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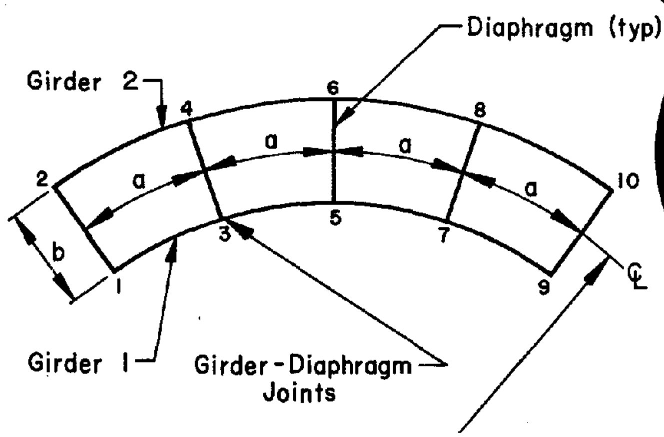
FATIGUE OF CURVED STEEL BRIDGE ELEMENTS

Design Recommendations for Fatigue of Curved Plate Girder and Box Girder Bridges

April 1980

Final Report



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Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
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FOREWORD

Horizontally curved steel plate and box girders are being used more frequently for highway structures, sometimes because of increased economy, and because of their esthetic appearance. The design of curved girders differs from that of straight girders in that torsional effects, including nonuniform torsion, must be considered. The resulting use of lateral bracing between curved plate girders and internal bracing and stiffening of curved box girders gives rise to complicated states of stress and to details which can be sensitive to repetitive loads. This situation prompted the FHWA to sponsor this research, the primary objective of which is to establish fatigue design guidelines for curved girder highway bridges in the form of simplified equations or charts.

This is the final report in a series of eight on the results of the research and is being distributed to the Washington and field offices of the Federal Highway Administration, State highway agencies, and interested researchers.

Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

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16. Abstract

Research on the fatigue behavior of horizontally curved, steel bridge elements was conducted at Fritz Engineering Laboratory, Lehigh University, under the sponsorship of the Federal Highway Administration (FHWA) of the U.S. Department of Transportation. The multi-phase investigation spanning nearly five years was performed in five Tasks: 1) analysis and design of five large scale horizontally curved steel twin plate girder assemblies and three large scale horizontally curved steel box girders, primarily for fatigue testing, 2) special analytical studies of the influences on fatigue of stress range gradient, heat curving, "oil canning" of webs and the spacing of internal diaphragms in curved box girders, 3) fatigue tests, to 2,000,000 cycles, of each of the above eight curved test girders, 4) ultimate strength tests of three of the curved plate girder assemblies and two of the curved box girders following the fatigue tests (composite reinforced concrete slabs were added to two of the three curved plate girder assemblies and to both curved box girders) and 5) development of design recommendations suitable for inclusion in the AASHTO bridge design Specifications.

This is the eighth and final report of the project and presents the results of Task 5 above. The entire project is described and the findings summarized which were presented in the previous project reports. The report concludes with suggested additions and modifications to the Tentative Design Specifications for Horizontally Curved Highway Bridges, prepared for the FHWA-DOT by CURT under Contract Number FH-11-7389, March 1975.

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The work was a part of Fritz Laboratory Research Project 398, "Fatigue of Curved Steel Bridge Elements" sponsored by the Federal Highway Administration (FHWA) of the United States Department of Transportation. The FHWA Project Manager is Mr. Jerar Nishanian. The Advisory Panel members are Mr. A. P. Cole, Dr. Charles G. Culver, Mr. Richard S. Fountain, Mr. Gerald Fox, Dr. Theodore V. Galambos, Mr. Andrew Lally, Mr. Frank D. Sears, and Dr. Ivan M. Viest.

The following members of the faculty and staff of Lehigh University made major contributions in the conduct of this work: Dr. R. G. Slutter, Dr. S. N. Iyengar, Dr. N. Zettlemoyer, J. M. Talhelm, D. du Plessis, Y. Erler, T. F. Cleary, R. Trevino, W. Herbein, R. P. Batcheler, M. Marzullo, J. Maurer, D. Abraham and T. A. Fisher.

Ms. Shirley Matlock typed the manuscript. The figures were prepared by Mr. John Gera and his staff.

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LIST OF ABBREVIATIONS AND SYMBOLS

a	=	diaphragm spacing, measured along centerline (inches)
b_f	=	flange width (inches)
d	=	transverse stiffener spacing (inches)
t_f	=	flange thickness (inches)
t_w	=	web thickness (inches)
w_i	=	initial out-of-flatness of web
D_w	=	web depth (inches)
F_y	=	yield stress (psi)
K	=	stress intensity
R	=	horizontal radius of curvature of test assembly (feet)
R	=	horizontal radius of curvature of a plate girder (inches)
S_r	=	stress range
I_o, II_{oa}, II_{ob}, etc.	=	welded detail types/subtypes for open section (plate girder) test assemblies
I_{ca}, I_{cb}, II_c, etc.	=	welded detail types/subtypes for closed section (box girder) test assemblies

U.S. Customary-SI Conversion Factors

To convert	To	Multiply by
inches (in)	millimeters (mm)	25.40
inches (in)	centimeters (cm)	2.540
inches (in)	meters (m)	0.0254
feet (ft)	meters (m)	0.305
miles (miles)	kilometers (km)	1.61
yards (yd)	meters (m)	0.91
square inches (sq in)	square centimeters (cm^2)	6.45
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
acres (acre)	square meters (m^2)	4047
square miles (sq miles)	square kilometers (km^2)	2.59
cubic inches (cu in)	cubic centimeters (cm^3)	16.4
cubic feet (cu ft)	cubic meters (m^3)	0.028
cubic yards (cu yd)	cubic meters (m^3)	0.765
pounds (lb)	kilograms (kg)	0.453
tons (ton)	kilograms (kg)	907.2
one pound force (lbf)	newtons (N)	4.45
one kilogram force (kgf)	newtons (N)	9.81
pounds per square foot (psf)	newtons per square meter (N/m^2)	47.9
pounds per square inch (psi)	kilonewtons per square meter (kN/m^2)	6.9
gallons (gal)	cubic meters (m^3)	0.0038
acre-feet (acre-ft)	cubic meters (m^3)	1233
gallons per minute (gal/min)	cubic meters per minute (m^3/min)	0.0038
newtons per square meter (N/m^2)	pascals (Pa)	1.00

1. INTRODUCTION AND DESCRIPTION OF RESEARCH

1.1 Background

In 1969 the Federal Highway Administration (FHWA) of the U.S. Department of Transportation (U.S. DOT), with the sponsorship of 25 participating state transportation departments, commenced a large research investigation of horizontally curved plate and box girder bridges. The investigation was conducted by four universities (Carnegie-Mellon, Pennsylvania, Rhode Island, and Syracuse) and was commonly referred to as the CURT (Consortium of University Research Teams) Project. The investigation was directed towards the development of specific horizontally curved steel girder design guidelines for possible inclusion in the AASHTO bridge design specifications.

The tentative specifications [1,2] resulting from the CURT investigation incorporate their findings as well as input from other simultaneous efforts such as from the University of Maryland. The CURT project also included an extensive literature survey. Prior curved girder work has therefore been taken into account. However, the tentative specifications do not suggest provisions related to fatigue. The CURT program concluded with a recommendation that future research investigate the fatigue behavior of horizontally curved steel bridges.

While the CURT investigation was in progress, considerable work was underway at Lehigh University in the area of straight girder fatigue. Two reports were produced which clarified the understanding of the fatigue performance of steel bridge members. [3,4] The findings of this research resulted in a major revision of fatigue design rules in 1974 and is now incorporated in the fatigue provisions of the Twelfth Edition (1977) of the AASHTO Standard Specifications for Highway Bridges. [5] Two other reports provide condensed commentary and guidance related to the application of the new provisions. [6,7] However, since no horizontally curved girders were analyzed or tested, direct applicability of the new specifications to these members was not assured. Furthermore, no previous fatigue research on horizontally curved girders can be found in the literature.

In 1973 the FHWA sponsored a research investigation at Lehigh University entitled "Fatigue of Curved Steel Bridge Elements". This is a multi-task analytical and experimental investigation into the fatigue behavior of horizontally curved steel plate and box girders for highway bridges. The investigation was completed in 1978. The five project tasks are shown in Appendix A. A list of all reports produced in this project is given in Appendix B. This is the final report of the project.

1.2 Problem Statement

Concern has been expressed that the extensive lateral bracing required in horizontally curved steel plate girder bridges, coupled with the unequal deflections of adjacent girders and the attendant complicated state of stress may lead to welded details which are sensitive to repetitive loads. Similarly, the internal bracing and stiffening required of horizontally curved steel box girders which are subjected to torsional, lateral and longitudinal forces may also produce welded details sensitive to repetitive loads.

Real fatigue problems have been encountered in straight girder bridges. Fatigue cracks have occurred primarily at cover plate ends as a result of extreme volumes of traffic and lower than estimated fatigue resistance, as well as at stiffeners, braces and other attachments, primarily as a result of displacement-induced stresses. Such problems can be avoided in straight girder design by adherence to the new fatigue provisions of the bridge design specifications.(5) There is concern that more fatigue problems may exist in curved girder bridges because bracing members and connections, for example, carry forces which are more complex than the nominal forces in straight girders.

Concern has also been expressed about the fatigue problems associated with "oil canning" of curved thin web plates with stiffeners. Oil canning is a term used to mean the lateral displacement of the web plate of a plate or box girder, straight or horizontally curved, under cyclic loads. Existing web slenderness (depth-thickness ratio) requirements for straight girders are based on buckling and fatigue considerations.(5) Prior fatigue tests indicated that for straight girders the web bending stresses due to initial imperfections in the web panel did not cause fatigue problems as long as the web was within the specified slenderness limit.[8] Due to curvature effects, the web of a curved girder which is subjected to bending tends to deform outward in the compression region and to flatten out in the tension region.[9,10,11] These deformations produce web bending stresses which are of concern. By using a simple physical model to account for the shell action at a curved panel, an expression was developed for the web slenderness ratio required to limit these web bending stresses However, no experimental results on the fatigue behavior of curved panels were available to substantiate this expression. Testing and examination of curved girders with respect to this aspect have been completed and the results summarized herein.

Questions have also been raised regarding the influences of three other factors on the fatigue behavior of curved plate and box girder bridges. These are (1) stress range gradients across the flanges of plate girders, (2) residual stresses due to heat curving plate girders and (3) spacing of internal diaphragms in curved box girders. These factors were examined analytically and the results are summarized herein.

1.3 Objectives and Scope

The objectives of this investigation are: (1) to examine the fatigue behavior of horizontally curved steel plate and box girder highway bridges, (2) to develop fatigue design guides in the form of simplified equations, charts or specification provisions suitable for inclusion in the AASHTO bridge specifications, and (3) to investigate the ultimate strength behavior of curved steel plate and box girder highway bridges. Before the second objective is carried out it is intended that the fatigue behavior of curved girders be compared with straight girder performance to determine if in fact revisions to the AASHTO specifications are necessary.

It has long been recognized that fatigue problems in steel bridges are most probable at details associated with bolted and welded connections in tensile stress range regions. Straight girder research has shown that welded details are more fatigue sensitive than bolted details. Modern bridge structures utilize welded connections extensively in the construction of main members and for securing attachments such as stiffeners and gusset plates. Therefore, the investigation is centered on the effect of welded details on curved girder fatigue strength. The investigation also examines other areas of concern such as the "oil canning" effect.

The work is broken down into five tasks as shown in Appendix A. In Task 1 the analysis and design of large scale horizontally curved plate and box girder test assemblies are performed, including bridge classification and selection of welded details for study. Task 2 concerns special studies on stress range gradients, heat curving residual stresses, web slenderness ratios and diaphragm spacing as related to fatigue performance. Fatigue tests of the large scale test assemblies are performed in Task 3. Ultimate strength tests of three curved plate girder assemblies and two curved box girders are performed in Task 4. Composite reinforced concrete slabs were added to two of the three curved plate girder assemblies and to both curved box girders. Design recommendations for fatigue of curved plate and box girder highway bridges are prepared as Task 5 based on the work of Tasks 1, 2 and 3.

2. REVIEW OF PROJECT TASKS

The following is a review of project Tasks 1, 2, 3 and 4 (Appendix A). This report constitutes the work done in Task 5. The project findings are reported in Chapter 3.

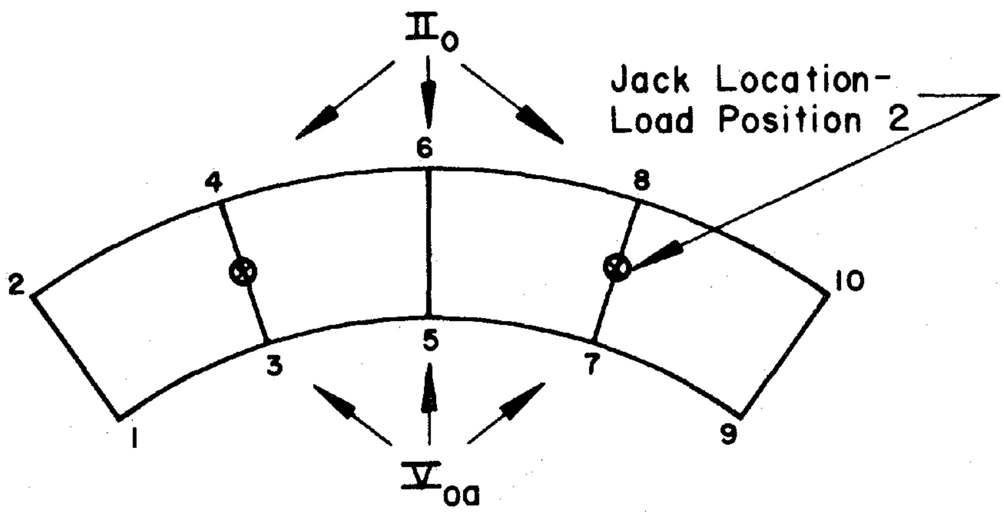
2.1 Task 1 - Analysis and Design of Large Scale Plate Girder and Box Girder Test Assemblies[13]

The experimental phase of the fatigue investigation (Task 3) required large scale test girders. Five plate girder assemblies and three box girder assemblies were designed and fabricated to investigate the fatigue behavior of typical welded details usually found in curved girders and to study the effect of web slenderness "oil canning" on the fatigue of curved girder webs.

