relatively short number of cycles necessary to propagate any buried unmelted portion of crack to the surface. As shown in Figure 56, successful retrofitting increased fatigue life by as much as 160 percent. Deeper cracks (greater than 0.2 in.) only yielded up

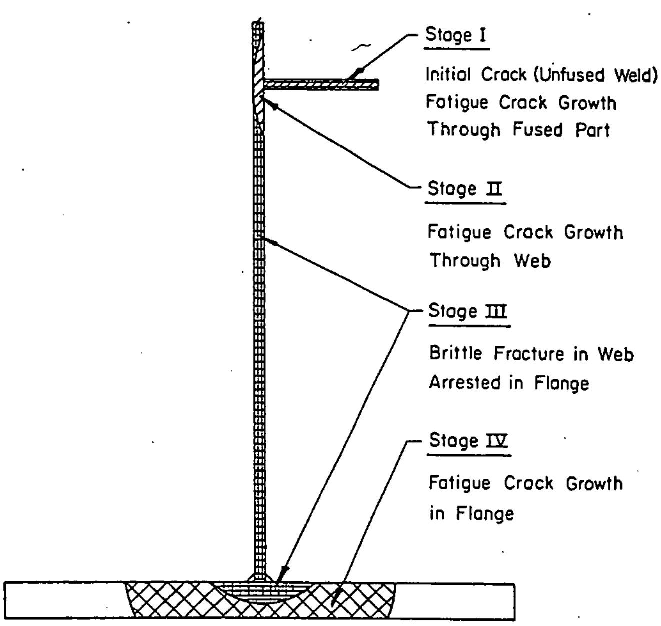


Figure 51. Stages of crack growth from longitudinal stiffener weld.

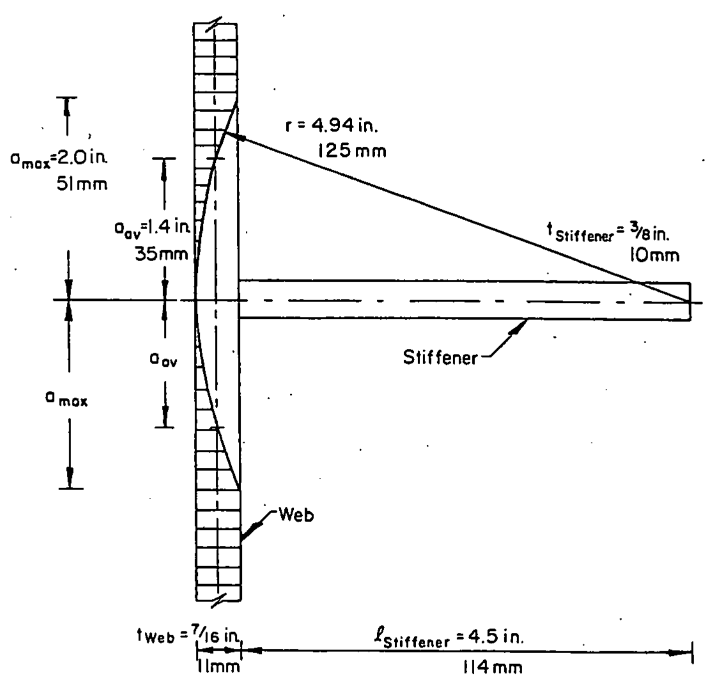


Figure 52. Analytical model of crack development.

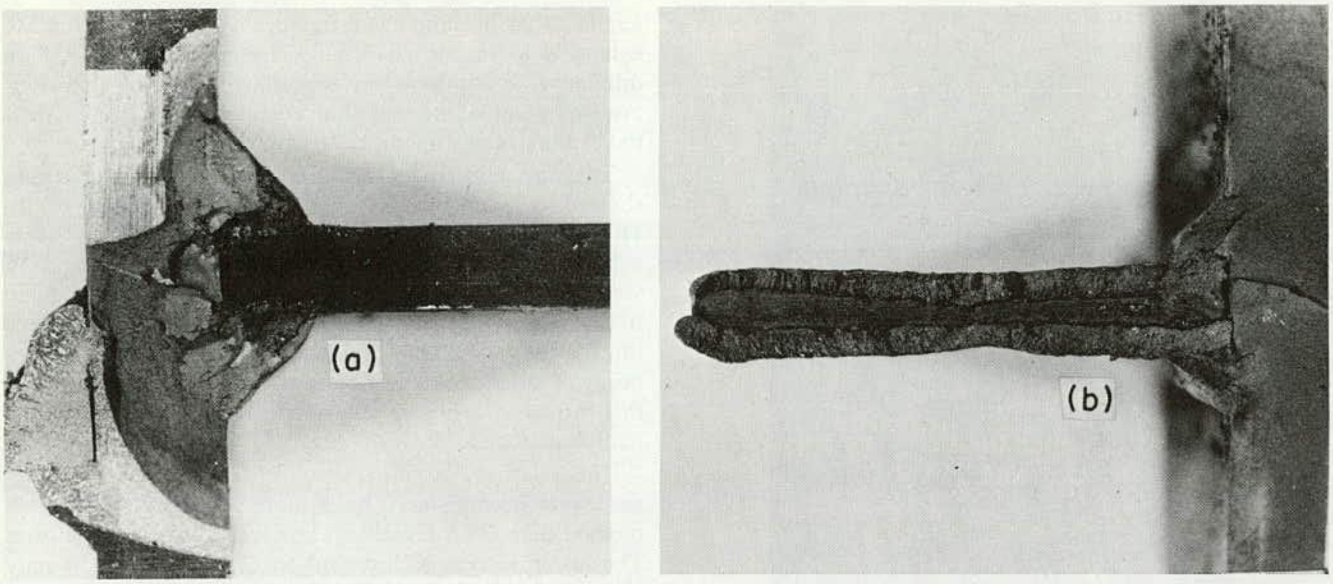


Figure 53. (a) Crack growth in web from longitudinal stiffener lack of fusion region; (b) lack of fusion in longitudinal stiffener.

