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FATIGUE OF CURVED STEEL BRIDGE ELEMENTS

ANALYSIS AND DESIGN OF PLATE GIRDER AND BOX GIRDER TEST ASSEMBLIES

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ABSTRACT

Research on the fatigue behavior of horizontally curved, steel bridge elements is underway at Lehigh University under the sponsorship of the Federal Highway Administration (FHWA) of the U. S. Department of Transportation. This multi-phase investigation involves the performance of five Tasks: 1.) analysis and design of large scale plate girder and box girder test assemblies, 2.) special studies of selected topics, 3.) fatigue tests of the curved plate girder and box girder test assemblies, 4.) ultimate load tests of the test assemblies, and 5.) development of design recommendations suitable for inclusion in the AASHTO design specifications. The first Task, analysis and design of horizontally curved plate girder and box girder test assemblies, is complete.

The research effort centered on fatigue crack propagation at welded details. Examination of design drawings of existing, curved, highway bridges indicated a variety of welded details in current use (see Tables 3 and 9). In view of the number of details to be tested and the desired test replication, five plate girder test assemblies and three box girder test assemblies were designed to provide stress and deflection conditions typical of actual bridges at the details to be tested. The test assemblies were analyzed using existing, available computer programs. Test assembly design was in accordance with the AASHTO design specifications as modified by the CURT tentative design recommendations. An account of the test assembly design process and the final designs of the test assemblies are included herein.

Later reports will document the execution of Tasks 2 through 5.

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LIST OF ABBREVIATIONS AND SYMBOLS

a = web panel length in Girder 1 of the plate girder test assemblies (inches)

b = web panel length in Girder 2 of the plate girder test assemblies (inches)

b_f = flange width (inches)

d = transverse stiffener spacing (inches)

f_b = bending stress (psi) at the web-to-compression flange intersection

f_v = average shearing stress (psi) in the gross section of a web plate (Art. 2.7)

f_v = $0.33\ F_y$ = assumed average shear stress (psi) due to torsion (Art. 3.6)

k = buckling coefficient for compression flange longitudinal stiffeners (Ref. 1)

n = number of longitudinal stiffeners

t_f = flange thickness (inches)

t_w = web thickness (inches)

w = width of flange between longitudinal stiffeners or distance from the web to the nearest longitudinal stiffener (inches)

D_w = web depth (inches)

F_b = allowable normal stress (psi) in the tension flange of a curved box girder

F_y = yield stress (psi)

I_s = recommended moment of inertia of a longitudinal stiffener of the compression flange of a curved box girder (in^4)

L = span length measured at centerline of the test assembly

L_1 = span length measured at centerline of Girder 1 of the test assembly

LIST OF ABBREVIATIONS AND SYMBOLS
(Continued)

L_2 = span length measured at centerline of Girder 2 of the test assembly

R = horizontal radius of curvature of test assembly (feet)

R = horizontal radius of curvature of a plate girder (inches)

S_r = stress range (ksi)

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

1. INTRODUCTION AND RESEARCH APPROACH

1.1 Background

Within the past decade there has been increased utilization of horizontally curved girders in highway bridges. In conforming to nonaligned roadway approaches, curved supporting members are more aesthetic than straight girder segments and can result in reduced construction costs. However, the design of curved girders is considerably more difficult partly due to a relative lack of design experience with such girders, and partly due to the more complicated structural behavior particularly with regard to torsion. Until recently, few design guidelines or specifications existed and comparatively little supporting research was performed.

In 1969 the Federal Highway Administration (FHWA) of the U. S. Department of Transportation (U. S. DOT), with the sponsorship of 25 participating state highway departments, commenced a large research project on curved girder bridges. The project involved four universities (Carnegie-Mellon, Pennsylvania, Rhode Island, and Syracuse) and was commonly referred to as the CURT (Consortium of University Research Teams) Project. All of the work was directed towards the development of specific curved steel girder design guidelines for inclusion in the AASHTO bridge design specifications. The curved girders studied included both open (plate girder) and closed (box girder) cross sections.

The tentative specifications[1,2] resulting from the CURT study incorporate their findings as well as input from other simultaneous efforts such as from the University of Maryland. The CURT program 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, under the direction of J. W. Fisher, 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 structures[3,4]. The outcome of these two reports was a major revision of the 1973 fatigue design rules and is now contained in the 1974 interim AASHTO bridge specifications[5,6]. Two other references provide condensed commentary and guidance related to the application of the new provisions[7,8]. However, since no curved girders were analyzed or tested, direct applicability of the new specifications to these members is not assured. Furthermore, no fatigue research on curved girders can be found in the literature.

The research reported herein is part of a multi-phase investigation of curved girder fatigue at Lehigh University entitled, "Fatigue of Curved Steel Bridge Elements", and is sponsored by the FHWA.

1.2 Objectives and Scope

The objectives of the investigation are: (1) to establish the fatigue behavior of horizontally curved steel plate girder 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 girder 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 required.

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 regions. Straight girder research has shown that welded details are more fatigue sensitive than bolted details. Modern bridge structures rely heavily on welded connections 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 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 the modified test assemblies are performed in Task 4. Design recommendations for fatigue are prepared in Task 5, based on the work of Tasks 1, 2 and 3.

This report presents the results of the work carried out in Task 1. Future reports will document the results of work performed in the other Tasks listed in Appendix A.

1.3 Basis for Test Assembly Designs

The intent of the investigation is to fatigue test full scale welded details using large scale test assemblies. This does not imply that the entire test assembly has to be full scale. It means, however, that the welded details should be full scale and the stresses and deflections imposed on the details should simulate full scale conditions. A natural extension of this concept is that the imposed forces and displacements should, in some cases at least, represent extreme values possible in curved girder bridges. The test assemblies are therefore designed to investigate the maximum deviation from straight girder fatigue behavior.

Since the test assemblies are large scale in geometry and cost, it is important to optimize the benefit-cost ratio of each test assembly. As many welded details as possible are therefore placed on each assembly. The number of test assemblies is dictated by the number of different details and the desired replication. All details on a given test assembly are designed to fail in fatigue at about the same cycle life in order to reduce testing time and to reduce problems associated with crack repair. A life of two million cycles was chosen which represents a desired life expectancy for many bridges and can be considered a bench mark figure for fatigue testing.

1.4 Test Assembly Design Constraints

Certain constraints exist in the design of large scale curved girder test assemblies for laboratory testing. One constraint relates to the geometry of the dynamic test bed located in Fritz Engineering Laboratory, Lehigh University. Although the reactions of the test assembly may be slightly off the test bed if necessary, the loading frames must be anchored to the bed. The desired span and centerline radius of the test assembly therefore are limited by the length and width of the test bed and the available opening through the loading frames.

The opening through the loading frames also limits the number of plate girders and box girders in each test assembly cross section.

Another limitation of the testing facility concerns the jack stroke. The maximum dynamic load capacity of each of the two available jacks is 110 kips. At that load capacity the maximum theoretical dynamic stroke of the jack is 0.45 inches, although expected deflections of the loading frame and support movements set the usable maximum stroke closer to 0.35 inches. This means that the vertical deflection of an assembly at the two jack positions can not exceed approximately 0.35 inches.

An equally important design constraint is the ratio of the jack forcing frequency to the natural frequency of a test assembly. The forcing frequency is constant at 250 cycles per minute or about 4 Hertz. The mass and stiffness of an assembly should be such that the minimum natural frequency is about 15 to 20 Hertz so that inertial stresses are minimized and resonance avoided.

1.5 Computer Programs Available for Analysis of Assemblies

Most curved girder structures are of such complexity that the computer is required for reasonably accurate and quick analysis. Very simple curved girders might be analyzed by hand using the "exact" method of Dabrowski[9] or the approximate V-load method developed by U. S. Steel for open sections[10]. However, Dabrowski's approach is difficult to use when diaphragms or bottom lateral bracing are present and both methods are too time consuming when optimization procedures require many design repetitions.

The objectives and tasks of this investigation specifically exclude the development of extensive computer programs for the overall analysis of curved girder bridges. All analyses and designs of the test assemblies are carried out using currently available design guides, methods, and computer programs. No attempt is made to duplicate the CURT effort or any other work in that area. As a result, considerable effort was spent searching for and adapting suitable computer programs to the needs of this investigation.

The needs of fatigue research with regard to computer programs are somewhat different from those of the typical bridge design process to which CURT addressed itself. In assessing fatigue it is important to accurately predict the stress range at a specific point on any given cross section[3,4]. Generalized stresses on the cross section which often suffice in design are not directly usable in estimating cycle life in fatigue. The accurate distribution of stress on the cross section must be known. Unfortunately, many existing curved girder programs do not provide the accuracy required for fatigue research, or for the prediction of fatigue behavior at a given point in a bridge member.

A survey of available computer programs, not all from CURT, was undertaken. The results are shown in Appendix B. Several programs are suitable for curved plate girder analysis but they have varying degrees of accuracy and some have limitations. Programs for curved box girder analysis are essentially limited to finite element and finite strip methods.

The philosophy adopted in the computer analyses of all the test assemblies was to use two different computer programs for the analysis of all of the plate girder assemblies and two different programs for the analysis of all the box girder assemblies. In this way a comparative check on stresses is available. Such a comparison is felt necessary due to the high number of welded details on each of the test assemblies, and to ensure that fatigue cracks develop at the welded details reasonably near the assumed design life of two million cycles.

Referring to Appendix B the Syracuse program[11] was used during the preliminary designs of the curved plate girder assemblies. This program is limited in that it provides only generalized cross section stresses. These were extended using hand calculations to determine stresses at welded details. The final designs of the plate girder assemblies were based on analyses using the Berkeley program, CURVBRG[12]. CURVBRG gave stresses at the welded details and required little hand computation. The Syracuse and Berkeley programs were used primarily because they were immediately available and their input is relatively simple. Also, they were adaptable to Lehigh's CDC 6400 computer system. The adaptation process, however, required considerable effort. This effort probably represents a minimum for programs of this type. Comparison of results of analyses using the Syracuse program and CURVBRG show reasonable agreement. Preliminary results of tests on plate girder assemblies 2 and 3, available in spring 1976 indicates that the experimental stresses are for the most part within about 15 to 20% of the theoretical predictions by CURVBRG. (See Chapter 2 for description of assemblies.)

The search for suitable programs for the analysis of the box girder assemblies was far more extensive. Referring to Appendix B the finite element program, SAP IV, from Berkeley[13], was selected for the primary and final analytical work. A finite strip program, CURDI, also from Berkeley[14], was chosen for the comparative analyses. CURDI became operational only in June 1975 when most of the final design work based on SAP IV was completed. Comparison of analytical results from both programs showed reasonable agreement. The major drawback with any finite element program is the expense and SAP IV proved relatively costly to run. However, in finite element work cost is related to accuracy which is largely a matter of the type and number of elements used.

2. DESIGN OF CURVED PLATE GIRDER TEST ASSEMBLIES

2.1 Curved Plate Girder Bridge Characteristics

Three sources of information were used to establish the characteristics of existing, horizontally curved, plate girder bridges. First, the results of a Federal Government survey, made available by the Federal Highway Administration (FHWA) and summarized in Tables 1 and 2, proved very valuable. Second, the results of a survey conducted by the AASHO-ASCE Committee on Flexural Members was helpful[15]. Finally, a sample of actual design drawings, made available by the State of New York Department of Transportation (SONYDOT), the Pennsylvania Department of Transportation (PennDOT), and other state transportation departments, was reviewed.

The information shown in Table 1 as well as that determined from Ref. 15 and the design drawings reveals that at present most bridges have girder radii greater than 150 feet. Only two percent of the bridges reported in Ref. 15 have a radius of 150 feet or less. The significance of 150 feet is that the 1973 AASHO bridge specification does not permit heat curving of members with radii below this level[5].

Table 2 shows the dimensionless parameters describing the bridges listed in Table 1. Although the steel yield strength is not available, the flange width-thickness ratios (b_f/t_f) are less than the maximum limit of 24 prescribed in AASHO Art. 1.7.69[5]. The web depth-thickness ratios (D_w/t_w) suggest that some designs follow allowable stress procedures (AASHO Arts. 1.7.70 and 1.7.71) while others conform to load factor requirements (AASHO Art. 1.7.124)[5]. In several cases longitudinal stiffeners are used. Reference 15 indicates that over 75 percent of existing curved steel bridges use A36 steel.

Most of the curved plate girder bridges in the survey have two to four girders per span. A large percentage of the bridges have only one span although a number of them also have two or three spans. The common span lengths are between 50 and 150 feet. About the same number of bridges have span lengths above and below this range.

AASHO Art. 1.7.21 for straight girders provides that "plate girder spans shall be provided with cross frames or diaphragms at each end and with intermediate cross frames or diaphragms spaced at intervals not to exceed 25 feet". This article also states that "diaphragms shall be at least 1/3 and preferably 1/2 the girder depth"(5). The CURT tentative design specifications, Art. 1.7.147, suggests that all cross frames or diaphragms should be full depth(1). The commentary to Ref. 1 suggests that a 25 foot diaphragm spacing should be used only when a radius exceeds 1000 feet. For radii under 200 feet a 15 foot diaphragm spacing is suggested.

The available information on diaphragms indicates that a large majority are full depth and of the truss type. The diaphragm spacings range from less than eight feet to over 25 feet. The majority of the diaphragms are spaced between 14 and 20 feet. Many existing curved girder bridges, therefore, conform to the recommendations of Ref. 1 with regard to diaphragms.

The spacing of parallel girders and the corresponding girder depths were examined in the available sample of design drawings. While variance does exist between the designs, most girders were found to be spaced between five and ten feet apart. Girder depths were generally in the vicinity of five feet. This means that a trused diaphragm, if present, forms an angle of between 25 and 45 degrees with the horizontal. Some states, such as New York and Missouri, recommend the 45 degree angle whenever possible.

Another feature of curved plate girder bridges is the use of lateral cross bracing. Reference 15 shows that about half of the existing bridges use a lateral bracing system. Usually it exists only at the bottom of the parallel girders. Except during the construction phase, top lateral bracing is automatically provided by the composite concrete slab.

The information obtained on typical curved girder designs is necessary to define the conditions under which actual welded details normally exist. However, as stated in Art. 1.3, it is not the intent of this investigation to test full size bridges. Rather, the welded details should be full scale and the boundary stresses and deformations imposed by the surrounding test assembly on the welded detail should characterize full scale conditions in actual bridges. These considerations required that each test assembly be large scale, within the limits of the testing facilities. The geometrical design of each test assembly therefore follows this philosophy and is based on the information from the surveys, summarized in Tables 1 and 2.

2.2 Welded Detail Classifications

The welded detail classifications contained in Refs. 4 and 7 form a basis for review of actual curved girder details. The objective of detail testing is twofold. First, to determine the fatigue performance of curved girder details which are also common to straight girders. This is not necessarily a duplication of the previous research work on straight girders since the details are now located in a curved girder stress and deformation environment. Second, to establish classifications for details which are found only in curved girders. As far as plate girders are concerned, the same details can be found in either straight or curved girders. However, due to shorter diaphragm spacing requirements, the number of welded details per curved girder is often greater.

Table 3 summarizes the welded details selected for investigation. There are five basic types (I_o to V_o) with subtypes for III_o and V_o. The detail type is shown by the Roman numeral in the upper left hand corner of each drawing. The first subscript, o, refers to open section. A second subscript a or b is given when there are subtypes. The corresponding straight girder category, relating to the 1974 interim AASHTO specifications, Table 1.7.3B, is shown by the capital letter in the upper righthand corner of each drawing in Table 3[6]. As far as plate girders are concerned all details of interest are either Category C or Category E. Below the category letter is the corresponding allowable stress range (ksi) for straight girders which represents the 95% confidence limit for 95% survival at two million cycles.

In all drawings in Table 3 a solid dot defines the location of the predicted fatigue crack. Often two or more such locations are possible depending on the stress distribution and/or initial flaw size. Only the welds relating to the details studied are shown. Groove welds are specifically identified. All welds shown without marking symbol are of the fillet type. Other welds such as those connecting webs to flanges are not shown for clarity. For any weld not shown in the drawings it can be assumed that the flaws and stress concentration associated therewith are not critical relative to those of the welds shown. Therefore, fatigue crack growth in these welds, although likely present, is not expected to limit the detail life.

For details III_o, IV_o, and V_o in Table 3 a detail dimension is shown. The length in the direction of the weld can be interpreted as a prerequisite for deciding in which category the detail should be placed(6). As length increases the detail category becomes more severe. Most details over four inches long fall into Category E. An exception is detail IV_o shown in Table 3, having smooth circular transitions which decrease the severity. Detail IV_o is actually not very common to straight or curved girders. It is included in the testing program for comparison with detail V_{oa} which is more common. Any dimensions not shown are assumed unimportant with regard to the fatigue life of the welded detail.

Fabrication of the plate girder assemblies requires the complete specification of individual girder cross sections plus all information pertaining to the diaphragms including the welded connections to the plate girders. All major design work therefore focused on the welded details located at diaphragm and bottom lateral bracing connections in the tensile stress region of the assemblies. Because no room for error exists once fabrication is complete, the stress conditions at these locations must be known as accurately as possible prior to fabrication and testing. On the other hand, because the individual plate girders are readily accessible after fabrication, additional details can be added between the diaphragms in Fritz Laboratory after initial static load tests have determined the actual stress conditions in the girders.

The welded details shown in Table 3 therefore fall into two basic groups depending on their positions along an assembly. Group 1 details consist of welded details at connections for diaphragms or bottom lateral bracing which are placed during fabrication of an assembly. These details are discussed further in Art. 2.5. Group 2 details consist of the additional details welded to an assembly in Fritz Laboratory after the initial static load tests of that assembly are complete but prior to fatigue testing that assembly. These details are discussed further in Art. 2.6. The welded details in group 2 provide replication of group 1 details as comparative results with straight girder behavior, thus increasing the benefit-cost ratio for each test assembly.

The welded details shown in Table 3 are associated with attachments which occur at two basic locations on the girder cross section. Detail types I_o, IV_o, and V_o occur on the flanges where both bending and warping normal stress range exists. Types II_o and III_o occur on the web where, due to the doubly symmetric section, warping is negligible.

2.3 Preliminary Designs of Plate Girder Assemblies

References 3 and 4 emphasize that only stress range and type of detail are critical in determining fatigue life. Mean stress, type of steel, and other variables have little noticeable affect on fatigue performance. Therefore the small dead load stress was ignored in the analysis. An early decision was made to design each curved plate girder test assembly for symmetrical quarter point loading using two hydraulic jacks operating at a maximum load range of 100 kips each. Only stress range produced by the two hydraulic loading jacks cycling between 5 and 105 kips is considered in the design of the plate girder details. In the preliminary designs all steel is assumed to be A36.

Before analyzing by computer, several sets of preliminary designs were made by hand using the V-load method[10] and available approximate design charts[16,17]. The objective was to determine the approximate overall geometry of the test assemblies within the constraints of the test bed dimensions (Art. 1.4). The results of the preliminary design are shown in Figs. 1 and 2. A single 40 foot span and 120 foot radius, both defined by the assembly centerline midway between the two plate girders, was selected in preference to a larger span and radius since the smaller radius tends to emphasize curved girder characteristics. An even smaller radius was considered but this would result in a smaller span, because of test bed width limitations, and fewer welded details. Stress range gradients in the flanges would also exceed typical values if the radius is much smaller than 120 ft. Twin plate girder test assemblies were selected because of width limitations of the test bed. A three girder assembly is not possible if reasonable girder spacing is to be maintained. Girder depths of about four to five feet together with a five foot girder spacing results in a diaphragm bracing angle (X-type) near the sometimes recommended 45 degrees (Art. 2.1).

Given a 40 foot span, five diaphragms at 10 foot spacing are possible, three of which are interior in the span. The resulting aspect ratio, d/D_w, is within the range of practical values as shown in Table 2. The preliminary designs considered 100 kip loads positioned over each of the two diaphragms at the quarter points (Fig. 1). The desired stress ranges are therefore attainable over more than half of each girder length. Spherical bearings, assumed for each of the four support points, will be simulated in the test set-up by sets of double rollers placed orthogonal to each other to allow tangential and radial motion at each support.

The number of welded detail types (Table 3) and the number of suitable locations for details in each test assembly suggested that five plate girder assemblies would be required. Since it is necessary to have replication in fatigue testing due to typical data dispersion, five assemblies provide between three and fifteen data points for each type of detail.

The preferred jack location (with respect to the limiting jack deflection discussed in Art. 1.4 and the desired stress ranges) along radial lines over the diaphragms at the quarter span positions was determined by analysis using the Syracuse computer program (Appendix B). For a given test assembly the vertical deflection under a hydraulic jack increases as the jack position is moved toward the outer girder of the assembly (Fig. 1). As the load moves towards the outer girder the bending stress in that girder also increases while bending stress decreases in the inner girder, even to the point of changing sign. Likewise, the end reactions of the outer girder increase with outward load movement while the inner girder reactions decrease and may also change sign. In all cases it was assumed in the preliminary designs that the simulated spherical supports will offer only vertical restraint and that only vertical loads are possible. The jack loads are also confined to locations between the inner girder and the assembly centerline, thus assuring compressive reactions (no uplift).

For jack positions between the inner girder and the assembly centerline, the warping stress on the centerline side of the tension flange of the inner girder is tensile adjacent to the interior diaphragms and adds to the primary bending tensile stress. Warping stress on the centerline side of the tension flange of the outer girder is compressive adjacent to the interior diaphragms and tends to cancel the primary bending tensile stress. It was therefore desirable to place most flange test details on the centerline side of the inner girders and most web test details on outer girders. The cross section geometry of each girder was then adjusted to make the design stress range along the inside edge of the inner girder tension flange and the design stress range at the web-to-tension flange junction along the outer girder reach the appropriate levels (Table 3) at test detail locations so that fatigue failure of the details will occur within the desired fatigue life of two million cycles.

Final analyses of each test assembly following the preliminary designs (Art. 2.5) revealed that the additional details in group 2 could be added to the flanges and webs of both girders at several locations where the stress range was suitable for the particular welded detail category.

2.4 Consideration of Composite Plate Girder Assemblies

Preliminary designs of one of the plate girder assemblies were performed considering both steel and composite steel-concrete assemblies. The objective was to determine if the addition of a composite reinforced concrete slab was necessary in order to provide realistic stress ranges and stress range gradients at the full scale welded detail locations on the assemblies. The concrete slab was assumed to be 84 in. wide and 6 in. deep. Complete interaction between the slab and the steel girders was assumed. The 28 day compressive strength of the concrete was assumed to be 4000 psi.

Comparative analyses showed that the required stress range conditions at all welded details could easily be obtained with or without a composite slab. Specifically, the required stress ranges were attained with very little alteration of the overall design (assembly layout and cross-section dimensions). Stress range gradients and displacement-induced stresses such as oil canning and flange raking were slightly higher when no slab was present, but were within the ranges of practical conditions.

The analyses also indicated that the fatigue test results for assemblies without a composite slab would tend to be upper bound with regard to the effects of diaphragm forces and warping stress range gradients in the flanges. Test results for steel assemblies will therefore tend to emphasize curved girder characteristics with respect to fatigue. This includes other behavior such as flange raking which, in turn, relates to fatigue crack propagation at web boundaries. While the amount of raking may remain constant with or without a composite slab, the neutral axis is higher in assemblies with a slab. Since cross bending stresses in the web are primarily displacement-induced, fatigue damage at web boundaries are more probable in the non-composite case since boundary stresses are a little higher.

An early decision was made, on the basis of the comparative analyses to design and test non-composite assemblies. This decision led to a simplification of test assembly fabrication. In addition, more accurate correlation between predicted and measured stresses will result during laboratory testing thus enabling increased accuracy in interpreting fatigue test results.

2.5 Final Designs of Plate Girder Assemblies

As soon as the Berkeley computer program, CURVBRG, became operational at Lehigh University (late spring 1974), analyses of each test assembly were made using CURVBRG and compared with the Syracuse analyses. After finding reasonable agreement on both stresses and deflections, CURVBRG was then used for the final designs of all five test assemblies. The stress range profiles and location of group 2 details for each plate girder test assembly are shown in Figs. C1 to C10 of Appendix C.

As discussed in Art. 2.2, Table 3 shows stress ranges for each category which correspond to 95% confidence of 95% survival. Thus, to ensure the formation of large fatigue cracks at about two million cycles and to allow for a margin of error between calculated and measured stress ranges it was necessary to use design stress ranges in the tests somewhat higher than those recommended in AASHTO[6]. For this reason a 10 ksi design stress range was selected for all Category E details and a 15 ksi design stress range was selected for all Category C details.

Except for the above modification of design stress ranges the CURT tentative design specifications or the 1973 AASHO bridge design specifications were followed[1,2,5,6]. Where discrepancies between the two specifications existed, the CURT recommendation was used. If CURT does not make a specific recommendation, AASHO was used. One exception was in the permissible web slenderness ratio. In this instance, a range of values was used. A second exception was in the selection of transverse stiffener spacing. Certain limits prescribed by CURT and AASHO were purposely exceeded at some locations and these are discussed further in Art. 2.6.

Figure 3 shows a schematic plan view of a typical plate girder test assembly. Girder and joint (girder-diaphragm intersections) numbering are the same for all plate girder assemblies. The loading range for each jack is 100 kips (between 5 kips minimum and 105 kips maximum). Two jacks are used for each test assembly. As shown in Fig. 3, the load positions are either directly over girder 1 at the quarter points (position 1) or midway between girders 1 and 2 at the quarter points (position 2). The jack load for position 2 is applied to a short spreader beam (Fig. 2) which is supported at the adjacent girder joints (3 and 4 or 7 and 8). For load position 1 the load is imposed directly on girder 1 at joints 3 and 7.

The plate girder test assembly program is summarized in Figs. 4 through 8. These figures show the cross section dimensions of both girders of each assembly as well as the locations of the hydraulic jacks and the locations of the group 1 details (Art. 2.2).

For girders 1 and 2 (Fig. 1) of each test assembly Table 4 gives the computed stress ranges at joints 3 to 6 (Fig. 3) on both edges of the tension flange, and at the web to flange intersection. Where two stress range values are shown in the flange for a particular girder, the interpretation is as follows: For girder 2 the upper value is the stress range at the exterior (outside the assembly) flange edge and the lower value is for the interior (towards assembly centerline) flange edge. For girder 1 the upper value is the stress range at the interior (towards assembly centerline) flange edge and the lower value is for the exterior (outside the assembly) flange edge. For both girders the stress range values for the web are those at the junction of the flange and the web. Stress ranges at joints 7 and 8 are equal to those at joints 3 and 4, by symmetry. The underlined stress ranges in Table 4 correspond to the locations of the group 1 details (Art. 2.2) shown in Fig. 4 througn 8.

By comparing the computed stress ranges shown in Table 4 with those required as shown in Table 3, it is apparent that reasonable agreement exists in most cases. It is not possible and is not necessary to achieve perfect agreement in any case. It is only necessary in these experiment designs to assure that the discrepancies between the actual and desired stress ranges at the details are not too large. The larger the discrepancies the larger the time interval between the formation of the first and the last crack in a given assembly. If the first cracks form too early, it may be difficult to repair or retrofit them so that later cracks may develop before the assembly itself is destroyed.

For detail types III_o, IV_o and V_o, the expected crack location is up to 16 in. from the diaphragm-to-girder joint. Examination of the stress range profiles in Appendix C indicates that, depending on the location of the detail, the stress range can be higher or lower than the stress range at the joint shown in Table 4. The discrepancies involved were not considered detrimental to the life of the assemblies. It is anticipated that early cracks can be retrofitted so that later cracks can develop. Therefore all group 1 details were located with reference to the stress ranges at the diaphragm-to-girder joints.

Examination of Table 4 also shows very small stress ranges at the Type I_o details at joints 4 and 6 of girder 2 of assembly 1. In this case these details are associated with stiffeners which are required for connecting the diaphragm members. Figure 4 shows this detail designation in parentheses since fatigue cracking there is not anticipated.

While stress ranges were determined for a simultaneous 100 kip load range for each jack, deflections at the jack location were determined for a 105 kip maximum load. In general, deflections are maintained at or below the 0.35 in. jack stroke limit (Art. 1.4). One exception is in test assembly 2 where a 0.37 in. deflection is permitted.

Bottom lateral bracing is shown in Figs. 7 and 8 for both center bays of test assemblies 4 and 5. Details III_{oa} and III_{ob} exist only in these assemblies where it is desirable to have bracing members in place during the tests. In all cases these bracing members are single angles, 3x3x3/8. The forces in the bottom lateral bracing will be of the displacement-induced type and are not expected to be large. Some racking of the web at the ends of the gusset plates is predicted because of restricted gusset plate movement and bottom flange raking.

Due to the different types of group 1 details provided on each test assembly, the final designs often required that extra details be provided for diaphragm connection purposes. For example, test assembly 2 shown in Fig. 5 has test detail V_{oa} on girder 1 and detail II_o on girder 2. These are not exactly compatible for attaching the diaphragm members unless the girder depths vary significantly. Thus, a detail identical to type V_{oa} was also welded to girder 2 of assembly 2 below the Type II_o test stiffener detail only to connect the girder to the bottom member of the diaphragm. Such auxiliary connections are not expected to fail in fatigue before the group 1 test detail since, based on the values shown in Table 4, the stress range is always somewhat less than that permitted for two million cycles.