In order to provide full scale welded details with realistic residual stress fields and stress range gradients in the vicinity of the welded details, as well as reasonable flange and web dimensions, large scale test girders were required. Examples of the test girders are shown in Figs. 1 through 4.

The test girders were analyzed using available computer programs and designed in accordance with the 1973 AASHTO Specifications and 1974 Interim, including the modifications suggested in the CURT tentative design specifications.[1,2]

Three major constraints governed the design of the plate girder assemblies and box girders for testing. First, it is desirable for all welded details on a given test girder to fail in fatigue at approximately the same cycle life in order to reduce testing time and the problems associated with fatigue crack repair. The test girders were designed to a fatigue life of two million cycles for all details tested. The required stress ranges were estimated using the AASHTO fatigue provisions. The allowable stress ranges specified by AASHTO represent the 95% confidence limit for 95% survival. Therefore, to ensure the formation of visible fatigue cracks and to allow for a small margin of error in the analysis the design stress ranges were selected approximately 2 ksi higher in the flanges and 1 ksi higher in the webs than the allowable stress ranges specified by AASHTO.



Girder	b_f	t_f	D_w	t_w	D_w/t_w
1	8"	1/2	58	3/8	155
2	10	3/4	58	5/16	186

b_f = flange width D_w = web depth

t_f = flange thickness t_w = web thickness

Fig. 1 Plate Girder Test Assembly 2 - Cross Section Dimensions and Locations of Group 1 Details on Tension Flanges

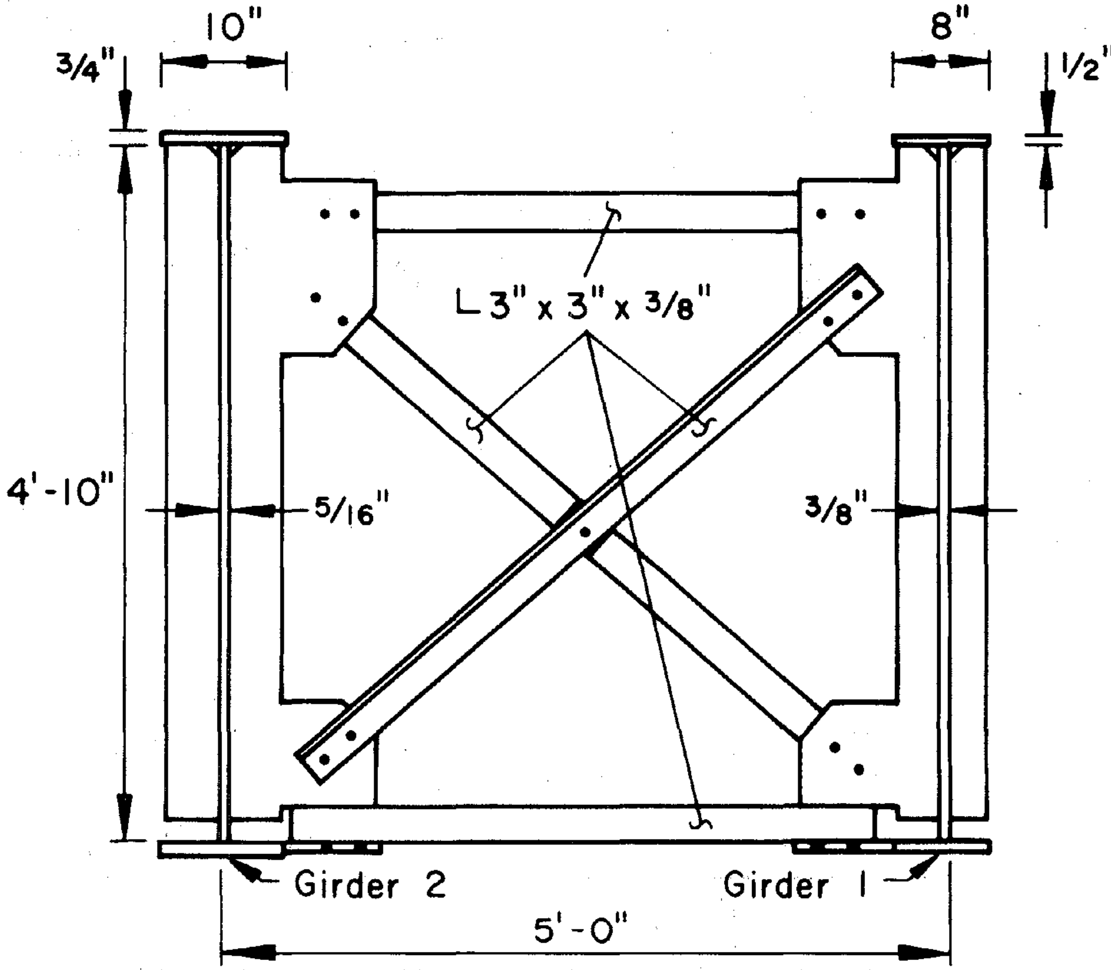


Fig. 2 Plate Girder Test Assembly 2 - Cross Section at Interior Diaphragms

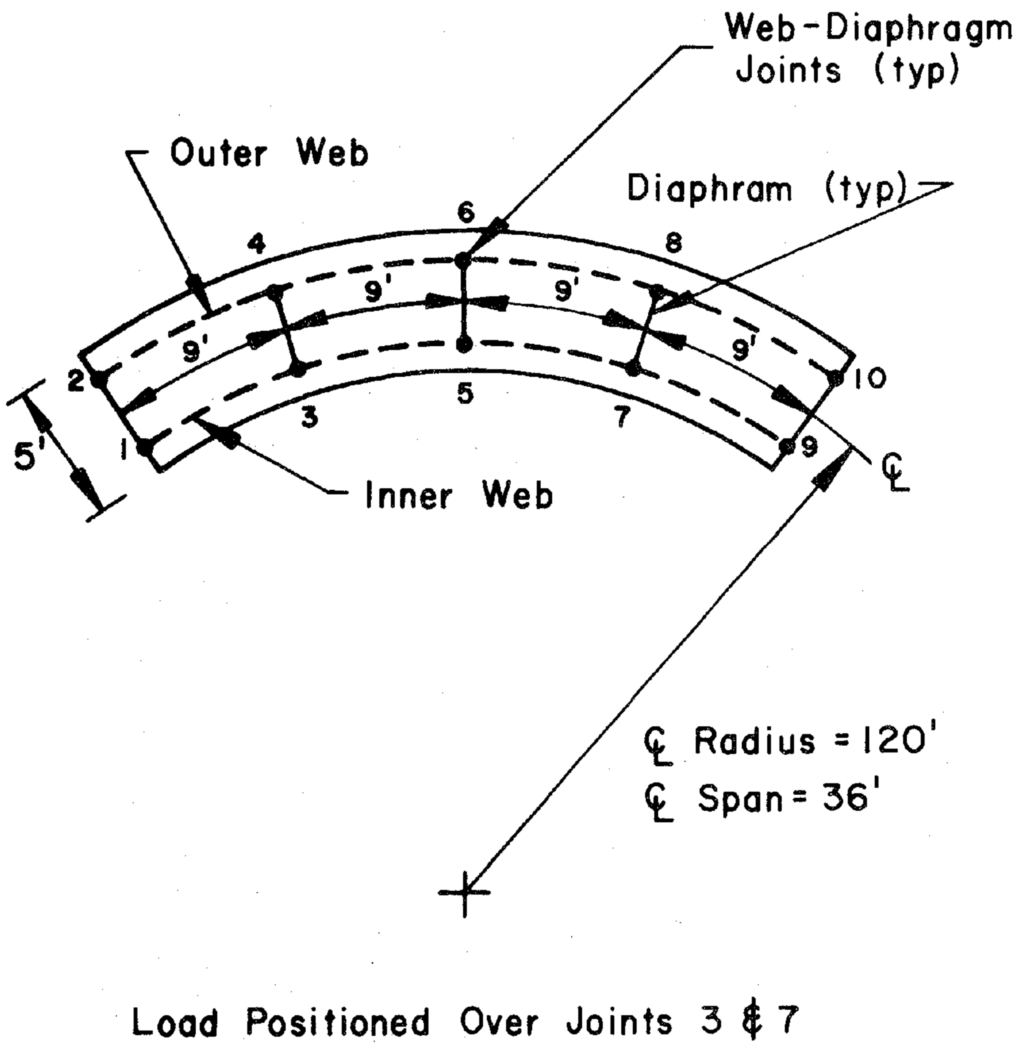


Fig. 3 Schematic Plan View of Typical Box Girder Test Assembly

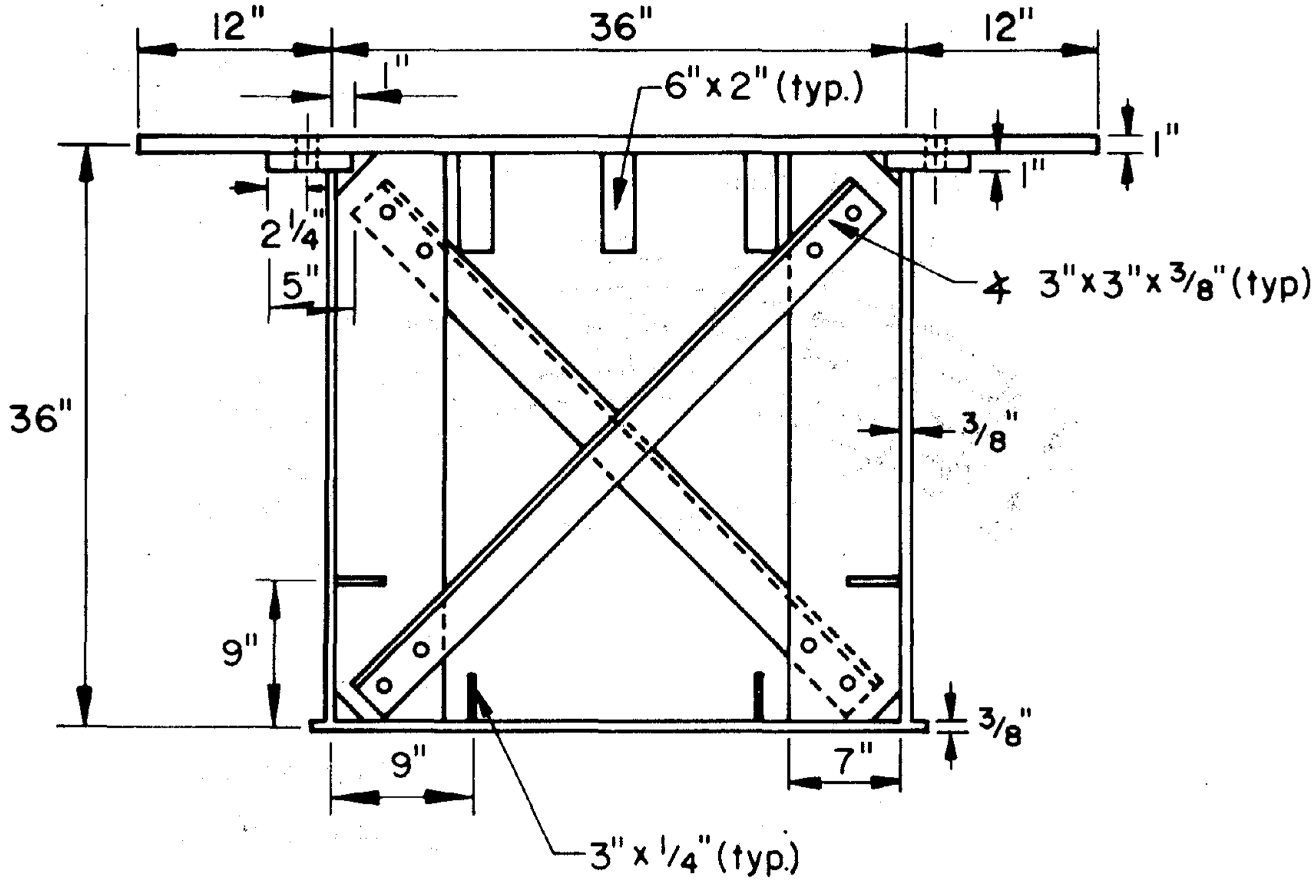


Fig. 4 Box Girder Test Assembly 1 - Cross Section at Interior Diaphragms

Second, the span length, centerline radius, and number of plate girders in each plate girder assembly and the cross section dimensions of the box girders are limited by the dimensions of the dynamic test bed in Fritz Engineering Laboratory and the method of loading.

Third, the maximum loading which can be applied to the test girders is limited by the maximum dynamic load capacity of the available pulsating jacks. Amsler pulsators and hydraulic jacks having 110 kip dynamic capacity and a maximum usable stroke of 0.35 inches were used.

Execution of Task 1 was carried out in light of these constraints through the following procedure:

1. An examination of the characteristics of existing curved girder bridges and a classification of welded details was performed.

2. A preliminary design of the test assemblies was performed.
   a) Preliminary design of the plate girder test assemblies was achieved using the V-load method.[14]
   b) Preliminary design of the box girders was accomplished by assuming that the assemblies were straight, and carrying out a simple structural analysis.