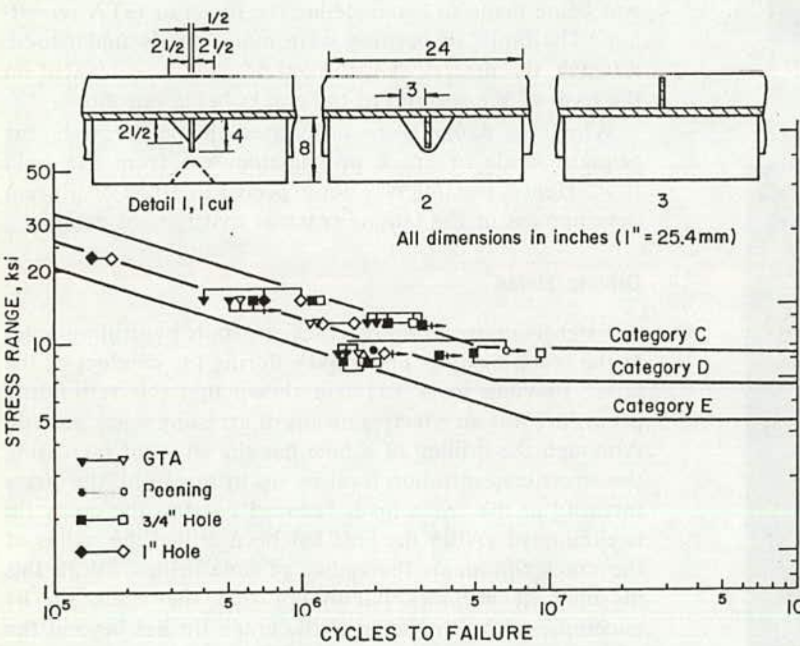


Figure 54. Increase in fatigue life of web gusset plates after retrofitting.

to 11 percent increase in life. The relatively longer 2-in. (5-cm) welds at the ends of the attached flanges as compared to the web gussets allowed a greater chance of detecting a shallow crack before it propagated a significant amount into the web. For this type detail, the typical increase due to retrofitting is equivalent to the increment to the next design category.

Peening

Peening of weld toes as a retrofitting technique is successful only on very small shallow cracks (i.e., crack length less than 2-in. (50-mm) and an estimated depth of less than 0.125 in. (3 mm), or no visible crack) and a low stress range with the simulated dead load applied during retrofitting (*5*). The level of dead load (minimum stress)

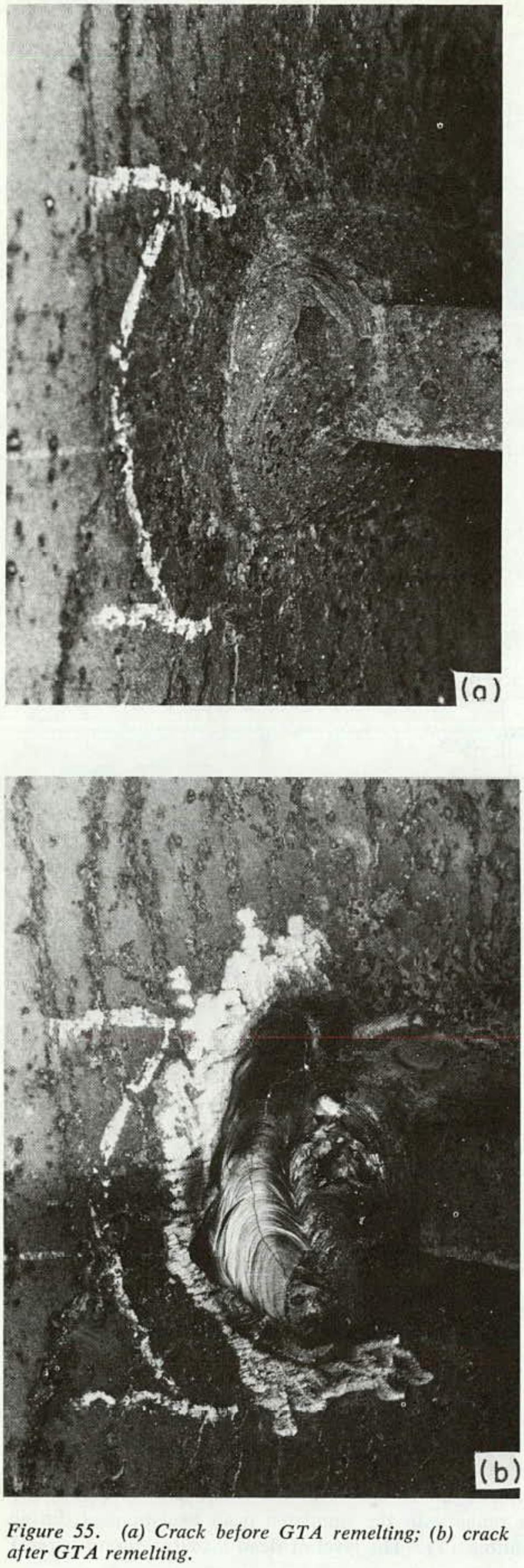


Figure 55. (a) Crack before GTA remelting; (b) crack after GTA remelting.

applied after peening has a significant influence on the life extension for a peened detail. The application of a high minimum stress decreases the effectiveness of the compression residual stresses that are introduced at the weld toe by the peening operation.

The web gusset plate detail when retrofitted by peening showed as much as a 240 percent increase in fatigue life (Fig. 56). This was a much greater improvement than was achieved on the 2-in. (5-cm) thick flange framing into web details that experience only up to a 31 percent fatigue life increase (Fig. 56). The lesser improvement of fatigue life for the latter detail is because of the relatively slower decay of stress concentration at the thick flanges. In effect, this results in a higher actual stress range than that normally calculated.

This relatively large increase in life due to peening suggests that peening may be a more successful retrofitting method than GTA remelting; however, this is not the case. The lower success with regard to GTA remelting is only because the depth limitations for peening were adhered to more rigidly than those for GTA remelt. For example, if a crack was deemed too deep to successfully retrofit by peening, it was most likely treated by GTA remelting without regard to depth considerations because an attempt was being made to better define the limits of GTA remelting. The limits of peening were more clearly understood. As such, the success of each method must be evaluated on the basis of the severity of the cracks being retrofitted.

When the flange plate penetrated the girder web, the primary mode of crack propagation was from the weld root. Hence, peening was not expected to arrest or prevent development of the fatigue crack at that type of detail.

Drilling Holes

Extensive retrofitting of cracked details by drilling holes at the crack tips was undertaken during the conduct of the tests. Previous work (5) had shown that this retrofitting procedure was an effective means of arresting crack growth. Although the drilling of a hole has the effect of increasing the stress concentration level by up to threefold, the stress intensity at the crack tip is reduced because the crack tip is eliminated. After the hole has been drilled the radius of the crack tip equals the radius of hole drilled. With this in mind, it becomes paramount that the crack tip be encompassed by the hole. If the crack tip lies beyond the hole, the crack tip will not be blunted and the stress concentration may accelerate the crack propagation. During the test program, the location for holes was changed from the hole center at the visually determined crack tip to locating the assumed crack tip at the near perimeter of the hole (see Fig. 57) to ensure that the actual tip falls within the hole.