A summary of the group 1 welded test details for the plate girder assemblies is shown in Table 5.

The arrangement of diaphragm members for each plate girder test assembly is shown in Figs. 9 through 13. The diaphragms for each assembly were designed to accommodate the group 1 details to be tested. Although nominal angle sizes were required for the fatigue test program, the angle sizes were modified slightly to conform to ultimate strength requirements discussed in Art. 2.8.

2.6 Welded Details Between Diaphragms - Group 2 Details

The difference between the group 1 and group 2 details was discussed in Art. 2.2. The final designs of the plate girders focused on the stress range conditions at the group 1 details only. However, stress range profiles along the length of the tension flanges of both girders of each assembly were developed and are shown in Appendix C. These profiles were used to position the group 2 details. A more complete discussion of the locations of these details is presented in Appendix C. The number of details possible, is based on the number of available locations as shown in Figs. C1 to C10 of Appendix C.

An attempt was made to locate as many group 2 details between diaphragms as reasonably possible, while maintaining sufficient separation of details to minimize any interference effects. In some instances more than one type of detail is possible at a given location. The selection was made on the basis of equalizing the number of data points for each detail type, if possible.

A summary of the group 2 welded test details is shown in Table 5. Subtype III_{oa} is not suitable as a group 2 detail. Extra transverse stiffeners are not provided at locations of type III_o details, so only type III_{ob}, attaching to the web, is possible as a Group 2 detail. Table 5 also shows the total number of group 1 and group 2 details provided and a summary on the basis of assembly and girder. The number of type V_o details provided is somewhat greater than for other details. The excess details provide for the possibility that during testing, more locations for this detail may be possible. This possibility is likely since the experimental stress range profiles along the tension flanges are not expected to correlate exactly with the analytical profiles shown in Appendix C.

Detail types I_o and II_o in group 2 also serve a double function. In addition to enabling more data to be obtained on Category C details, they also serve to modify the web panel aspect ratio, thus providing more data on the web boundary fatigue problem (oil canning).

Since no diaphragm or bottom lateral bracing members are connected to group 2 details, a comparison of fatigue data for group 1 and group 2 details of the same type will show the role of diaphragm and bracing member forces in the fatigue process.

The group 2 flange attachment details are also located in regions of different warping stress range gradients (Appendix C), enabling the importance of this parameter on crack growth to be investigated experimentally.

2.7 Web Slenderness Ratios and Transverse Stiffener Spacing

Articles 1.7.70 and 1.7.71 of AASHO, discuss allowable web slenderness ratios and maximum transverse stiffener spacing[5]. For A36 steel the maximum slenderness ratio (D_w/t_w) is 165. The AASHO requirement for transverse stiffener spacing (inches), d, is interpreted in this investigation as follows:

a) $d \le \dfrac{11{,}000 t_w}{\sqrt{f_v}}$ but not greater than D_w

where f_v = average shearing stress (psi) in the gross section of the web plate at the point considered

t_w = web thickness (inches)

D_w = web depth (inches)

b) The first two transverse stiffener spaces at a simply supported end shall be one half that calculated in a).

c) Certain transverse stiffeners may be ommitted if the web slenderness ratio, D_w/t_w, does not exceed $7500/\sqrt{f_v}$ or 150, whichever is less, and the maximum spacing of remaining stiffeners does not exceed D_w.

If load factor design provisions are followed, Art. 1.7.124(E) of AASHO limits D_w/t_w in A36 steel to 192 provided transverse stiffeners are used[5].

The CURT recommendation was derived from Ref. 18[1,2]. This recommendation modifies Art. 1.7.70 of AASHO by virtue of the curvature and introduces a link between Arts. 1.7.70 and 1.7.71. The formula given below is taken from Art. 1.7.151 (a) and applies when d/R exceeds 0.02.

$$\frac{D_w}{t_w} \le \frac{23000}{\sqrt{f_b}} \left[1.19 - 10\left(\frac{d}{R}\right) + 34\left(\frac{d}{R}\right)^2\right] \quad \text{but not greater than 170}$$

where R = horizontal radius of curvature of the girder (inches)

f_b = calculated compressive bending stress (psi) where the web intersects the compression flange

The requirements of AASHO Art. 1.7.71 mentioned previously still apply to stiffener spacing, d. However, the sizing of transverse stiffeners is somewhat altered in the CURT recommendation[1,2].

Since transverse stiffeners exist at all diaphragm locations, the above recommendations were reviewed to decide where additional transverse stiffeners should be located. In all cases the stress condition corresponding to a jack load of 105 kips was used. The philosophy followed was to stay within prescribed allowable stress provisions in some instances and to exceed them in others. In this way some comparative results on the suggested provisions, as related to fatigue can be obtained. Also, fatigue data on the so-called "oil canning" effect is thereby acquired.

Figure 14 shows the placement of extra transverse stiffeners required for the fatigue tests as discussed above. Table 6 compares the actual and allowable values of web slenderness ratios and transverse stiffener spacing. In this table the word panel refers to the length of web between diaphragm locations. Since the web slenderness ratio provided is constant for a given girder, the ratio allowed is the minimum found for any section of the girder.

2.8 Ultimate Strength Tests

Ultimate strength testing of the assemblies is not within the original scope of this investigation. However, it is possible to repair most of the fatigue cracks and continue testing the assemblies under static load until the ultimate load is reached thereby increasing the benefit-cost ratio of the test assemblies. With the possibility that such tests may be performed after completion of the fatigue test program a decision was made to design diaphragms and bearing stiffeners for the anticipated ultimate loads.

No attempt was made to analyze the plate girder test assemblies for their actual ultimate load capacities. Such a determination is possible only after decisions are made as to the limit states of interest and the assemblies are examined and modified if necessary to achieve those limit states. It is likely that most if not all the assemblies would at least have a composite reinforced concrete slab added to them. The load positions also may be different from those considered in the fatigue test program (Figs. 4 to 8).

Nevertheless, an estimate was made of the largest forces which might occur in the diaphragms and at the four bearing points of each assembly. The diaphragm forces were computed assuming that each assembly was straight, instead of curved, and that a simple plastic moment condition is achieved. The corresponding estimates of diaphragm forces were such that only minor modification of diaphragms was necessary in all but assembly 1. In that case the angle sizes were increased substantially (Art. 2.5).

3. DESIGN OF CURVED BOX GIRDER TEST ASSEMBLIES

3.1 Curved Box Girder Bridge Characteristics

Very little information is available on existing, horizontally curved, box girder bridges. A survey by the AASHO-ASCE Committee on Flexural Members showed only 9 of the 507 horizontally curved bridges reported were box girder bridges[15]. A survey by the CURT Project of State Highway Departments and consulting engineers showed 19 of 32 replies reported no experience with curved box girder bridges[21]. In addition to the above surveys, however, two sources were helpful in establishing the characteristics of existing, horizontally curved, box girder bridges. First, a state-of-the-art survey of horizontally curved bridges by McManus, Nasir, and Culver was helpful[22]. Second, a review of actual design drawings from the Pennsylvania Department of Transportation provided much information.

Table 7 shows the available information on the overall characteristics of existing, horizontally curved, box girder bridges. Table 8 shows the values of dimensionless parameters considered important in curved box girder design. These values were obtained by a review of available design drawings.

The AASHO-ASCE survey reported that 2 of the 9 curved box girders included were heat curved. It was noted, however, that the radius of curvature of both heat curved box girders was 1,973 ft.[15]. It is expected that heat curving of box girders would be rare except in the case of an extremely long radius of curvature. The remaining curved box girder bridges surveyed were cut to the required curvature[15].

The CURT tentative design specifications, Art. 1.7.171, require intermediate diaphragms or cross frames to limit the normal stresses and transverse bending stresses due to distortion of the box section. The CURT recommendations do not specify the spacing or stiffness of such diaphragms. These parameters are to be determined by a rational analysis. Both sets of plans available for review indicated internal diaphragms spaced at 10 feet. The diaphragms were full depth and of the truss type in both cases.

Curved box girder cross-sections may take a variety of forms. Rectangular and trapezoidal sections may be used as well as other configurations involving the number of girders in each span and the number of cells in each girder. Examples of such configurations are the single cell, single box girder bridge; the single cell, multiple box girder bridge; and the multiple cell, single box girder bridge. The available design drawings were all of curved box girders with rectangular cross sections but with a variety of girder configurations.

Another feature of curved box girder bridges is the use of top lateral cross bracing. Reference 21 shows that of the respondents reporting experience with curved box girder bridges 92 percent used top lateral bracing systems. One fourth of the top lateral bracing systems were temporary, provided only for shipping and erection.

3.2 Detail Classification

The welded detail classifications contained in Refs. 3, 4 and 7 form an initial basis for the review of curved box girder details. These classifications were extended after examination of details in actual use to include those common only to box girders. Unlike the situation discussed in Art. 2.2 for plate girders, there is no information presently (1976) available on the fatigue behavior of straight box girder details. Therefore a correlation between the fatigue behavior of details common to both straight and curved box girder details is not possible. As far as box girders are concerned the same details can be found on either straight or curved girders. However, since internal and external diaphragm spacing on curved box girders may be shorter, the number of welded details per curved girder is often greater.

Table 9 summarizes the welded details selected for investigation. There are eight basic types (I_c to $VIII_c$) with subtypes for all but types II_c, V_c and $VIII_c$. The detail type is shown by a Roman numeral in the upper left hand corner of each drawing. The first subscript, c, refers to closed section. A second subscript, a or b, is given when there are subtypes. The corresponding straight girder category, relating to the 1974 Interim AASHTO Specifications, Table 1.7.3B, is shown by the capital letter in the upper righthand corner of each drawing in Table 9[6]. As far as box girders are concerned most details of interest are either Categories C, D or E. There are three details for which the exact category is not known. These are expected to be in the range of category C to D. One detail (VII_{cb}) is Category B. Below the category letter is the corresponding allowable stress range (ksi) for straight girders which represents the 95% confidence limit for 95% survival at two million cycles.

In all drawings in Table 9 a solid dot defines the location of the predicted fatigue crack. Often two or more such locations are possible depending on stress distribution and/or initial flaw size. Only the welds relating to the details studied are shown. Groove welds are specifically identified. All welds shown without marking symbol are of the fillet type. Other welds such as those connecting webs to flanges are not shown for clarity. For any weld not shown in the drawings it can be assumed that the flaws and stress concentration associated therewith are not critical relative to those of the welds shown. Therefore, fatigue crack growth in these welds, although likely present, is not expected to limit the detail life.

Fabrication of the box girder assemblies requires the complete specification of the assembly cross sections plus all information pertaining to the diaphragms including connections to the flange and web plates. All major design work therefore focused on the welded details located at diaphragm connections in the tensile region of the box girder assemblies. Because no room for error exists once fabrication is complete, the stress conditions at these locations must be known as accurately as possible prior to fabrication and testing.

As discussed in Art. 2.2 in connection with the plate girder assemblies, it is possible to place many additional details between the internal diaphragms and on the exterior surfaces of the box girder assemblies. But unlike the plate girder assemblies where the interior is readily accessible, access to the interior of a box girder assembly is somewhat restricted. As a result welding of additional details to the interior surfaces, in the laboratory, following initial static load tests could be a difficult, time-consuming, and costly procedure. A decision was made therefore to also place all of the additional welded details on the box girder assemblies during fabrication.

The welded details shown in Table 9 are divided into four basic groups depending on the type of transverse or longitudinal attachment with which they are associated in each of the box girder assemblies.

Group 1 welded details are associated with the type of interior diaphragm. Three diaphragm types were selected in order to examine the effect of diaphragm rigidity and cross section distortion on the fatigue behavior of the box girder assemblies. The diaphragms are discussed further in Arts. 3.3 and 3.5. If only transverse web stiffeners plus cross bracing (no transverse flange stiffeners) are used as diaphragms the influence of relatively high distortions on fatigue strength can be examined. With reference to Table 9, details I_{ca} and II_c are associated with this type of diaphragm. Because the web distortions may seriously impair the fatigue strength, a second type of diaphragm consisting of both transverse web and flange stiffeners plus cross bracing is also provided. The type I_{cb} detail is associated with this diaphragm configuration. The stress at the flange surface may increase slightly with this type of diaphragm. However, this increase should be more than offset by a reduction in stress due to smaller distortion at the stiffener-to-web connection. The third type of diaphragm is essentially a plate type which is expected to provide the greatest diaphragm rigidity. Detail I_{cb} is also associated with this diaphragm. Because the diaphragms in curved box girders are subjected to higher forces than those in straight girders the significance of the distortional effects on the fatigue strength will be evaluated by looking for cracks that are expected to form either in the web when the transverse web stiffener is touching (not welded to) the bottom flange (detail II_c) or in the flange when the transverse web and flange stiffener or the plate type diaphragm is welded to the bottom flange surface (details I_{ca} and I_{cb}). The details at the diaphragm locations are expected to govern the overall fatigue strength of curved box girder bridges. For this reason they are of primary interest to designers. Detail I_{cb}, similar to that used on the more rigid plate type diaphragm, will provide a comparison with details associated with the more flexible diaphragms to indicate whether or not a more rigid type diaphragm is desirable from the point of view of fatigue strength.

Group 2 welded details are associated with the connections of longitudinal stiffeners to the flanges and webs of curved box girder assemblies. Referring to Table 9, detail types and subtypes of III_c, IV_c, V_c, and VI_c are contained in this group. There are no experimental fatigue results directly available on either straight or curved elements for these types of details. Simulated beam specimens have indicated that a severe AASHTO category E detail may be expected when a longitudinal stiffener is abruptly discontinued such as shown in details III_{cb} and V_c of Table 9. Because a substantial reduction in fatigue strength is expected, possible improvements in fatigue strength will be examined for a modified V_c detail in the form of a curved radius transition at the weld toe termination shown as detail IV_{ca}, or a 1:2.5 sloped transition shown as detail IV_{cb}. Because longitudinal stiffeners are likely to occur in box girders the application of the radius or straight transition appears to be reasonable for application to both straight and curved box girder configurations. Details III_{cb} and V_c are very similar. An improvement in the III_{cb} detail is shown as detail III_{ca} where the longitudinal stiffener is welded to the transverse stiffener, providing continuity. Other variations of detail V_c such as details VI_{ca} and VI_{cb} will also be examined to determine the influence of weld terminations with a bolted splice to reduce the stress concentration effect, and the influence of intermittent welds.

Group 3 welded details are associated with the continuous longitudinal web-to-flange welds. Both fillet and single beveled groove welds will be examined as shown in details VII_{ca} and VII_{cb} of Table 9. These welds are not unique to curved girders but an excellent opportunity is provided to examine the influence of both continuous and discontinuous back-up bars with continuous fillet welds. These were and are being used and no information is presently (1976) available on their fatigue bahavior. These studies will be equally applicable to both curved and straight box girder configurations.

Group 4 welded details are short exterior attachments used for connecting exterior (between box) diaphragms. A clip angle, as shown in detail $VIII_c$ of Table 11 was selected for study.

3.3 Preliminary Designs

References 3 and 4 emphasize that only stress range and type of detail are critical in determining fatigue life. Mean stress, type of steel, and other variables have little noticeable affect on fatigue performance. Therefore, the small dead load stress was ignored in the analysis. An early decision was made to design each curved box girder test assembly to about the same span and loading conditions as for the plate girder assemblies (Art. 2.3). The assemblies are designed for symmetrical quarter point loading using two hydraulic jacks operating at a maximum load range of 100 kips each. Only stress range produced by the two hydraulic loading jacks cycling between 5 and 105 kips is considered in the design of the box girder details. In the preliminary design all steel is assumed to be A36.

Early in the preliminary design several cross-section geometries were considered. These included a single cell, single box girder assembly, a two-cell single box girder assembly, and a single cell, twin box girder assembly each having either rectangular or trapezoidal cross-section. A decision was made to design each assembly as a single cell, single box girder with rectangular cross-section. There were five factors present in this decision: (1) minimum cross-section dimensions of about 3 ft. per side are required to provide sufficient interior space for a person to crawl through the assembly to inspect for interior fatigue cracks; (2) the rectangular cross section is easier to fabricate; (3) the objectives of the fatigue test program do not justify more elaborate analysis and design; (4) a large single box cross-section allows more realistic size welded details to be tested in fatigue; and, (5) test bed dimension limitations dictate a single rather than twin box girder assembly.

Contrary to the preliminary design phase of the plate girder assemblies, no "simple" computer program was available for preliminary analyses of the box girder assemblies. As a result these analyses were carried out using SAP IV, a finite element computer program[13]. Each trial box girder was discretized into plate, shell, beam and truss elements. Considerable time and effort was required using many trial girders to establish flange and web thicknesses, cross-sectional dimensions and preferred load positions. The primary objective is to design the box girder assemblages such that the deflection under the jack loads is not more than 0.35 inches (stroke limit) but with stress ranges throughout most of the girder such as to optimize the number of fatigue details to be investigated.

Preliminary designs indicated that the desired stress ranges would be obtained using curved box girders having a 36-ft. centerline span. The results of the preliminary designs are shown in Figs. 15 and 16. All trial sections were analyzed as simply supported (spherical supports at the four corners) single span box girders subjected to concentrated loads at the quarter points. The loads are applied normal to the plane of curvature and directly above the inner web. This position was determined after several trial locations along radial lines through the quarter point positions. Because of curvature and eccentricity of load, both bending and torsion is introduced. Deformation of the cross section results in additional stress resultants. This effect varies with the number, rigidity and location of internal diaphragms.

The number of welded detail types (Table 9) and the number of suitable locations in each test assembly for details suggest that three box girder assemblies are required. Since it is necessary to have replication in fatigue testing due to typical data dispersion, three assemblies provide between four and sixteen data points for each detail type. The three box girder assemblies are identical except for the interior diaphragm configurations and the types of attachments and associated welded details.

3.4 Consideration of Composite Assemblies

It was recognized at the outset that some type of concrete top slab or steel plate is required to produce a torsionally closed cross-section. As far as the objectives of the fatigue test program are concerned it makes no difference whether a composite concrete slab or a steel plate, welded or bolted to the webs, is used, providing realistic stress range conditions are obtained at the welded details under test.

A decision was made to use a fairly thick top flange plate which is bolted to the two webs of the box girder. This decision was based on four factors: (1) the plate would simulate the effect of a composite concrete slab, (2) stress range data at welded detail locations would likely be more predictable if a steel plate rather than a concrete slab were used, (3) the top flange would be removable to allow major access to the interior of the box girder when necessary for weld repair, and, (4) only one top flange plate need be fabricated for use with all three box girder assemblies.

3.5 Final Designs

A schematic plan view of a typical box girder assembly is shown in Fig. 17. Cross-section views of the three assemblies are shown in Figs. 18, 19 and 20.

All assemblies are 36" x 36" in cross-section with a centerline span of 36 ft. and a radius of 120 ft. The bottom flange and webs are 3/8" in thickness. A 1" x 60" top flange plate with three 2" x 6" longitudinal stiffeners is bolted to the webs. A single flange plate will be fabricated for use on all three assemblies. This plate is provided with lifting hooks for use with the overhead crane in Fritz Laboratory. The longitudinal stiffeners serve two functions: (1) to help raise the neutral axis to a realistic level, and, (2) to stiffen the top flange plate during lifting.

Since two of the three assemblies will be shipped to Fritz Laboratory without a top flange, a top lateral bracing system shown in Fig. 21 will be used during shipping and handling.

The final designs of the curved box girder assemblies were also performed using SAP IV(13). Figure 22 shows the discretization used in the finite element program. A second program CURDI, a finite strip program, was used to perform a comparative check on the SAP IV results(14). Figure 23 shows the discritization used in the finite strip program. Stress and deflection results produced by the two programs differ by about 15 to 20 percent. Some difference is expected due to the inevitable difficulties in modelling the box girders for either finite element or finite strip analyses. Smaller stress and deflection results were obtained using CURDI. Due to the greater experience with SAP IV in Fritz Laboratory, the final designs were based on the SAP IV results.

Stress range profiles and locations of group 2 and 3 welded details for the curved box girder assemblies are shown in Figs. D1 to D7 of Appendix D. For all three assemblies the profiles are shown at the intersection of the flange and web stiffeners, the points of interest for details in group 2 (Art. 3.2). For assembly 3 a stress range profile along the web-to-bottom flange intersection is also shown. This is the location of details in group 3 (Art. 3.2).

The three curved box girder test assemblies differ basically in the type and location of welded details to be tested in fatigue and the type of interior diaphragms used as shown in Figs. 18, 19 and 20. Three types of diaphragms are used: X type in assembly 1 (Fig. 18); V type in assembly 2 (Fig. 19) and plate type in assembly 3 (Fig. 20). The end diaphragms of all three assemblies are of the plate type. The maximum stresses and deflections of the three test assemblies are nearly identical. The designs of the test assemblies are optimized to obtain fatigue failure at as many locations as possible at approximately 2 million cycles. Table 9 shows stress ranges corresponding to 95% confidence of 95% survival. Thus, to ensure failure at two million cycles it is necessary to design for a stress range slightly higher than indicated. For this reason all fatigue details to be tested were placed in a region where the stress range is 2 ksi more than the stress range specified in Table 9.

In most instances the tentative design specifications from CURT were followed[1,2]. The tentative specifications were developed from both analytical and experimental studies of curved box girder members as well as from field tests of existing curved bridges. It should be noted that the current AASHO design specification does not distinguish between straight and curved box girder bridges[5]. The CURT recommendations do not specify the spacing of interior diaphragms for curved box girders. Diaphragm spacing may be different depending on whether a static or fatigue design condition is assumed. The diaphragm type and spacing was, of course, considered when generating the stress range profiles shown in Appendix D.

Table 10 shows the stress ranges at the center of an element, 4-1/2" away from the web-to-bottom flange intersection, at each diaphragm location. The stress range at the web-to-bottom flange intersection, computed by extrapolation from the computed element stresses, is also shown. The stress ranges shown in Table 10 apply to the first group of welded details (Art. 3.2) at the diaphragm connections in the tension region. These details are Category C requiring 15 ksi (13 ksi plus 2 ksi). Thus fatigue cracks are expected to occur in the vicinity of joints 3, 6 and 7 (Fig. 17) but not at joints 4, 5 and 8. Fatigue details exist at these joints because of the required diaphragm connections. The locations of details in the second group (Art. 3.2) are shown in Figs. D1 to D6 to Appendix D. In each case the detail is located at a point where the stress range is 2 ksi larger than the category stress range shown in Table 9.

The locations of details in the third group (Art. 3.2) are shown in clip angle details contained in group four (Art. 3.2).

Table 11 summarizes the number of details to be tested and shows the replication achieved. Some details of type III_{ca}, III_{cb}, I_{ca} and I_{cb} which occur because of a physical condition (intersection of a longitudinal stiffener with a transverse stiffener) or because of a required diaphragm connection, in a location of insufficient stress range, and where fatigue damage is not expected are shown in parenthesis in Table 11.

3.6 Proportioning the Flanges and Webs

The CURT tentative design specifications for the tension flange of a curved box girder are based on first yield. Both the normal stresses due to bending and warping and the torsional shear stress contribute to yielding of the tension flange. The Mises yield criterion is used to determine the combination of shear and normal stresses required to cause yielding.

Neglecting the small radial stresses in the plane of the flange plate and incorporating a factor of safety of 1.82, the Mises yield condition establishes the allowable normal stress, F_b, in the tension flange of a curved box girder as

$$F_b = 0.55\, F_y \left[1 - 9.2 \left(\frac{f_v}{F_y}\right)^2 \right]^{\frac{1}{2}}$$

where $f_v = 0.33\, F_y$ = shear stress due to torsion (psi)

F_y = yield stress (psi)

This expression is given in the CURT tentative design specifications Art. 1.7.170(A)[1,2].

The derivation of the above expression assumes that the shear and normal stresses are approximately uniform across the flange width. This implies that the entire tension flange will yield simultaneously. However, in the curved box girder test assemblies there is a significant stress gradient across the tension flanges. First yield of the tension flange would not constitute overall failure of the box girder. Redistribution of stress and strain hardening would prevent overall failure even to the point of complete yielding of the tension flange. Therefore, the provisions of Art. 1.7.170(A) are conservative.

The design of the compression flange of a curved box girder must consider instability as well as yielding. Therefore, the CURT tentative design specifications for unstiffened and stiffened compression flanges are based primarily on elastic plate buckling theory[1,2]. In the case of compression flanges with longitudinal stiffeners two modes of instability are possible: (1) local buckling of the flange plate between the longitudinal stiffeners with stiffeners remaining straight; and (2) buckling of the complete flange involving both the longitudinal stiffeners and flange plate. For practical box girders, it is usually not economical to provide longitudinal stiffeners with sufficient rigidity to prevent overall flange buckling prior to local plate buckling between the stiffeners[19]. The specification equations are therefore expressed in terms of a buckling coefficient and the width-thickness ratio for the flange plate between stiffeners. The CURT tentative design specifications Art. 1.7.170(C) recommends longitudinal stiffeners be placed at equal spacings across the flange width[1,2]. It is recommended that each longitudinal stiffener have a moment of inertia, I_s, about an axis perpendicular to the stiffener and through the stiffener-to-flange juncture such that:

$$I_s = \phi\, t_f^3\, w$$

where $\phi = 0.07\, k^3 n^4$ for values of n greater than 1

$\phi = 0.125\, k^3$ for a value of $n = 1$

w = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener (in.)

n = number of longitudinal stiffeners

k = buckling coefficient which shall not exceed 4 (a value of k between 2 and 4 is suggested)[1]

t_f = flange thickness (in.)

This equation is also given in AASHO Sect. 1.7.129 and was developed by approximating the relationship between the buckling coefficient and stiffener rigidity given by Timoshenko[20].

For the compression flange of a curved box girder, stiffened according to the above provisions, the allowable normal stress is the same as for the tension flange if the width-to-thickness ratio of the flange, w/t, is such that:

$$\frac{w}{t} \leq \frac{3070\sqrt{k}}{\sqrt{F_y}}\, X$$

where $X = 1 + 0.5\left(\frac{f_v}{F_v}\right)^2 \left[2.6 - \left(\frac{k}{k_s}\right)^2\right]$

$$k_s = \frac{5.34 + 1.28 \sqrt[3]{10.92 \frac{I_s}{wt^3}}}{(n+1)^2} \leq 5.34$$

$f_v \leq 0.33\ F_y$ = shear stress due to torsion (psi)

F_y = yield stress (psi)

The width-to-thickness ratios of the stiffened compression flanges of all the box girder assemblies are less than this limiting value.

No direct statement is given in the current AASHO specifications with regard to the proportioning of web plates for box girders(5). It is implied that webs of box girders are to be designed referring to the provisions for webs of plate girders with due consideration given to the inclination of the web. Webs of curved box girders are subjected to both bending and shear, the latter due to flexural loading and torsion. After determining the total shear force, the webs are proportioned (depth-thickness ratio, transverse stiffener spacing) in the same manner as in the curved plate girders.

The effect of web slenderness ratio on fatigue will not be investigated here. All curved box test assemblies have the same web slenderness ratio of 91.5.

The bending stress is low in the end panels and no data on web fatigue can be expected there. But the bending stress is higher in the interior panels and it is probable that fatigue failure will occur by two million cycles.

3.7 Ultimate Strength Tests

Ultimate strength testing of the assemblies is not within the scope of this investigation. However, it is possible to repair most of the fatigue cracks and continue testing the assemblies under static load until the ultimate load is reached.

As with the plate girder assemblies (Art. 2.7) no attempt was made to analyze the box girder assemblies for their ultimate load capacities. However, certain details such as bearing stiffeners were examined and designed for an estimated ultimate load of 2 to 3 times the maximum fatigue load of 105 kips.

4. CONCLUSIONS

Task 1 of Appendix A, the analysis and design of horizontally curved steel test assemblies, is complete. Five plate girder test assemblies and three box girder test assemblies were designed and analyzed in preparation for the fatigue testing program of Task 3.

The test assemblies must provide stress and deflection conditions typical of full scale bridges at the welded details selected for testing. Therefore, the test assemblies are designed in accordance with the AASHTO bridge specifications[5] as modified by the CURT tentative design specifications[1].

Several constraints governed the design of the plate girder and box girder test assemblies:

First, it is desirable for all details on a given assembly to fail at approximately the same cycle life in order to reduce testing time and the problems associated with crack repair. The assemblies are designed to produce a fatigue life of two million cycles for all details tested. The stress ranges required to produce a fatigue life of two million cycles are estimated from the AASHTO fatigue provisions[6,7,8]. The allowable stress ranges given by the AASHTO fatigue provisions represent the 95% confidence limit for 95% survival. Therefore, to ensure the formation of large fatigue cracks and to allow a small margin for error, the design stress ranges are 2 ksi higher than the allowable stress ranges specified in the AASHTO fatigue provisions.

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

Third, the maximum dynamic load capacity of the two available jacks is 110 kips with a usuable maximum stroke of 0.35 inches.

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 (Tables 1, 2, 7 and 8) and classification of welded details (Tables 3 and 9) was performed.