3. Following the preliminary designs, the test girders were analyzed using the most recently available computer programs. Two different programs were used to analyze each plate girder test assembly and another two were used to analyze each box girder. Thus, a check on the accuracy of the stress range and deflection conditions as required by the first and third design constraints was provided.
   a) The plate girder test assemblies were analyzed using the Syracuse[15] and CURVBRG[16] computer programs. Reasonable agreement between the results of both programs was obtained.
   b) The box girder test assemblies were analyzed using the SAP IV[17] and CURDI[18] computer programs. Reasonable agreement between the results of these programs was also obtained.

4. Revised preliminary designs were prepared as required by the results of the computer analyses.

5. The revised preliminary designs were analyzed as outlined in Step 3. Several cycles of design (Step 4) and analysis (Step 5) were required to satisfy the above design constraints.

6. Final designs of each of the test girders were made and included complete designs of the required diaphragms, transverse stiffeners, bearing stiffeners, etc. and preparation of design drawings for fabrication. Fabrication contracts were let on the basis of competitive bids.

Each of the five plate girder assemblies had a centerline span length of 40 feet and a centerline radius of 120 feet, and were loaded at the quarter points either directly over the inner girder or at the test assembly centerline. The applied load range was from 5 kips to 105 kips and the maximum deflection under the load was approximately 0.35 inches. The dimension of the girders are summarized in Table 1.

Each of the three box girders has a centerline span length of 36 feet and a centerline radius of 120 feet, and are loaded at the quarter points directly over the inner web. The applied load range and deflection conditions are similar to those for the plate girder assemblies.

The work of Task 1 also included the selection of typical welded details which are fabricated into the test girders. Tables 2 and 3 show the welded details selected for the plate girder assemblies and box girders (13). The detail type is given by the Roman numerals and subscripts. The AASHTO fatigue category and allowable stress range for 2,000,000 cycles of load application is also shown for each detail.

Some of the results of the Task 1 studies will be summarized in Chapter 3 on the findings of this project.

2.2 Task 2 - Special Studies

The special studies contemplated at the start of the project are listed in Appendix A. As the project progressed, it became necessary to enlarge the scope of the special studies. Some of this additional work was performed outside the project. The findings from all the special studies are reported in Chapter 3.

The significance of a fatigue crack growing across the width of a flange in the presence of a stress range gradient was studied. The

TABLE 1 PLATE GIRDER ASSEMBLY AND BOX GIRDER GEOMETRIES

Type	Assembly No.	Girder No.	b_f (in)	t_f (in)	D_w (in)	t_w (in)
Plate Girder Assembly	1	1	12	1	54	3/8
		2	12	1	54	9/32
	2	1	8	1/2	58	3/8
		2	10	3/4	58	5/16
	3	1	8	1/2	58	3/8
		2	10	3/4	58	3/8
	4	1	8	1/2	52	3/8
		2	12	1	52	3/8
	5	1	8	1/2	52	3/8
		2	12	1	52	3/8

Type	Girder No.	Web	b_f [a] (in)	t_f (in)	D_w (in)	t_w (in)
Box Girder	1	inner web	36	3/8	36	3/8
		outer web	36	3/8	36	3/8
	2	inner web	36	3/8	36	3/8
		outer web	36	3/8	36	3/8
	3	inner web	36	3/8	36	3/8
		outer web	36	3/8	36	3/8

[a] Bottom flange width

CENTERLINE DATA

Type	Radius (ft)	Span (ft)
Plate Girder Assembly	120	40
Box Girder	120	36

TABLE 2 SUMMARY OF WELDED DETAILS FOR PLATE GIRDER TEST ASSEMBLIES

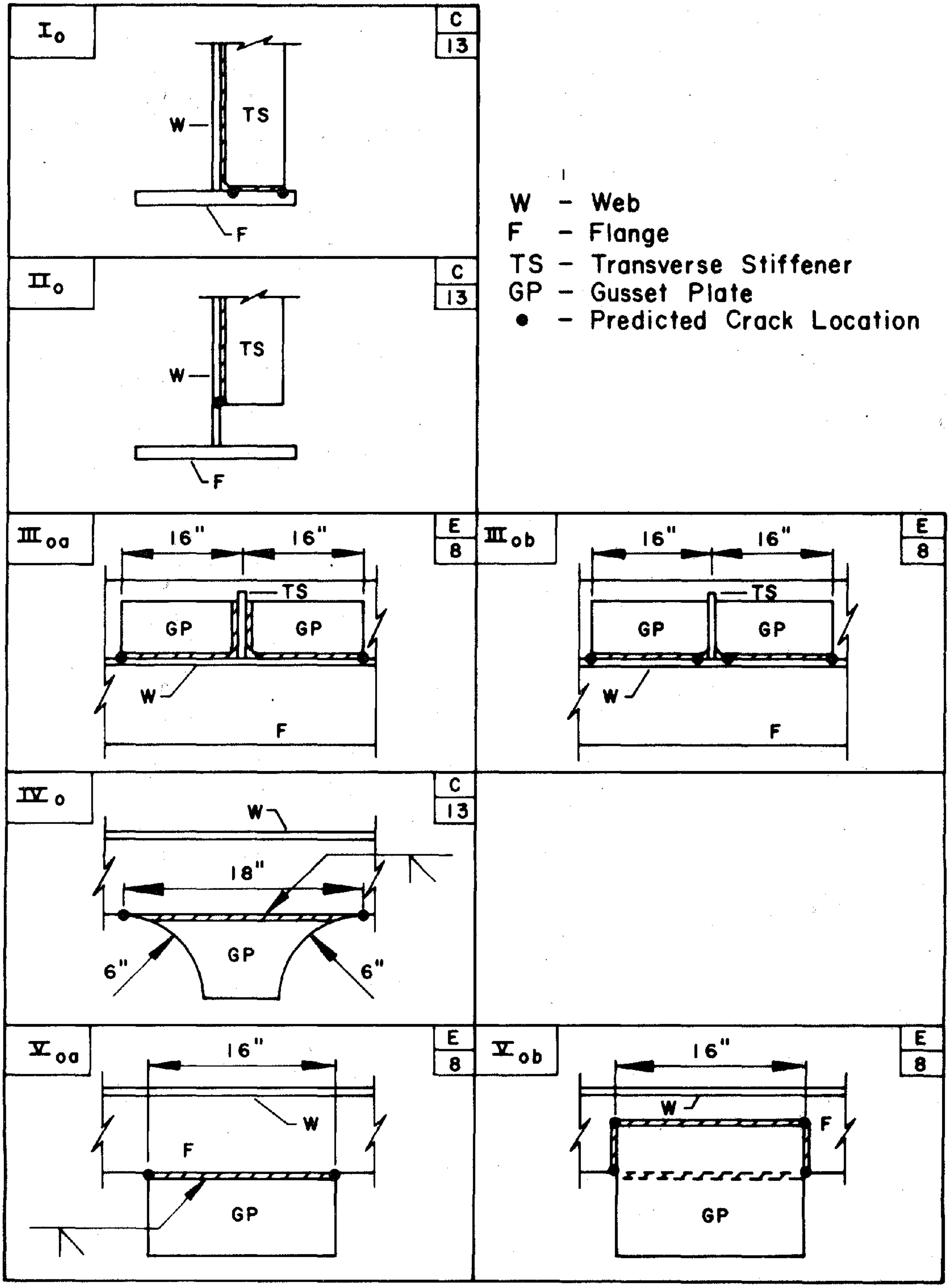


TABLE 3 SUMMARY OF WELDED DETAILS FOR BOX GIRDER TEST ASSEMBLIES

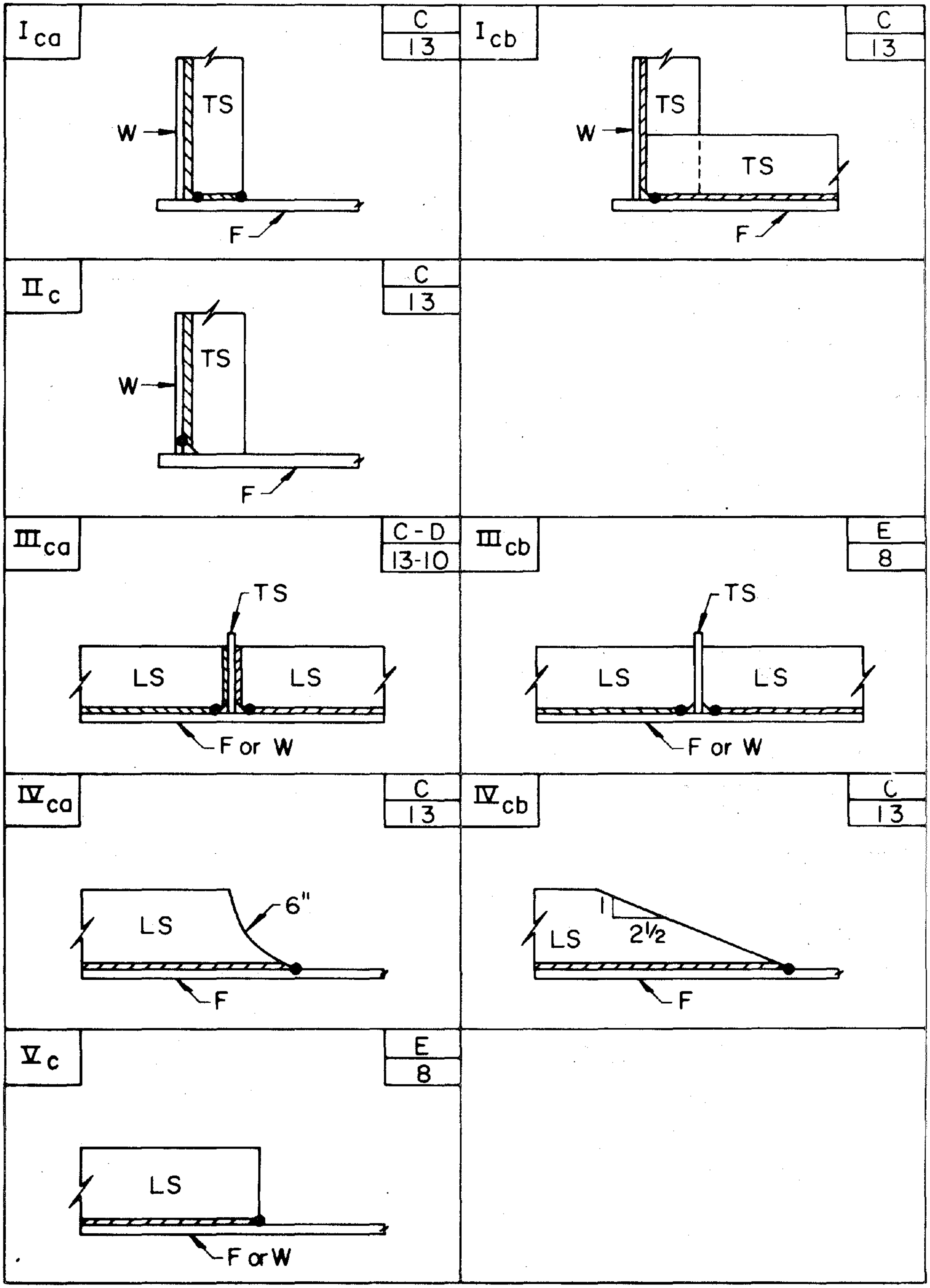
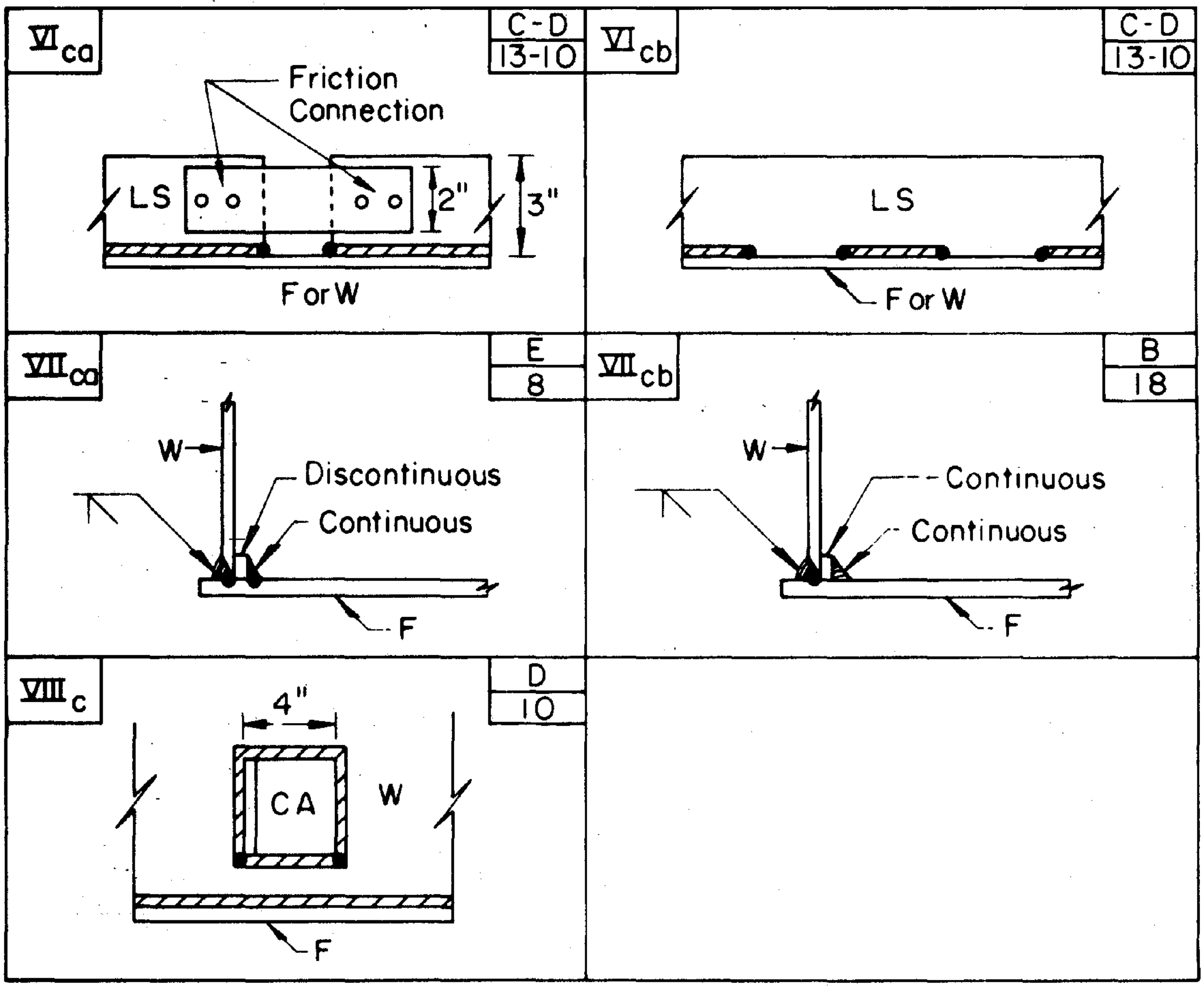


TABLE 3 SUMMARY OF WELDED DETAILS FOR BOX GIRDER TEST ASSEMBLIES (CONTINUED)



W - Web

F - Flange

TS - Transverse Stiffener

LS - Longitudinal Stiffener

GP - Gusset Plate

CA - Clip Angle

● Predicted Crack Location

primary emphasis of this work is to improve the analytical capability in predicting fatigue life. The improvement centers on the influence of stress range gradient and local stress concentration on the fracture mechanics stress intensity factor K. Stress concentration plays a significant role in distinguishing between the AASHTO fatigue categories. Any condition which raises the stress concentration increases the likelihood of crack growth. The results of these studies are reported in detail in Refs. 19 and 20.