This method of retrofitting has a greater advantage over the previous methods discussed because much larger cracks can be successfully treated. The method was used after the crack path was clearly defined, and a 0.75-in. (1.9-cm) or 1.0-in. (2.5-cm) diameter hole was drilled. Improved resistance was obtained by placing tightened high strength bolts in the drilled holes and thereby producing a zone of

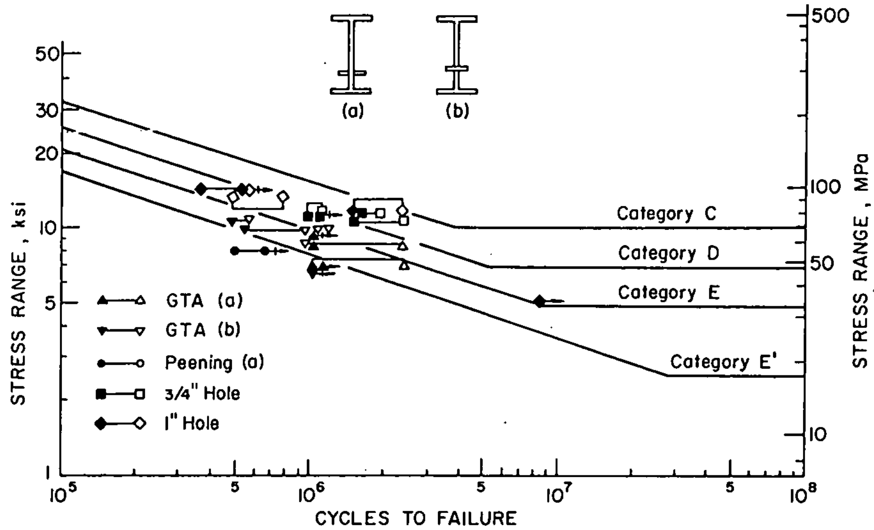


Figure 56. Fatigue life increases by retrofitting details that frame into or are inserted through the web.

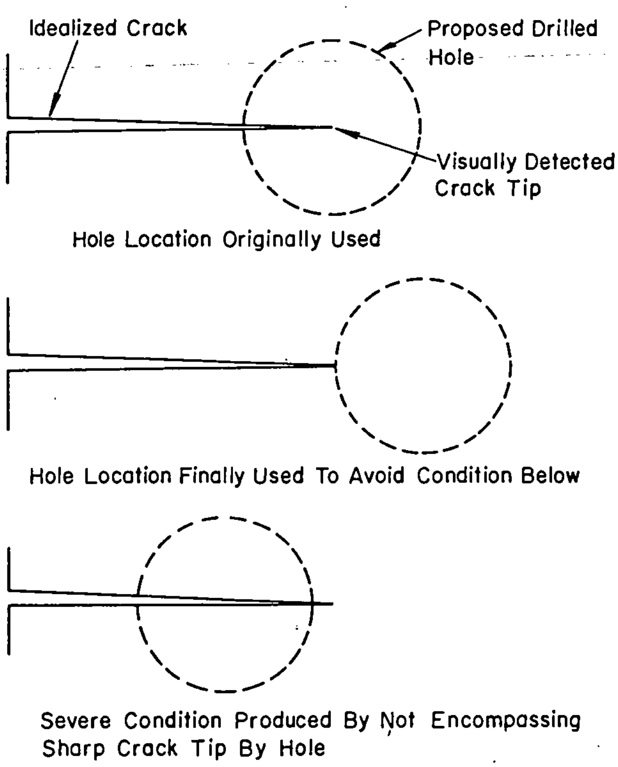


Figure 57. Crack tip and hole perimeter locations.

compressive stress. However, the presence of bolts and washers made it difficult to determine when a crack reinitiated from the holes. Once the crack would grow long enough to detect, the rate of propagation was extremely fast (see Fig. 58).

Through the observation of reinitiation of fatigue cracks from drilled holes, a threshold value of $\Delta K/\sqrt{\rho}$ was established using the procedure described by Rolfe and Barsom (*10*). Plate specimens had indicated that the crack would not reinitiate if $\Delta K/\sqrt{\rho} < 10\sqrt{\sigma_y}$ (for σ_y in ksi). The stress intensity factor, K, was calculated for a fatigue crack with length equal to outer edges of the drilled holes. The variable, ρ, is the radius of the drilled holes. The values of $\Delta K/\sqrt{\rho}$ provided by these tests are plotted in Figure 59 as a function of the cycles needed to reinitiate the crack. These results suggest that threshold value of $\Delta K/\sqrt{\rho} \simeq 4\sqrt{\sigma_y}$ (σ_y in ksi) (10.5 $\sqrt{\sigma_y}$ (σ_y in MPa)) is required to prevent crack reinitiation.

It is apparent that the welded detail has influenced the level of $\Delta K/\sqrt{\rho}$ necessary to reinitiate the fatigue crack. The thinner web beams (W27 x 114) and thinner details, such as the web gussets which permitted the holes to be closer together thereby yielding lower ΔK values, provided a more effective retrofit (see Fig. 59).

The beam tests (plotted in Figs. 54 and 56) indicate that the drilled holes were an effective means of extending the fatigue resistance.

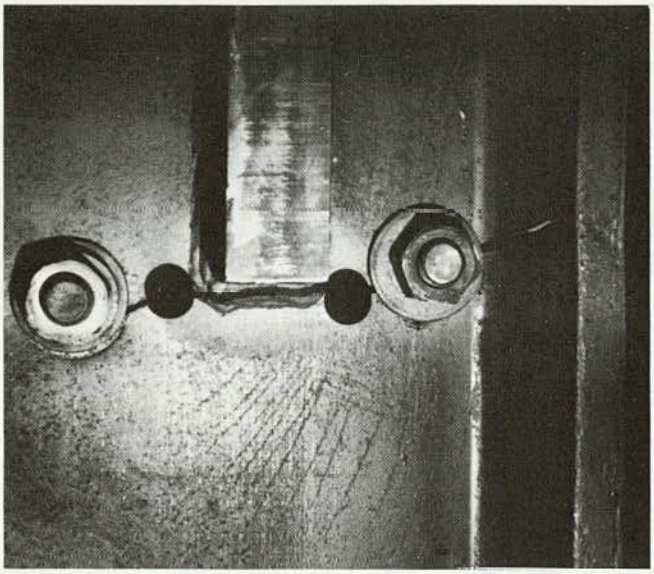


Figure 58. Crack repropagated from holes.