2. Initial preliminary design of the test assemblies was undertaken.

a) Initial preliminary design of the plate girder test assemblies was achieved using the V-load method[10].

b) Initial preliminary design of the box girder test assemblies was accomplished by assuming the assemblies were straight, and carrying out a simple strength of materials analysis.

3. The preliminary designs of the test assemblies were analyzed using existing, available computer programs. Two different programs were used for the analysis of each plate girder test assembly and each box girder test assembly. Thus, a check on the stress ranges and deflection conditions required by the first and third design constraints was achieved.

a) The plate girder test assemblies were analyzed using the Syracuse[11] and CURVBRG[12] computer programs. Reasonable agreement between the results given by both programs was obtained.

b) The box girder test assemblies were analyzed using the CURDI[14] and SAP IV[13] 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. Many cycles of design (Step 4) and analysis (Step 5) were required to achieve the optimum preliminary designs satisfying the design constraints mentioned previously.

6. Based on the optimum preliminary designs, final designs for each of the test assemblies were prepared. Final design of the test assemblies included complete detailing of the required diaphragms, transverse stiffeners, and bearing stiffeners and preparation of design drawings for fabrication of the test assemblies.

All five plate girder test assemblies have a centerline span length of 40 feet and a centerline radius of 120 feet. Typical cross sections of the plate girder test assemblies are shown in Figs. 9 through 13. The plate girder test assemblies are loaded at the quarter points either directly over the inner girder or at the test assembly centerline. The applied load ranges from 5 kips to 105 kips and the maximum deflection under the load is approximately 0.35 inches.

All three box girder test assemblies have a centerline span length of 36 feet and a centerline radius of 120 feet. Typical cross sections of the box girder test assemblies are shown in Figs. 18 through 20. The box girder test assemblies are loaded at the quarter points directly over the inner web. The applied load range and deflection conditions are similar to those for plate girder test assemblies.

Future reports will present the results of the fatigue testing program and the special studies, as well as recommendations for the fatigue design of horizontally curved, steel bridges.

5. TABLES

TABLE 1 OVERALL CHARACTERISTICS OF EXISTING HORIZONTALLY CURVED, PLATE GIRDER BRIDGES

(From Survey by FHWA)

Bridge Number	State	No. Spans	Girders Per Span	Maximum Span Length	Minimum Radius of Curvature	Diaphragm
1	New York	1	4	102 ft.	150 ft.	A
2	New York	2	5	106	250	A
3	New York	3	4	86	1000	A
4	New York	2	4	182	500	A
5	New York	2	4	250	230	A
6	New York	2	4	124	151	A
7	New York	2	6	127	157	A
8	New York	1	4	133	1200	A
9	New York	1	4	126	464	A
10	Pennsylvania	2	2	88	938	B
11	Pennsylvania	2	2	110	198	B
12	Pennsylvania	3	2	119	198	B
13	Pennsylvania	4	2	172	293	B
14	Pennsylvania	2	2	131	293	B
15	U.S.Steel Ex.	2	2	115	288	B
16	Texas	3	2	155	1146	B
17	Texas	3	2	109	1093	B
18	Texas	2	4	110	714	C
19	Texas	2	4	192	1526	A
20	Connecticut	2	3	178	1022	---
21	Massachusetts	1	4	78	180	A
22	New Hampshire	1	8	113	413	---
23	New Hampshire	4	6	153	7617	---
24	New Hampshire	4	6	147	5713	---
25	New Jersey	4	4	75	239	A
26	New Jersey	2	6	67	173	A
27	Vermont	4	10	100	4713	C
28	Vermont	3	9	72	1407	B

A = Trussed X Bracing Diaphragm
B = Wide Flange Diaphragm
C = Channel Diaphragm

TABLE 2 DIMENSIONLESS PARAMETERS DESCRIBING EXISTING HORIZONTALLY CURVED, PLATE GIRDER BRIDGES

(From Survey by FHWA)

Bridge Number	$\frac{b_f}{R}$	$\frac{b_f}{t_f}$ max.	$\frac{D_w}{t_w}$	$\frac{d}{D_w}$
1	.0120	24.0	160	1.00
2	.0060	15.8	160	1.00
3	.0026	24.0	160	1.00
4	.0044	22.4	156	1.00
5	.0082	24.0	160	0.90
6	.0142	24.0	139	0.90
7	.0114	24.0	145	0.80
8	.0030	17.4	160	1.30
9	.0032	12.0	138	1.20
10	.0016	18.0	205	0.52
11	.0100	19.2	120	0.53
12	.0100	24.0	154	1.60
13	.0084	18.4	256	0.47
14	.0060	24.0	154	0.62
15	.0074	13.2	128	0.85
16	.0024	19.2	192	0.90
17	.0018	12.0	192	0.64
18	.0018	10.6	128	0.95
19	.0012	16.0	128	0.72
20	.0020	12.0	284	0.38
21	.0080	13.0	136	0.41
22	.0040	8.0	144	1.00
23	.0002	20.4	160	0.86
24	.0002	20.4	160	0.86
25	.0040	15.0	56	--
26	.0058	7.0	44	--
27	.0002	10.2	47	0.67
28	.0008	15.0	56	0.53
Average	.0050	17.2	147	0.83

b_f = flange width

R = horizontal radius of curvature

t_f = flange thickness

D_w = web depth

t_w = web thickness

d = transverse stiffener spacing

TABLE 3 SUMMARY OF WELDED DETAILS FOR PLATE GIRDER TEST ASSEMBLIES

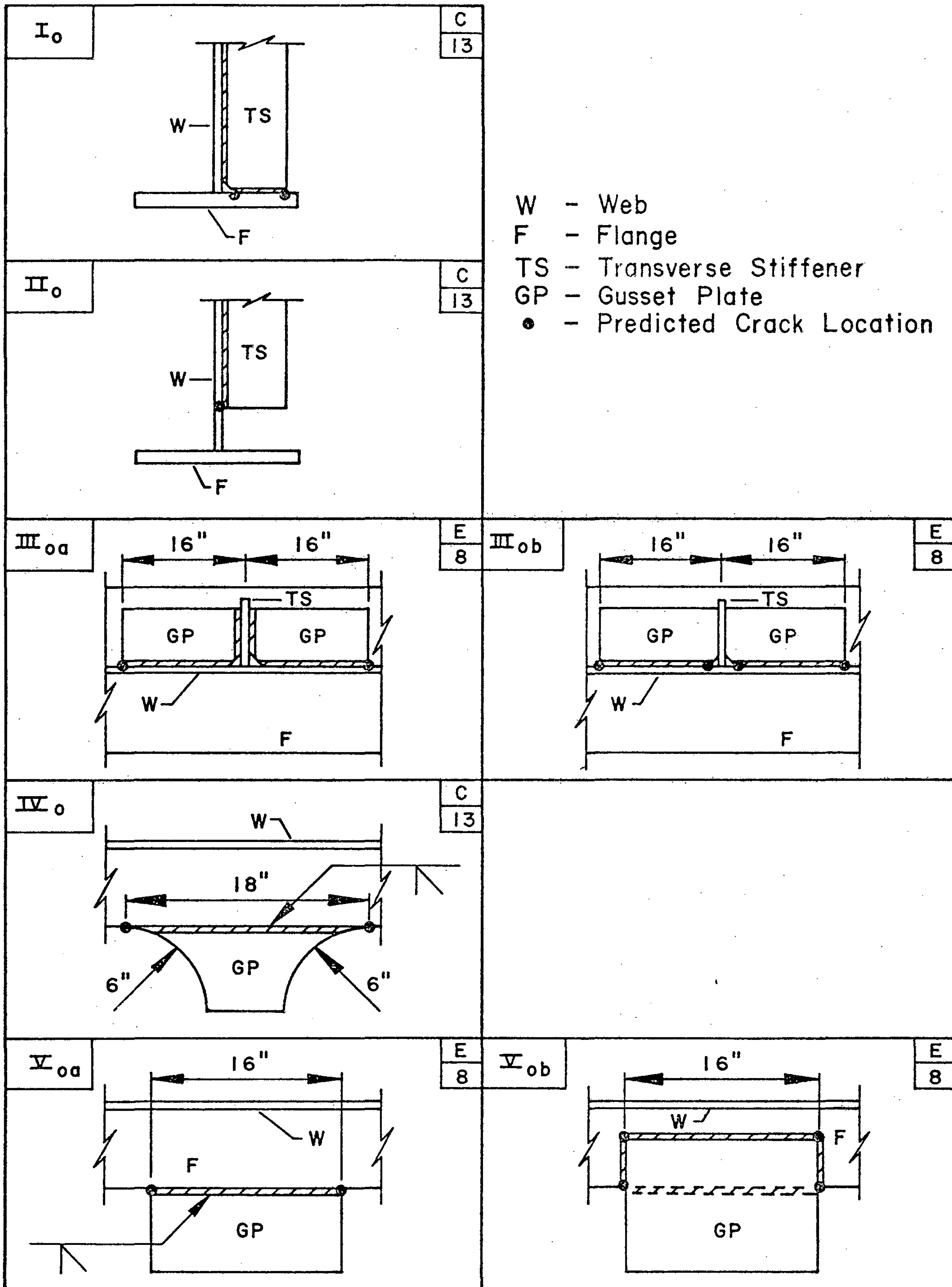


TABLE 4 COMPUTED STRESS RANGES AT GROUP 1 DETAILS - PLATE GIRDER TEST ASSEMBLIES

Assembly No.	Girder	Joint 3 Flange	Joint 3 Web	Joint 5 Flange	Joint 5 Web	Joint 4 Flange	Joint 4 Web	Joint 6 Flange	Joint 6 Web
1	2					6.3 ksi	3.6	7.4	4.8
						1.0		2.4	
	1	16.4	10.5	15.1	9.4				
		5.0		4.0					
2	2					23.0	14.5	27.0	16.2
						6.4		5.8	
	1	12.8	7.0	10.4	4.7				
		1.3		-0.9					
3	2					22.3	13.7	26.2	15.3
						5.5		4.9	
	1	12.9	7.0	10.4	4.7				
		1.2		-1.0					
4&5	2					16.0	11.0	18.5	12.3
						6.5		6.6	
	1	15.1	8.2	12.3	5.5				
		1.3		-1.2					

TABLE 5 - SUMMARY OF WELDED TEST DETAILS
FOR PLATE GIRDER ASSEMBLIES

Detail Type/ Subtype	Assembly Number	Girder Number	Number of Group 1 Details	Number of Group 2 Details	Total Number Provided
I_o	1	1 2	3 -	- -	12
	2	1 2	- -	- 2	
	3	1 2	- -	- 4	
	4	1 2	- -	- -	
	5	1 2	3 -	- -	
II_o	1	1 2	- -	- -	12
	2	1 2	- 3	- 4	
	3	1 2	- 3	- 2	
	4	1 2	- -	- -	
	5	1 2	- -	- -	
III_{oa}	1	1 2	- -	- -	3
	2	1 2	- -	- -	
	3	1 2	- -	- -	
	4	1 2	- 3	- -	
	5	1 2	- -	- -	
III_{ob}	1	1 2	- -	2 -	9
	2	1 2	- -	- 2	
	3	1 2	- -	- 2	
	4	1 2	- -	- -	
	5	1 2	- 3	- -	

TABLE 5 - SUMMARY OF WELDED TEST DETAILS
FOR PLATE GIRDER ASSEMBLIES
(Continued)

Detail Type/ Subtype	Assembly Number	Girder Number	Number of Group 1 Details	Number of Group 2 Details	Total Number Provided
IV_o	1	1	–	–	9
		2	–	–	
	2	1	–	–	
		2	–	–	
	3	1	–	–	
		2	–	–	
	4	1	3	–	
		2	–	4	
	5	1	–	–	
		2	–	2	
V_{oa}	1	1	–	2	15
		2	–	–	
	2	1	3	2	
		2	–	–	
	3	1	–	–	
		2	–	2	
	4	1	–	2	
		2	–	–	
	5	1	–	2	
		2	–	2	
V_{ob}	1	1	–	4	15
		2	–	–	
	2	1	–	–	
		2	–	2	
	3	1	3	2	
		2	–	–	
	4	1	–	2	
		2	–	–	
	5	1	–	2	
		2	–	–	

TABLE 6 WEB SLENDERNESS AND STIFFENER SPACING FOR PLATE GIRDER TEST ASSEMBLIES

		WEB SLENDERNESS		STIFFENER SPACING			
				INTERIOR PANELS		END PANELS	
ASSEMBLY NO.	GIRDER	$\frac{D_w}{t_w}$ actual	$\frac{D_w}{t_w}$ allow.	d actual	d allow.	d actual	d allow.
1	1	144	134	118 in.	54	47 / 71	54
	2	192	170	123	54	123	54
2	1	155	170	118	58	118	58
	2	186	107	123	58	61	53
3	1	155	170	118	58	118	58
	2	155	110	123	58	61	58
4	1	139	161	118	52	118	52
	2	139	123	123	52	61	52
5	1	139	161	118	52	118	52
	2	139	123	123	52	61	52

D_w = web depth

t_w = web thickness

d = stiffener spacing

TABLE 7 OVERALL CHARACTERISTICS OF EXISTING HORIZONTALLY CURVED, BOX GIRDER BRIDGES

BRIDGE NUMBER	LOCATION	NO. SPANS	GIRDERS PER SPAN	MAXIMUM SPAN LENGTH	MINIMUM RADIUS OF CURVATURE
1	Germany	NA	NA	144 ft.	NA
2	New York	2	1	111	67 ft+
3	Japan	NA	NA	131	NA
4	Japan	3	1	107	NA
5	Pennsylvania	4	2	160	2000
6	Pennsylvania	2-4	1-4	154	650

\+ Roadway ramp between levels of the Port Authority Bus Terminal, New York City.

TABLE 8 DIMENSIONLESS PARAMETERS DESCRIBING EXISTING HORIZONTALLY CURVED, BOX GIRDER BRIDGES

BRIDGE NUMBER	$\frac{L}{D_w}$	$\frac{L}{R}$	$\frac{b_f}{t_f}$	$\frac{b_f}{D_w}$	$\frac{D_w}{t_w}$
5	20.0	0.080	384	1.50	192
6	30.8	0.237	320	2.00	160

L = span length

D_w = web depth

R = horizontal radius of curvature

b_f = flange width

t_f = flange thickness

t_w = web thickness

TABLE 9 SUMMARY OF WELDED DETAILS FOR BOX GIRDER TEST ASSEMBLIES

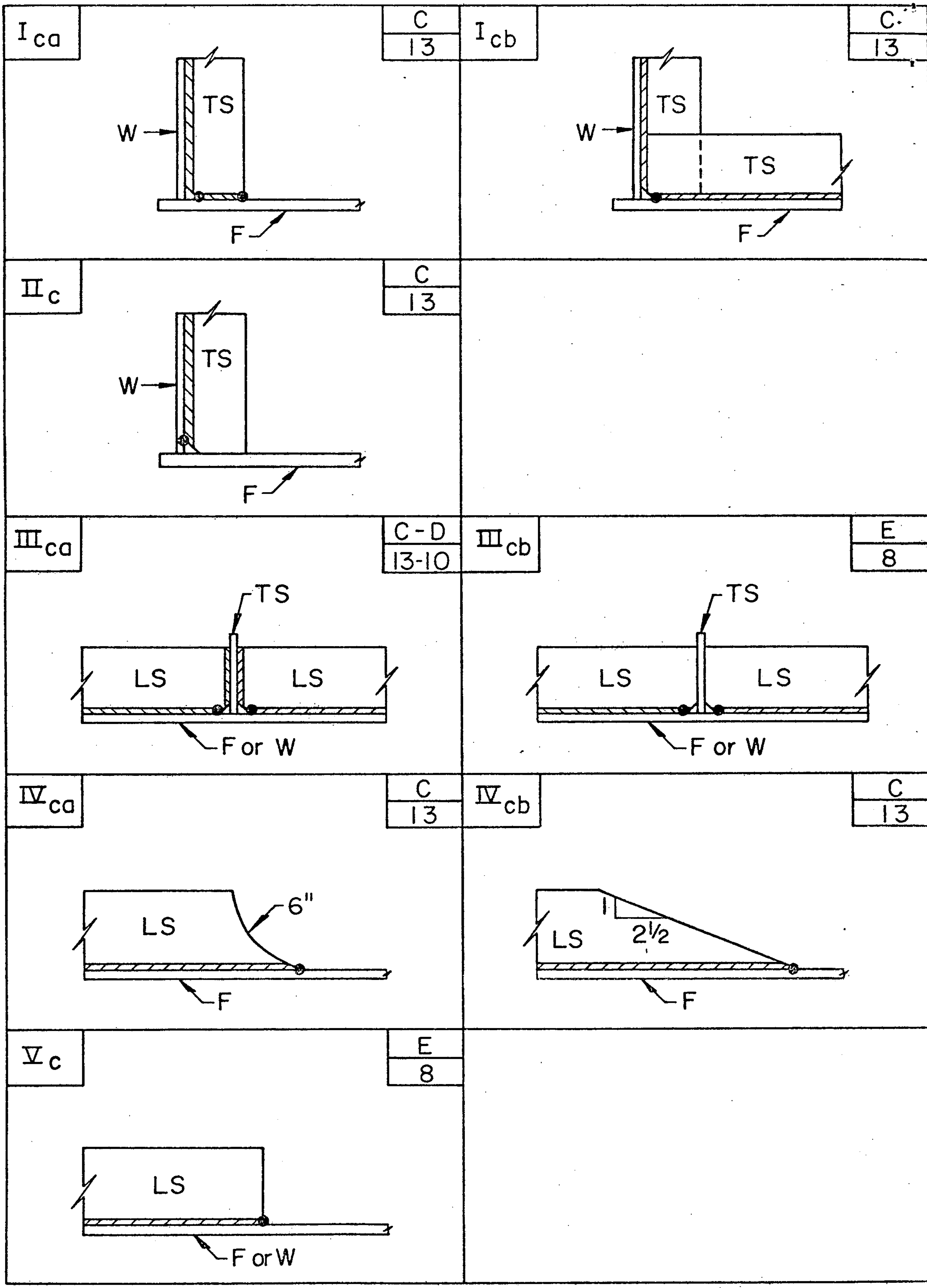
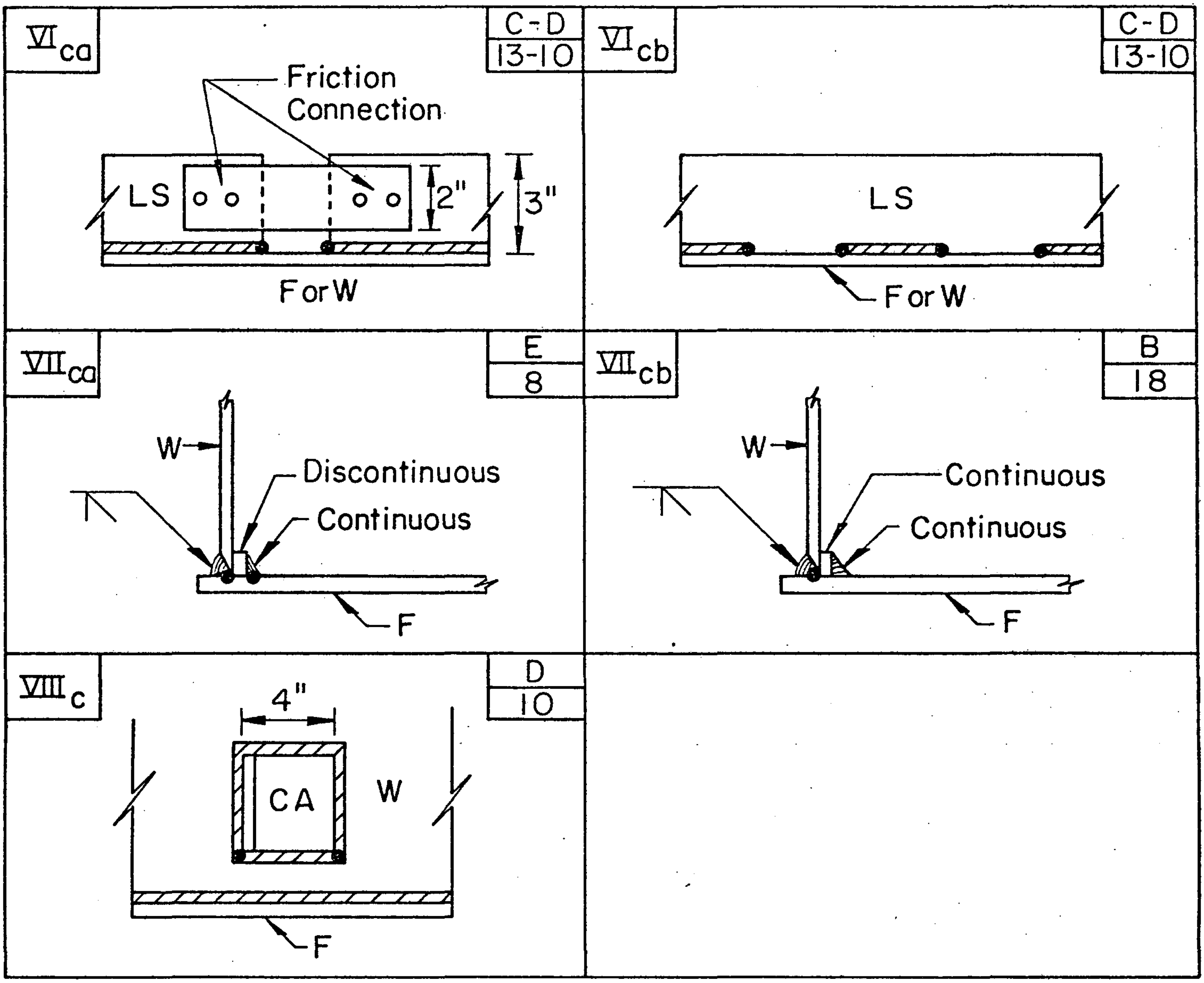


TABLE 9 SUMMARY OF WELDED DETAILS FOR BOX GIRDER TEST ASSEMBLIES (continued)



W - Web

F - Flange

TS - Transverse Stiffener

LS - Longitudinal Stiffener

GP - Gusset Plate

CA - Clip Angle

● Predicted Crack Location

TABLE 10 - COMPUTED STRESS RANGES AT INTERIOR DIAPHRAGMS - BOX GIRDER ASSEMBLIES

	Joint 3			Joint 5			Joint 4			Joint 6		
Assembly No.	Flange	Flange-Web Inter-Section	Web	Flange	Flange-Web Inter-Section	Web	Flange	Flange-Web Inter-Section	Web	Flange	Flange-Web Inter-Section	Web
1	18.3 ksi	21.0 ksi	16.7 ksi	8.3 ksi	6.8 ksi	5.4 ksi	9.5 ksi	8.5 ksi	6.8 ksi	20.1 ksi	22.6 ksi	18.0 ksi
2	18.2	20.6	16.6	8.8	7.3	5.8	9.7	8.8	7.0	19.7	22.2	17.7
3	16.9	18.9	15.2	7.6	5.9	4.7	10.0	9.6	7.5	19.9	22.6	18.1

TABLE 11 DETAIL REPLICATION FOR BOX GIRDER TEST ASSEMBLIES

Detail	Assembly No.	No. Provided	Total
I_{ca}	1	- (3)	2 (3)
	2	2	
	3	-	
I_{cb}	1	-	2 (1)
	2	- (1)	
	3	2	
II_{c}	1	1	6
	2	1	
	3	4	
III_{ca}	1	1	2 (4)
	2	- (2)	
	3	1 (2)	
III_{cb}	1	- (3)	2 (9)
	2	1 (3)	
	3	1 (3)	
IV_{ca}	1	2	2
	2	-	
	3	-	
IV_{cb}	1	-	4
	2	2	
	3	2	

Detail	Assembly No.	No. Provided	Total
V_{c}	1	8	18
	2	6	
	3	4	
VI_{ca}	1	4	8
	2	2	
	3	2	
VI_{cb}	1	-	4
	2	2	
	3	2	
VII_{ca}	1	-	4
	2	-	
	3	4	
VII_{cb}	1	-	4
	2	-	
	3	4	
$VIII_{c}$	1	3	9
	2	3	
	3	3	