Reference 21 presents the results of a study of the influence on fatigue of residual stresses introduced by heat curving.

Reference 22 presents the results of a study of the influence on fatigue of the spacing of internal diaphragms in curved box girders. Diaphragm spacing has a direct influence on the plate bending stresses due to distortion of box girders.

Task 2 required the study of lateral web deflections, or "oil canning", on the fatigue behavior of curved webs. The CURT tentative design specifications for slenderness ratios for curved girder webs suggest values lower than the current permissible slenderness ratios for straight girder webs (1, 5).

For the purpose of studying web performance under fatigue loading, various web slenderness ratios and transverse stiffener spacings are used for the five curved plate girder assemblies (13, 23). Table 4 summarizes the web slenderness ratios and stiffener spacings for the curved plate girder assemblies. Also listed are the allowable web depth to thickness ratios and the stiffener spacings for straight girder web panels, as well as the CURT proposed slenderness limits for curved plate girders.

The actual D_w/t_w ratios selected stayed within the allowable limits for straight girders but exceeded the CURT proposed values in all cases. For all web panels, the spacing of transverse intermediate stiffeners was larger than that allowed for straight girders. The behavior of these web panels under repeated application of loads on the girders provided information for comparing the proposed provisions.

TABLE 4 SUMMARY OF WEB SLENDERNESS RATIOS AND STIFFENER SPACINGS FOR PLATE GIRDER ASSEMBLIES

					AASHTO D/t (straight)		CURT D/t (curved)	Internal d			End Panel d_o		
				D/t					Permitted			Permitted	
Assembly	Girder	D	t	Act.	ASD	LFD	LFD*	Act.	ASD	LFD	Act.	ASD	LFD
1	1	54	3/8	144	170	192	100	118	81		71	27	54
	2	54	9/32	192	170	192	100	123	81		123	27	54
2	1	58	3/8	155	170	192	100	118	87		118	29	58
	2	58	5/16	186	170	192	100		87		61	29	58
3	1	58	3/8	155	170	192	100	118	87		118	29	58
	2	58	3/8	155	170	192	100		87		61	29	58
4	1	52	3/8	139	170	192	100	118	78		118	26	52
	2	52	3/8	139	170	192	100	123	78		61	26	52
5	1	52	3/8	139	170	192	100	118	78		118	26	52
	2	52	3/8	139	170	192	100	123	78		61	26	52

*All d_o larger than 1.5 D, the maximum allowed spacing.

Backup bars are a common feature of box girders and were included in the study. Although studies of backup bars were not contemplated when the project began, the importance of exploring the fatigue strength of this detail became evident while the project was in progress. These studies were undertaken outside the project scope and are reported in Ref. 24. Detail Types VII_{ca} and VII_{cb} in Table 2 show discontinuous and continuous backup bars. The results of this work will be included in Chapter 3 as part of the findings of this project.

While the work of Task 1 was underway, it became desirable to investigate the stress distribution in actual curved girder bridges. Field studies of a curved box girder bridge and a curved plate girder bridge were therefore included as part of the work of another related fatigue project. Reference 25 documents a stress history study of a curved box girder bridge, while Ref. 26 reports on the stress distribution in a curved plate girder bridge. The results of these studies will also be included in Chapter 3 as part of the findings of this project.

2.3 Task 3 - Fatigue Tests of Curved Plate Girder and Box Girder Test Assemblies

Complete details of the work done in Task 3 are contained in Refs. 23 and 27.

Each of the five curved plate girder assemblies was subjected to approximately 2,000,000 cycles of load application using two coupled 110 Kip capacity Amsler hydraulic jacks. Each assembly contained from 11 to 20 welded details from among the 7 types and subtypes shown in Table 2. The five assemblies contained a total of 75 welded details. Each detail was designed and located such that it would exhibit visible fatigue cracks at a life of approximately 2,000,000 cycles.

Each of the three curved box girders designed was also subjected to approximately 2,000,000 cycles of load application using the same test set-up as for the plate girder assemblies. Each box girder contained from 27 to 34 welded details from among the 13 types and subtypes shown in Table 3. The three box girders contained a total of 90 welded details. Each detail was designed and located in each box girder such that it would exhibit visible fatigue cracks at a life of approximately 2,000,000 cycles.

Prior to fatigue testing of a plate girder assembly or box girder, static loading was applied up to the level of the maximum fatigue load. Stresses and deflections were measured at numerous locations and compared with predicted values. The nominal stress range at each welded detail was computed and adjustments in loading and/or girder alignment were made to ensure that the actual stress range was as close as possible to the design stress range at the location.

For the five plate girder assemblies, the details between diaphragms were welded in place in Fritz Laboratory to ensure that they were located accurately with respect to actual stress range.

During the fatigue tests of all eight test girders stress ranges were regularly monitored using analog trace recorders. Deflections were also regularly monitored. Cyclic loading was stopped at frequent intervals especially between 1,000,000 and 2,000,000 cycles so that the girder could be thoroughly inspected for fatigue cracks. All cracks detected were noted as to size, location and number of cycles.

Although all details were expected to exhibit visible fatigue cracks at approximately 2,000,000 cycles, the first cracks generally were detected somewhat earlier. This usually was due to a higher actual stress range at the detail than the design value, a situation which was unavoidable. Occasionally, early fatigue cracking also occurred as expected, due to out-of-plane bending of webs, at locations such as below cut-short transverse stiffeners (Detail type II_O, Table 2 for example). Once a fatigue crack was detected at a welded detail, its further growth was arrested so that additional fatigue cracks could develop elsewhere under further cyclic loading. Arresting of crack growth was accomplished by either 1) drilling a small hole at the crack tip; 2) drilling a hole at the crack tip and inserting a high strength bolt; 3) drilling out the crack tip and bolting a steel plate over the crack; 4) clamping steel plates on both sides of the crack if the crack existed in a flange; or 5) welding the crack closed.

The type of method used to arrest crack growth depended upon 1) type of crack; 2) range of the stress intensity factor, ΔK, at the crack tip; and 3) the number of cycles of load application remaining to reach the desired life of 2,000,000 cycles.

The results of the Task 3 studies will be included in Chapter 3 on the findings of this report.

2.4 Task 4 - Ultimate Load Tests of Curved Plate and Box Girder Assemblies

Complete details of the work done in Task 4 are contained in Ref. 28.

The ultimate strength investigation was very limited in scope and was intended to ensure that no unexpected behavior of curved girders would develop prior to the attainment of their carrying capacity. The investigation also serves as a pilot study into the ultimate strength capacity of curved plate and box girders.

Five previously tested girders were selected and retrofitted for the ultimate strength tests. These girders were Plate Girder Assemblies 1, 4 and 5 and Box Girders 1 and 3. The latter four were made composite by the addition of reinforced concrete slabs connected to the top flanges with shear connectors. Plate Girder Assembly 1 was tested without a slab.

The results of these tests are expected to be combined with those from further studies for the purpose of developing an ultimate strength theory, and are not included in Chapter 3 of this report.

2.5 Task 5 - Design Recommendations

The objective of Task 5 is to formulate design provisions against fatigue failure of horizontally curved steel plate and box girder bridges based on the analytical and experimental work performed in Tasks 1, 2 and 3. Specification provisions are to be formulated in a manner consistent with that for straight girders for presentation to the AASHTO Bridge Committee. These proposed specification provisions are presented in Chapter 4.

3. FINDINGS

The principal findings of the investigation are summarized in this chapter. The findings are discussed with respect to their implications for design against fatigue of curved steel plate and box girders for bridges. Design recommendations are discussed. Further details of the project findings can be found in the project reports: Refs. 13, 19, 21, 22, 23 and 27 (see also Appendix B) and in Refs. 24, 25 and 26.

3.1 Analysis of Curved Plate and Box Girders

Most curved steel girder bridges are of such complexity that a computer is required for reasonably accurate and quick analysis. Very simple curved girders might be analyzed using Dabrowski's "exact" method or the approximate V-Load method for open sections (plate girders).[29,14] However Dabrowski's method is very difficult to use when diaphragms or bottom lateral bracing are present.

In the design of bridge girders against fatigue it is very important to accurately predict the nominal stress range at any specific point on any given cross section of the girder under all conditions of loading.[3,4,5,7] Although generalized stresses or approximate "safe" stress distributions often suffice in the design of bridge girders for allowable stress conditions these stresses are not adequate enough for predicting cycle life in fatigue. Allowable stress design also often employs simplified models of the superstructure on the assumption that the increased complexity of the real structure (diaphragms, skew, bracing, etc.) cannot decrease the static strength. This same complexity can and frequently does decrease the fatigue resistance of the real structure.

Unfortunately, many existing methods of analysis for curved girders, including the V-Load method, and most of the available computer programs do not provide the accuracy required for the design of curved girders against fatigue. Appendix C of Ref. 13 lists, and comments on the possible suitability of, 15 computer programs available for the analysis of curved steel bridge girders. At the time the analyses of the 8 large scale curved steel test girders were performed (1973-1975) these were the only programs known to be available. Only a few of them give sufficiently accurate stress magnitudes for the analysis of curved girders against fatigue. Four were selected for the design and analyses of the project test girders.

The Syracuse program, Ref. 15, was used during the preliminary allowable stress designs of the five curved plate girder assemblies. This program is not suitable for the final design against fatigue since it provides only generalized cross section stresses. These must be extended by hand calculation or other means to provide stresses at points in the girder cross sections.

Final designs of the five curved plate girder assemblies were performed using the Berkeley program, CURVBRG, described in Ref. 16. CURVBRG provides stresses at any point in the girder cross section as well as in diaphragms and bracing members. Comparison of the results of the Syracuse program and CURVBRG show reasonable agreement. Test results indicated that the experimental stresses deviated,on the average, less than 15 to 20% from the theoretical predictions by CURVBRG.

The finite element program SAP IV,which is described in Ref. 17, was used for the analyses and final designs of the three curved box girders. A finite strip program, CURDI, Ref. 18, was used for comparative analyses. Comparison of the results from the SAP IV and CURDI programs showed reasonable agreement. Again, the test results indicated that the experimental stresses differ less than 15% from the theoretical predictions by SAP IV.

SAP IV, in its present form, was not available to the project at the time the five curved plate girder assemblies were designed. It is capable of the analysis of curved plate and curved box girders. SAP IV was used, however, for the elastic analysis of the five curved plate and box girders which were tested to ultimate strength and reported in Ref. 28.

3.1.1 Design Recommendation

The design against fatigue of horizontally curved steel plate and box girders for bridges requires accurate stress analyses, under the appropriate loading conditions, to determine the expected nominal stress ranges at all points of potential fatigue crack growth. The computer program(s) for performing the analyses should be carefully selected. Accurate nominal stress ranges can be determined only by realistically modelling the three-dimensional structure consisting of girders, diaphragms, attachments, bracing, etc., and by a careful discretization of the structure.

3.2 Fatigue Strength of Welded Details

A complete description of the fatigue performance of the welded details on all eight test girders is given in Refs. 23 and 27. The following is a brief summary of the findings.

The five curved plate girder assemblies and three curved box girders contained a total of 142 welded details of the types shown in Tables 2 and 3. Altogether, the eight test girders contained a total of 284 anticipated fatigue crack locations. Fatigue cracks were observed at 107 of these locations of which 61 were through-thickness fatigue cracks. Of the 61 through-thickness cracks, 48 had fatigue resistances equal to or greater than their respective categories for straight girders as stipulated by the present AASHTO Specifications. The measured nominal stress range for each of these 48 cracks is plotted according to category in Figs. 5 and 6 against cycle life. Some of the plotted points represent more than one fatigue crack. That the fatigue resistance lines of categories C and E serve well as the lower bound for these cracks is evident from the figures.