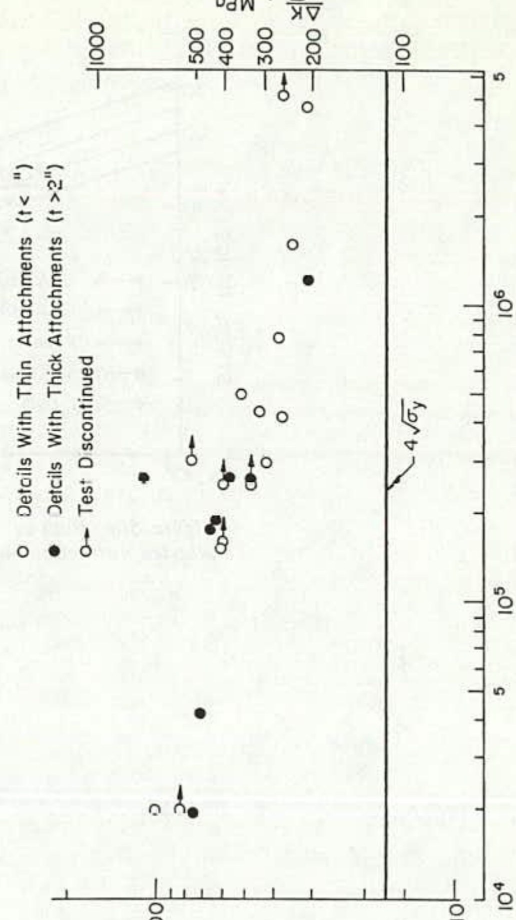


Figure 59. $\Delta K/\sqrt{\rho}$ vs. fatigue life increase.

CHAPTER FOUR

APPLICATION OF RESULTS

The findings from this study should be of value to structural engineers involved in the design of welded steel girders, researchers working in the subject area, and members of specification writing bodies. The suggested revisions to the AASHTO *Standard Specifications for Highway Bridges* included here warrant consideration. These suggested revisions can also be applied to other specifications, such as those of the American Institute of Steel Construction and the American Railway Engineering Association. These findings result from a comprehensively designed and executed experimental effort verified by analyses of crack propagation and fracture mechanics and need consideration for immediate adjustments in design specifications.

FATIGUE BEHAVIOR OF WEB GUSSET PLATES

1. This study has shown that the fatigue classification (Category E) assigned to the ends of lateral connection plates connected to the girder web is satisfactory. Four types of web lateral connection plates were evaluated and all provided about the same fatigue resistance. Two types were welded to a transverse stiffener as well as to the girder

web. One type of gusset plate was welded to the girder web opposite the stiffener, and the fourth type was framed around the transverse stiffener without direct attachment.

None of the gusset plates produced undesirable secondary stresses or deflections in the girder web. Significant changes in the stiffness of the lateral bracing and in the end reactions for the bracing had no influence on the fatigue behavior.

2. Cracks were detected in the load-carrying fillet welds that attached the gusset plates to the transverse stiffeners. This cracking developed in undersize welds adjacent to the girder web and was not detected throughout tests. The results of these tests indicate that it is difficult to detect this type of cracking. Furthermore, the behavior observed indicates that intersecting welds in the corner of the gusset-web-transverse stiffener connection is extremely undesirable. Such conditions are likely to lead to cracking in the transverse connection that can propagate into the girder web as has been observed at several groove-welded attachments that had lack of fusion in this same connection.

3. No cracking was detected in the web gaps of any of the three types of gusset connections that were tested with such gaps. A destructive examination of these regions after completing the cyclic testing verified this.

FATIGUE BEHAVIOR OF GUSSET PLATES ATTACHED TO FLANGE TIPS

1. The results of this study and other test results available in the literature indicate that the classification assigned to flange tip gusset plates attached by groove welds is satisfactory. None of the details with 2-in. (50-mm) and 6-in. (150-mm) end radii developed fatigue cracking. They indicated that the classification provided by Category C was satisfactory.

2. Groove-welded gussets with no radius transition at the ends but that were ground and resulted in a short radius (0.2 to 0.4 in. (5 to 10 mm)) achieved a fatigue resistance compatible with Category D. The cracks that formed at these "0" radius details all developed from subsurface discontinuities in the longitudinal weldments in the transition region provided by the short radius. Hence, grinding the ends of Category E groove-welded attachments can provide a significant improvement in fatigue resistance.

3. Attachment of the lateral bracing to the flange gusset plates produced no ill effects as a result of distortion. No indication of cracking was detected in the weldment or gusset at the flange tip connection.

4. A review of the laterature and the behavior observed in these tests indicated that fillet-welded connections with radius end transitions have a high probability of discontinuities at the fillet weld roots. Fatigue cracking developed from subsurface discontinuities in the transition radius near the point of tangency, and the difference in behavior of the 2-in. (50-mm) groove-welded details and the 4-in. (100-mm) fillet-welded details suggests that fillet-welded transition radius connections should not be used for Category C and B design conditions.

5. It is recommended that the specifications limit the use of fillet-welded connections with radiused end transitions to a Category D classification.

FATIGUE BEHAVIOR OF GUSSETS ATTACHED TO FLANGE SURFACE

Gusset plates welded to the surface of a girder flange by transverse welds alone did not provide satisfactory fatigue resistance. Fatigue cracking developed from the weld root and severed the gusset plates from the flange surface at fatigue lives well below Category E. In one of the test girders, cracking also developed in the girder flange as a result of the gusset plate connection.

It is recommended that gusset plates with only transverse end welds that are perpendicular to the cyclic stress not be used.

FATIGUE BEHAVIOR OF FLANGES FRAMING INTO OR PIERCING THROUGH GIRDER WEBS

1. A significant difference was observed between 2-in. (50-mm) thick flange plates fillet or groove welded to a girder web and ¾-in. (20-mm) thick flanges. The heavier flange plates resulted in a fatigue resistance comparable to large cover-plated beams (Category E′). The smaller flange plates were comparable to the web gusset plates and provided a fatigue resistance in agreement with Category E.

It is recommended that, when girder flanges with thickness equal to or greater than 1 in. (25 mm) are fillet or groove welded to a girder web, they be designed in accord with Category E′. Flanges less than 1 in. (25 mm) thick can continue to be treated as Category E connections.

2. When girder flanges greater than 1 in. (25 mm) pierce through a web and are fillet welded to each side of the web, a fatigue resistance comparable to Category E′ results. The resistance of such connections is also dependent on the thickness of the web plate through which such flanges pass. Three-fourth-in. (20-mm) thick flanges that were fillet welded to a ¼-in. (6.4-mm) web yielded the same fatigue resistance whether they passed through the web or were welded to its surface. Category E was found to be applicable for such cases.

3. Welding to each side of a flange that penetrates through a girder web significantly improves the fatigue resistance. Previous studies had demonstrated that "seal" welds on one side of a girder web were very low fatigue resistant details. Such connections provided a fatigue strength well below Category E′ and should not be used in cyclic tension areas.

These studies and the earlier studies have indicated that it is undesirable to penetrate a web plate thicker than ¼ in. (6.4 mm) and weld around leaving a large unfused region. The designer is urged to avoid such details.