8. FIGURES

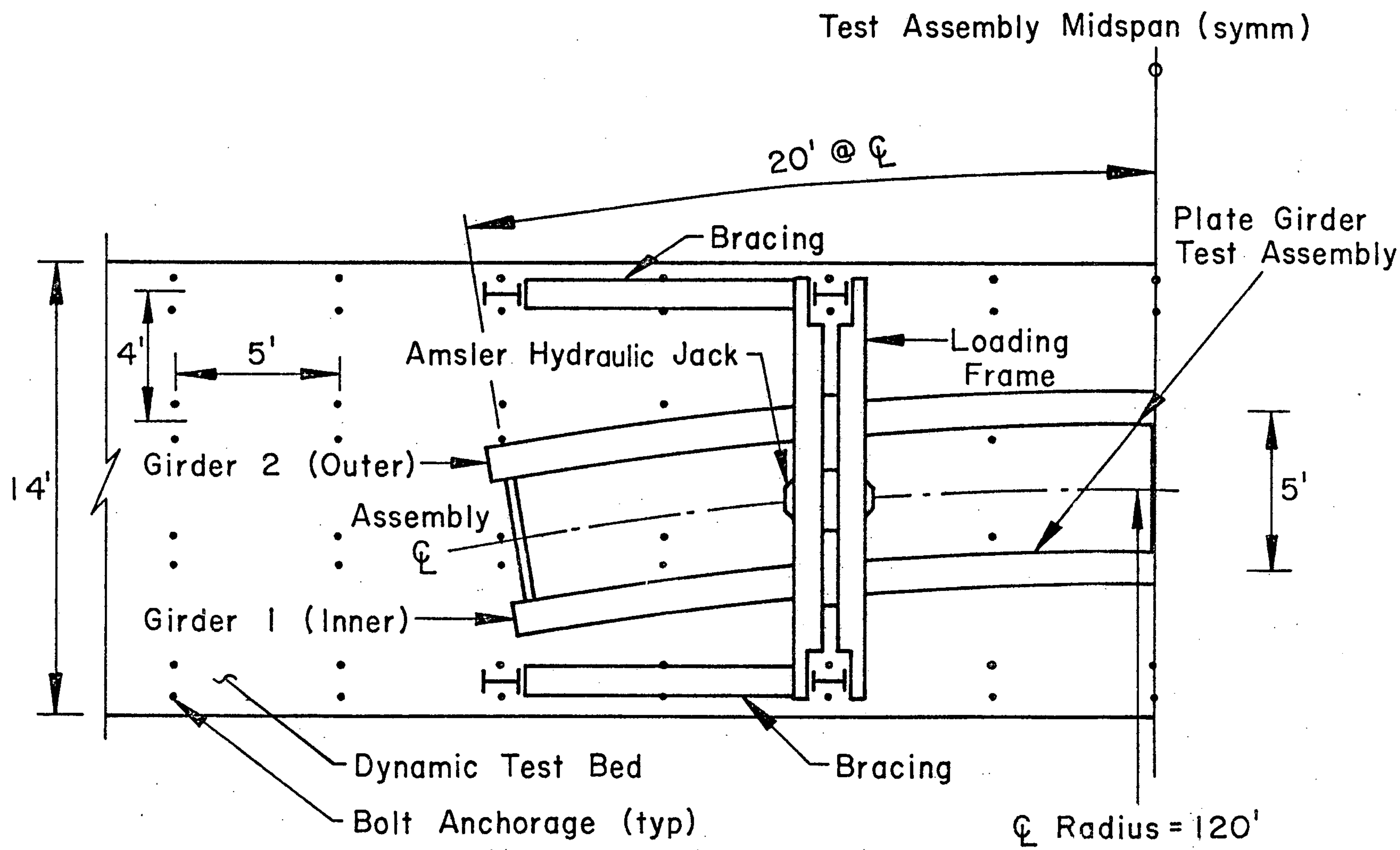


Fig. 1 Preliminary Design of Plate Girder Test Assemblies - Schematic Plan View

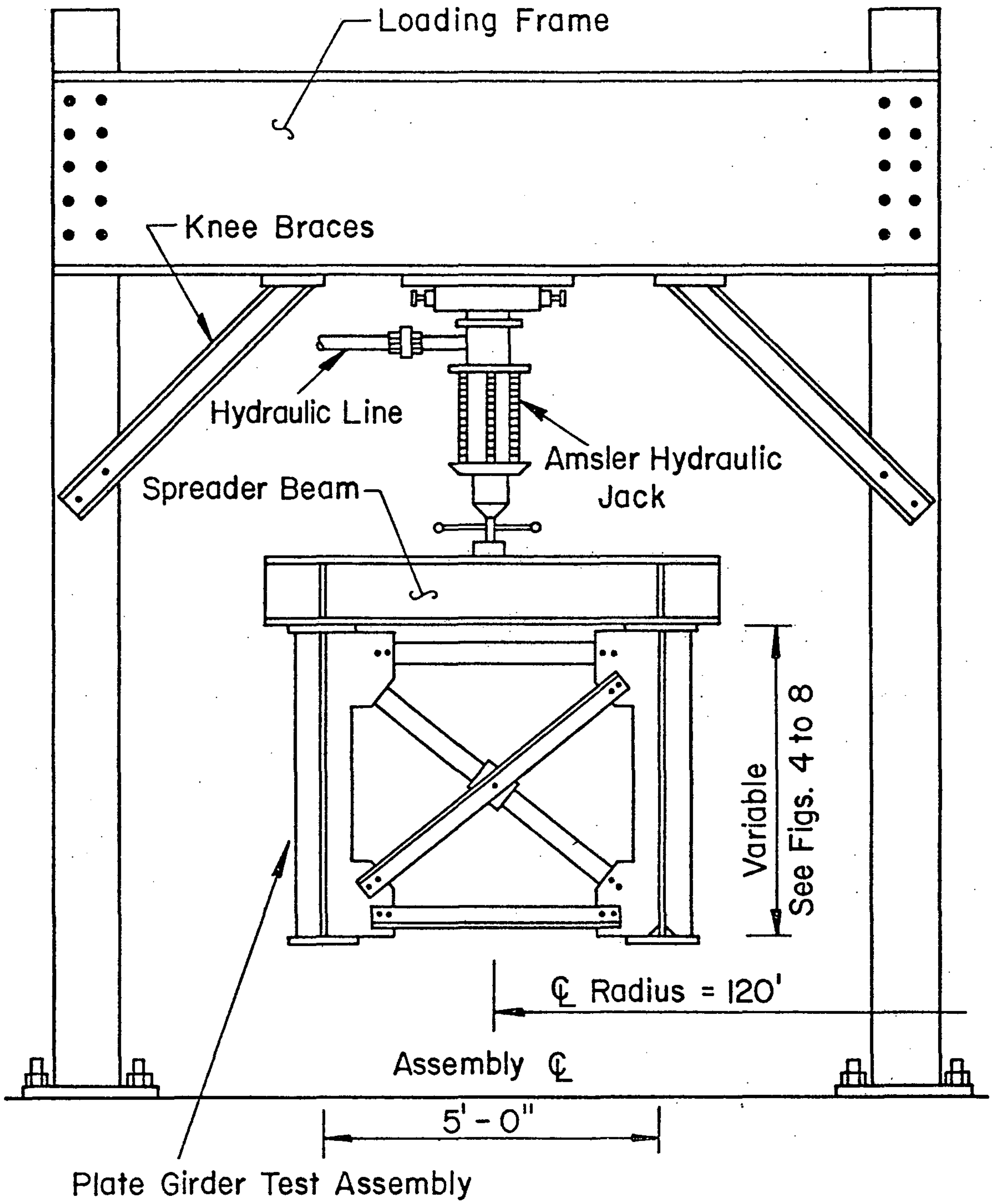
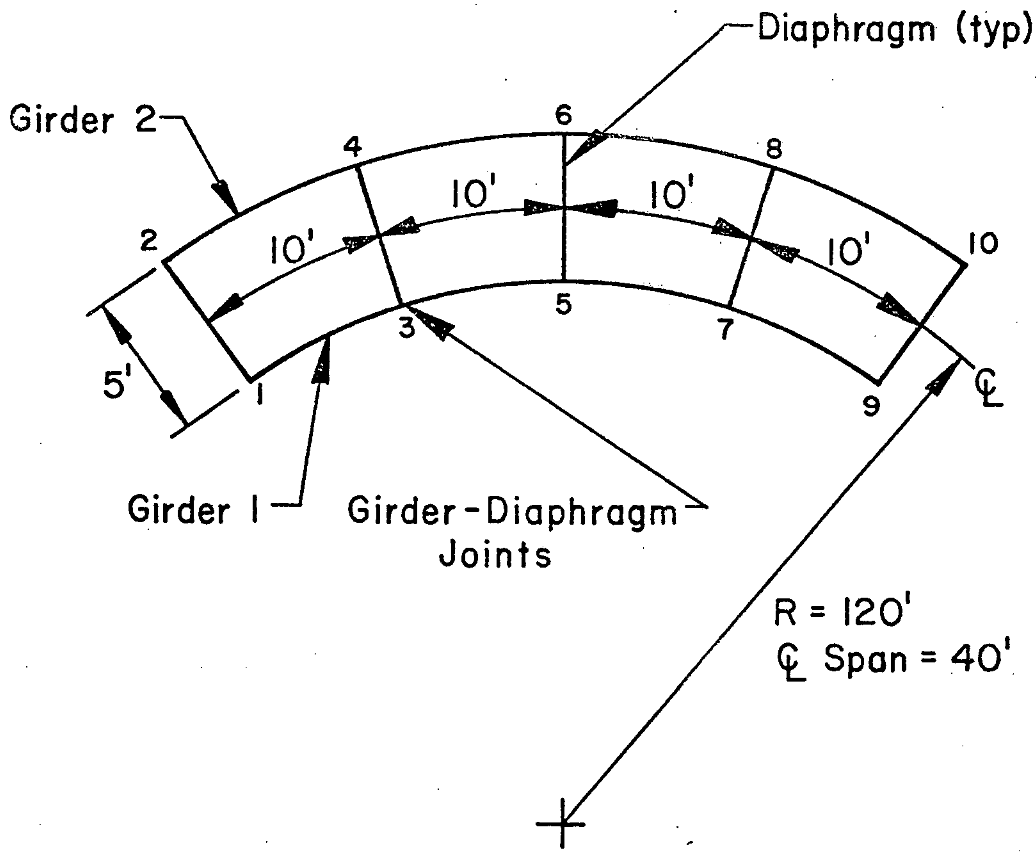


Fig. 2 Preliminary Design of Plate Girder Test Assemblies - Schematic Section at Loading Frame

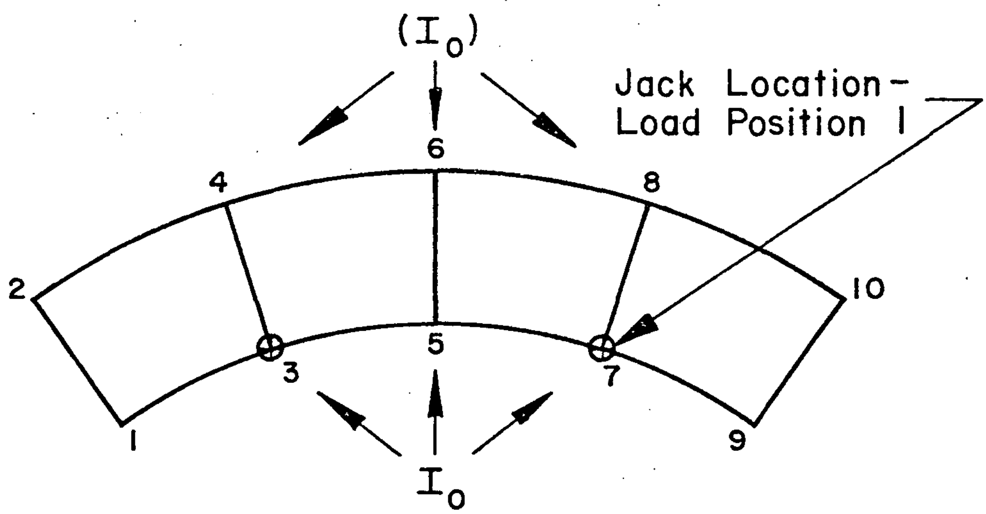


Load Position 1 - Over Joints 3 & 7

Load Position 2 - Midway Between Joints 3 & 4

Midway Between Joints 7 & 8

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

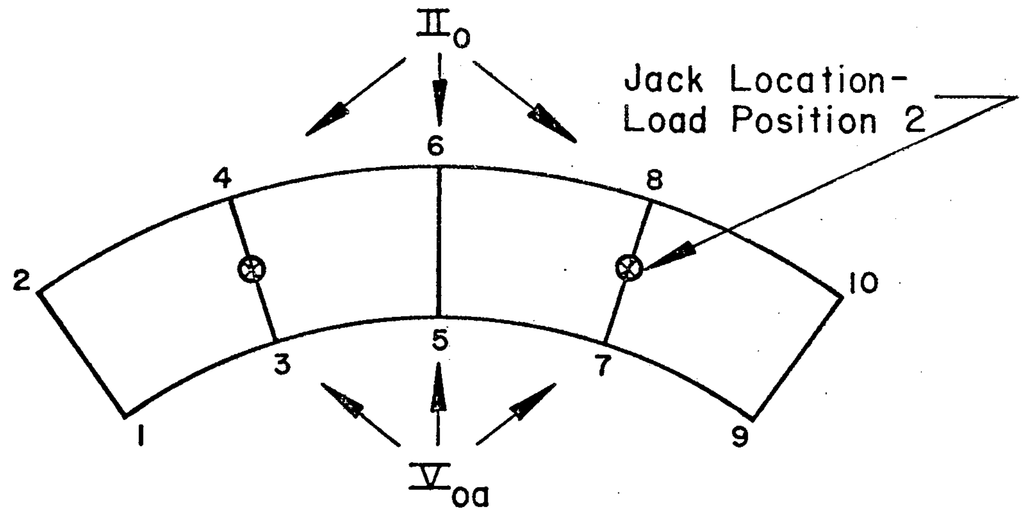


Girder	b_f	t_f	D_w	t_w	D_w/t_w
1	12"	1	54	3/8	144
2	12	1	54	9/32	192

b_f = flange width D_w = web depth

t_f = flange thickness t_w = web thickness

Fig. 4 Plate Girder Test Assembly 1 - Cross Section Dimensions and Locations of Group 1 Details on Tension Flanges

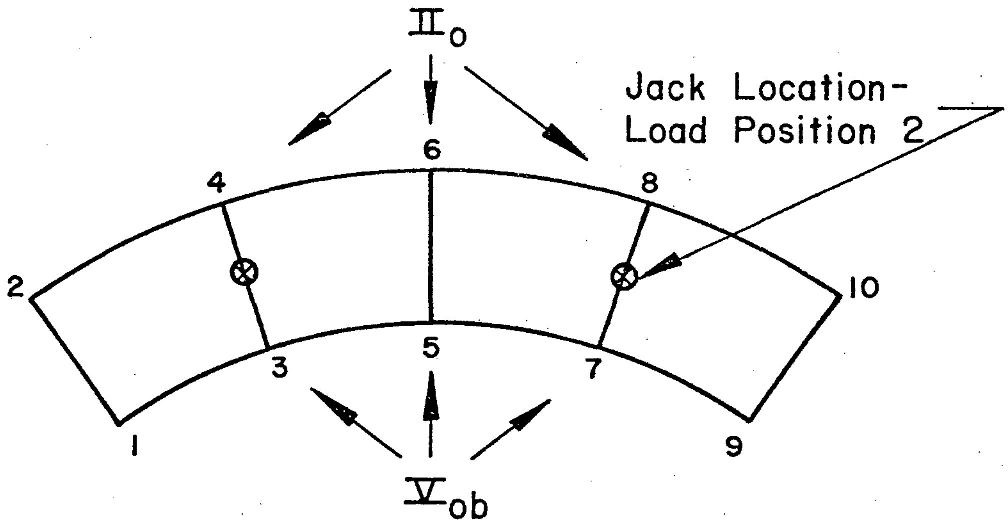


Girder	b_f	t_f	D_w	t_w	D_w/t_w
1	8"	1/2	58	3/8	155
2	10	3/4	58	5/16	186

b_f = flange width $\quad$ D_w = web depth

t_f = flange thickness $\quad$ t_w = web thickness

Fig. 5 Plate Girder Test Assembly 2 - Cross Section Dimensions and Locations of Group 1 Details on Tension Flanges

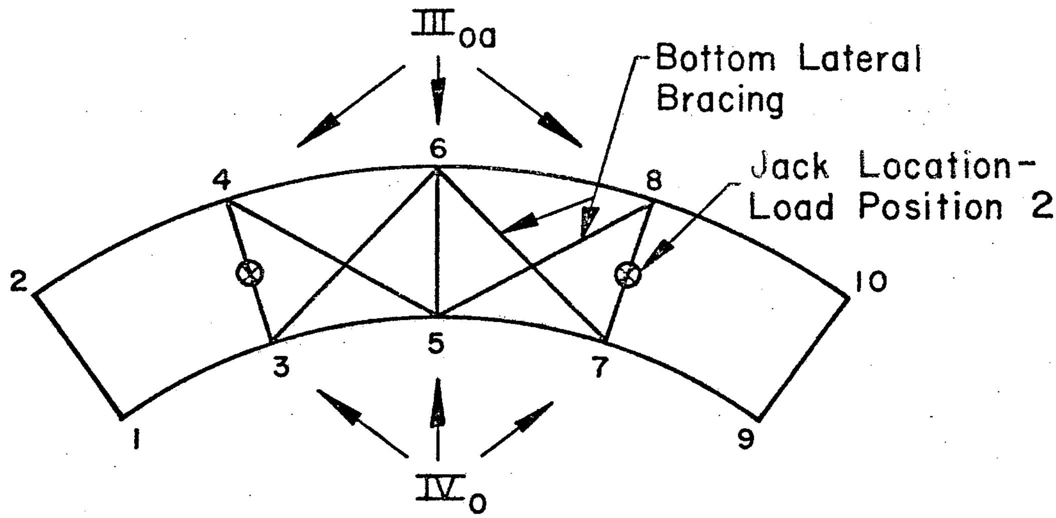


Girder	b_f	t_f	D_w	t_w	D_w/t_w
1	8"	1/2	58	3/8	155
2	10	3/4	58	3/8	155

b_f = flange width　　D_w = web depth

t_f = flange thickness　　t_w = web thickness

Fig. 6 Plate Girder Test Assembly 3 - Cross Section Dimensions and Locations of Group 1 Details on Tension Flanges

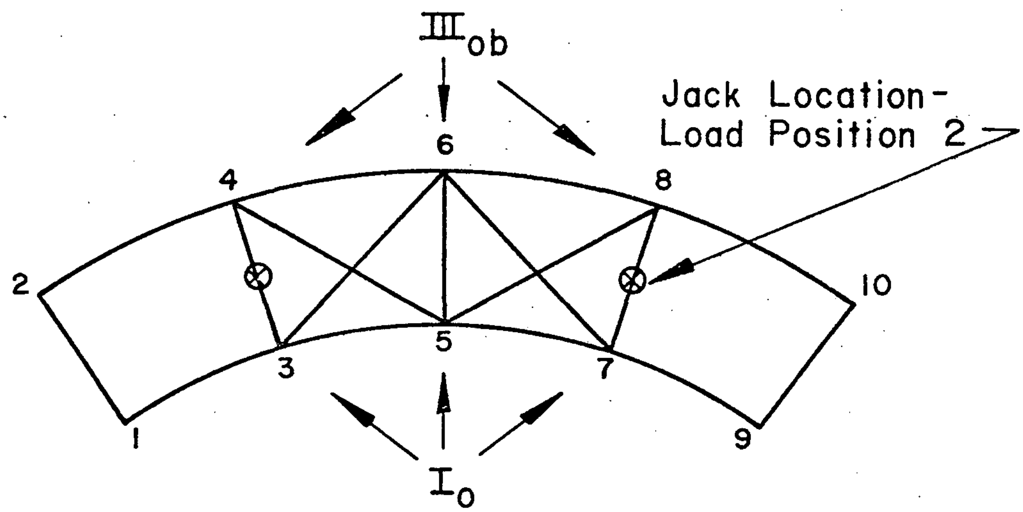


Girder	b_f	t_f	D_w	t_w	D_w/t_w
1	8"	1/2	52	3/8	139
2	12	1	52	3/8	139

b_f = flange width $\quad$ D_w = web depth

t_f = flange thickness $\quad$ t_w = web thickness

Fig. 7 Plate Girder Test Assembly 4 - Cross Section Dimensions and Locations of Group 1 Details on Tension Flanges



Girder	b_f	t_f	D_w	t_w	D_w/t_w
1	8"	1/2	52	3/8	139
2	12	1	52	3/8	139

b_f = flange width D_w = web depth

t_f = flange thickness t_w = web thickness

Fig. 8 Plate Girder Test Assembly 5 - Cross Section Dimensions and Locations of Group 1 Details on Tension Flanges