The fatigue resistances of the remaining 13 through-thickness cracks were less than their respective categories as given by the AASHTO Specifications. The correlation of measured nominal stress range and cycle life for each of these 13 cracks is plotted in Figs. 7 and 8 according to categories. In each case the nominal stress range has incorporated the curvature of the girder, therefore the reduced fatigue resistance could not be attributed to curvature of the girder per se. In each case the reduction was attributed to factors such as unsatisfactory weld quality, particular flaw conditions, and stress and deflection conditions which were not considered in designing the girders.

The following is a brief recapitulation of the reasons for the relatively early fatigue crack occurrence of each case shown in Figs. 7 and 8.

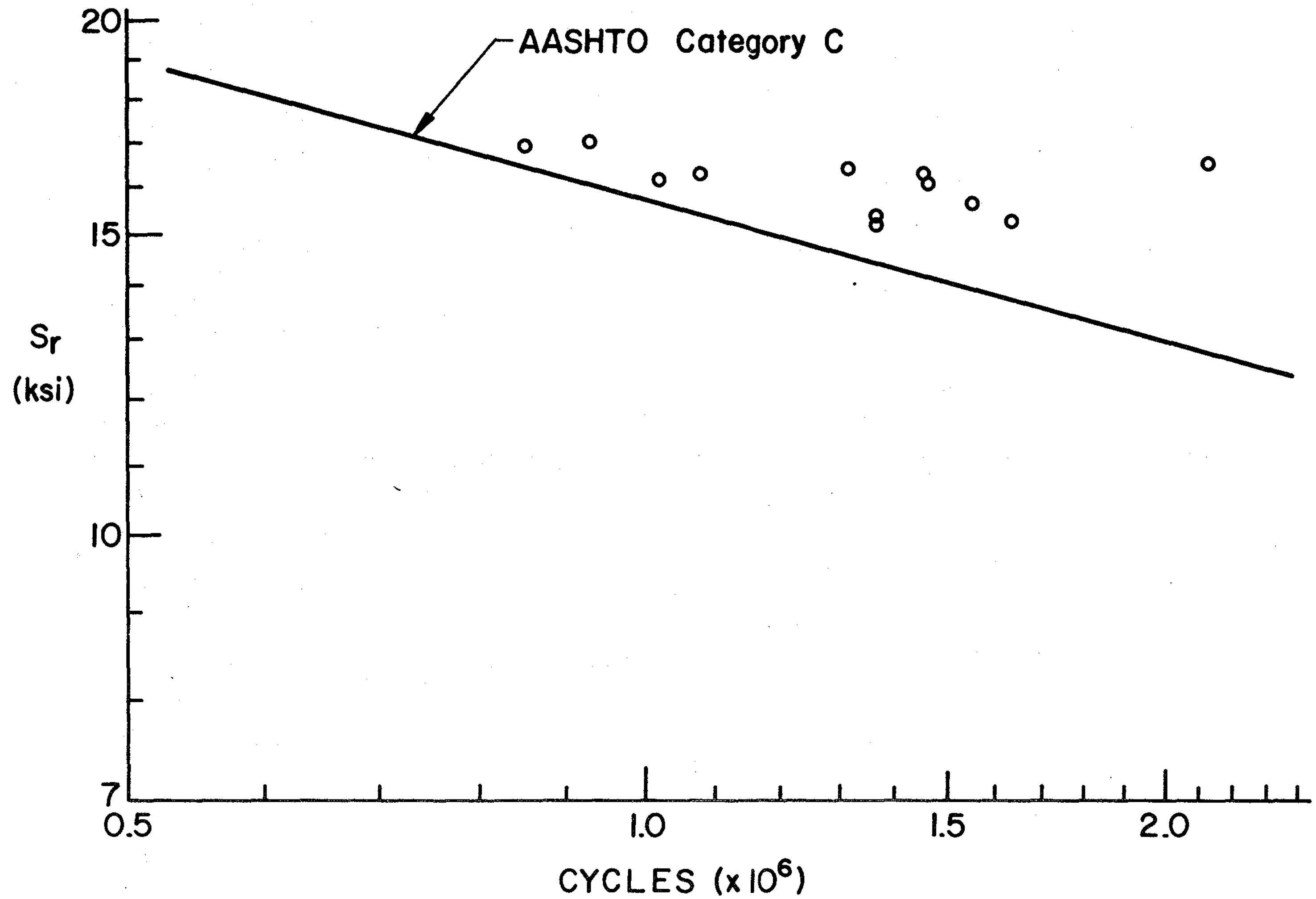


Fig. 5 Fatigue Results - Category C Details

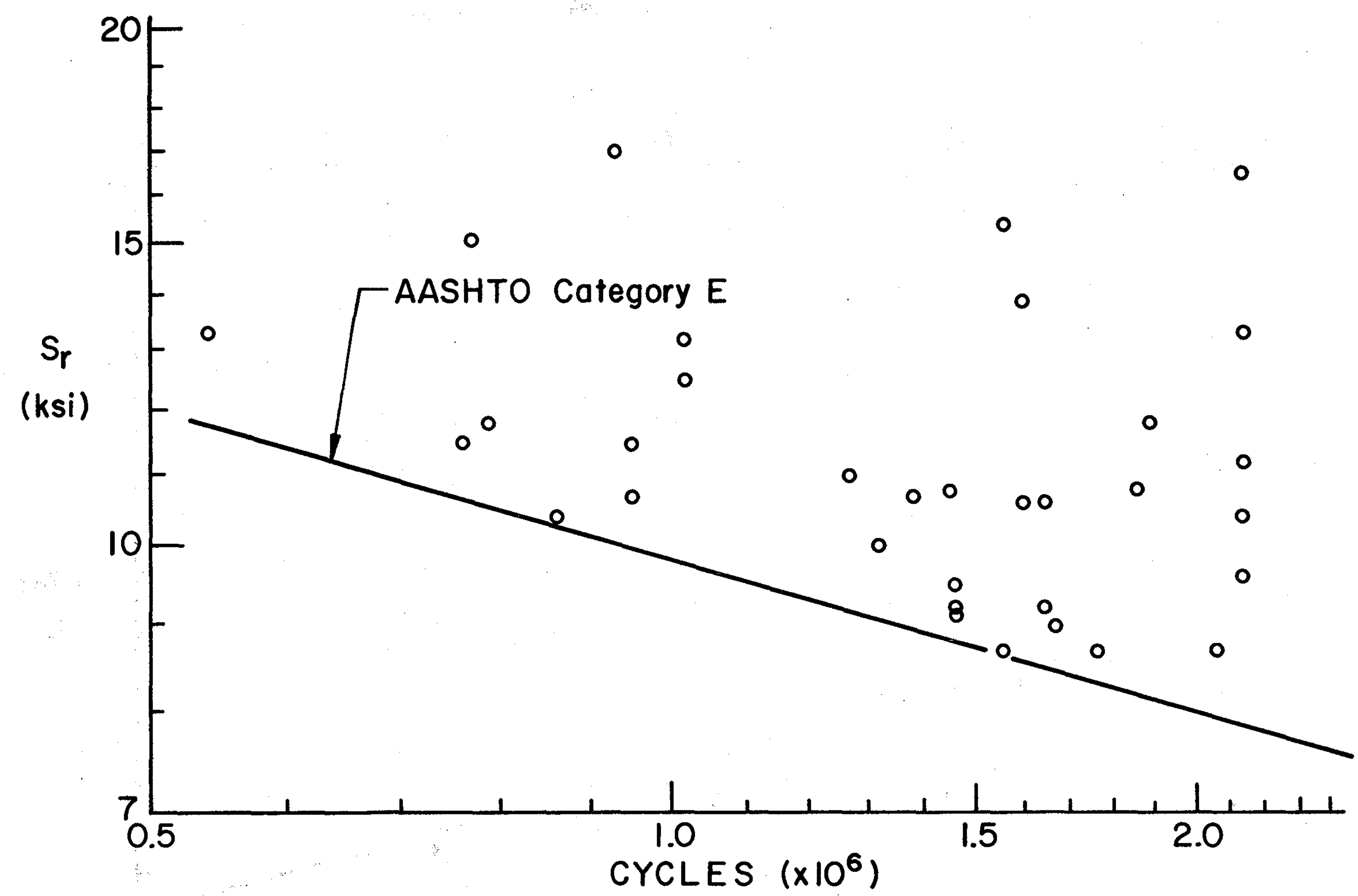


Fig. 6 Fatigue Results - Category E Details

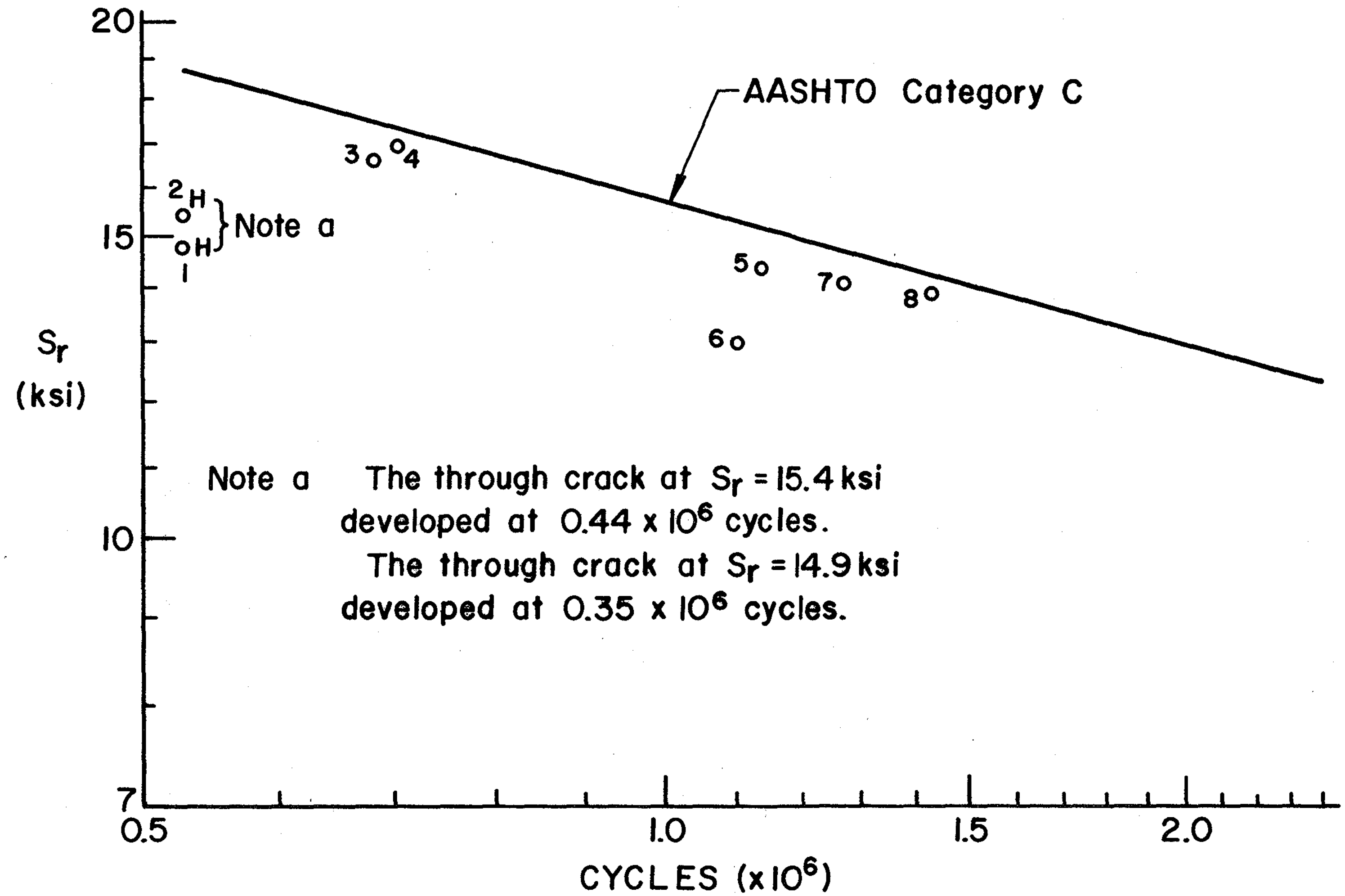


Fig. 7 Fatigue Results - Category C Details

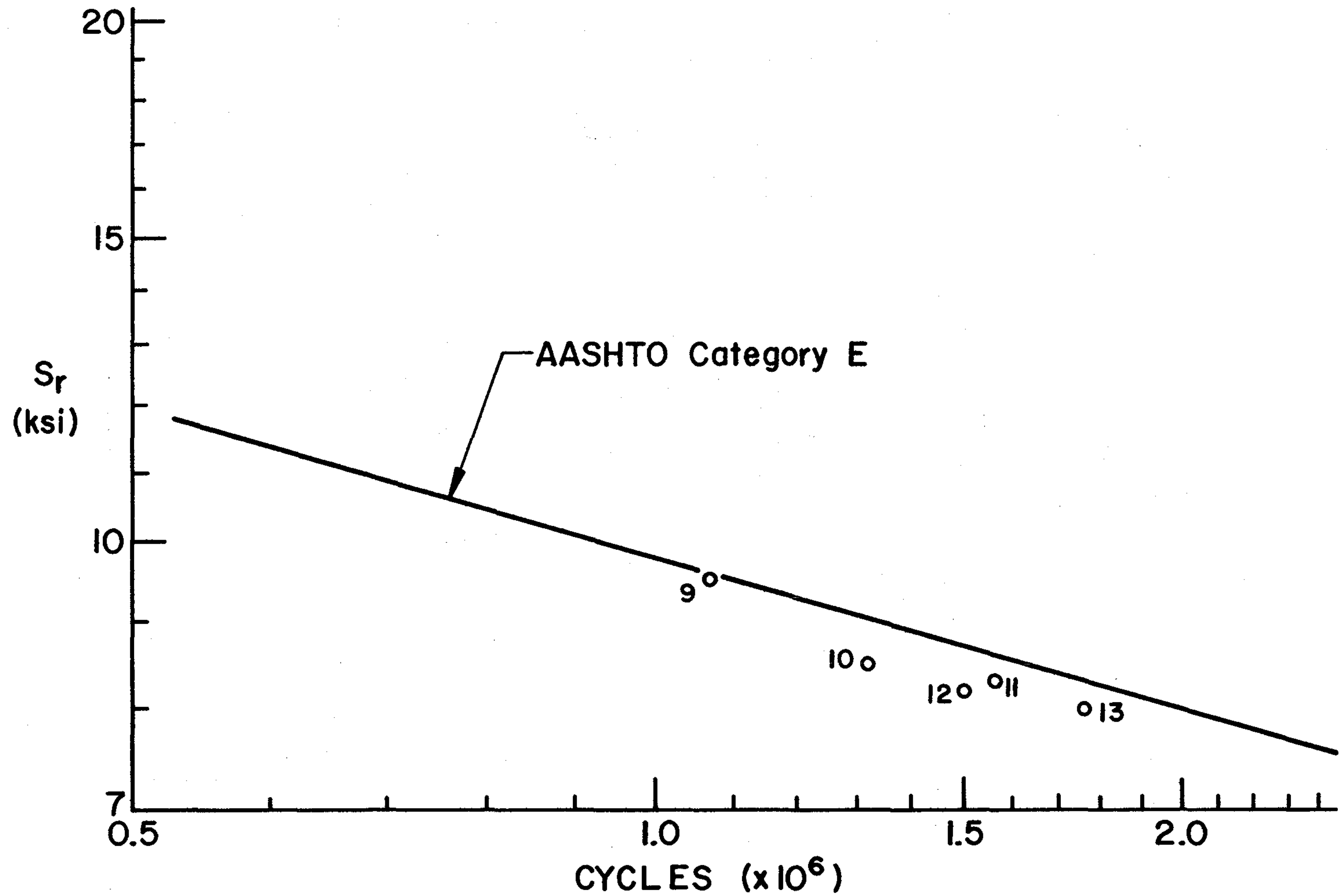


Fig. 8 Fatigue Results - Category E Details

Points 1, 2 and 3 (Type IV_o Category C)

Presence of large discontinuities in the groove welds near the point-of tangency (Ref. 23, Art. 7.4).

Point 4 (Type IV_{ca} Category C)

Weld discontinuities at the point-of-tangency (Ref. 27, Art. 7.1.4).

Points 5 and 8 (Types II_c and I_{ca} Category C)

Out-of-plane displacement of web due to transverse forces from the girder diaphragm (Ref. 27, Arts. 7.1.1 and 7.1.2).

Points 6 and 7 (Type I_{cb} Category C)

Undercut at a weld toe and a severe weld geometry condition (Ref. 27, Art. 7.1.1).

Point 9 (Type V_c Category E)

Fatigue strength slightly below Category E without evident reason. Since the AASHTO categories are the 95% survival lower confidence limits from experimental studies, occasional results below the categories are to be expected (Ref. 27, Art. 7.1.5).

Point 10 (Type V_{oa} Category E)

The nominal stress range at this detail was not measured but assumed to be the same as that at a symmetrical detail (Ref. 23, Art. 7.5).

Point 11 (Type III_{cb} Category E)

Cold lap and porosity of the weld (Ref. 27, Art. 7.1.3).

Points 12 and 13 (Type VII_{ca} Category E)

These points are associated with the discontinuous backup bar detail. The results are consistent with the findings reported in Ref. 24 which shows that the fatigue resistance of this detail is below Category E (Ref. 27, Art. 7.1.7).

3.2.1 Design Recommendation

The fatigue tests of the eight large scale curved girders confirm that for the detail categories examined, no modification of the present AASHTO fatigue provisions is necessary. The design of horizontally curved steel bridge girders against fatigue can proceed under the present AASHTO Specifications.[5] The essential condition is that the nominal stress ranges at details are accurately calculated through considering the superstructure as a three-dimensional structure.

The fatigue tests also underscore the need for weld quality control and adherence to nondestructive inspection of welds as required by AASHTO. The groove welded flange attachment with 6 in. transition radius (Type IV_o, Category C; see Table 2) is particularly sensitive to weld discontinuities near the point-of-tangency. Careful grinding of the weld ends and nondestructive inspection to establish weld soundness as required by AASHTO (Table 1.7.2A2, Ref. 5) is necessary.

A similar fillet welded detail with transition radius (Type IV_{ca}, Category C; see Table 3) is also particularly sensitive to weld discontinuities near the point-of-tangency. Careful grinding of the weld end and nondestructive inspection of the weld should be required.

Discontinuous backup bars (Type VII_{ca}; see Table 3) have inadequate fatigue strength and are prohibited in structures by the AWS Structural Welding Code.[30] Adherence to the AWS Code is essential.

3.3 Stress Concentration, Stress Range Gradient and Principal Stress Effects on Fatigue Life

A complete description of the analytical investigation of stress concentration, stress range gradient and principal stress effects on the fatigue of curved steel bridge elements is contained in Ref. 19. The following is a brief summary of the findings.

3.3.1 Stress Concentration

Stress concentration factors were established for transverse fillet-welded details on girder flanges and webs and groove-welded gusset plates.

Typical stiffener-to-flange fillet welds have values of stress concentration factor between 3.0 and 4.0. Gusset plates with circular transitions generally have values of stress concentration factor below 3.0 except when the transition radius is less than the plate thickness.

3.3.2 Stress Range Gradient

Stress range gradient in flanges of curved girders has little effect on fatigue life and can reasonably be disregarded.

3.3.3 Principal Stress Effects

It is not necessary to calculate principal stress in curved girder flanges. The maximum principal and maximum bending stresses are comparable. The crack path in a fillet-welded detail is likely to be perpendicular to the weld toe. The crack path at a groove-welded detail is likely to follow the minimum principal stress trajectory away from the stress concentration region. However, this was observed to have a negligible effect on fatigue life.

3.3.4 Design Recommendation

The theoretical investigation on the effects of curvature on the fatigue strength of curved plate and box girder bridges has confirmed that no modification of the present AASHTO fatigue provisions is necessary. The design of horizontally curved steel bridge girders against fatigue can proceed under the present AASHTO Specifications.[5]

3.4 Effect of Heat Curving on the Fatigue Strength of Plate Girders

A complete description of the analytical investigation of the effect of heat curving on the fatigue strength of horizontally curved steel plate and box girder bridges is presented in Ref. 21.

The investigation focused on the effect of residual stresses and strains produced by heat curving on fatigue strength. Two primary mechanisms are studied.