RETROFITTING METHODS

1. This study has shown that the two methods used to retrofit cover-plated beams at weld toes—peening the weld toe region or applying a gas tungsten arc remelt pass—can be applied successfully to other types of welded details, such as web gusset plates, stringer flanges welded to girder webs, and other comparable details.

Both methods were effective when crack growth was developed from the weld toe and the crack was discovered when its depth was less than 0.125 in. (3 mm) deep.

2. Generally the crack shape at the end of short transverse welds at the ends of the web gusset plates was not as favorable for detection as the longer but shallower cracks that formed at the ends of the thicker attachments. As a result it was easier to detect the cracks at the ends of the thicker attachments.

3. It was not possible to retrofit cracks that developed from the weld root. Several of the fillet-welded flanges that were attached to or pierced the beam web developed cracking from weld root, and attempts to retrofit such cracks were not successful.

4. Drilling holes at the tips of fatigue cracks that formed at the ends of the web attachments was found to be a very effective means of arresting fatigue crack growth. It was found that such fatigue cracks could be prevented from reinitiating at the drilled holes when the relationship

$$\frac{\Delta K}{\sqrt{\rho}} < 4\sqrt{\sigma_y} \quad (\sigma_y \text{ in ksi})$$

$$\left[\frac{\Delta K}{\sqrt{\rho}} < 10.5\sqrt{\sigma_y} \quad (\sigma_y \text{ in MPa})\right]$$

was satisfied. The relationship developed from simple plate specimens by Rolfe and Barsom (*10*) was found to be unconservative.

5. When holes were drilled at the ends of fatigue cracks it was found desirable to place the hole so that its perimeter was at the apparent crack tip to ensure that the crack did not extend beyond the hole (Fig. 57).

CHAPTER FIVE

CONCLUSIONS

The conclusions in this chapter are based on an analysis and evaluation of the test data acquired during this study, on the results of other work, or on theoretical studies based on the application of the fracture mechanics of fatigue crack growth.

FATIGUE BEHAVIOR OF WEB GUSSET PLATES

1. All web gusset plates resulted in fatigue cracks developing at the ends of the gussets at all levels of stress range. The fatigue strength was found to be compatible with Category E.

2. No cracking was detected in the girder web in the gaps at the transverse stiffeners.

3. The web gusset need not be welded to the transverse stiffener, provided an adequate gap is provided (approximately 3 in. or greater).

4. Web gussets that are welded to the transverse stiffener risk development of fatigue cracking from the weld root of the load-carrying fillet adjacent to the girder web.

5. Connecting lateral bracing to the gusset had no influence on its fatigue behavior.

FATIGUE BEHAVIOR OF GUSSET PLATES ATTACHED TO FLANGE

1. The flange gusset plates in this study confirmed the applicability of the AASHTO specifications for transition radius details attached to the flange tip.

2. Groove-welded gusset plates with 2-in. (50-mm) and 6-in. (150-mm) transition radii provided superior fatigue behavior compared to other studies on fillet-welded web gussets with 4 in. (100 mm).

3. Grinding a small 0.2 to 0.4-in. (5 to 10-mm) radius at the ends of rectangular groove-welded gussets resulted in fatigue behavior compatible with Category D details.

4. There was no detectable effect of connecting the lateral bracing members to the flange gusset plates.

5. Gusset plates welded to the surface of the flange with transverse welds alone did not provide adequate fatigue resistance. All details cracked at cycle lives below Category E.

FATIGUE BEHAVIOR OF FLANGES FRAMING INTO OR PIERCNG THROUGH GIRDER WEBS

1. Flanges greater than 1 in. (25 mm) thick that frame into girder webs should be assigned to Category E′ of the AASHTO specifications.

2. When flanges pierced the girder web and fillet welds were placed around the opening on each side of the web, an improvement in fatigue resistance was observed. The improvement in fatigue resistance was found to be dependent on the web thickness and the depth of fusion at the flange tip.

3. Semicircular end copes (see Fig. 47) at the tip of thick flanges (2 in. (50 mm)) that pierced the web and attached to it by fillet welds provided a fatigue resistance less than Category E′.

RETROFITTING PROCEDURES

1. Shallow surface cracks that form at the ends of welds attaching gusset plates or flanges to the girder web were successfully retrofitted by peening or gas tungsten arc remelting the cracked area.

2. Cracks forming at the ends of plates less than 1 in. (25 mm) thick were more difficult to detect at an early stage because of the length of the crack and its relationship to its depth. Cracks that formed at the end of the thick flange plates were more readily detected and therefore more successfully retrofitted.

3. It was not possible to retrofit cracks that formed at the weld root using the peening or gas tungsten arc remelting.

4. The placement of holes through the girder web at the tip of cracks that formed at welded details was found to be a satisfactory means of preventing further fatigue crack growth providing the relationship

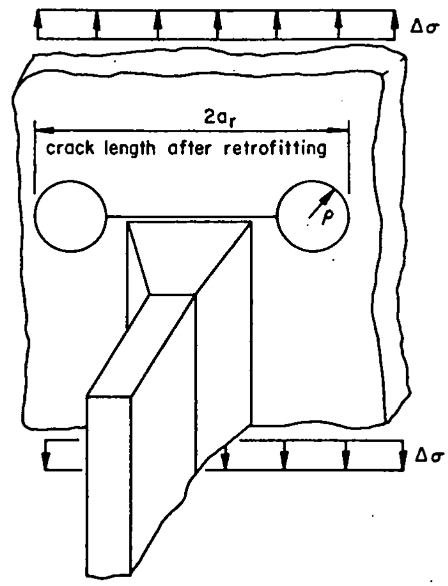
$$\frac{\Delta K}{\sqrt{\rho}} < 4\sqrt{\sigma_y}\ (\sigma_y \text{ in ksi})$$

$$\left[\frac{\Delta K}{\sqrt{\rho}} < 10.5\sqrt{\sigma_y}\ (\sigma_y \text{ in MPa})\right]$$

is satisfied. An example of the application of this relationship is shown in Figure 60.

5. Placing tightened (turn-of-nut) high strength bolts in the drilled holes improved the fatigue resistance. However, the bolts made it more difficult to inspect the hole.

Example Problem : Cracked detail fabricated of A36 steel, $\Delta\sigma = 6$ ksi, $\rho = 1/2''$



To Prevent Reinitiation,

$$\frac{\Delta K}{\sqrt{\rho}} < 4\sqrt{\sigma_y}$$ Empirical relationship developed from these experimental results.

but $$\Delta K = \Delta\sigma\sqrt{\pi a}$$ From fracture mechanics

$$\frac{\Delta\sigma\sqrt{\pi a_r}}{\sqrt{\rho}} < 4\sqrt{\sigma_y}$$

$$2a_r < \frac{32\,\sigma y\,\rho}{\pi(\Delta\sigma)^2}$$

For our example,

$$2a_r < 5.1''$$

Figure 60. Example problem—$\Delta K/\sqrt{\rho}$ threshold.