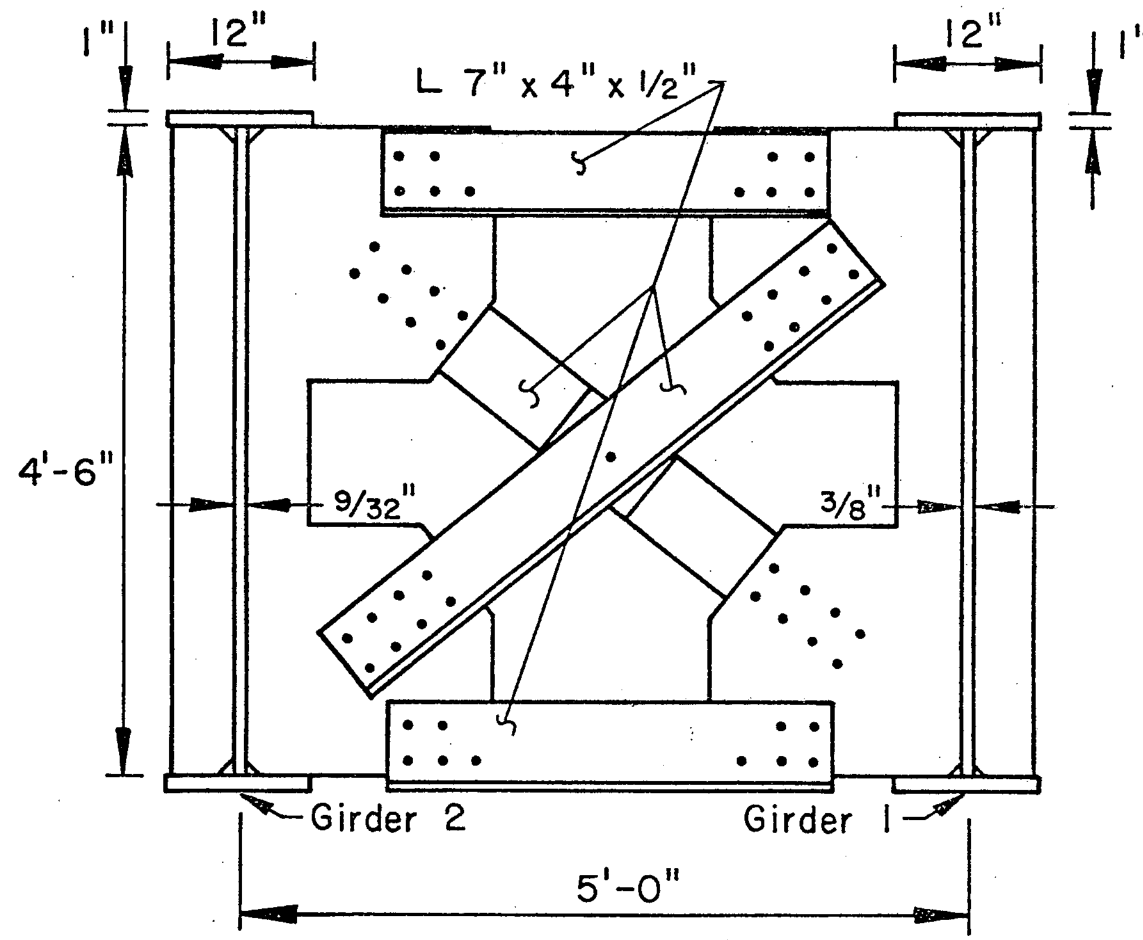


Fig. 9 Plate Girder Test Assembly 1 - Cross Section at Interior Diaphragms

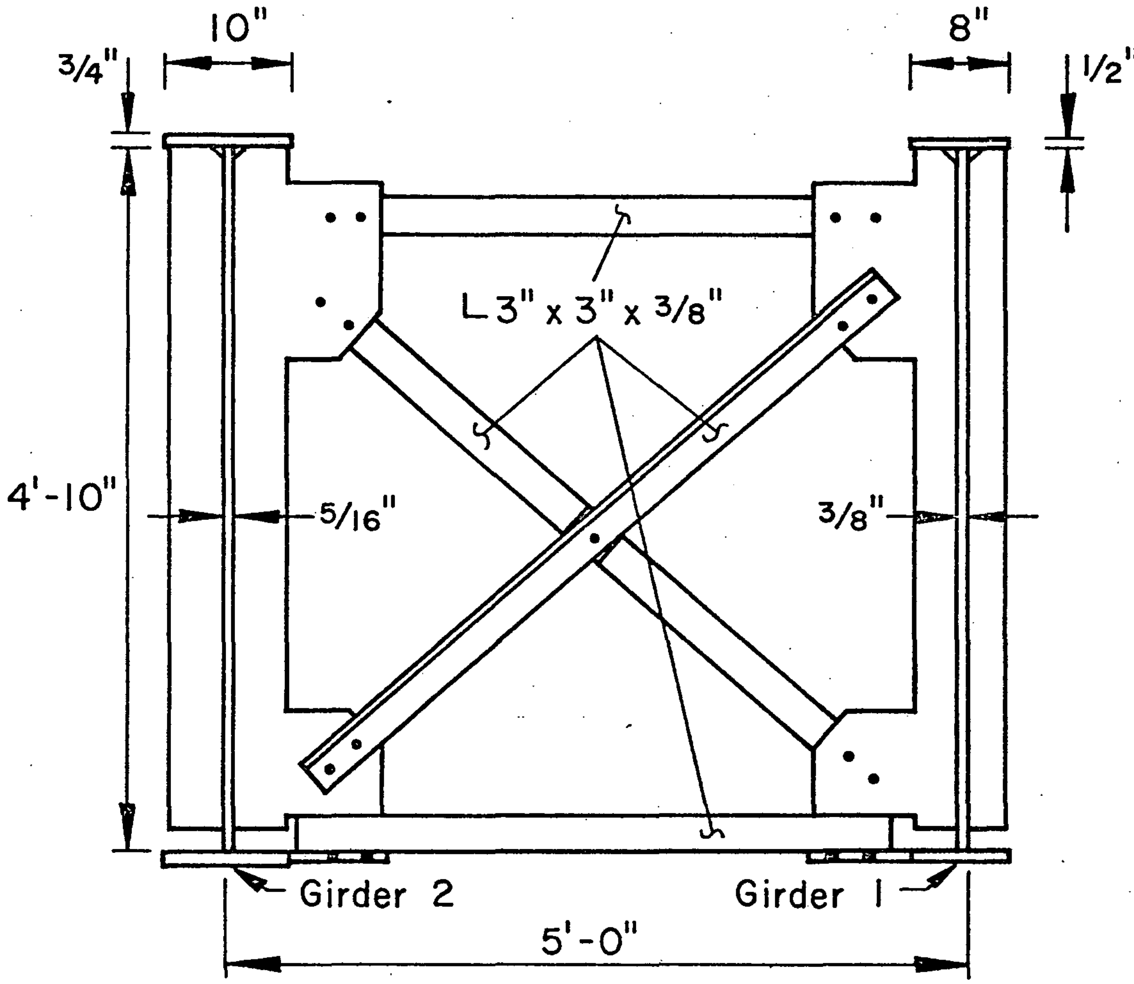


Fig. 10 Plate Girder Test Assembly 2 - Cross Section at Interior Diaphragms

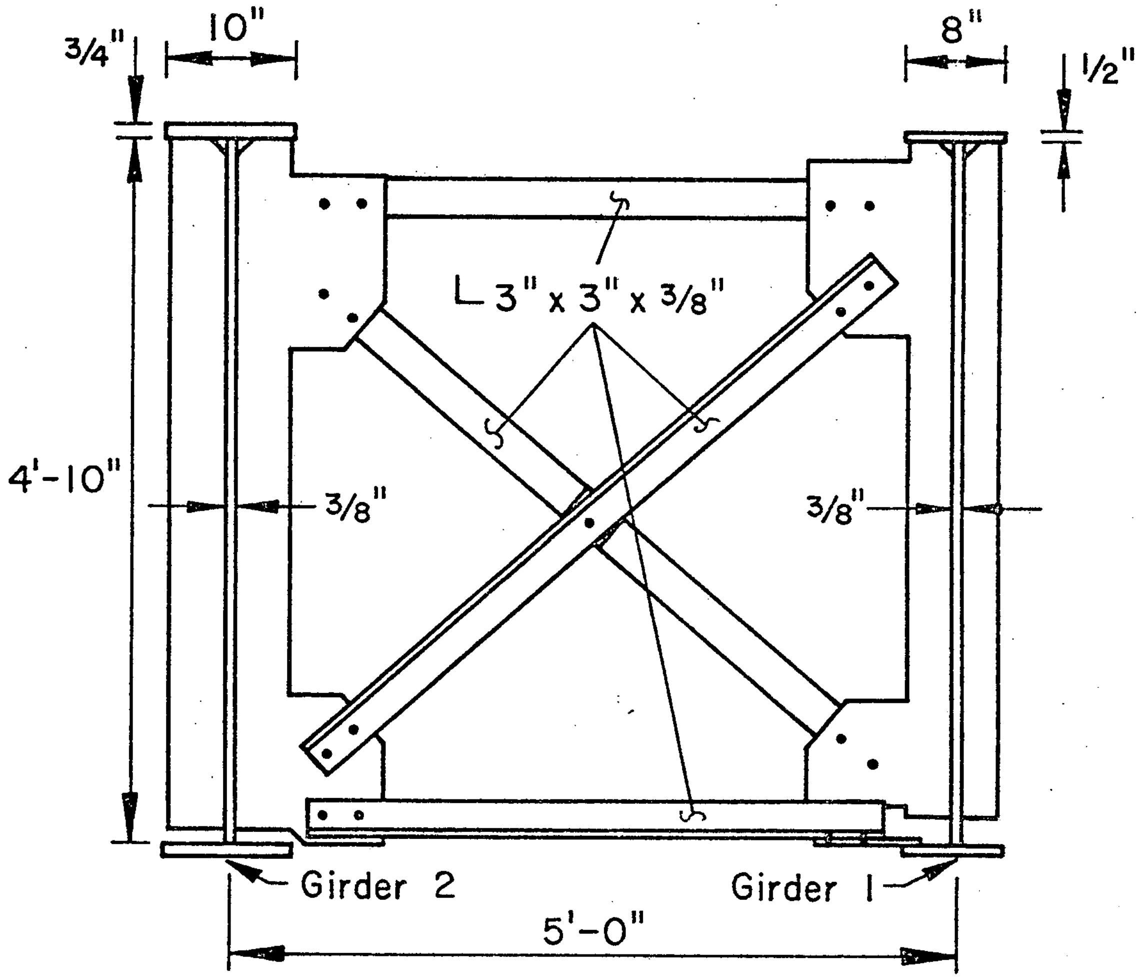


Fig. 11 Plate Girder Test Assembly 3 - Cross Section at Interior Diaphragms

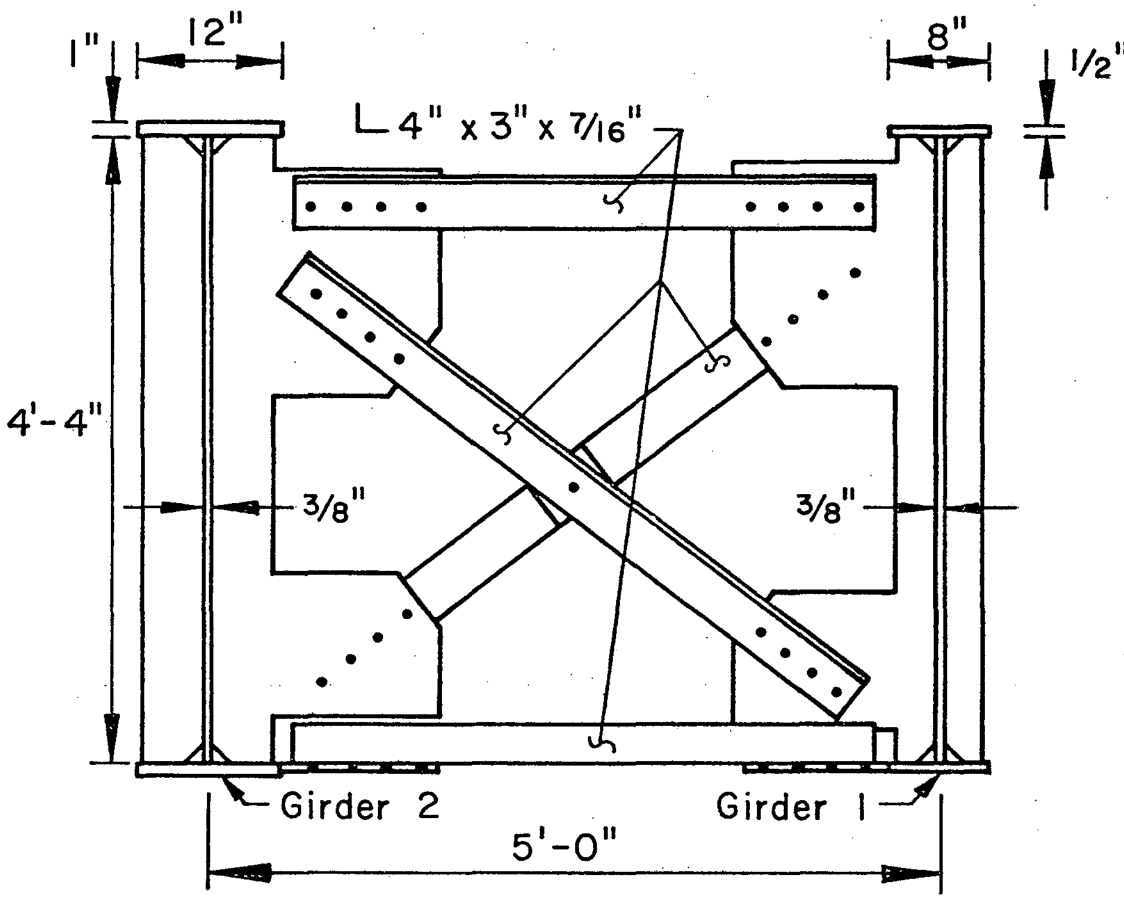


Fig. 12 Plate Girder Test Assembly 4 - Cross Section at Interior Diaphragms

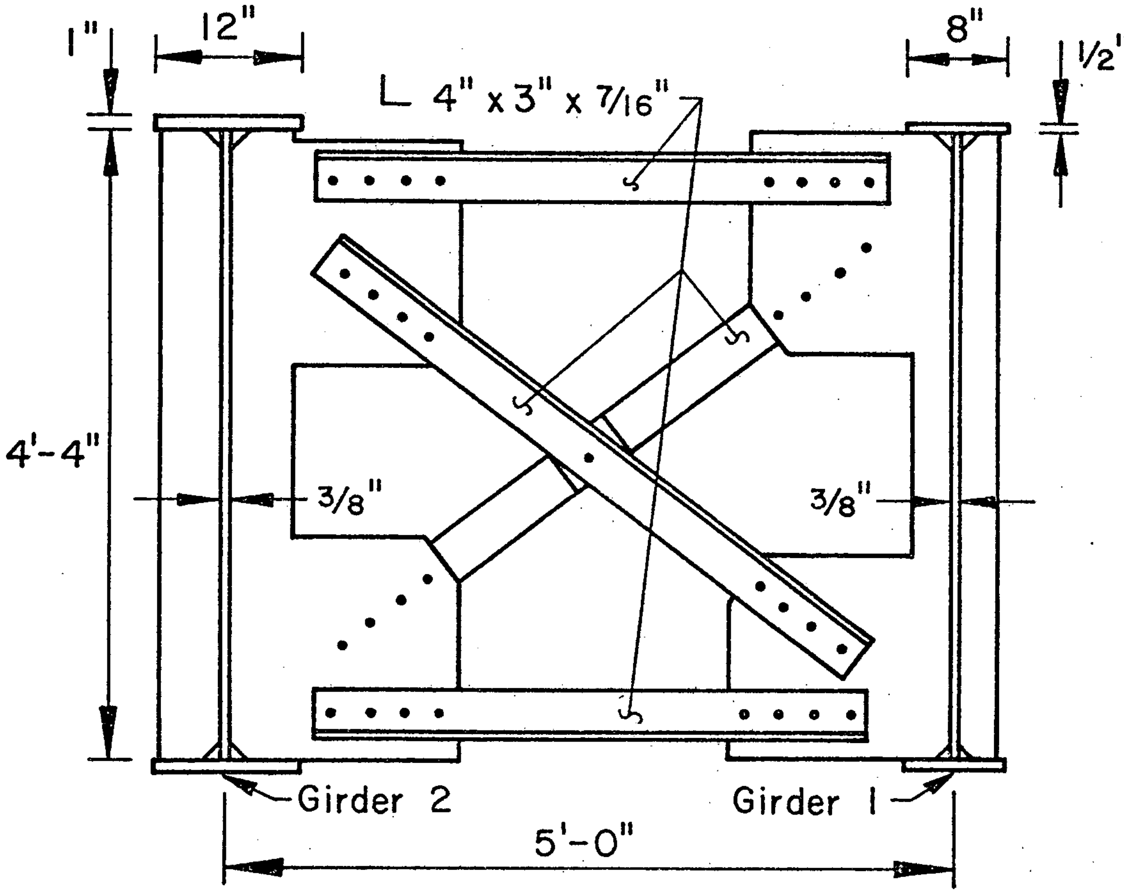


Fig. 13 Plate Girder Test Assembly 5 - Cross Section at Interior Diaphragms

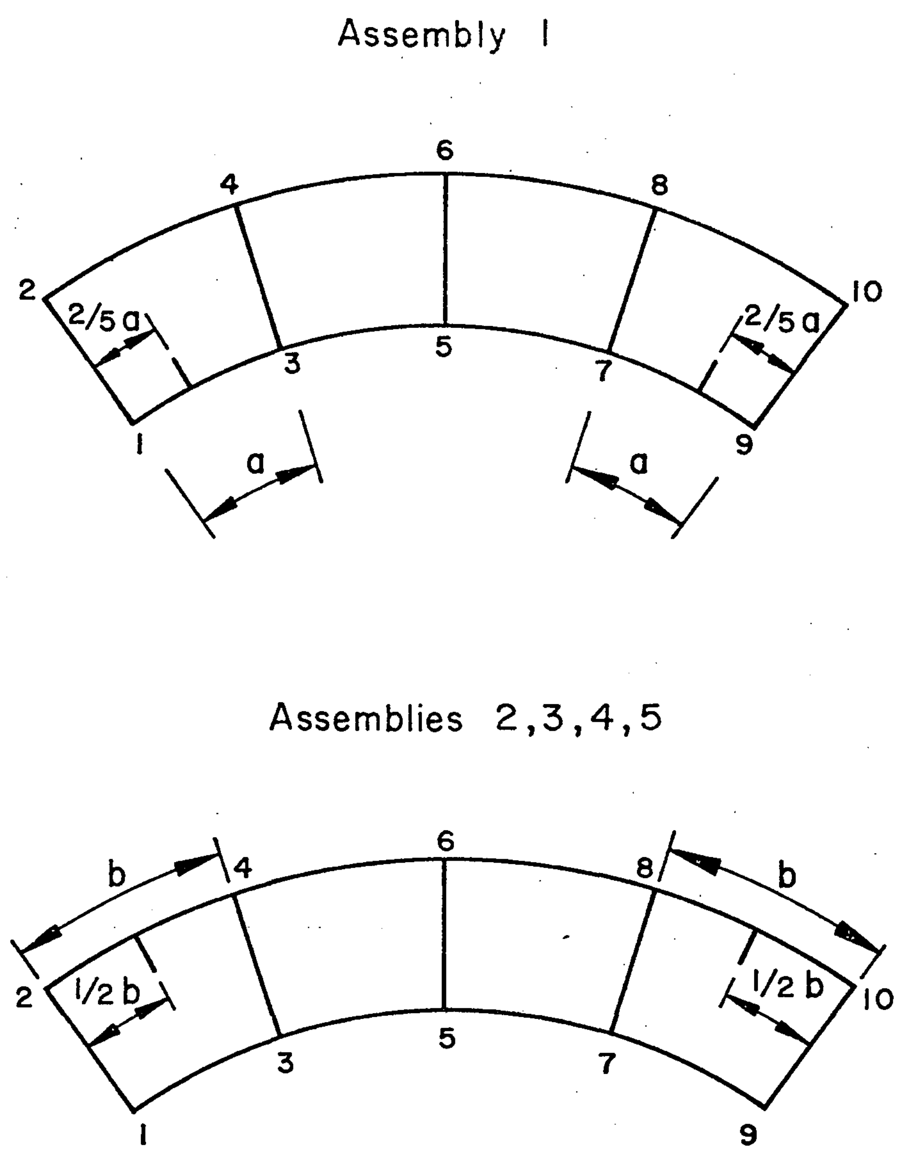


Fig. 14 Additional Transverse Stiffeners Required in Plate Girder Test Assemblies

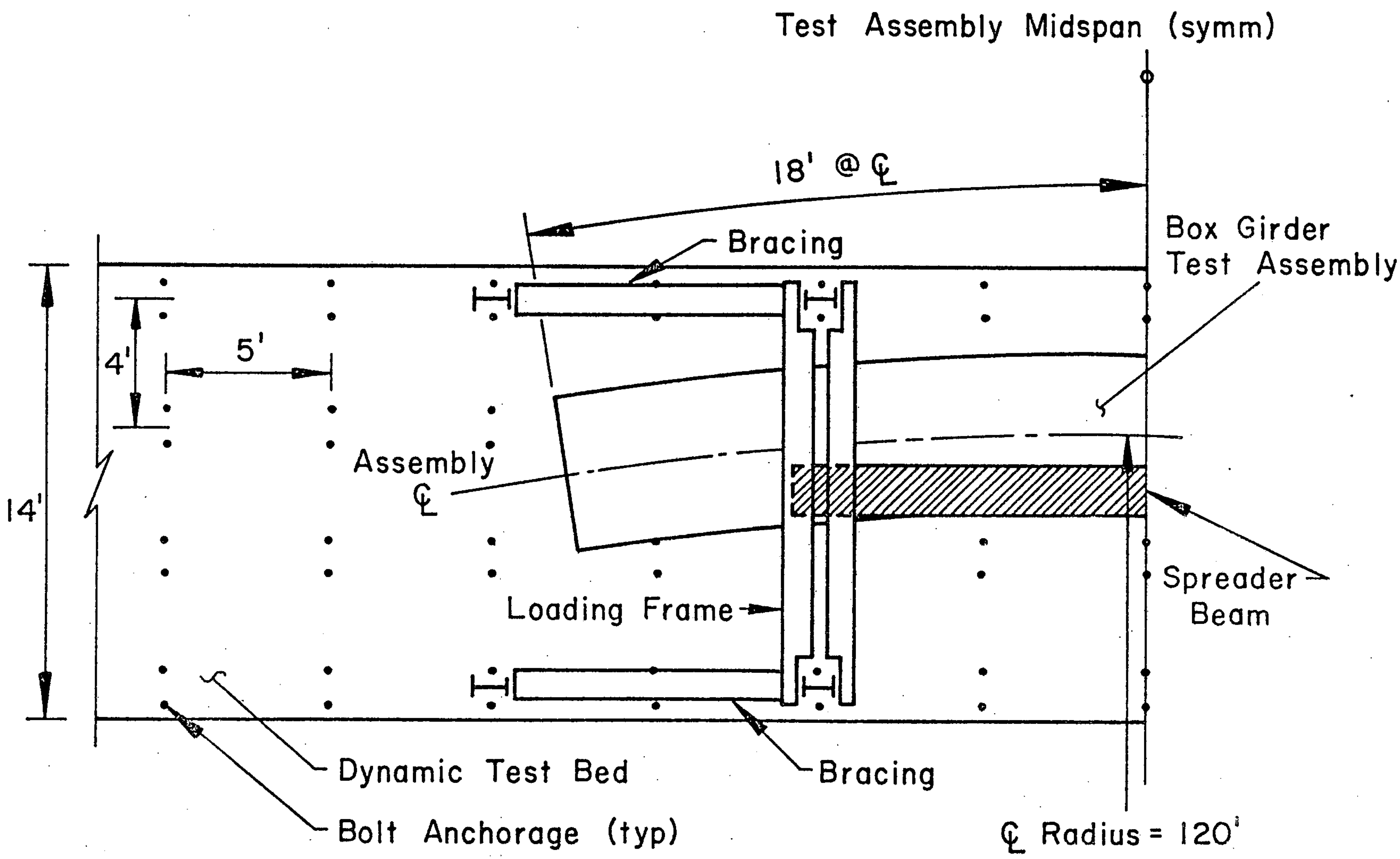


Fig. 15 Preliminary Design of Box Girder Test Assemblies - Schematic Plan View

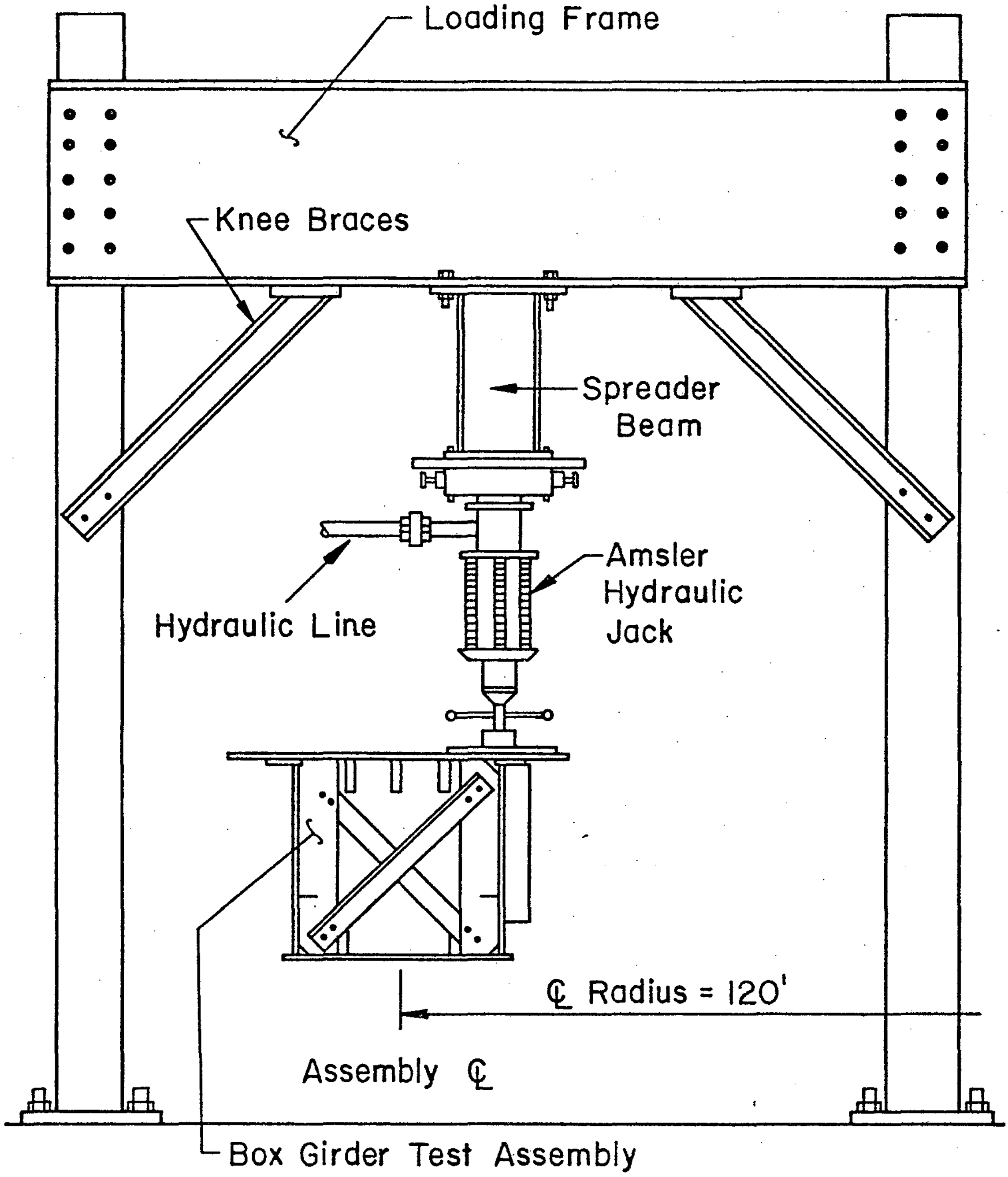


Fig. 16 Preliminary Design of Box Girder Test Assemblies - Schematic Section at Loading Frame