1. Mean stress effects on fatigue crack growth, and

2. Fatigue crack growth at web boundaries due to excessive web bowing under compressive residual stresses.

The study found that there should be no significant effect on the fatigue strength due to either mechanism.

3.4.1 Design Recommendation

The study confirms existing design provisions which indicate that if any stress reversal occurs, the entire stress range (including compressive stress excursions) should be considered effective in promoting fatigue crack growth. These provisions apply to heat-curved girders just as they do to straight girders.

3.5 Effect of Internal Diaphragms on the Fatigue Strength of Curved Box Girders

A complete description of the analytical investigation of the effect of the spacing of internal diaphragms on the fatigue strength of curved box girders is presented in Ref. 22. The following is a brief summary of the findings.

The primary stresses in horizontally curved box girders are a function of cross-sectional geometry, span length, curvature, support conditions and loading. However, the distortional stresses are affected by the presence of internal diaphragms. The use of internal diaphragms directly reduces the distortional normal and plate bending stresses, thus increasing fatigue strength of box girders. The relationship of stress range (S_r) versus diaphragm spacing-to-radius ratio (a/R) was found to be a practical guide for selecting diaphragm spacing.

3.5.1 Design Recommendation

Design of curved box girders against fatigue can proceed in the normal fashion according to the present AASHTO Specification.[5] Control of the distortional stresses can be achieved by providing internal diaphragms. The spacing of internal diaphragms can be determined for a desired fatigue life by using a conservative linear S_r versus a/R relationship as illustrated in Ref. 22.

3.6 Effect of Web Slenderness on Fatigue Strength

When slender web plates are used in girders, the main concern with regard to fatigue failure is that the lateral, out-of-plane deflection of slender webs may generate high magnitudes of plate bending stresses. These plate bending stresses could cause fatigue cracks along web panel boundaries.[8] In consideration of this condition, reductions in permissible web slenderness ratios (D/t) have been recommended by CURT.[1]

For Allowable Stress Design procedure

$$\frac{D}{t} = \frac{23{,}000}{\sqrt{f_b}}\left[1.19 - 10\left(\frac{d_o}{R}\right) + 34\left(\frac{d_o}{R}\right)^2\right] \leq 170 \qquad (1)$$

For Load Factor Design procedure

$$\frac{D}{t} = \frac{36{,}500}{\sqrt{F_y}}\left[1 - 8.6\left(\frac{d_o}{R}\right) + 34\left(\frac{d_o}{R}\right)^2\right] \leq 192 \qquad (2)$$

In these equations, the quantities in square brackets are the reduction factors and the fractions before them are the permissible slenderness ratios for straight girder webs as specified by AASHTO. The bending stress f_b is the maximum total (dead load plus live load) stress in the flange of a panel, F_y is the yield stress of the plate material; d_o is the actual spacing between transverse stiffeners; and R is the radius of the curved girder. Initial deflections of web plates have been assumed and are incorporated in the reduction factor.

For all test panels of the curved plate girder assemblages the actual web slenderness ratios and stiffener spacings were both higher than those proposed by CURT. No fatigue crack was detected anywhere along the web panel boundaries of these test girders.

The actual web slenderness ratios of all plate girder assemblies, the corresponding AASHTO permissible values for straight girders, and the proposed CURT limits are all listed in Table 4. Only Load Factor Design values for curved girders are listed. The D/t values for Allowable Stress Design are not computed from Eq. 1 because f_b magnitudes were not defined for the test girders. In designing the test girders, the stress ranges (Δf_b) were controlled, not the total bending stresses.

The actual and permissible spacing of transverse stiffeners are also listed in Table 4.

To compare the initial lateral deflections (w_i) of the web plates, the nondimensionalized deflection values (w_i/t where t is the web thickness) are plotted against the web slenderness ratio (D/t) in Fig. 9. Web deflection data from straight plate girders are also presented. These latter set of data were used in the development of web slenderness ratio limits for straight plate girders.[31] From the figure, it is clear that the curved plate girders of this study had higher initial out-of-straightness than the straight girders.

Examination of web deflections of the test girders indicated that relatively low magnitudes of lateral deflections took place under load. Table 5 lists the maximum measured deflections of the test girders and the corresponding nondimensionalized nominal stress ranges at the panels. The stress ranges in terms of the respective nominal allowable stresses are indications of the load ranges. For straight girders of similar web geometry and under about the same proportion of loads, the web deflections were of higher magnitudes.[31,32] This is partly due to the condition that the outward deflection of the compression flange and the inward deflection of the tension flange of curved girders force a double curvature in the web deflection. The resulting web deflections with respect to the web-flange boundaries are relatively small. Examples are shown in Fig. 10.

These lateral deflections of the curved webs did not cause fatigue cracks along the web boundaries of the test girders. The control of web slenderness ratios and stiffener spacing effectively reduces lateral deflections of the curved webs, and thus reduces the cause of fatigue failure.

3.6.1 Design Recommendation

The curved web panels of the test girder had web slenderness ratios equal to or lower than those permitted for straight girders, but exceeded the CURT-proposed limits in all cases. The distances between stiffeners were all higher than permitted. The web deflections under load were

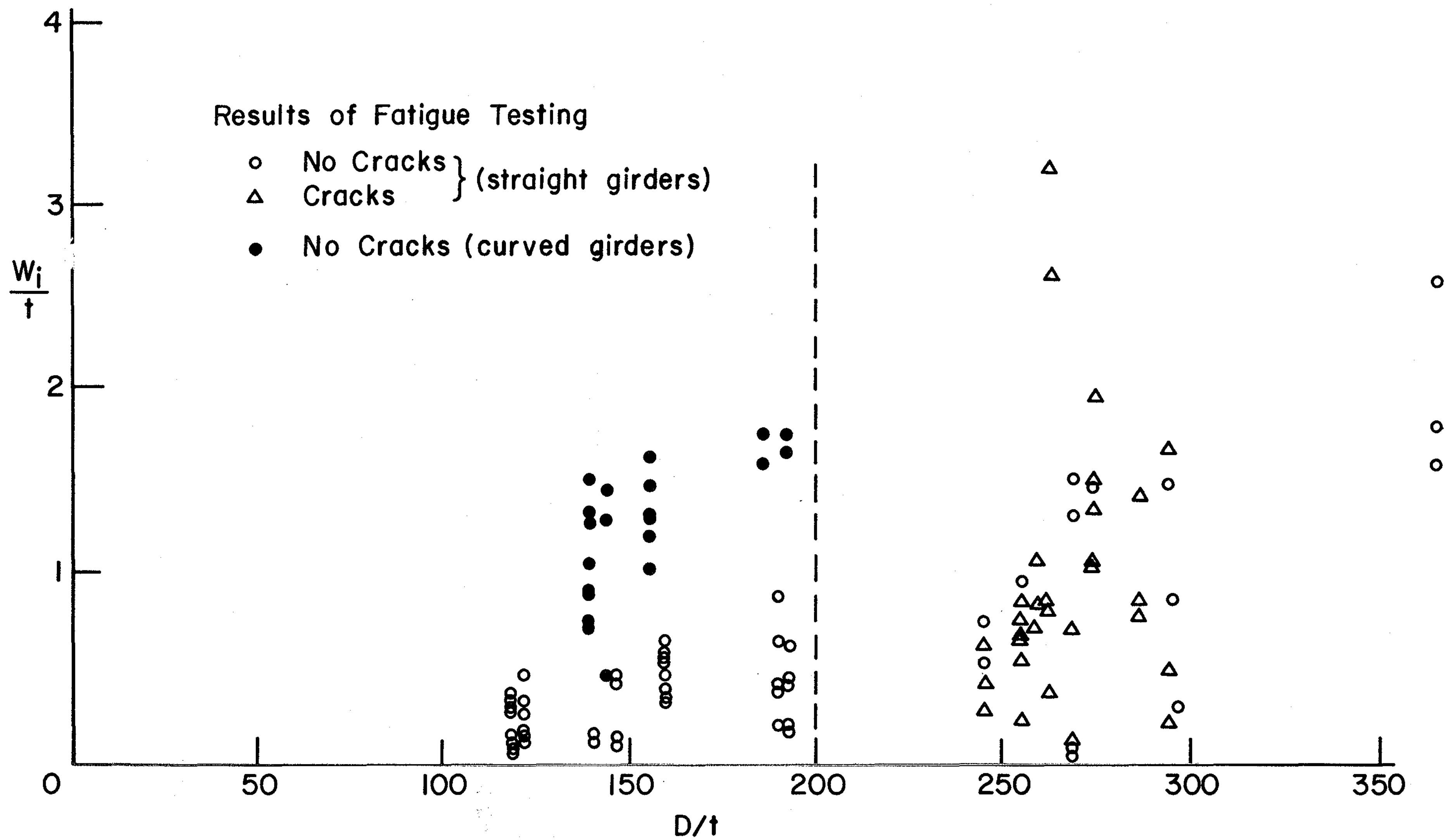


Fig. 9 Initial Deflection vs. Slenderness Ratio for Straight and Curved Girder Webs