CHAPTER SIX

RECOMMENDATIONS FOR FURTHER RESEARCH

The experimental work documented in *NCHRP Report 102* and *NCHRP Report 147* has indicated that the primary factors influencing fatigue strength are the range of stress and the type of detail. These studies show that the cover-plated beam represents one of the most severe conditions that can be expected and yields a lower bound condition. A number of other details were shown to exhibit comparable behavior.

In the studies documented in *NCHRP Report 206,* two methods of strengthening severe notch producing details, such as the cover-plated beams, were found to be successful. Peening the weld toe region was observed to strengthen details when peening was done in the presence of the minimum stress (dead load). The application of a gas tungsten arc remelt pass at the weld toe was also observed to provide a satisfactory method of retrofitting or improving the fatigue strength of as-welded and precracked details when the depth of crack penetration did not exceed 0.188 in. (5 mm). In the studies reported herein, it was shown that these two techniques could be successfully applied to other attachments and details providing the depth of crack penetration did not exceed 0.1 in. (2.5 mm). Groove-welded and fillet-welded attachments to the webs of girders could be retrofitted when the cracks formed at the weld toe.

The studies on thick flanges framing into or piercing through the girder web continued to confirm that certain details were susceptible to size effects. The thicker flange attachment tended to decrease the fatigue resistance.

These studies have pointed out the need for additional research concerning improvement procedures. It is recommended that consideration be given to the following studies, so that appropriate design criteria and retrofit procedures and techniques can be developed. These recommendations are not provided in a priority listing.

IMPROVEMENT TECHNIQUES

1. Preliminary studies indicate that the gas tungsten arc remelt procedure does not metallurgically damage the repaired toe area; however, the remelt procedure should be applied to other structural steels to determine if this is also true for the higher strength heat-treated materials. Some high strength steels, such as those in the A514 group, are much more sensitive to the heat of welding. The extensive heating of the flange area necessary to eliminate existing deep cracks may have an adverse effect on the properties of the repaired area. Research extending the tungsten arc remelt procedure to one or more of those steels would establish the suitability of this repair method to newer materials.

2. Further studies are needed on the effectiveness of arc air gouging and subsequent rewelding of fatigue-cracked details. Gouging and rewelding should be similar to GTA remelting; moreover, it could be applied to much deeper cracks. Inasmuch as this procedure involves the deposition of weld metal, it may also lead to high residual stresses and weld defects. Existing laboratory experience favors this as a repair method; however, no systematic study of this repair procedure has been undertaken.

3. Further studies are needed on groove-welded attachments and cross beams because these are commonly used details. The groove-welded plate attached to a tension flange tip or beam web is commonly used in bridge construction to attach lateral bracing and diaphragms. Often, nothing is done with the weld-toe termination on the flange tip or web, and crack growth can be expected at these weld toes under high cyclic stress. Further experimental studies are needed on this detail; such studies would provide an opportunity to examine the improvement provided by peening or the gas tungsten arc remelt dressing of the weld toe as well as changes in transition geometry. The ground details used in this study prevented examination of these techniques.

SIGNIFICANCE OF MANUFACTURING AND FABRICATION DISCONTINUITIES

1. Studies are also needed to evaluate the significance of a variety of manufacturing and fabrication discontinuities. For example, the influence of seams, laminations, inclusions, and weld discontinuities that are parallel to the stress field need to be examined systematically. Although an inclusion, seam, or other comparable discontinuity may be critical when oriented so that the applied forces are perpendicular to the discontinuity, available information indicates that such discontinuities have a minor or negligible effect when located parallel to the line of stress. Seams have been observed in cover-plated beams with the longitudinal welds attaching the cover plate to the beam flange adjacent or over the longitudinal seams, and, in some cases, shrinkage has opened the seams. Other conditions that have developed are in rolled plates where seams have opened along the edge when welds have been placed along the plate surface near the edge. Although analytical indications and available information point out that such observations are not significant, tests are desirable to confirm this.

A number of fabricated beams are occasionally identified as having discontinuities on the surface and are rejected. An attempt could be made to acquire rejected members and to obtain from the mills beams having discontinuities. Fatigue tests on these beams would provide an indication as to their severity and significance.

Other details that penetrate the web of a girder may exist. A careful review and experimental assessment of such details are needed. For example, pipe penetrations through box girders and other comparable conditions that have been "seal" welded into place should be examined.

2. Longitudinal groove welds are commonly made with a backup bar when fabricating box girder sections. Studies are needed on this weld configuration to determine whether or not the flaw condition and fatigue strength under these welding conditions are more severe than they are under the plain welded beam. Many other forms of welded plate construction, such as orthotropic bridge decks, use welding from one side as well.

Various means exist for making the one-sided groove weld, including the use of backup bars and glass tape. If permanent backup bars are used, it has been common practice to attach them to one of the plates by tack welding prior to making the groove weld. It has also been the practice in the past to use several individual lengths of backup bar when long groove welds are made. These individual lengths have not been groove welded together, in many cases, and this discontinuity is believed to cause a substantial reduction in fatigue strength.

Individual lengths of bars also exist in box members when interior attachments such as diaphragms, girder webs, or brackets interfere with the continuity of the bar. These create notchlike conditions where the bar has been butted up against the detail. The significance of these conditions needs further study.

At present, there are no known studies on beams having welds made with a glass tape for the backup. However, glass tape backups can produce very smooth contours and may prove superior. This study could define some of the conditions under which backup tapes may be used and any limitations on their use in this configuration. The results of this study would yield information vital to assessing the strength of numerous bridges that have been fabricated with discontinuous backup bars. In addition, the examination of intermittent tack welds would provide much needed information on this detail, which is currently classified as a Category E condition, because so little test data are available.

ADDITIONAL DETAILS

1. Further studies are needed to define the behavior of a variety of attachments to beams and girders. These details should be placed on deeper girders and thicker flanges. Among factors requiring consideration are the relationship of stiffener or other attachment thickness to the flange or web thickness, the geometrical configuration of the weld, and sizes of copes. These variables can be examined, and the effectiveness of the suggested repair and improvement procedures can be evaluated on the same specimens.

2. Details not readily classified by existing fatigue design provisions need to be fatigue tested to verify the suitability of an assumed category. This would minimize the possibility of low fatigue strength being used in bridge structures.

3. The currently used AASHTO Category F provides design values for welds in shear. This category was developed from test data acquired in the 1940's. It results in the only stress range cycle life relationship that is not compatible with other details because it deviates substantially from the slope of all other details. This is particularly unsatisfactory at the lower stress range—long design life region. Here, greater fatigue strength is implied than may be available.