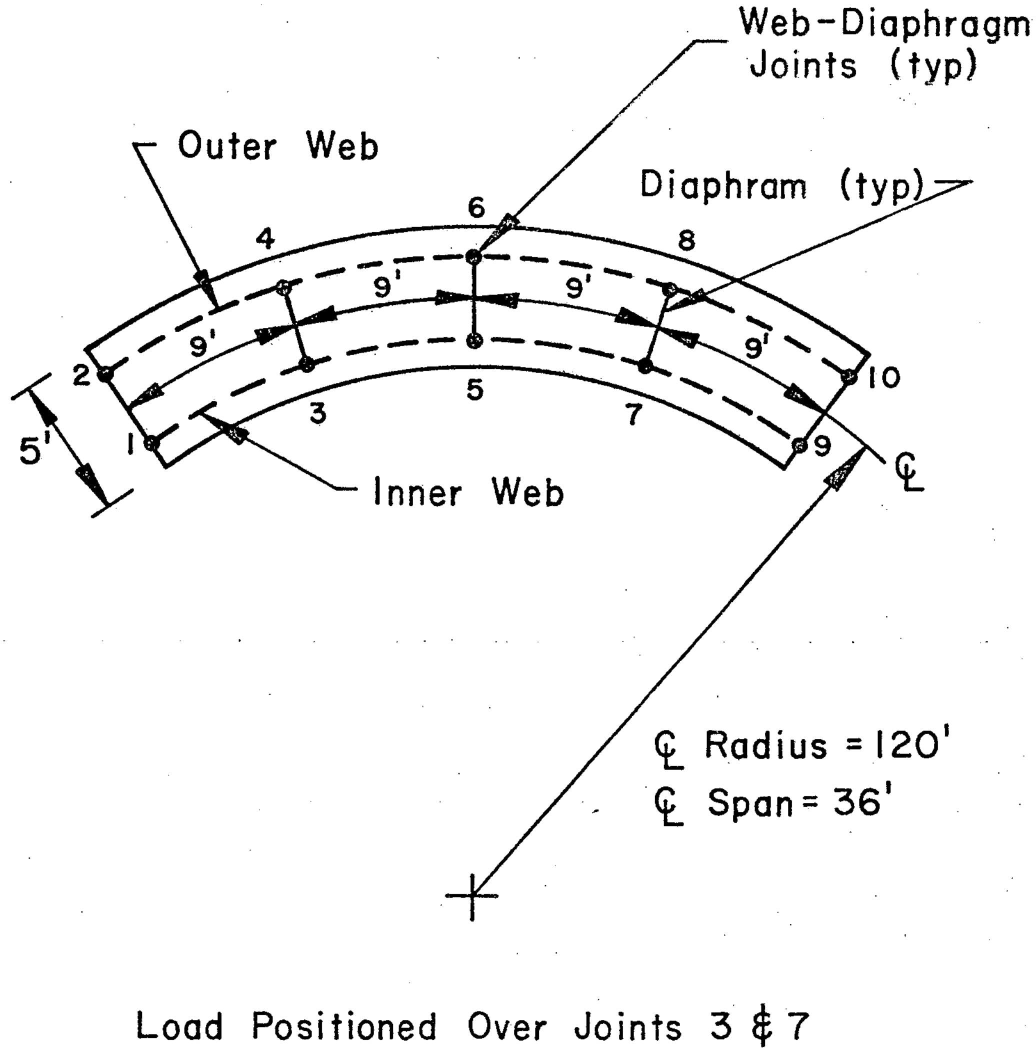


Fig. 17 Schematic Plan View of Typical Box Girder Test Assembly

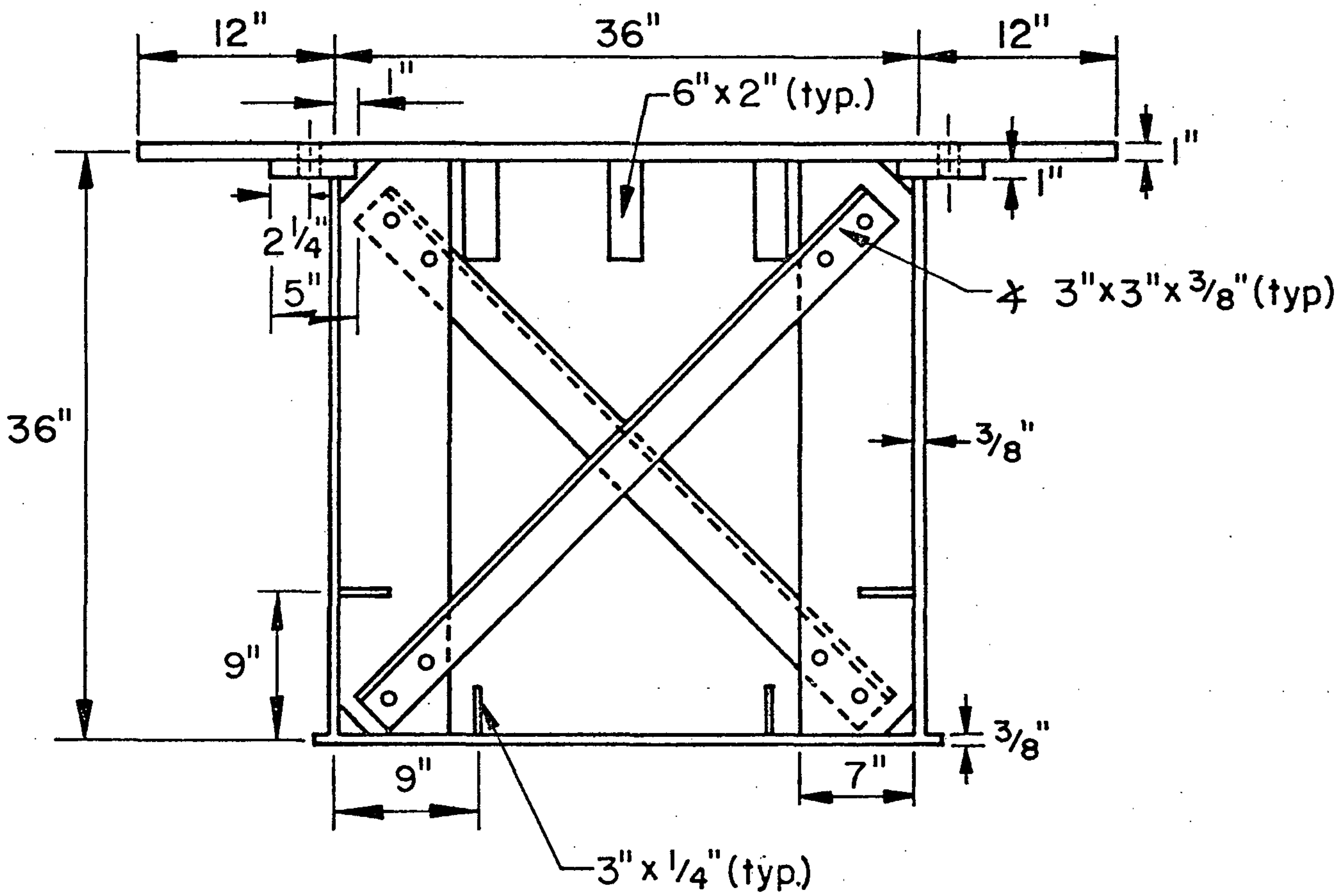


Fig. 18 Box Girder Test Assembly 1 -
Cross Section at Interior Diaphragms

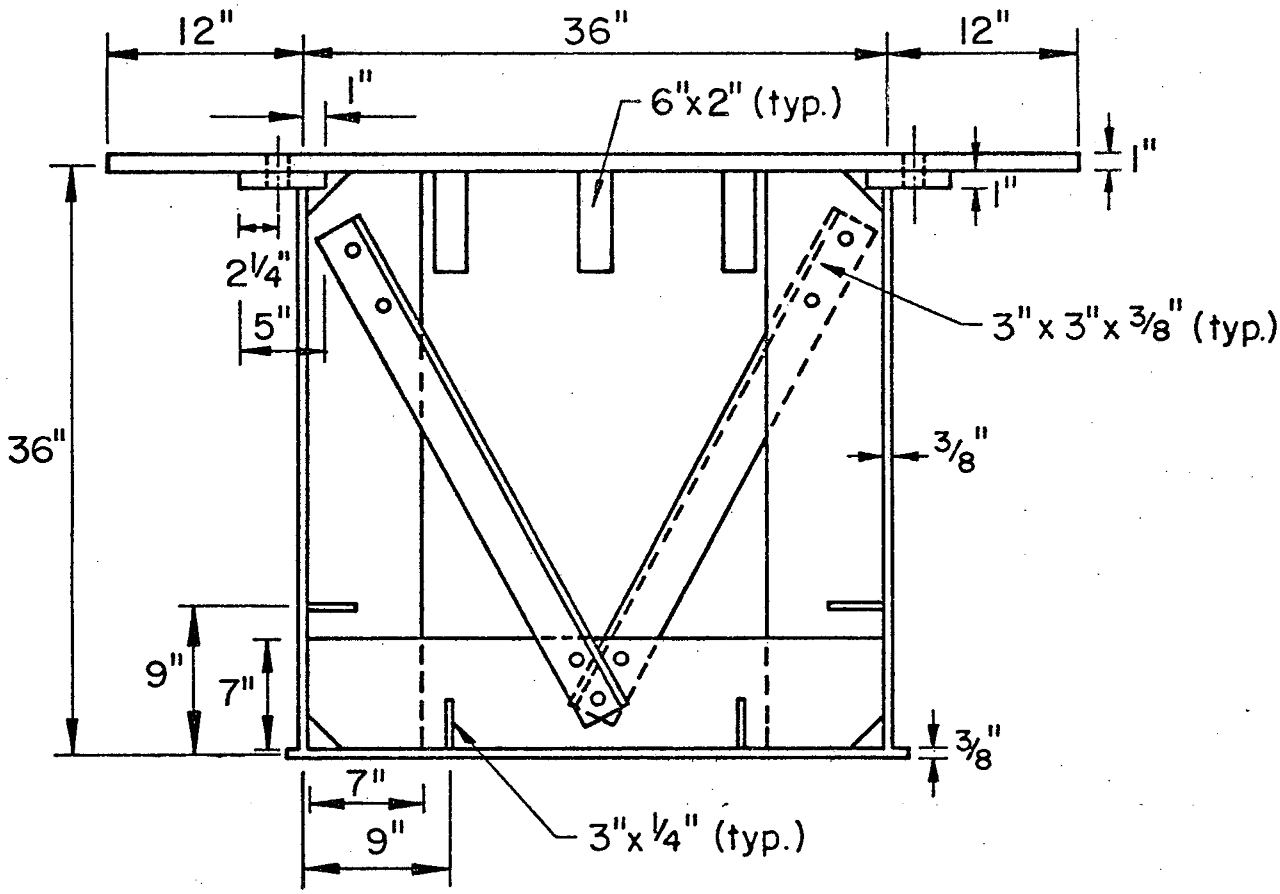


Fig. 19 Box Girder Test Assembly 2 -
Cross Section at Interior Diaphragms

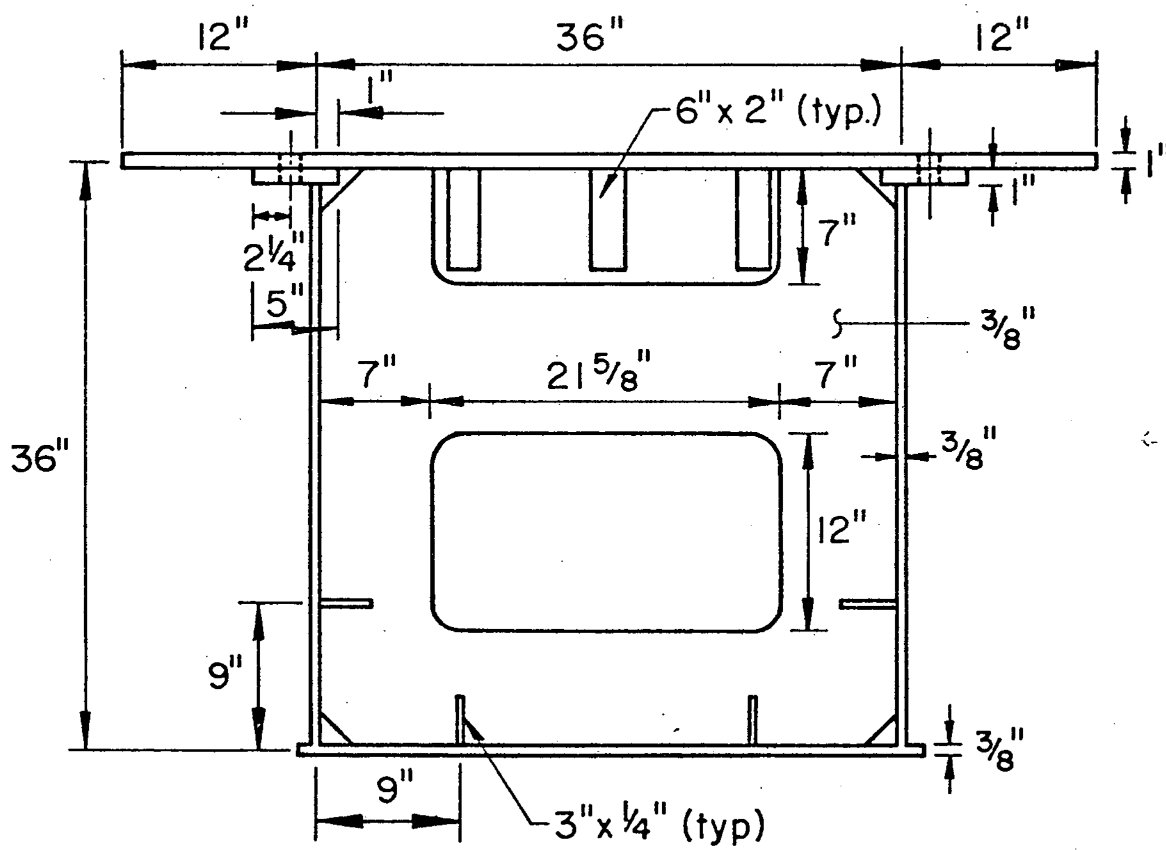


Fig. 20 Box Girder Test Assembly 3 -
Cross Section at Interior Diaphragms

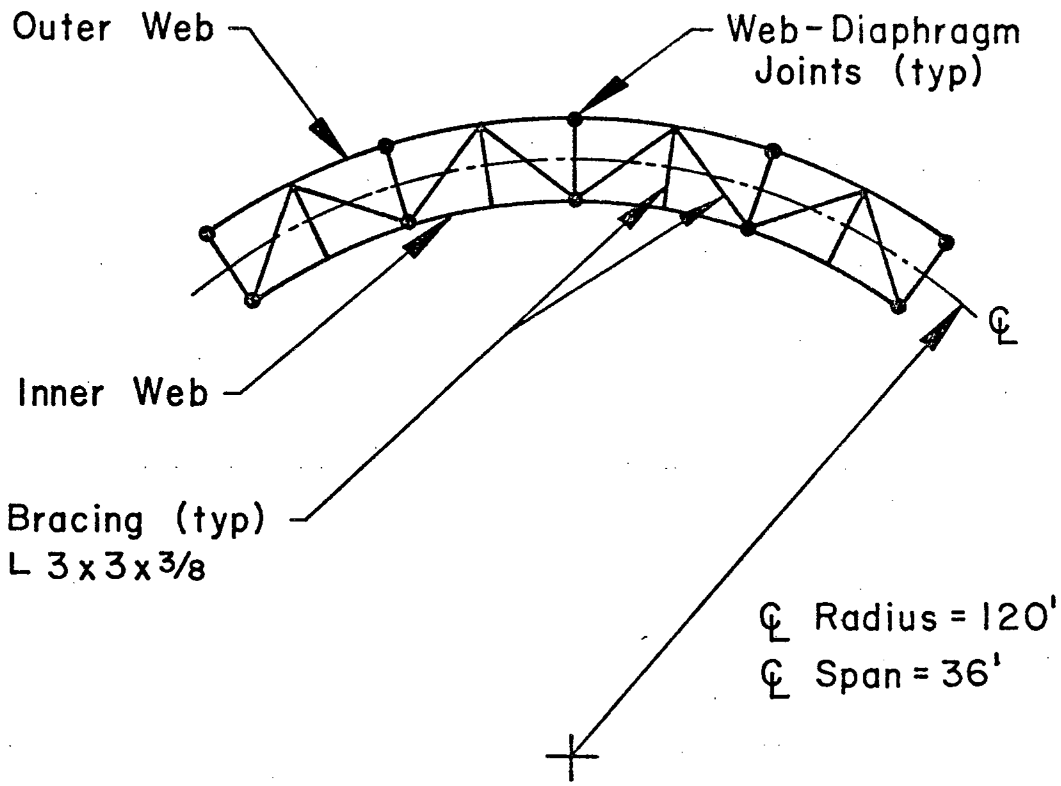


Fig. 21 Temporary Top Lateral Bracing System for Shipping and Handling

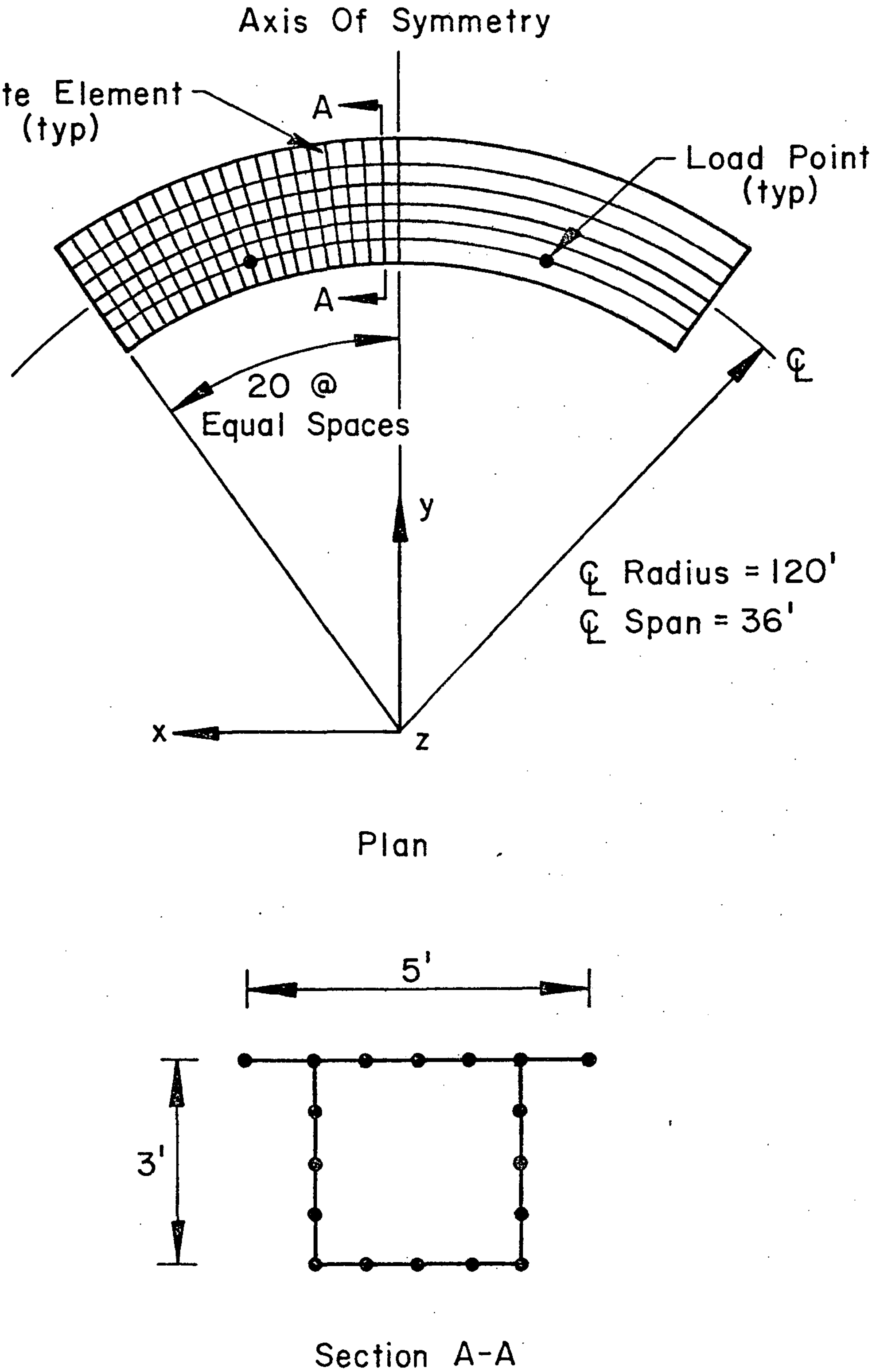
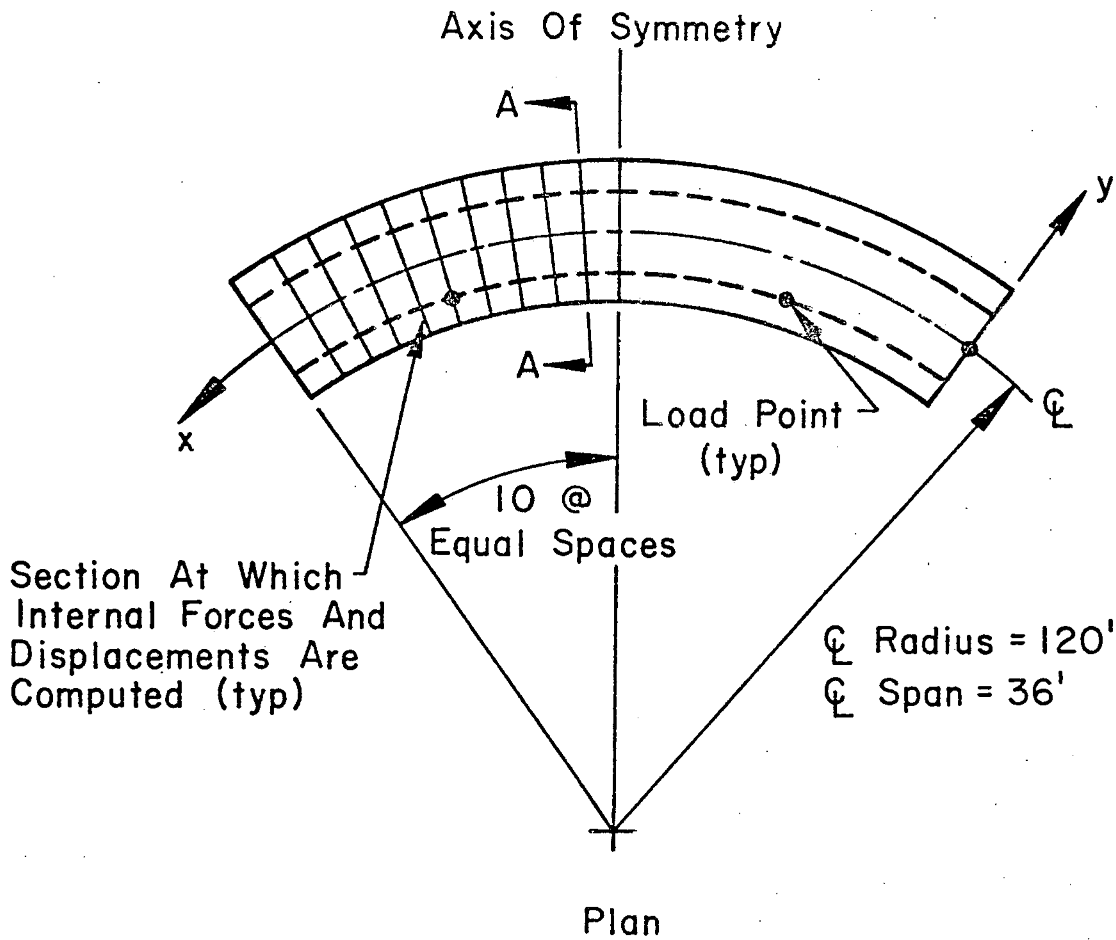


Fig. 22 Finite Element Discretization for Analysis of Box Girder Test Assemblies by SAP IV



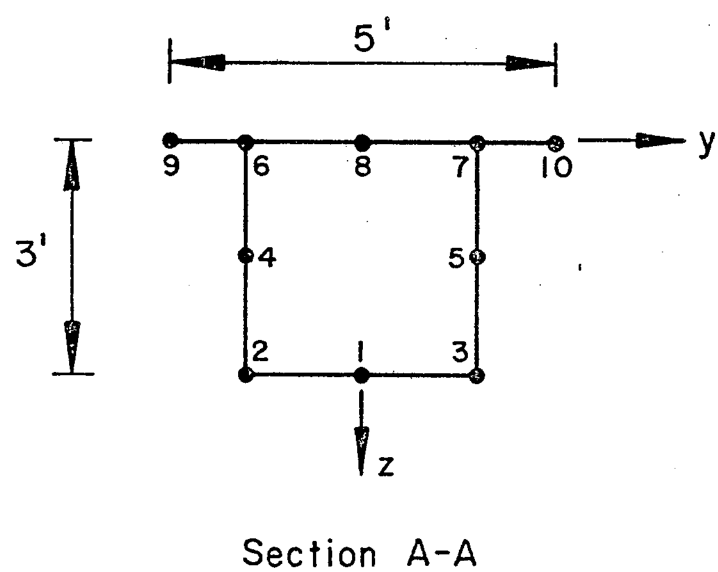


Fig. 23 Finite Strip Discretization for Analysis of Box Girder Test Assemblies by CURDI

7. APPENDIXES

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 span, 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 purposes of performing in-depth analyses, designs, 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: COMPUTER PROGRAM SURVEY

The following computer programs were examined for the purpose of selecting four programs suitable for the analysis of the large scale laboratory test assemblies (2 each for the plate girder and box girder test assemblies - Art. 1.5).

A. CURVED PLATE GIRDER ANALYSIS PROGRAMS

1. Reference - B6062, by D. R. Schelling
   J. E. Greiner Co.
   Baltimore, Md.

   Analysis - Grid method, IBM 1130

   Capability - 11 spans (continuous), 14 girders, uniform and concentrated loads, bending and warping stresses generated.

   Commentary - Diaphragms are non-composite. Only standard AASHTO or interstate loading is permitted. Box capability not yet included. Can only be used where a slab is present.

2. Reference - COBRA I, II, and III, by C. P. Heins
   University of Maryland

   Analysis - Fourier series slope deflection method, IBM 7094, FORTRAN IV

   Capability - 3 spans (continuous), 7 girders; any number of concentrated and uniformly distributed loadings can be combined. Incorporates bending, warping and pure torsion effects. Automatic interpolation of stresses between nodes.

   Commentary - The Roman numeral designation of the COBRA programs relates to the maximum number of continuous spans which the program can handle. Output includes pure and warping torsion as well as bending effects. Only generalized stresses available.

3. Reference - CURSYS, by C. P. Heins
   University of Maryland

   Analysis - Finite difference method combined with matrix stiffness for diaphragms, Univac 1108, FORTRAN V.

   Capability - Limitless continuous spans and limitless girders.

Limitless uniform and concentrated loading which can be combined. Diaphragm forces computed.

Commentary - Output only at nodes. Includes pure and warping torsion as well as bending effects.

4. Reference - CURVBRG, by D. P. Mondkar and G. H. Powell
University of California at Berkeley
(Report No. UC SESM 74-17)

Analysis - Direct stiffness method, CDC 6400/6600, IBM 360/370, FORTRAN IV

Capability - 10 girders, simple or continuous span. Unlimited concentrated, uniform and wheel train loadings which can be combined. Includes beam and truss diaphragms and bottom lateral bracing.

Commentary - Program can be used with or without a concrete slab, for composite and non-composite analyses. Employs automatic nodal point and section property generation. Automatic interpolation of stresses between nodes.

5. Reference - CUGAR 1 and 2, by F. H. Lavelle, et al.
University of Rhode Island
(Eng. Bull. No. 14 and CURT Final Report, Proj. HPR-2(111))

Analysis - Grid method, IBM 350, Model 50, also IBM 1130, FORTRAN IV (E-level)

Capability - 10 spans (continuous), 10 girder lines; plate girders and rolled beams; composite and non-composite; 200 loads; uniform, concentrated, partial uniform and combinations.

Commentary - CUGAR 1 does not include truss diaphragms. Truss diaphragms must be approximated as equivalent beams. CUGAR 2 includes truss or beam type diaphragms. Output only at nodes. Only generalized stresses available.

6. Reference - HORIZONTAL CURVED GIRDER ANALYSIS
by L. J. Yoder
Richardson, Gordon and Associates
3 Gateway Center
Pittsburgh, Pa. 15222

Analysis - U. S. Steel V method, IBM 1130, FORTRAN

Capability - 2 to 5 continuous spans, 6 girders, uniform loading.

Commentary - Does not accept concentrated loadings. Output of only generalized stresses at tenth points.

B. CURVED BOX GIRDER ANALYSIS PROGRAMS

1. Reference - CURDI, by A. C. Scordelis
University of California at Berkeley
(Report No. UC SESM 74-10)

Analysis - Finite strip, CDC 6400, FORTRAN IV

Capability - Continuous and simple spans with beam type diaphragms

2. Reference - CURSTR, by A. C. Scordelis
University of California at Berkeley

Analysis - Finite strip, CDC 6400, FORTRAN IV

Capability - Simple spans, without diaphragms

C. CURVED PLATE GIRDER AND BOX GIRDER ANALYSIS PROGRAMS

1. Reference - CUGAR 3, by F. H. Lavelle, et al.
University of Rhode Island
(CURT Final Report, Proj. No. HPR-2(111))

Analysis - Grid method, IBM 350, FORTRAN IV

Capability - 10 spans, 10 girder lines, simple or continuous spans. Includes diaphragms.

2. Reference - CURSEL, by C. P. Heins
University of Maryland

Analysis - Finite difference (Vlasov equations), Univac 1108, FORTRAN V

Capability - Limitless continuous spans, limitless loading.

Commentary - Output only at nodal points. Only generalized stresses available. Only single cell box capability.

3. Reference - NONSAP, by E. Wilson, et al.
University of California at Berkeley
(Report No. UC SESM 74-3 and 74-4)

Analysis - Finite element, CDC 6400, FORTRAN V

Capability - Static and dynamic response of nonlinear systems. Program capacity determined by the total number of degrees of freedom of the structure. Two and three dimensional elements.

4. Reference - SAP IV, by E. Wilson, et al.
University of California at Berkeley
(Report No. EERC 73-11)

Analysis - Finite element, CDC 6400, FORTRAN IV

Capability - Static and dynamic response of a linear three dimensional system. Program capacity is defined by the number of nodal points in the system.

5. Reference - SSAP (SOLID SAP), by E. Wilson
Denver Mining Research Center
Bureau of Mines
U. S. Dept. of the Interior

Analysis - Finite element, CDC 6400, FORTRAN IV

Capability - Performs static, linear, elastic analyses of three-dimensional structural systems. Program capacity depends on the number of nodal points in the system.

6. Reference - STACRB, by S. Shore, et al.
University of Pennsylvania
(CURT Report No. T0173, Proj. HPR-2(111))

Analysis - Finite element, IBM 370/165, FORTRAN IV

Capability - Simple or continuous spans; static loading; truss and beam diaphragms; composite and non-composite action.

7. Reference - 3-D GRID, by P. J. Brennan, et al.
Syracuse University
(CURT Project HPR-2(111))

Analysis - Three dimensional, IBM 360 or 370, FORTRAN IV

Capability - Simple and continuous spans. Truss diaphragms. Composite and non-composite action. With or without slab. Bottom lateral bracing.

Commentary - Output of only generalized stresses. No automatic generation of section properties or node locations.

APPENDIX C: STRESS RANGE PROFILES AND GROUP 2 DETAIL LOCATIONS FOR PLATE GIRDER TEST ASSEMBLIES

Figures C1 through C10 show the stress range profiles for the bottom flange of each girder. In the figures S_r is the stress range corresponding to a load range of 5 and 105 kips at each quarter point. The abscissa represents fractions of span length L, with X being the position along the assembly centerline. The lengths of girders 1 and 2 of each assembly are, respectively, L_1 and L_2. Profiles are shown for only half of each girder length because of symmetry of geometry and group 2 details about midspan. The curved stress range profiles in each figure are for the flange tips. The profile consisting of two straight line segments is for the web-to-tension flange junction. The web profile curve does not cross the flange tip curves at precisely the same location since web stress range values are plotted for the bottom of the web, not for the middepth of the flange.

The locations of group 2 details is governed by two considerations. First, an attempt is made to provide an equal number of various detail types and subtypes. Second, a diversity of stress range gradients for a given type is sought. The resulting locations of the group 2 details are shown by a vertical line on the stress range profiles in Figs. C1 to C10. The type of detail is also shown (refer to Table 3).

The vertical line representing the location of a given group 2 detail crosses either one or two profile curves depending on the attachment position. For example, detail III_{ob} is a web detail and its vertical line always cross the web-to-flange junction profile near a stress range of 10 ksi (i.e. that required for failure at two million cycles). Similarly detail type IV_o is attached only to the flange tips and its vertical line always crosses a flange tip profile at about 15 ksi. On the other hand detail types V_{ob} and I_o for example are attached across nearly half of the flange width. Thus, their vertical lines cross the web profile and the flange tip profile of the side on which they are placed.

For several of the girders little or no choice exists with regard to placement of group 2 details if fatigue failure is desired at two million cycles. Girder 1 of assemblies 2 through 5 can only use detail type V_o

(V_{oa} or V_{ob}) because only the one flange tip stress range profile reaches the 10 ksi level. The stress range for girder 2 of assembly 1 nowhere reaches 10 ksi. Therefore, there is no possibility of placing group 2 details on that girder. Finally, detail type II_o can only be placed on girder 2 of assemblies 2 and 3 since the web-to-flange stress range reaches 15 ksi only on these girders.

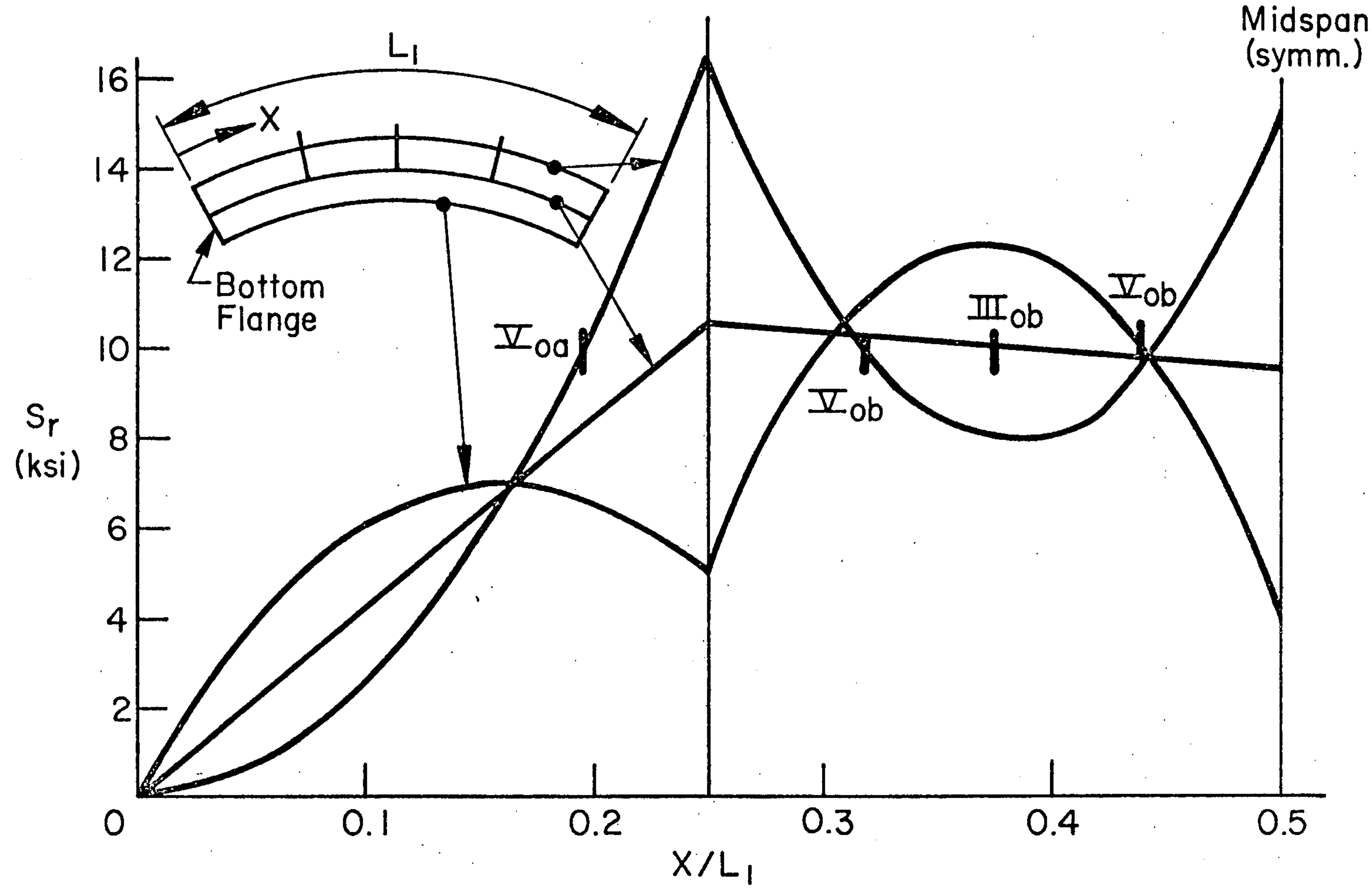


Fig. C1 Stress Range Profiles for Bottom Flange
Plate Girder Test Assembly 1 - Girder 1

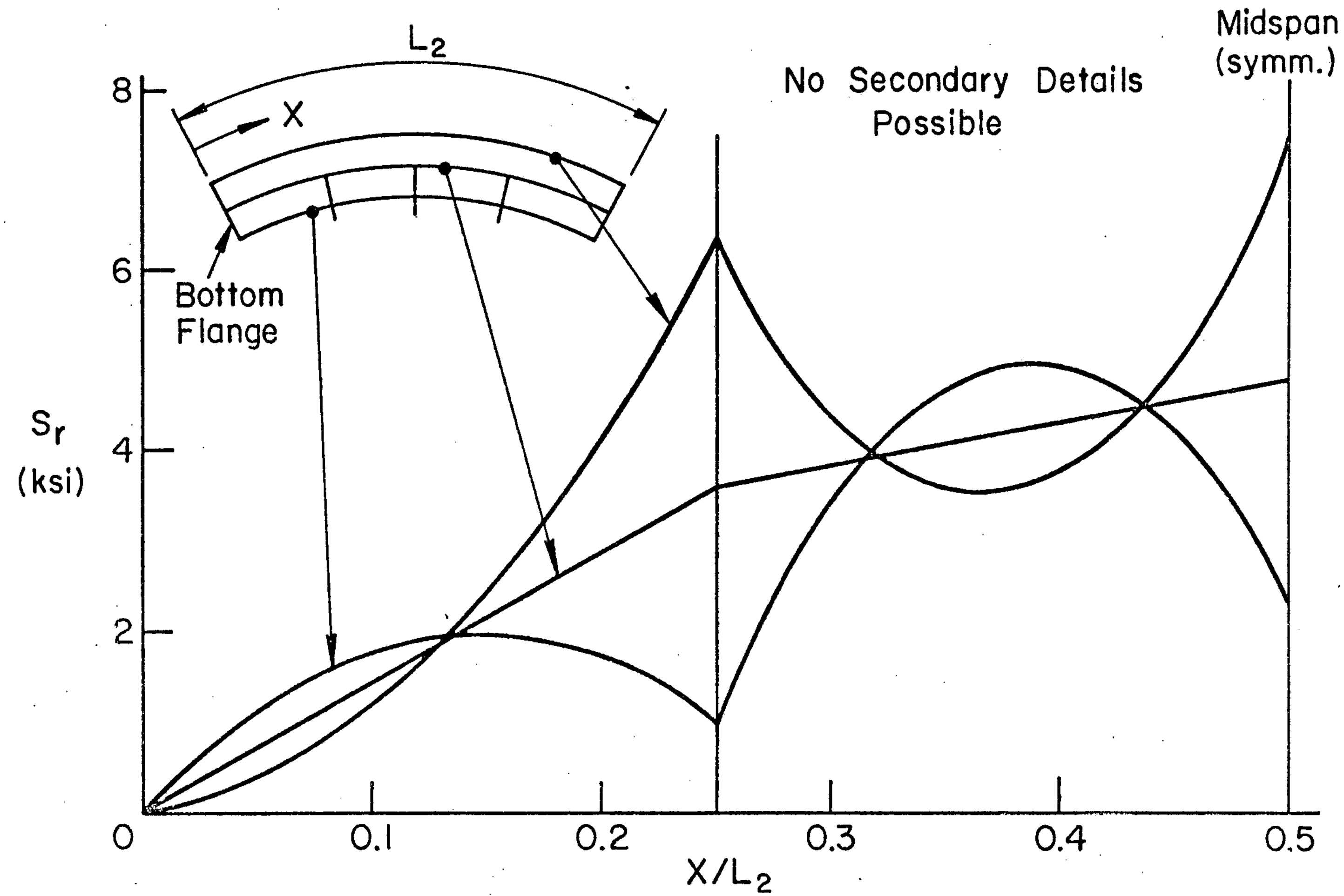


Fig. C2 Stress Range Profiles for Bottom Flange Plate Girder Test Assembly 1 - Girder 2