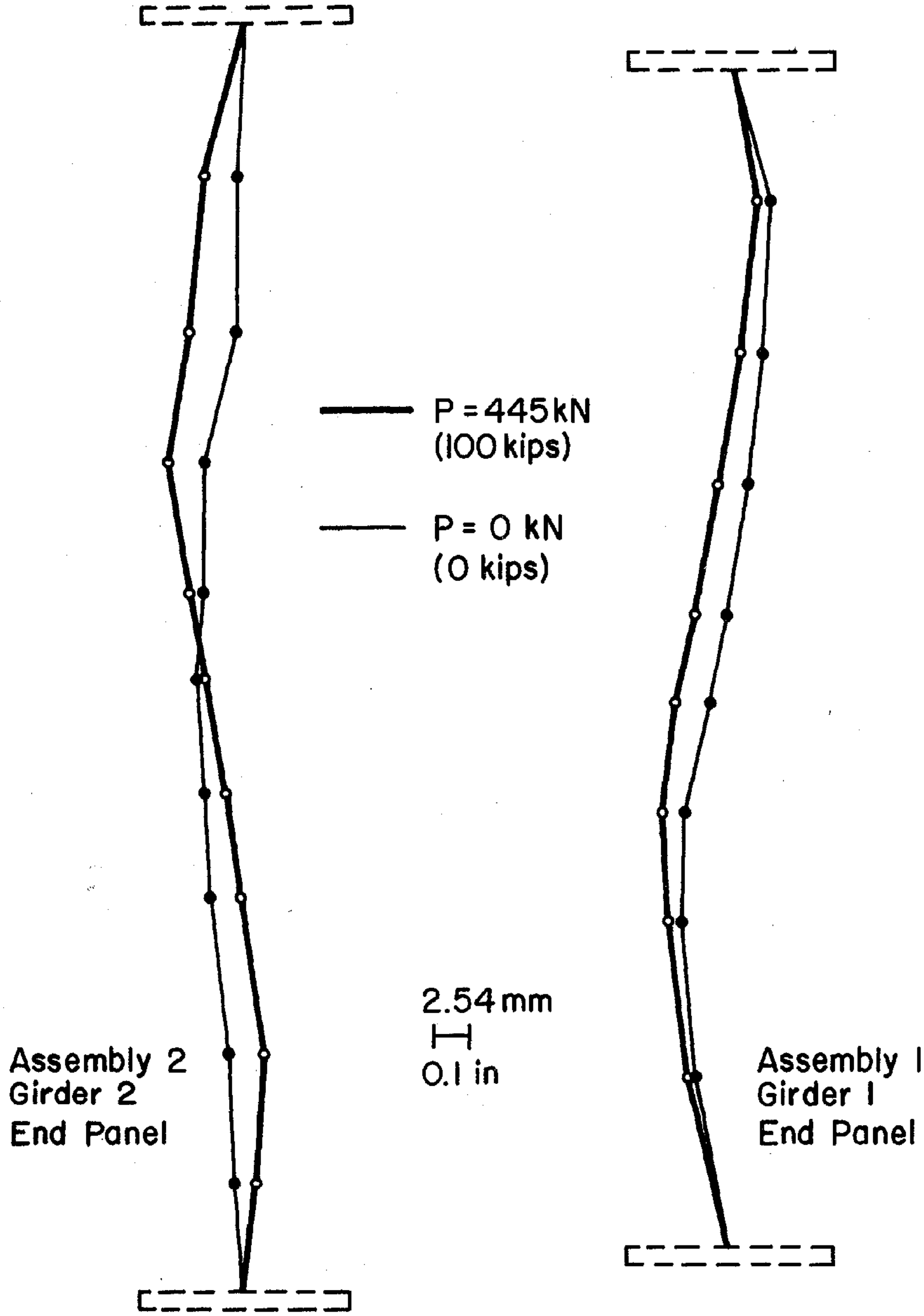


Fig. 10 Relative Web Deflections

TABLE 5 MAXIMUM WEB DEFLECTIONS

Assembly	Girder	Web Panel	$\frac{D}{t}$	$\frac{d}{D}$	$\frac{\Delta w}{t}$	$\frac{\Delta f_b}{20}$*	$\frac{\Delta f_v}{12}$
1	1	End	144	1.31	0.28	0.52	0.33
	2	End	192	2.28	0.28	0.22	0.14
2	1	Center	155	2.03	0.27	0.31	--
	2	End	186	1.05	0.43	0.81	0.35
3	1	Center	155	2.03	0.28	0.31	--
	2	End	155	1.05	0.28	0.76	0.30
4	1	Center	139	2.27	0.18	0.36	--
	2	Center	139	2.37	0.20	0.61	--
5	1	Center	139	2.27	0.19	0.36	--
	2	Center	139	2.37	0.15	0.61	--

*Δf_b and Δf_v in ksi; for end panels, Δf_b is the maximum at end of panel; assumed F_b = 20 ksi and F_v = 12 ksi.

relatively small, and no fatigue crack developed along the web boundaries. These above results strongly indicate that the CURT-proposed slenderness limits are too severe.

In the formulation of the slenderness reduction factors, a simplified model was used which, in effect, did not consider the deflection of the web-to-flange boundaries.[12] The computed plate bending stresses at web-to-stiffener junctions were evaluated as a function of stiffener spacing (d_o/R), arriving at the quantities in the square brackets of Eqs. 1 and 2. Since boundary deflections reduce the relative web deflections and the subsequent plate bending stresses, a less severe reduction factor is justified.

To estimate the lateral deflections of the web boundaries would be mathematically highly involved, if not impossible. Thus, a rigorous reexamination of the web boundary bending stresses is not warranted for the sake of establishing web slenderness ratios. A relatively simple although empirical way to liberate the slenderness reduction factor is to reduce the (CURT) adopted initial out-of-straightness. In other words, it is assumed here that a reduced initial web deflection can trade off the effects of web boundary deflections.

By assuming an initial web deflection of $w_i/t = 0.033$, one half of that adopted in Ref. 12, and assuming that the resulting reduction factor can be expressed as a linear function of d_o/R, the following equation is obtained for the Load Factor Design Procedure.

$$\frac{D}{t} = \frac{36,500}{\sqrt{F_y}}\left[1 - 4\left(\frac{d_o}{R}\right)\right] \leq 192 \qquad (3)$$

The web slenderness ratio from Eq. 3 for girders with $F_y = 36$ ksi are compared with that from Eq. 1 in Fig. 11. The limiting web slenderness ratios of the curved test girders are compared in Table 6. When the actual spacing between stiffeners ($d_o = 120$ in) is used, Eq. 3 gives a value of 128 as compared to 100 from CURT-proposed Eq. 1.

Because the actual stiffener spacings of the test girders were higher than permitted, the limiting slenderness ratios of 100 and 120 are not

TABLE 6 WEB SLENDERNESS RATIOS BY AASHTO LOAD FACTOR DESIGN*

			CURT	Lehigh	CURT	Lehigh
Assembly	Girder	Actual	Actual d_o		$d_o = 1.5$ D	
1	1	144	100	128	120	149
	2	192	100	128	120	149
2	1	155	100	128	116	146
	2	186	100	128	116	146
3	1	155	100	128	116	146
	2	155	100	128	116	146
4	1		100	128	122	150
	2		100	128	122	150
5	1	139	100	128	122	150
	2	139	100	128	122	150

*$F_y = 36$ ksi
Permissible D/t for straight girders is 192.

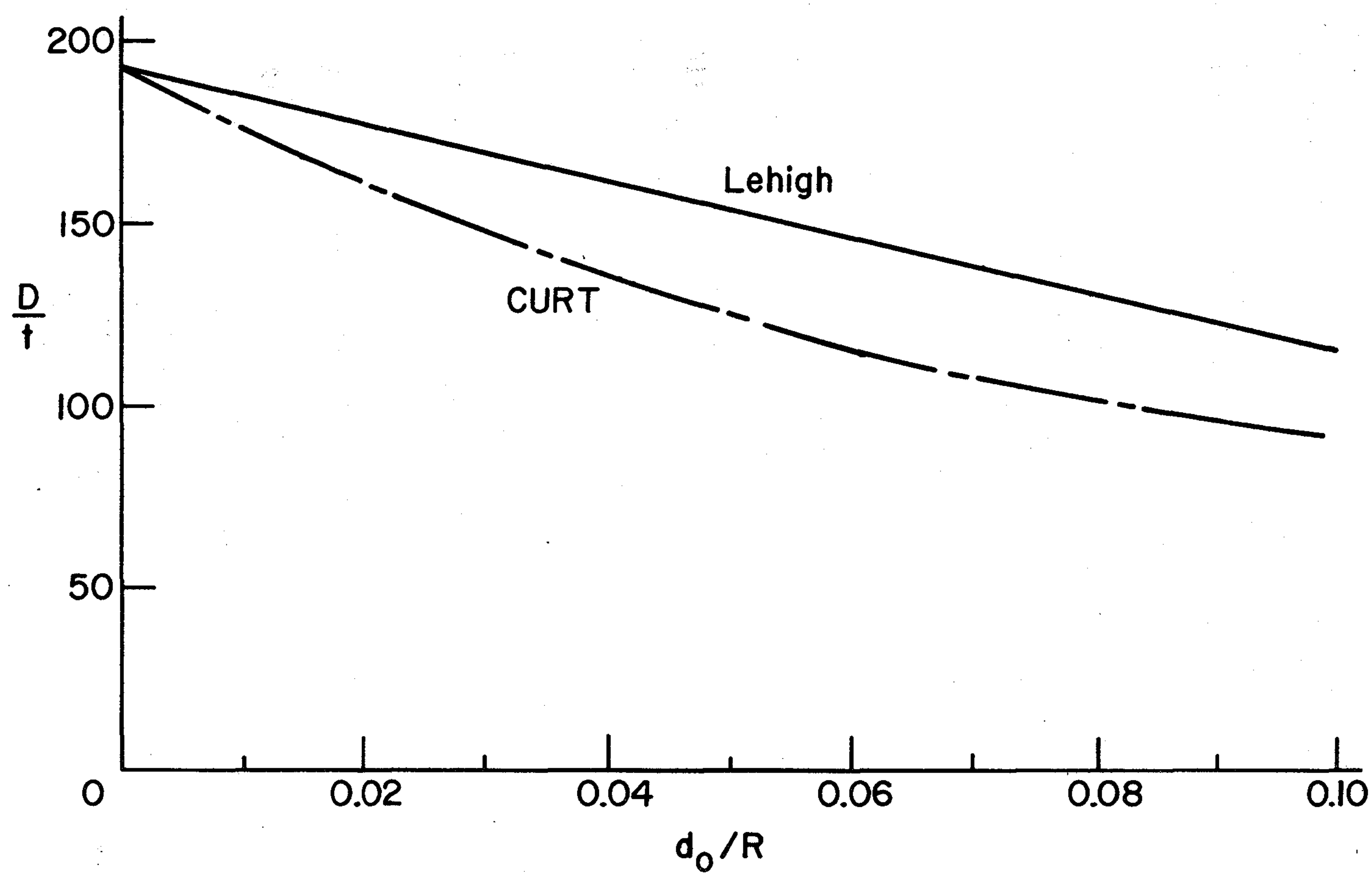


Fig. 11 Comparison of Reduction Factors

"correct" values. If the maximum spacing of d = 1.5 D is used in Eqs. 1 and 3, the proposed slenderness ratios are higher. These values are also listed in Table 6. The Lehigh-proposed limits are less severe than the CURT values.

For the Allowable Stress Design Procedure, it is proposed that the same reduction factor be used.

$$\frac{D}{t} = \frac{23,000}{\sqrt{f_b}}\left[1 - 4\left(\frac{d_o}{R}\right)\right] \leq 170 \tag{4}$$

This equation permits slenderness ratios consistent with those for straight girders when d_o/R is zero. (If, in the future, modifications are made to correlate the permissible slenderness ratios by the Load Factor Design and the Allowable Stress Design Procedure, a simple change of the fraction in front of the square bracket in Eq. 4 will be sufficient.)