This study should involve not only supplemental fatigue tests but also a comprehensive review of existing test data that have developed since 1950. A large number of tests have been made on cruciform joints that have failed in the weld. However, additional experimental work is needed near the crack growth threshold and on longitudinal welds, as well as at this lower stress level. In addition, a satisfactory design procedure should be developed because a fracture mechanics solution is not readily applied to design.

The root cracking that developed in the full-size cover-plated beams and the earlier studies on smaller scale beams suggest a need for further studies in this area. The cracking that developed from the root of the welded connection between the gusset plate and transverse stiffener further confirmed this need.

4. Further studies of groove-welded flange gussets are needed to investigate their behavior over wider stress range levels. This may permit a relaxation in the required radius transition dimensions.

HIGH CYCLE FATIGUE

1. Additional work is needed in the extreme life region of most fatigue categories. Only the cover-plated beam detail has been subjected to stress cycles in the 10^7 to 10^8 region. The suitability of existing design criteria is dependent on the adequacy of fatigue resistance in this region.

2. Further studies are needed on randomly applied variable stress cycles near the crack growth threshold (fatigue limit region). The existing laboratory and field experience suggests that more fatigue damage than anticipated will develop when some stress cycles in the spectrum exceed the crack growth threshold. It is not known what frequency of occurrence of such events is necessary to result in fatigue cracking.

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APPENDIX A

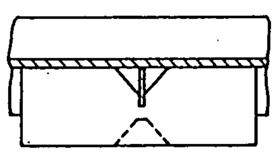
TABLES OF TEST RESULTS

Tables A-1 through A-9 summarize all data points acquired in this study. A sketch at the top of each table shows the different combinations of details and the beams to which they were attached. The column in the tables identified as "No. of Cycles to Crack" indicates the time at which first visual detection was made with a 10X magnifier. "No. of Cycles to Retrofit" provides the number of cycles to a through thickness crack at which time fatigue resistance is nearly exhausted and failure is considered to have occurred. The retrofit methods are those described in Chapter Three. If a crack reinitiated after retrofit, the first visual detection of the crack is listed under "Cycles to Retrofit Failure." If the crack was consequently re-retrofitted, the data are continued onto a second line in the table. "No of Cycles to End of Test" represents the maximum number of cycles to which the beam was subjected when it was removed from the testing bed after it was concluded that no more useful data could be obtained from any of the details.

The tables pertain to individual details, and only those details that contributed data points are listed.

TABLE A-1

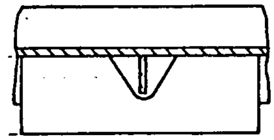
TEST RESULTS ON TYPE 1 AND TYPE 1-CUT WEB GUSSETS



Cross-Section	Beam No.	Stress MPa	Range (ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X145		62.1	(9)	4.68	4.98	X					9.60	9.60	Weld Toe
W27X114		89.6	(13)	1.26	--						--	1.46	Weld Toe 1
		89.6	(13)	1.26	--						--	1.46	Weld Toe
		103.4	(15)	1.02	1.02	X					1.09	1.09	Weld Toe
		103.4	(15)	1.02	1.02	X					1.09	1.09	Plates clamped to lower flange
W36X160		82.7	(12)	1.17	1.17				X		1.18	--	Consecutive Retrofits
		82.7	(12)	1.18	1.18		X				1.67	1.91	
		144.8	(21)	0.14	0.14						--	0.16	
		144.8	(21)	0.14	0.14		X				0.16	0.16	Reinitiation from lower hole

TABLE A-2

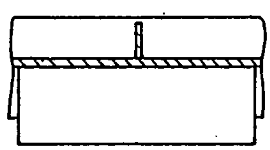
TEST RESULTS ON TYPE 2 WEB GUSSETS



Cross-Section	Beam No.	Stress MPa	Range (ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X145	4	82.7	(12)	1.77	2.11	X						2.34	Weld Toe
	4	82.7	(12)	1.81	2.11	X						2.34	Weld Toe
W27X114	18	62.1	(9)	1.37	1.37				X		1.70	--	Consecutive Retrofits
	18	62.1	(9)	1.70	1.90		X				--	6.99	
	18	62.1	(9)	1.37	1.37				X		1.90	--	Consecutive Retrofits
	18	62.1	(9)	1.90	1.90					X	--	6.99	
	16	103.4	(15)	0.50	0.50				X		0.57	0.80	Weld Toe
W36X160	7	62.1	(9)	3.50	3.50	X					--	3.84	Weld Toe
	6	82.7	(12)	1.60	1.85				X		2.00	--	
	6	82.7	(12)	2.00	2.00	X					2.78	--	Consecutive Retrofits
	6	82.7	(12)	2.78	2.78	X		X			--	3.50	
	6	82.7	(12)	1.60	1.85				X		--	3.50	

TABLE A-3

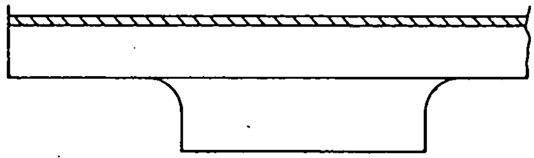
TEST RESULTS ON TYPE 3 WEB GUSSETS



Cross-Section	Beam No.	Stress MPa	Range (ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X145	9	82.7	(12)	1.50	1.89	X					--	1.98	Weld toe
W27X114	10	62.1	(9)	1.37	1.37	X					1.78	--	Consecutive Retrofit
	10	62.1	(9)	1.78	1.78	X		X			--	2.45	
	10	62.1	(9)	1.78	1.78	X		X			2.08	2.45	Weld toe
	11	103.4	(15)	0.57	0.63	X					--	0.78	Weld toe
W36X160	12	103.4	(15)	0.56	0.84		X				0.99	1.29	
	12	103.4	(15)	0.40	0.40				X		0.70	--	Consecutive Retrofits
	12	103.4	(15)	0.70	0.84		X				1.13	1.29	

TABLE A-4

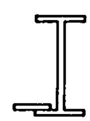
TEST RESULTS ON GROOVE-WELDED FLANGE GUSSETS