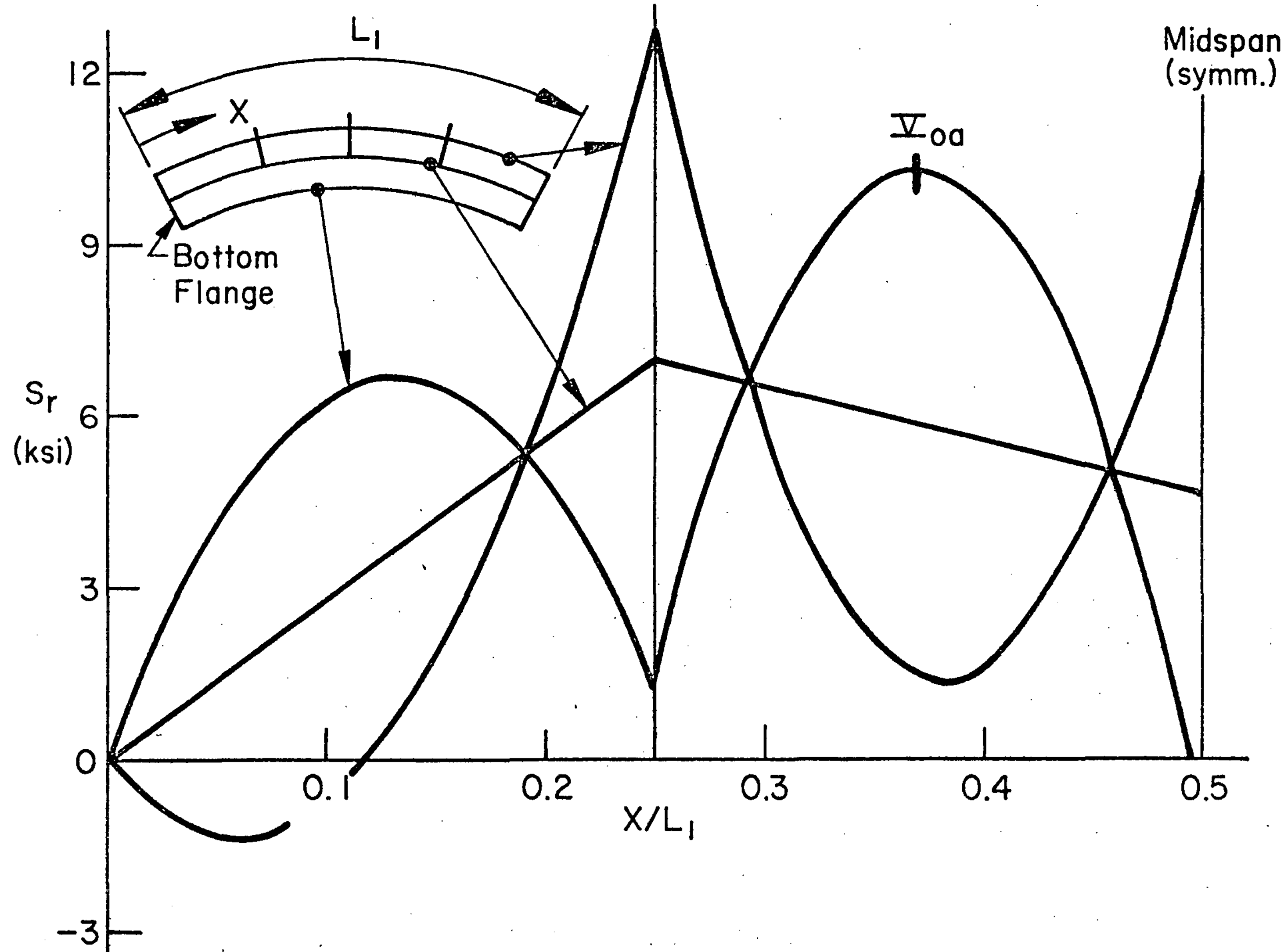


Fig. C3 Stress Range Profiles for Bottom Flange Plate Girder Test Assembly 2 - Girder 1

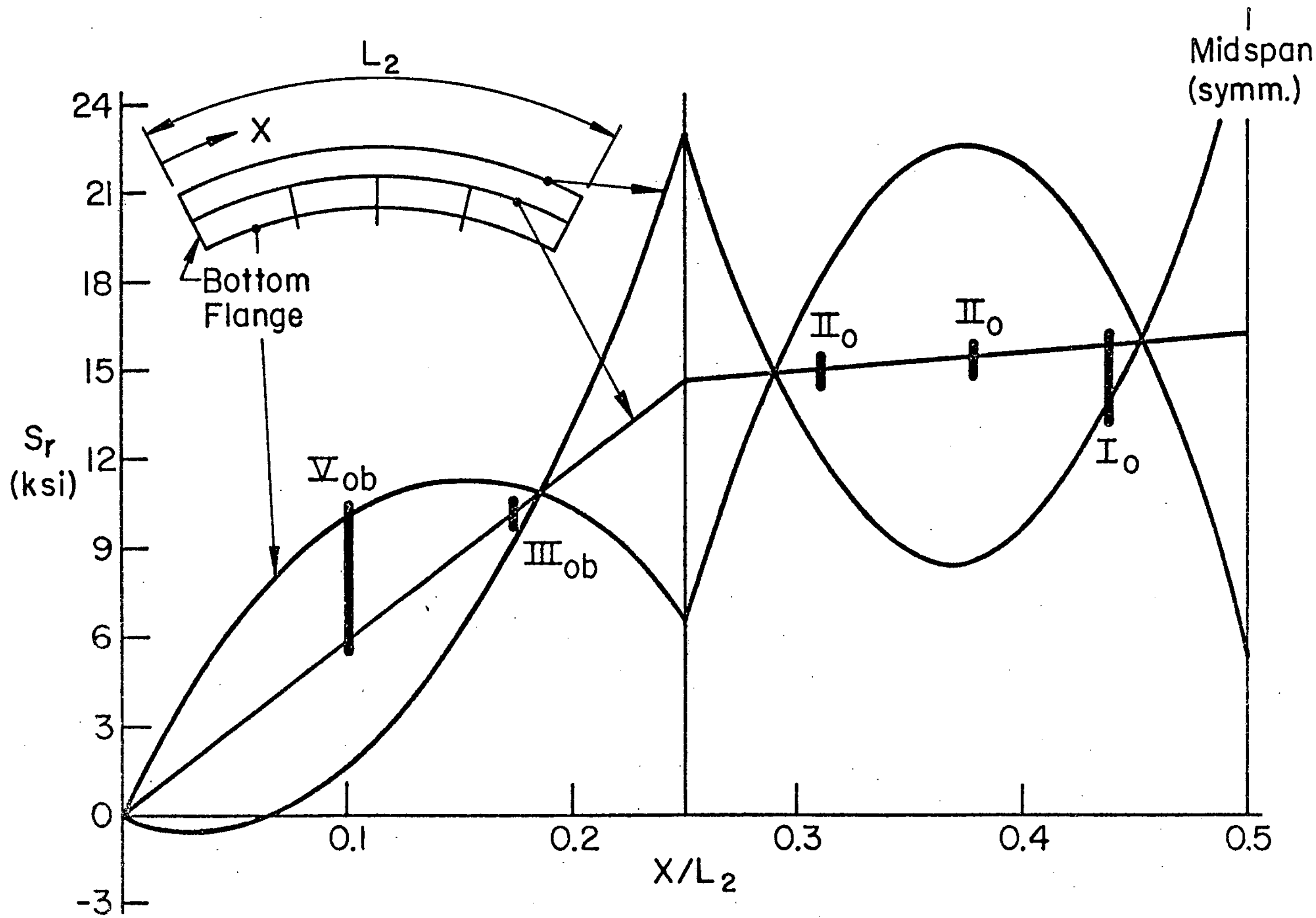


Fig. C4 Stress Range Profiles for Bottom Flange
Plate Girder Test Assembly 2 - Girder 2

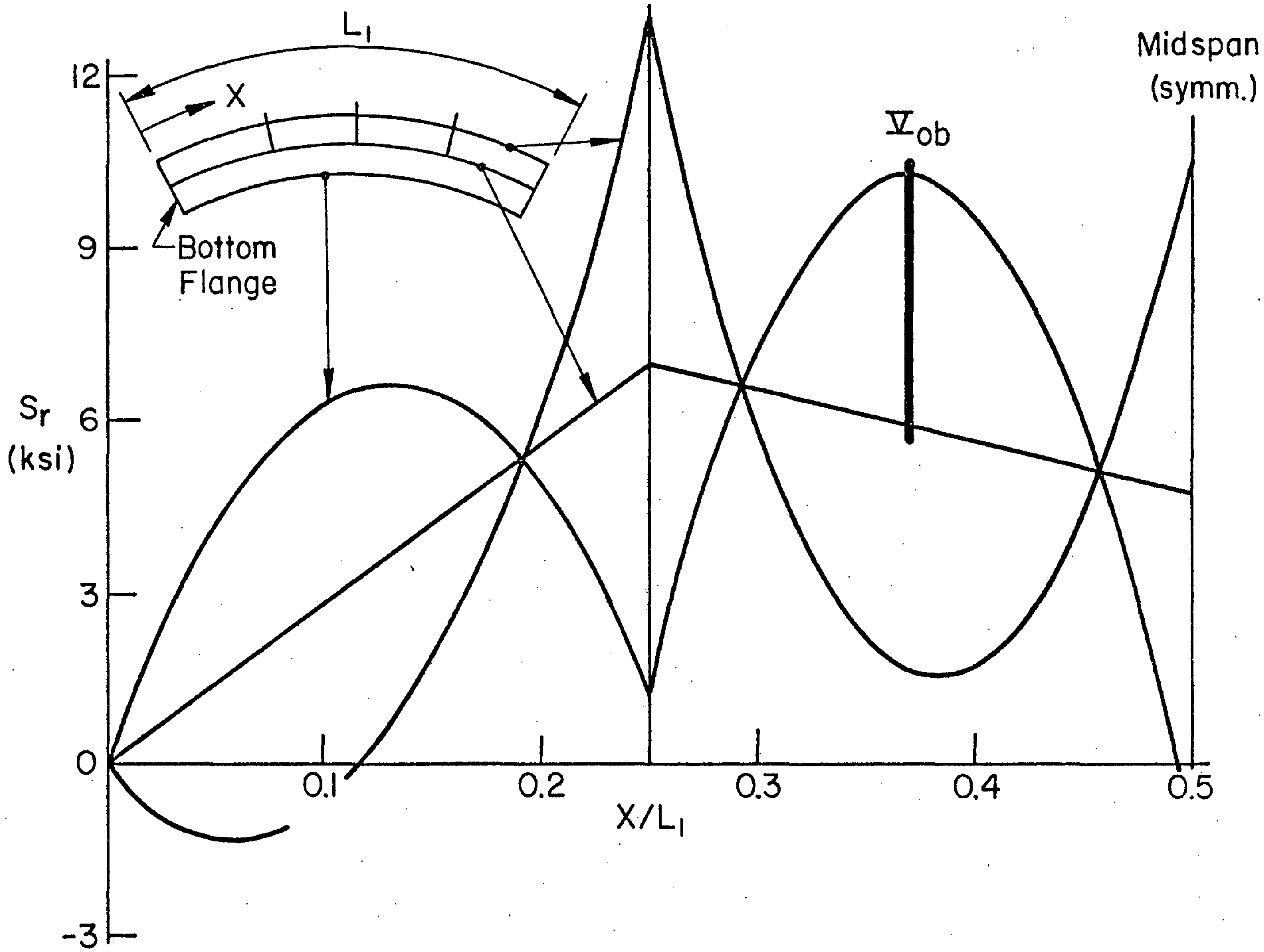


Fig. C5 Stress Range Profiles for Bottom Flange Plate Girder Test Assembly 3 - Girder 1

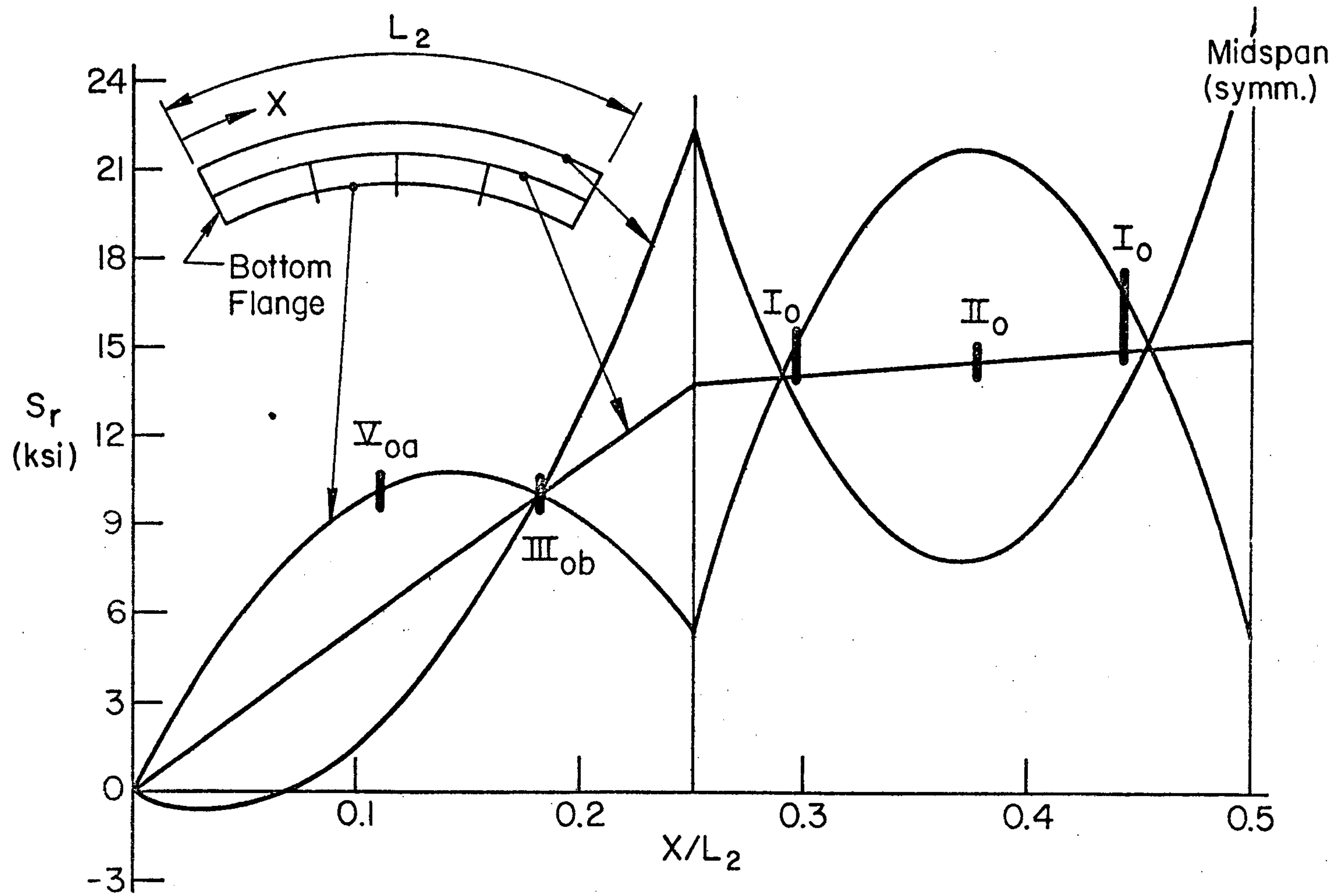


Fig. C6 Stress Range Profiles for Bottom Flange Plate Girder Test Assembly 3 - Girder 2

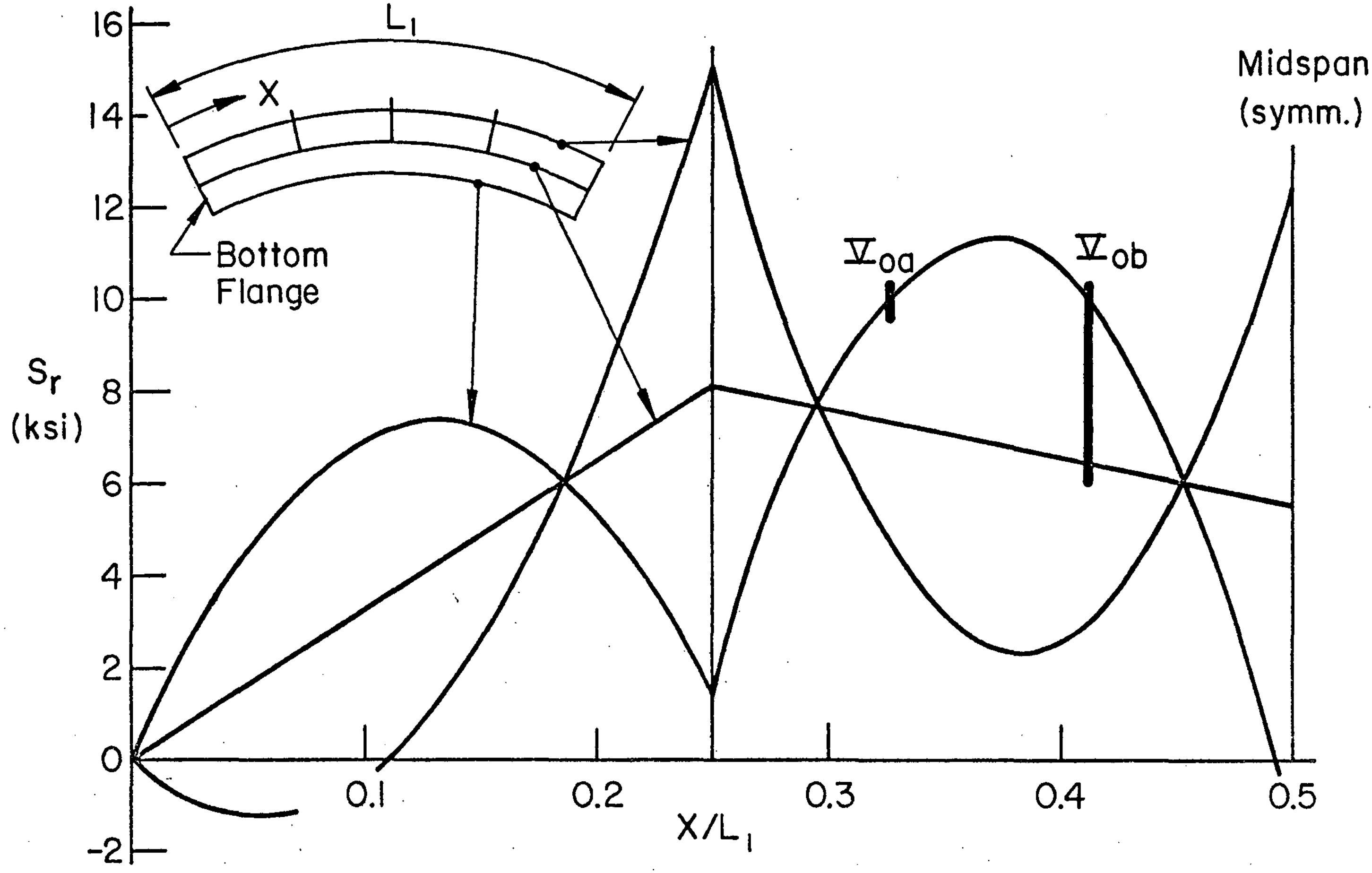


Fig. C7 Stress Range Profiles for Bottom Flange Plate Girder Test Assembly 4 - Girder 1

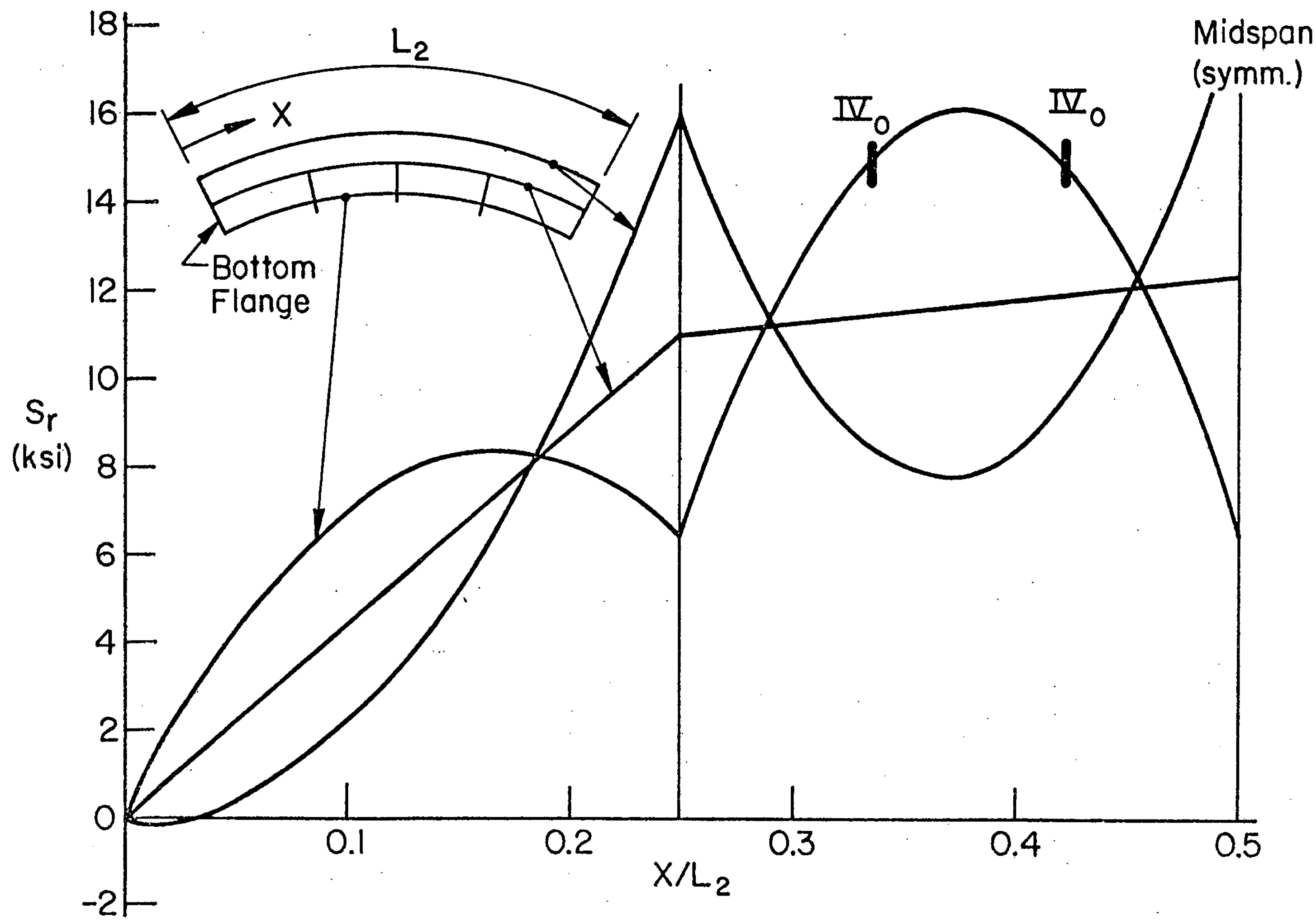


Fig. C8 Stress Range Profiles for Bottom Flange
Plate Girder Test Assembly 4 - Girder 2

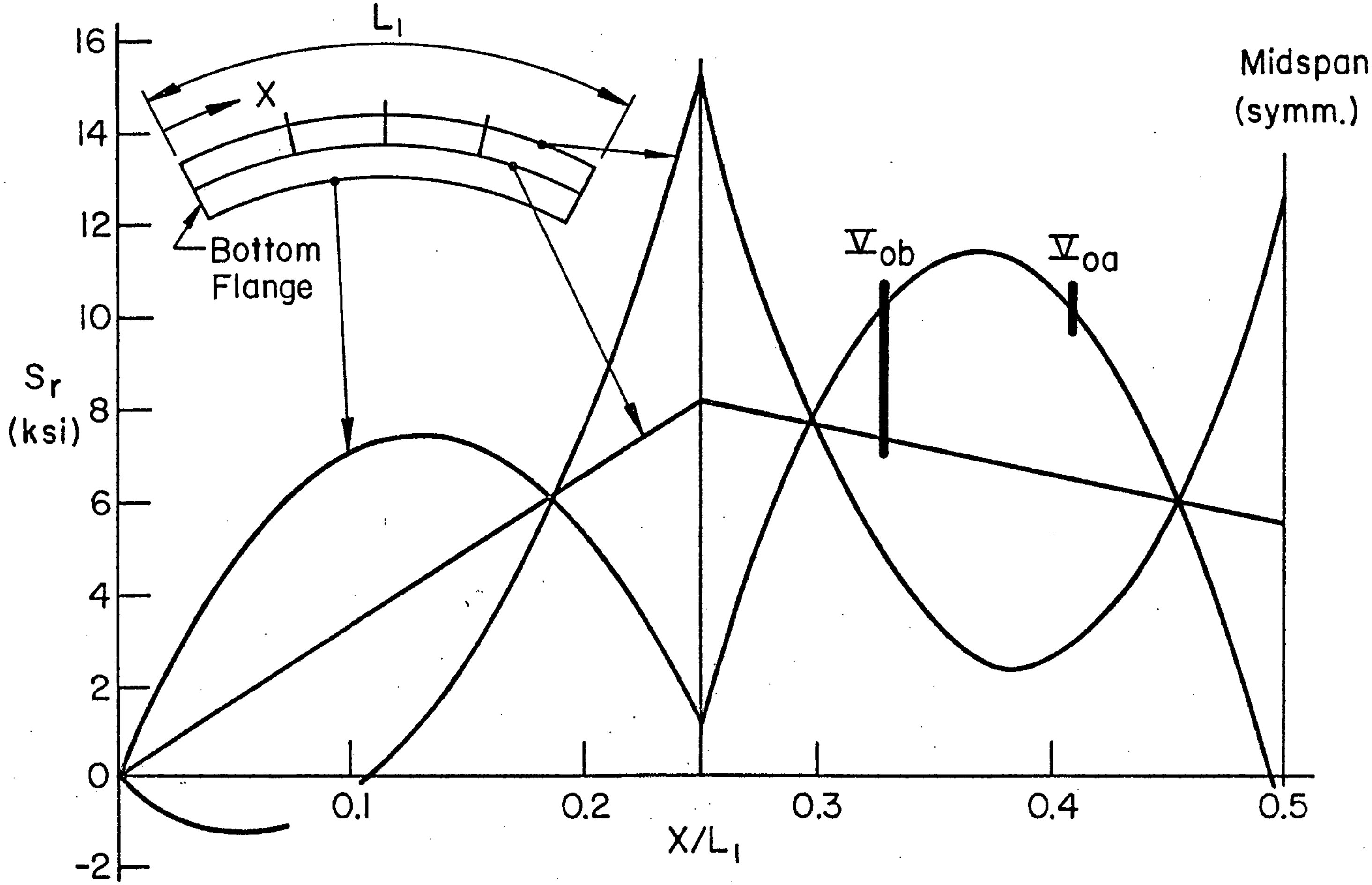


Fig. C9 Stress Range Profiles for Bottom Flange
Plate Girder Test Assembly 5 - Girder 1

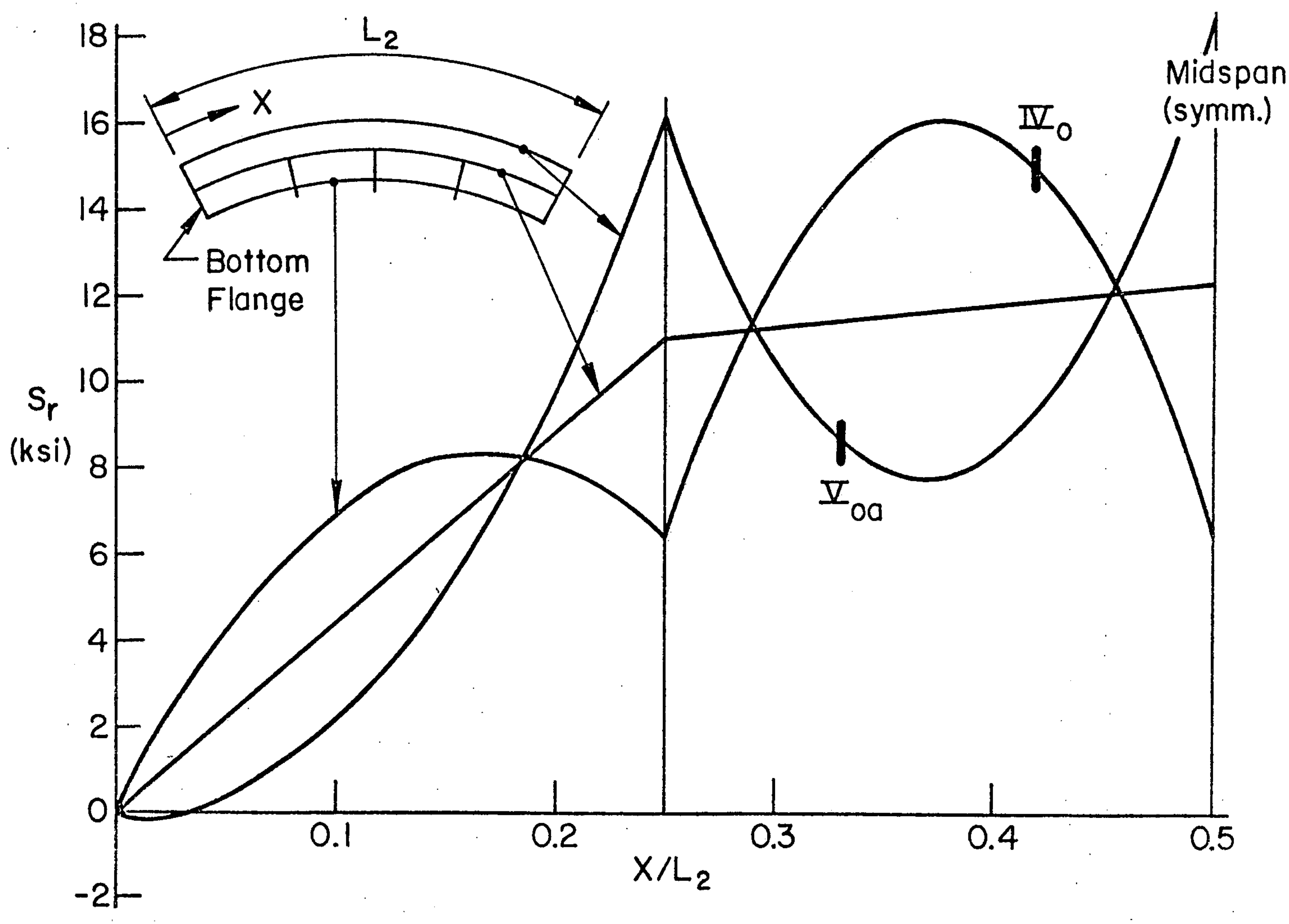


Fig. C10 Stress Range Profiles for Bottom Flange
Plate Girder Test Assembly 5 - Girder 2

APPENDIX D: STRESS RANGE PROFILES AND GROUP 2 AND 3 DETAIL LOCATIONS FOR BOX GIRDER TEST ASSEMBLIES

Figures D1 through D7 present stress range profiles for the curved box girder test assemblies. In the figures S_r is the stress range corresponding to a load range of 5 to 105 kips at each quarter point. The abscissa represents fractions of span length L, with X being the position along the assembly center line. Stress range profiles are shown for one-half the length of each assembly because of symmetry of geometry loading <u>and</u> fatigue details about mid-span.