4. PROPOSED SPECIFICATIONS AND COMMENTARY

The following proposed specifications are suggested modifications to Ref. 1:

> TENTATIVE DESIGN SPECIFICATIONS FOR HORIZONTALLY CURVED HIGHWAY BRIDGES, part of Final Report, Research Project HPR2-(111) "Horizontally Curved Highway Bridges" prepared for U.S. DOT by CURT, March 1975.

The underlined portions are additions to or modifications of the above tentative specifications.

1.7.144 FATIGUE

The proposed specification should be ammended to read:

> The requirements of Article 1.7.3 concerning fatigue stresses shall apply to <u>horizontally</u> curved steel <u>plate and box girder</u> bridges. Due consideration should be given to the evaluation of stresses in curved members to insure that accurate stress magnitudes are obtained at connection details. <u>The calculated stress shall include the contribution from torsion</u>.

Commentary to proposed addition to Art. 1.7.144:

> The primary difference between the stress distributions in horizontally curved versus straight girders is the contribution of torsional stresses. The resulting complex stress distribution, particularly in curved plate girders requires accurate stress analyses under the appropriate loading conditions when designing against fatigue.

1.7.147 DIAPHRAGMS, CROSS FRAMES, AND LATERAL BRACING

Add the following paragraph to the end of the proposed specification:

> <u>The diaphragm or cross frame connection plates attached to the girder web shall be connected to flange(s) as well in a manner that will prevent distortion of the web at each end of the connection plate. All connection plates shall be coped for a length of 4 to 6 times the web thickness, at points of intersection with longitudinal weldments</u>.

Commentary to proposed addition to Art. 1.7.147:

Out-of-plane web distortion causing severe displacement-induced transverse bending stresses in the web can result in very low fatigue strength unless the connection plates are also attached to the adjacent flange(s) of the plate girder. Coping is required to avoid intersecting welds.

1.7.148 GENERAL

The proposed specification should be ammended to read:

The provisions in Articles 1.7.149 to 1.7.153 are not applicable to riveted or bolted girders. Curved beams and girders shall be proportioned to resist normal stress due to bending and nonuniform torsion (also known as lateral flange bending). The normal stress due to bending shall be determined using the moment of inertia method. The normal stress due to nonuniform torsion shall be determined by any rational method of analysis. <u>For design against fatigue, a stress analysis shall be performed using a rational method of analysis capable of accurately determining stresses under combined bending and nonuniform torsion</u>.

Commentary to proposed addition to Art. 1.7.148

Fatigue strengths can be seriously under or overestimated, especially at intersections of diaphragms with curved plate girders, unless an accurate stress analysis of the three-dimensional structure is performed.

1.7.151 THICKNESS OF WEB PLATES

(A) Girders Not Stiffened Longitudinally

The web plate thickness of curved plate girders <u>shall not be less than that determined by the formula</u>:

$$t = \frac{D\sqrt{f_b}}{23,000} \frac{1}{1 - 4\left(\frac{d_o}{R}\right)} \qquad (a)$$

but in no case shall the thickness be less than D/170.

Where the calculated compressive bending stress in the flange equals the allowable bending stress, the thickness of the web plate in Eq. a shall not be less than the limiting values in Article 1.7.70(A).

Commentary to proposed change to Art. 1.7.151(A)

The CURT recommended deduction of web slenderness ratio (from values for straight girders webs) is too severe, as it is borne out from test results on curved plate and box girder webs. The Lehigh-proposed web slenderness limit incorporates a reduction factor which is expressed as a linear function of spaciny of transverse stiffener, instead of a quadratic form from CURT. The same simple mathematical mode is used for CURT-recommended and Lehigh-proposed reduction factor. However, a smaller initial out-of-straighteners has been assumed here. It is considered that a smaller assumed initial out-of-straightener accounts for the effects of deflections of the curved web boundary.

1.7.168 DESIGN THEORY

(D) Warping Stresses

The proposed specification should be ammended to read:

The effect of warping normal stresses due to nonuniform torsion and cross-sectional deformation shall be included in the design of curved box girder bridges, <u>especially for fatigue</u>, unless a rational analysis indicates that these effects are small <u>and will not reduce the fatigue strength</u>.

Comentary to proposed addition to Art. 1.7.168(D)

As previously stated, fatigue strength can be seriously under or over-estimated unless an accurate stress analysis of the three-dimensional structure is performed.

1.7.169 DESIGN OF WEB PLATES

(B) Secondary Bending Stresses

The proposed specification should be ammended to read:

Web plates may be plumb (90° to bottom of flange) or inclined. The inclination of the web plates to a plane normal to the bottom flange shall not exceed 1 to 4. Transverse bending stresses resulting from distortion of the girder cross section shall be determined using a rational method of analysis. The sum of these distortional stresses plus any transverse bending stress due to supplementary loading, such as utilities shall not exceed 20,000 psi.

<u>Where distortional stresses result from cyclic loads, the web plates shall be attached to the flanges with either full penetration groove welds or fillet welds on each side of the web plate</u>.

Commentary to proposed addition to Art. 1.7.169 (B):

Where a one-sided weld is used to attach the web plate to the flange a crack-like condition exists on the other side which can result in crack propogation. Placement of a full penetration weld or fillet welds both sides of the web can eliminate this condition.

1.7.171 DIAPHRAGMS WITHIN THE BOX

Add the following paragraph to the end of the proposed specification:

<u>The diaphragm or cross frame connection plates shall be attached to the box girder web and flanges in a manner that will prevent distortion of the web at each end of each connection plate. All connection plates shall be coped for a length of 4 to 6 times the web thickness at points of intersection with longitudinal weldments</u>.

Commentary to proposed addition to Art. 1.7.171:

Attachment to flanges is required, as stated above, to prevent low fatigue strengths arising from severe displacement induced transverse web bending stresses. Copies is required to prevent intersecting welds.

5. CONCLUSIONS AND RESEARCH NEEDS

A five year research investigation into the fatigue behavior of horizontally curved, steel bridge elements was conducted at Fritz Engineering Laboratory, Lehigh University, under the sponsorship of the Federal Highway Administration (FHWA) of the U.S. Department of Transportation. The multi-phase investigation was performed in 5 tasks which are listed in Appendix A. Appendix B summarizes the project reports.

The significant conclusions are as follows:

1. It is common practice to reduce complex three dimensional bridge supperstructures (straight or horizontally curved) to simpler planar structures for analysis purposes, on the assumption that the calculated strength will be a lower bound. From a static strength point of view this assumption is usually correct. However, this assumption can lead to sharp reductions of the fatiuge strength of the structure. For design against fatigue the superstructure should be analyzed in its three-dimensional configuration using a method of analysis capable of acurately predicting the stress at any point in the structure under all loading conditions. This is especially true for horizontally curved plate and box girder bridges.
2. Theoretically analyses of curved bridge elements indicates that within the configurational and geometrical limits established by AASHTO, curvature itself has little effect on the fatigue strength of elements and can be disregarded.
3. The results of the theoretical analyses of curved bridge elements plus the fatigue test results on a number of Category C and E details has shown that the present AASHTO fatigue provisions, developed for straight girders, are satisfactory for use with horizontally curved welded steel plate and box girder bridges as long as stresses are adequately evaluated considering the total curved structure.
4. Curved girder bridges are subjected to larger forces from diaphragm and lateral bracing than do straight girder bridges. Consequently curved girders may develop relatively early fatigue cracking due to out-of-plane displacement induced stresses particularly in the

webs. Web stiffeners and attachment plates connected to diaphragm or lateral bracing members should be connected to the flanges in a manner that will prevent out-of-plane motion of the web.

5. If the design relies on the highest fatigue strength allowed by AASHTO for a gusset or other attachment which has a transition radius, extra care must be taken to ensure a smooth, transition curve with no oversized discontinuities in the weldments near the point-of-tangency. Careful grinding of the radius is required together with nondestructive inspection of the groove weld to ensure weld soundness.

6. For heat-curved girders, if any stress reversal occurs, the entire stress range (including compressive stress excursions) should be considered effective in promoting fatigue crack growth in the flanges.

7. The existing AWS web flatness requirements should be strictly applied to curved girders after heat curving.

8. Distortional stresses in curved box girders can be controlled with internal diaphragms. The required spacing of internal diaphragms with respect to fatigue strength can be computed using a straight line S_r versus a/R relationship and the AASHTO fatigue provisions.

9. The CURT-recommended web slenderness ratio reductions from limits for straight girder webs are too severe. New formulas for web slenderness limits are proposed. These formulas contain a linear reduction factor which is expressed as a function of stiffener spacing and radius of the curved web.

The following topics, among many others, need to be studied for future development of design recommendation.

1. The effects of longitudinal stiffeners on the lateral deflection of curved webs, particualrly on the plate bending stresses along the web boundary and the longitudinal and transverse stiffeners. The results will enhance the formulation of a more rational web slenderness limits.

2. Development of theories for the prediction of load carrying capacity of curved plate and box girders. Confirmation of predictions by tests is necessary. The load carrying capaicty is the necessary basis for the development of recommendations for LFD of curved plate and box girders.

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APPENDIX A: STATEMENT OF WORK
"Fatigue of Curved Steel Bridge Elements"

OBJECTIVE

The objectives of this investigation are: (1) to establish the fatigue behavior of horizontally curved steel plate and box girder highway bridges, (2) to develop fatigue design guides in the form of simplified equations or charts suitable for inclusion in the AASHTO Bridge Specifications, and (3) to establish the ultimate strength behavior of curved steel plate and box girder highway bridges.

DELINEATION OF TASKS

Task 1 - Analysis and Design of Large Scale Plate Girder and Box Girder Test Assemblies

Horizontally curved steel plate and box girder bridge designs will be classified on the basis of geometry (radius of curvature, span length, number of spans, girders per span, diaphragm spacing, types of stiffener details, type of diaphragm, web slenderness ratios and loading conditions). This will be accomplished through available information from existing literature and other sources, as required.

Current research on the fatigue strength of straight girders has identified and classified those welded details susceptible to fatigue crack growth. This classification shall be extended to include critical welded details peculiar to curved open and closed girder bridges. These welded details shall be examined with respect to their susceptibility to fatigue crack growth and analyses shall be made to estimate the conditions for fatigue crack growth.

Based on the analyses described above, a selected number of representative open and closed section curved bridge girders shall be defined for the purposes of performing in-depth analyses, design, and laboratory fatigue tests of large scale test assemblies. These girders shall be typical and will characterize commonly used girders, to include the use of welded details. The assemblies shall be analyzed and designed using currently available design guides, methods, and/or computer programs. Each test assembly shall be designed to incorporate the maximum number of welded details susceptible to fatigue crack growth. Stresses in all components of the cross section shall be examined so that the significance of each stress condition can be evaluated. An assessment of the significance of flexural stress, principal stress, stress range and stress range gradient shall be determined at each welded detail. The significance of curved boundaries on the stresses shall be examined. Stress states in welded details equivalent to those used in straight girders shall be examined.

Curved plate and box girder test assemblies shall be designed so that ultimate strength tests can be carried out following the planned fatigue tests, with a minimum of modification.

Task 2 - Special Studies

In addition to but independent of the analyses and designs described in Task 1, certain other special studies shall be performed. These special studies are specifically directed towards those problems peculiar to curved girder bridges, as follows: (1) the significance of a fatigue crack growing across the width of a flange in the presence of a stress range gradient shall be studied, (2) the effect of heat curving on the residual stresses and fatigue strength of welded details shall be examined, (3) newly suggested web slenderness ratios for curved girder webs reduce present slenderness ratios of unstiffened webs. These slenderness ratios shall be examined in terms of fatigue performance of curved webs, and (4) the effect of internal diaphragms in box beam structures will be examined with regard to fatigue behavior.

Task 3 - Fatigue Tests of Curved Plate Girder and Box Girder Test Assemblies

The plate and box girder test assemblies designed in Task 1 shall be tested in fatigue. Emphasis shall be placed on simulating full-scale test conditions. The test results shall be correlated with the analyses made in Task 1 and the results of the special studies performed in Task 2.

Task 4 - Ultimate Load Tests of Curved Plate and Box Girder Assemblies

Following the fatigue tests of Task 3, each plate and box girder test assembly shall be tested statically to determine its ultimate strength and mode of behavior. Fatigue cracks shall be repaired, where necessary, prior to the static tests. Consideration shall be given to providing a composite reinforced concrete slab on each test girder prior to the static tests.

Task 5 - Design Recommendations

Design recommendations for fatigue based on the analytical and experimental work shall be formulated in a manner consistent with that for straight girders. Specification provisions shall be formulated for presentation to the AASHTO Bridge Committee.

APPENDIX B: LIST OF REPORTS PRODUCED UNDER DOT-FH-11-8198

"Fatigue of Curved Steel Bridge Elements"

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FEDERALLY COORDINATED PROGRAM OF HIGHWAY RESEARCH AND DEVELOPMENT (FCP)

The Offices of Research and Development of the Federal Highway Administration are responsible for a broad program of research with resources including its own staff, contract programs, and a Federal-Aid program which is conducted by or through the State highway departments and which also finances the National Cooperative Highway Research Program managed by the Transportation Research Board. The Federally Coordinated Program of Highway Research and Development (FCP) is a carefully selected group of projects aimed at urgent, national problems, which concentrates these resources on these problems to obtain timely solutions. Virtually all of the available funds and staff resources are a part of the FCP, together with as much of the Federal-aid research funds of the States and the NCHRP resources as the States agree to devote to these projects.*

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Safety R&D addresses problems connected with the responsibilities of the Federal Highway Administration under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

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Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by keeping the demand-capacity relationship in better balance through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

* The complete 7-volume official statement of the FCP is available from the National Technical Information Service (NTIS), Springfield, Virginia 22161 (Order No. PB 242057, price $45 postpaid). Single copies of the introductory volume are obtainable without charge from Program Analysis (HRD-2), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

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Maintenance R&D objectives include the development and application of new technology to improve management, to augment the utilization of resources, and to increase operational efficiency and safety in the maintenance of highway facilities.