Cross Section	Beam No.	Radius	Stress Range MPa	(ksi)	Cycles to Initial Cracking (10^6)	Defect Size 2a mm	(in.)	2b mm	(in.)	Cycles to end of Test (10^6)	Remarks
W27X114	14	0	103.4	(15.2)	1.09	2.7	(0.108)	4.3	(.170)	1.09	Found by destructive Inspection
	14	0	103.4	(15.0)	1.09	1.5	(0.060)	5.0	(0.195)	1.09	Destructive Inspection
	11	6	103.4	(15.0)	0.78	12.6	(0.496)	16.6	(0.654)	0.78	Destructive Inspection
W36X160	8	0	82.7	(12.0)	1.91	22.9	(0.902)	31.5	(1.240)	1.91	Destructive Inspection
	8	0	82.7	(12.0)	1.91	--	--	--	--	1.91	Visual Inspection
	8	0	47.9	(7.0)	1.91	14.2	(0.558)	25.8	(1.016)	1.91	Destructive Inspection
	15	0	144.8	(21.0)	0.16	42.0	(1.652)	42.2	(1.660)	0.16	Fatigue Failure Visual Inspection
	15	0	83.8	(12.2)	0.16	3.4	(0.134)	8.8	(0.346)	0.16	Destructive Inspection

TABLE A-5

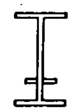
TEST RESULTS ON FLANGE GUSSETS WITH ONLY TRANSVERSE END WELDS



Cross-Section	Beam No.	Stress Range MPa	(ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X114	13	73.1	(10.6)	1.22	--						--	1.46	Instantaneous Cracked entire length of weld
	11	84.1	(12.2)	0.42	--						--	0.78	
	14	34.5	(5.0)	0.35	--						--	1.09	
	14	84.1	(12.2)	0.35	0.56		X				--	1.09	Hole drilled in flange plates clamped to flange
W36X160	12	88.3	(12.8)	0.07	0.12					X	0.14	1.29	Initial crack 76 mm (3 in.)

TABLE A-6

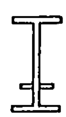
TEST RESULTS ON 2-IN. THICK FLANGE PLATES FILLET WELDED TO GIRDER WEB



Cross-Section	Beam No.	Stress MPa	Range (ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X145	3	39.3	(5.7)	4.33	--						--	17.73	
	2	78.9	(11.4)	0.78	0.78	X	X				0.97	0.98	Failure at both holes
	4	60.9	(8.8)	1.77	--						--	2.36	
	4	78.9	(11.4)	1.09	1.43	X	X				1.52	2.36	
W27X114	18	42.7	(6.2)	4.94	4.94		X				--	6.99	
	18	55.8	(8.1)	4.94	4.94		X				6.15	6.99	
W36X160	6	64.1	(9.3)	0.49	0.68					X	0.94	--	Consecutive Retrofits
	6	64.1	(9.3)	0.94	0.94					X	--	3.50	

TABLE A-7

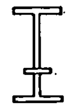
TEST RESULTS ON 2-IN. THICK FLANGE PLATES GROOVE WELDED TO GIRDER WEB



Cross-Section	Beam No.	Stress MPa	Range (ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X145	9	78.9	(11.4)	1.50	1.50	X					1.96	1.96	
W27X114	10	59.3	(8.6)	0.96	0.96				X		2.45	2.45	
	10	45.5	(6.6)	0.96	0.96				X		2.45	2.45	
W36X160	7	35.2	(5.1)	3.50	--						--	3.84	Weld toe
	7	48.0	(7.0)	3.50	--						--	3.84	Weld toe
	8	47.0	(6.8)	1.17	1.17				X		--	1.91	
	8	64.1	(9.3)	1.17	1.17				X		--	1.91	
	8	64.1	(9.3)	1.06	1.06				X		--	1.91	

TABLE A-8

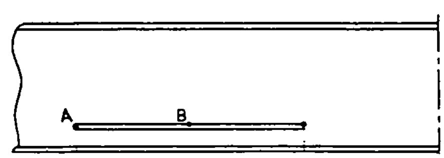
TEST RESULTS ON 2-IN. THICK FLANGE PLATES INSERTED THROUGH GIRDER WEB



Cross-Section	Beam No.	Stress Range MPa	(ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X145	9	76.0	(11.4)	1.08	1.08	X		X			--	2.00	Plates clamped to lower flange
W27X114	13	80.7	(11.7)	0.91	0.91	X					1.17	1.46	Plates clamped to lower flange after retrofit failure
	11	98.6	(14.3)	0.35	0.35		X				0.52	--	Consecutive Retrofits
	16	71.6	(10.4)	0.50	0.50				X		0.57	0.80	
	16	92.8	(13.5)	0.43	0.50		X	X			0.80	0.80	Lower hole only bolted
W36X160	8	47.0	(6.8)	1.17	1.17				X		--	1.91	
	8	64.1	(9.3)	1.06	1.06				X		1.18	1.91	
	12	50.5	(7.3)	0.97	--						--	1.29	
	12	69.0	(10.0)	0.55	0.55				X		0.97	1.29	
	15	80.0	(11.6)	0.14	0.14		X				0.16	0.16	Coped Ends
	17	35.3	(5.1)	8.25	8.63		X				--	--	Coped Ends

TABLE A-9

TEST RESULTS ON LONGITUDINAL STIFFENERS WITH PARTIAL PENETRATION BUTT WELDS



Cross-Section	Beam No.	MPa	(ksi)	Cycles to Initial Cracking (10^6)	Cycles to Retrofit (10^6)	Retrofit Method: 19 mm (0.75 in) Holes	25 mm (1.0 in) Holes	Bolted	GTA	Peened	Cycles to Retrofit Failure (10^6)	Cycles To End of Test (10^6)	Remarks
W27X145	5	59.3	(8.6)	1.15	2.00	X					2.85	9.30	Initial Crack 3/4" into web at 1 complete penetration weld (b)
	5	70.3	(10.2)	4.68	4.68	X					6.30	9.30	Stiffener end (a)
W27X114	13	96.5	(14.0)	1.07	--						--	1.46	Incomplete penetration weld (b)
	13	115.1	(16.7)	1.34	1.34		X				--	1.46	Stiffener end (a)
W36X160	12	97.2	(14.1)	0.11	0.11				X		0.52	--	Stiffener end (a)
	12				0.52		X				0.99	1.29	Stiffener end (a)
	12	67.1	(9.7)	0.31	1.13		X				--	1.29	Incomplete penetration weld (b).

APPENDIX B

NOMENCLATURE

a = crack size, minor semidiameter of elliptical crack
a_f = final crack size
a_i = initial crack size
a_r = crack size after retrofitting by drilling holes
A_F = fused area of weld
A_O = overlapping area of web-flange intersection
b = major semidiameter of elliptical crack
C = crack growth coefficient, constant
da = crack growth increment
$E(k)$ = complete elliptical integral of the second kind
F_E = elliptical crack front correction
F_G = stress gradient correction factor
F_S = front free surface correction factor
F_W = finite width correction factor
ΔK = stress-intensity-factor range
ΔK_{th} = stress intensity threshold
N = fatigue life
SCF = maximum stress concentration factor
S_r = stress range
t_w, T_w = web thickness
T_F = flange thickness
ρ = diameter of drilled hole
σ = stress
σ_y = yield strength

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