At the top of Figs. D1 through D6, a schematic view of the longitudinal stiffeners on a web or on the bottom (tension) flange are shown, and correlated with the schematic view of the assembly cross-section. The interior web denotes the web with the smaller radius. The welded details in group 2 (Art. 3.2 and Table 9) associated with these attachments are also shown. The shaded triangles indicate the cross-secitons, referred to the assembly centerline, on which the details occur.

Similarly Fig. D7 shows the location of welded details in group 3 (Art. 3.2 and Table 9) which are associated with the continuous and discontinuous back-up bars.

In order to ensure fatigue failure a welded detail is generally positioned where the stress range is equal to or not more than 2 ksi greater than the allowable stress range for that Category[6]. The allowable stress ranges represent the 95% confidence limit for 95% survival at two million cycles. Welded details that do not have sufficient stress ranges to expect fatigue failure are shown in the figures in parentheses. For clarity the welded details in groups one and four (Art. 3.2 and Table 9) are not shown in the figures.

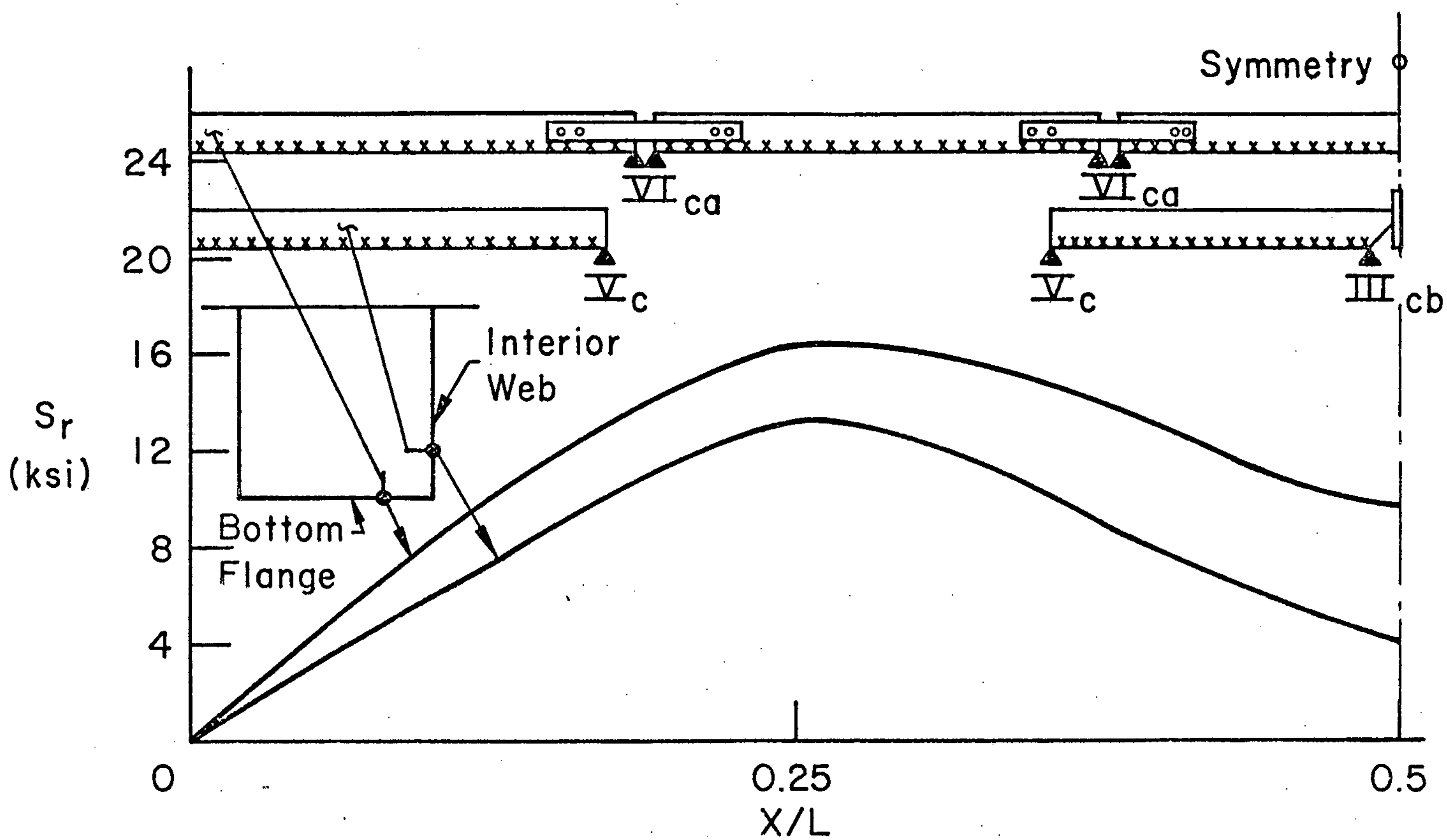


Fig. D1 Stress Range Profiles for Box Girder Assembly 1

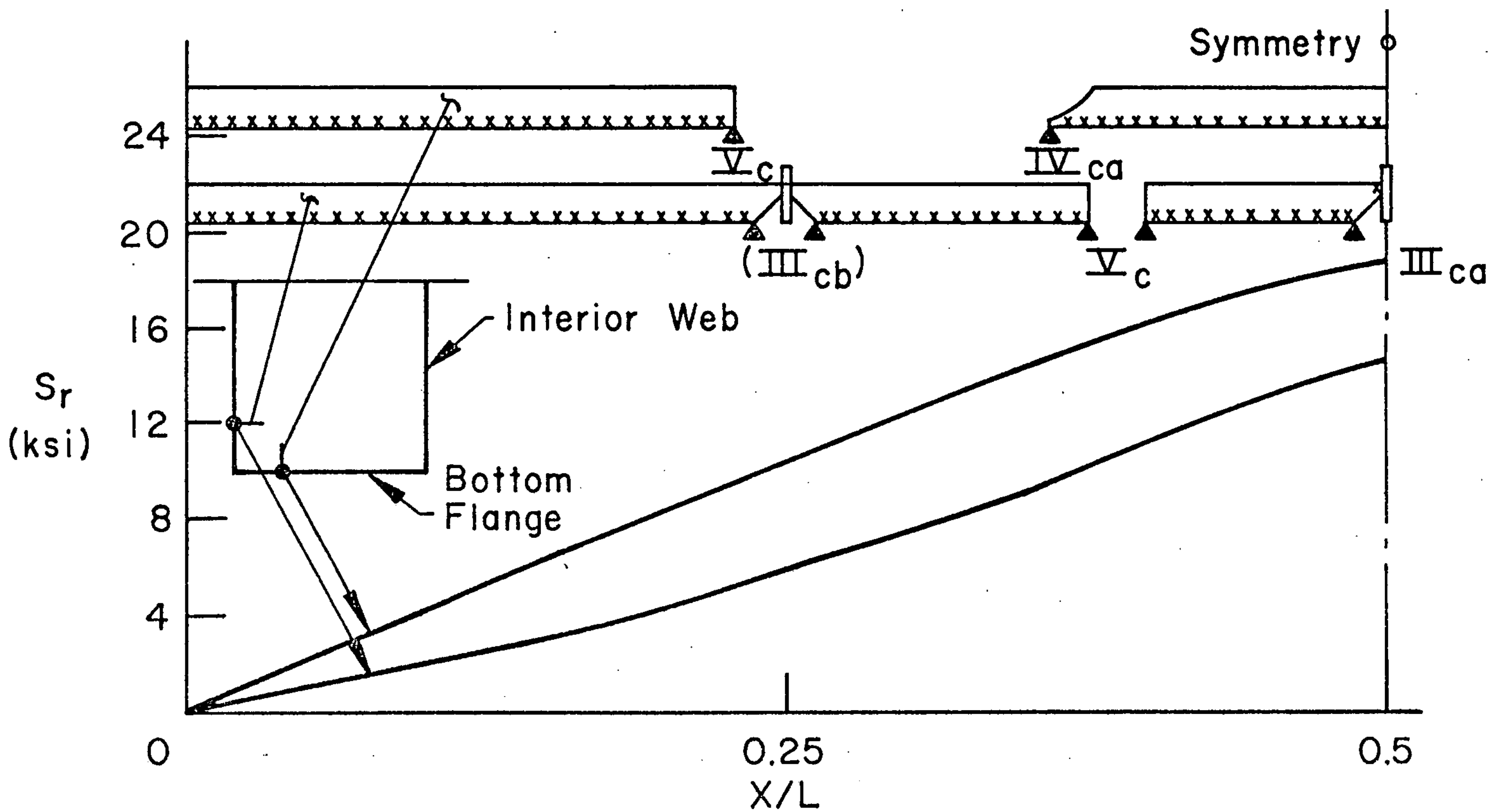


Fig. D2 Stress Range Profiles for Box Girder Assembly 1

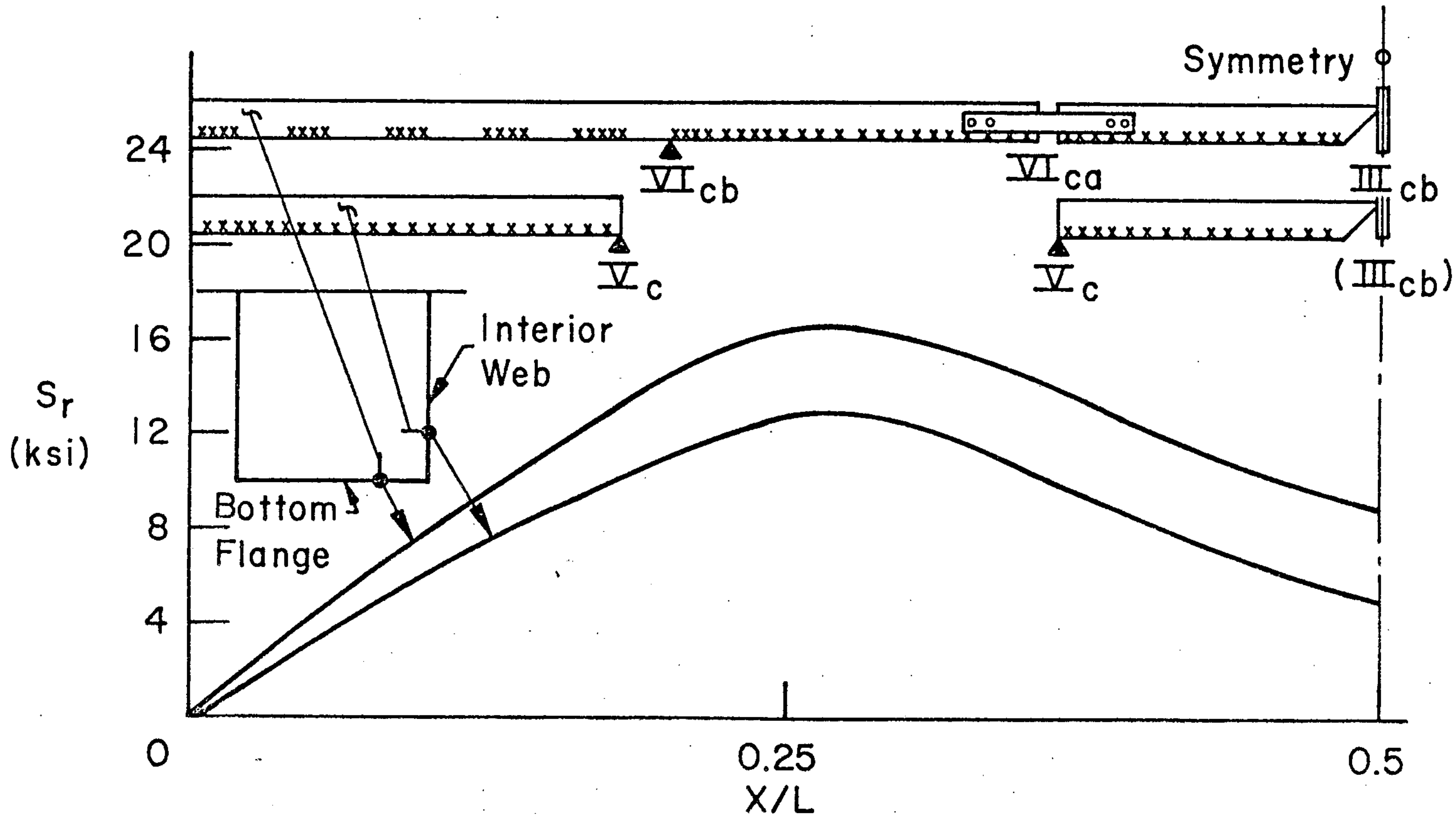


Fig. D3 Stress Range Profiles for Box Girder Assembly 2

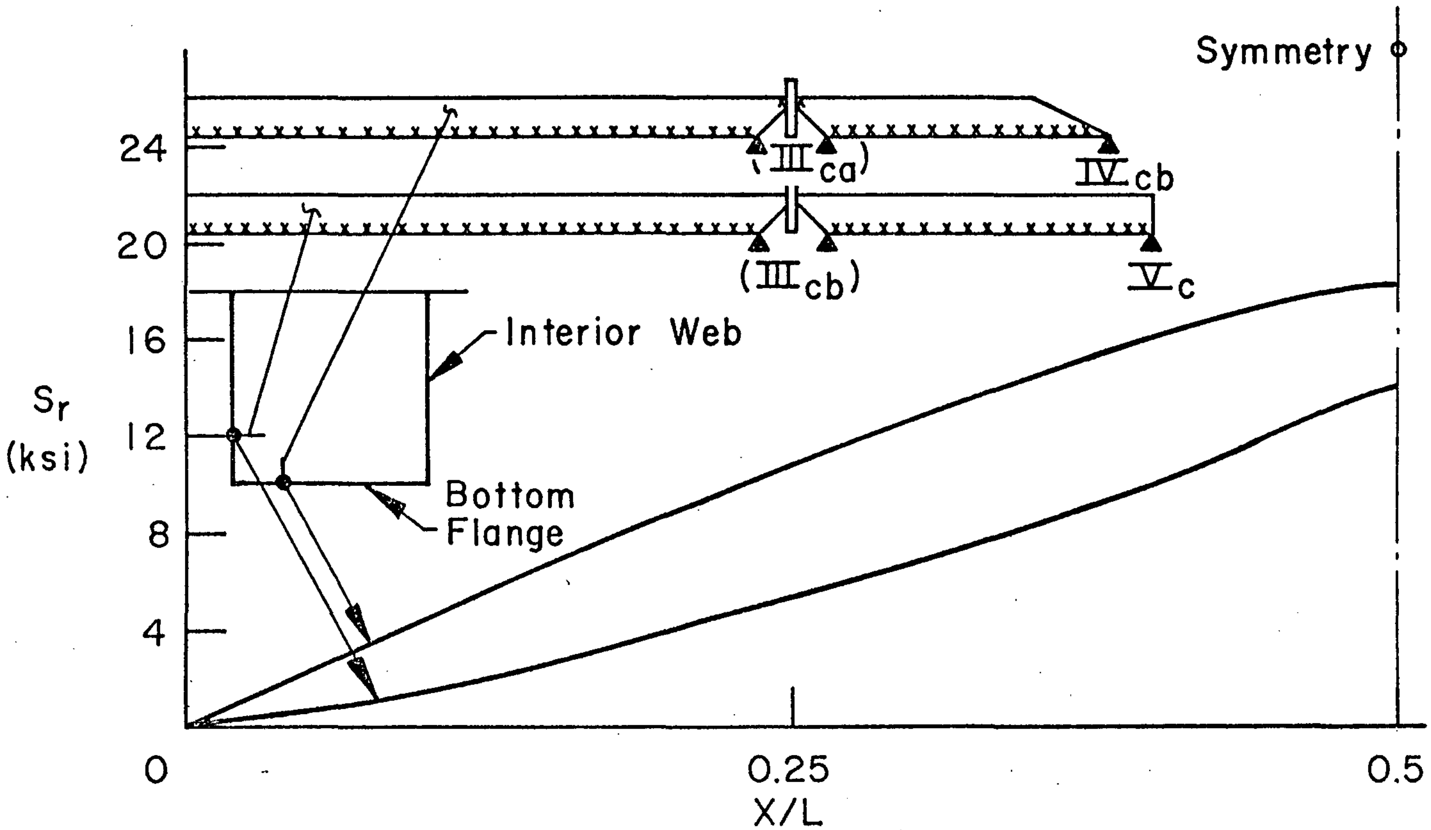


Fig. D4 Stress Range Profiles for Box Girder Assembly 2

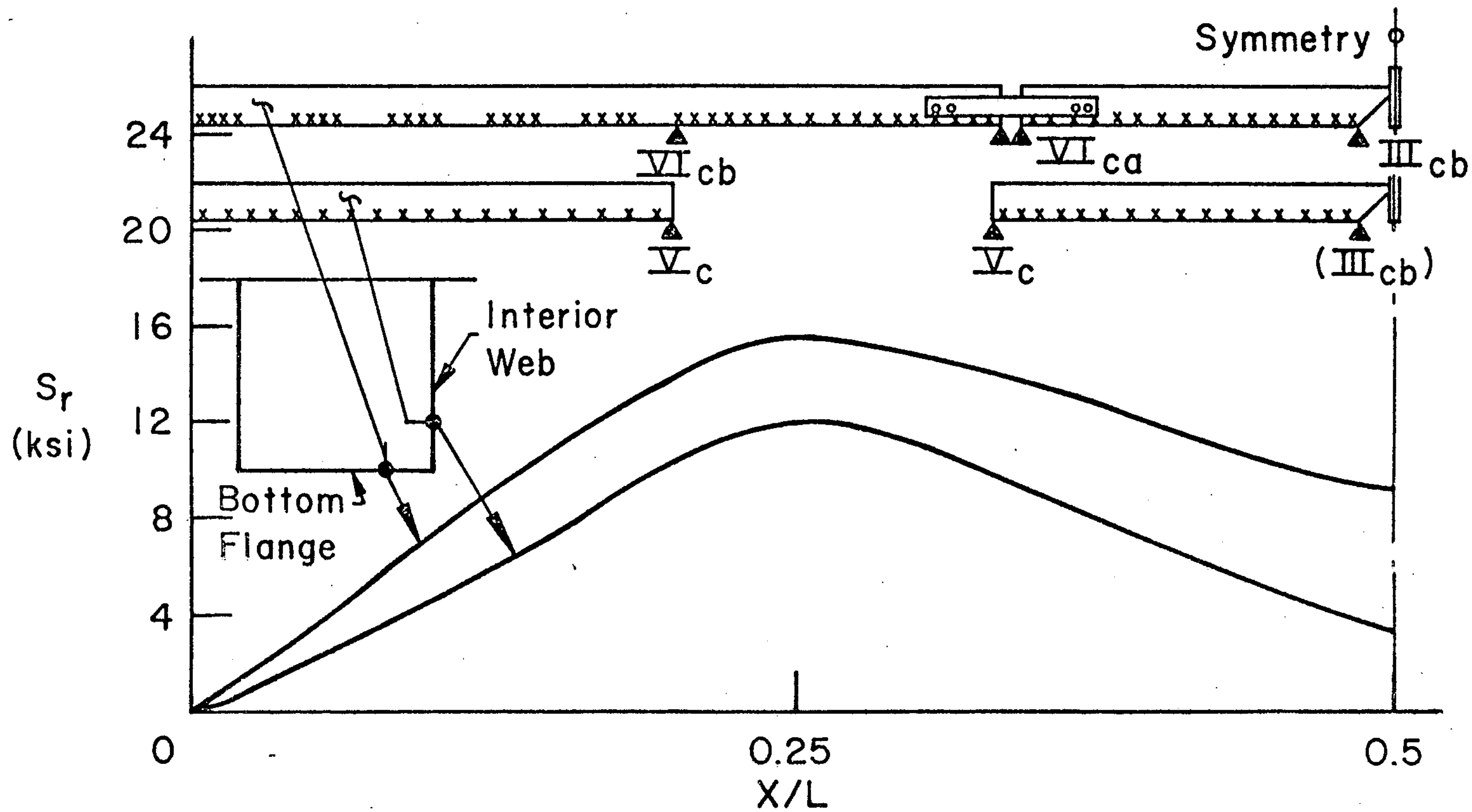


Fig. D5 Stress Range Profiles for Box Girder Assembly 3

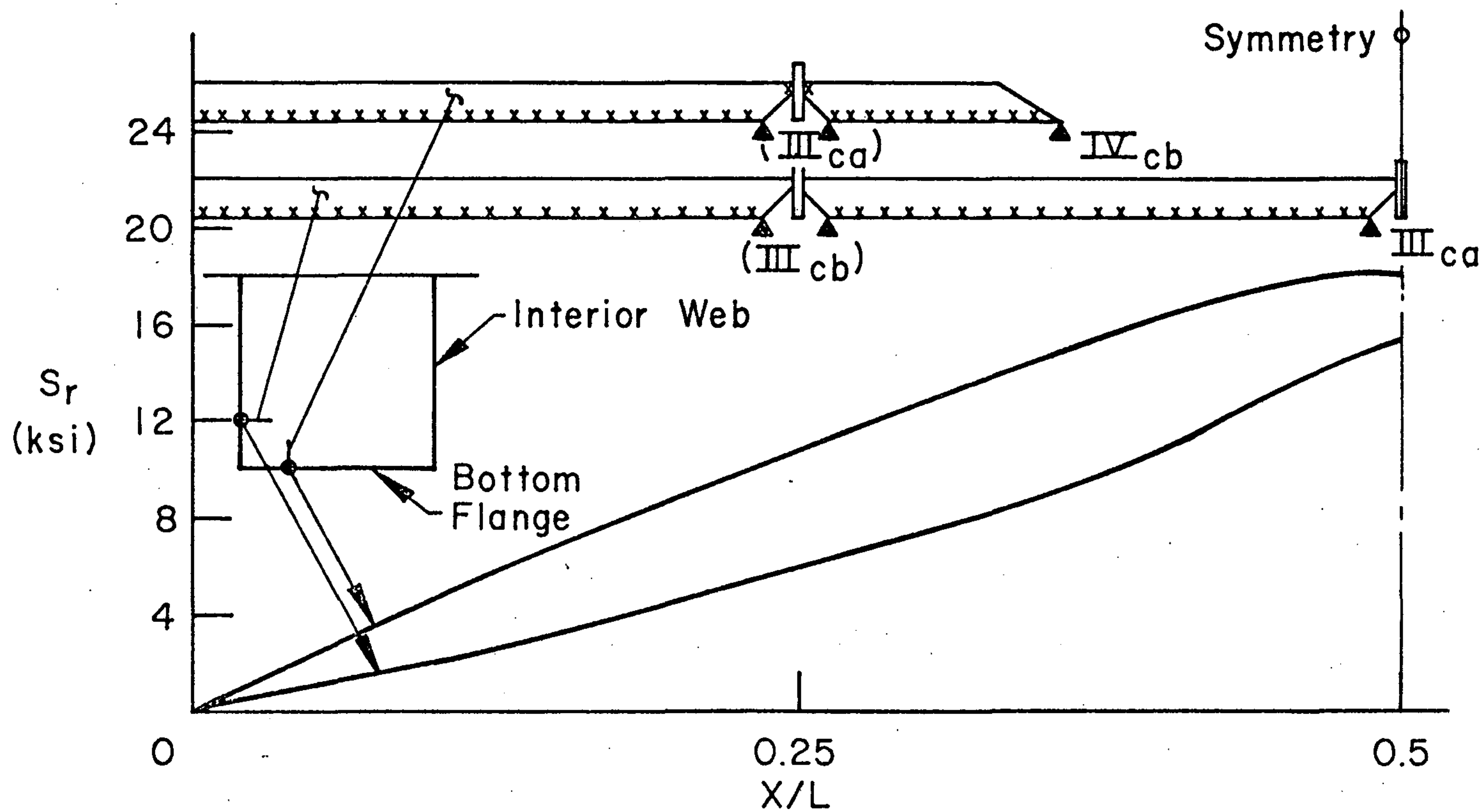


Fig. D6 Stress Range Profiles for Box Girder Assembly 3

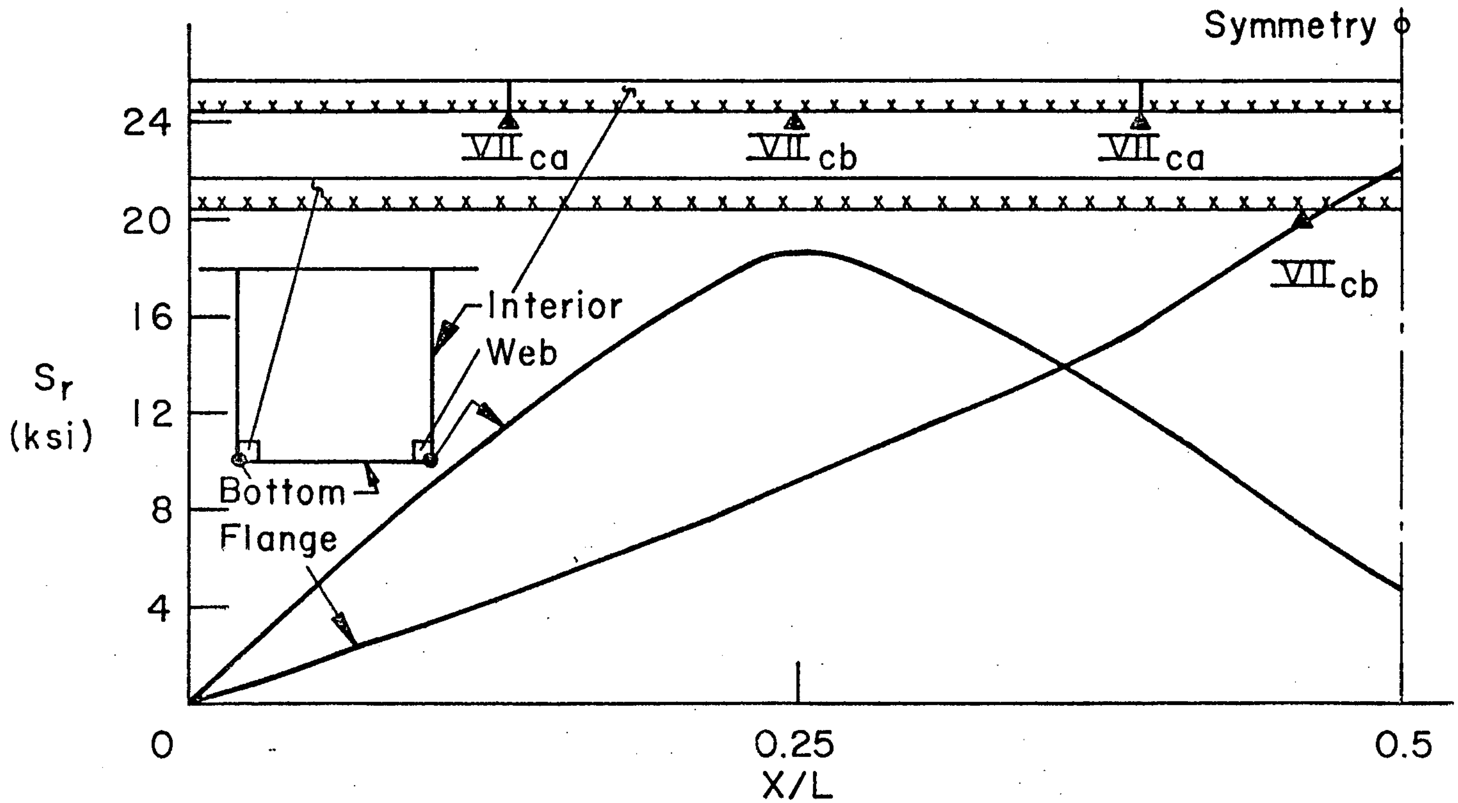


Fig. D7 Stress Range Profiles for Box Girder Assembly 3

8. ACKNOWLEDGMENTS

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T. V. Galambos

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The manuscript was typed by Ms. Antoinette Larkin and Ms. Mary Snyder. The figures were prepared under the direction of John M. Gera.

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