THIN WEB GIRDER FATIGUE BEHAVIOR
AS INFLUENCED BY BOUNDARY RIGIDITY

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UNIVERSITY OF ILLINOIS
URBANA, ILLINOIS
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Laurence Robert Hall, Ph.D.
Department of Civil Engineering
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The purpose of this investigation was to study the fatigue behavior of thin web girders. The investigation was directed primarily toward the determination of the effects of flange rigidity and vertical stiffener rigidity on the fatigue behavior of individual panels of thin web girders. A considerable amount of attention was also devoted to the initial lateral web deflections in the webs of the girders.

A qualitative analysis was made of the fatigue behavior of individual thin web girder panels with initial lateral web deflections. Two types of loading were considered: pure shear and combined shear and bending. The analysis was based on theoretical and experimental data on the post buckling behavior of girder web panels which have been reported in the literature.

In the experimental part of the investigation twenty fatigue tests were conducted on scale model thin web all-welded girders. Fifteen of these tests were conducted on girders in which the only geometric variable was the size of the flanges. Nine of the fifteen tests were carried out under conditions approximating pure shear and the remaining six tests were conducted under the condition of combined shear and flexure. Five additional tests were completed on girders in which the only variable was the rigidity of the vertical stiffeners.
The investigation revealed the manner in which thin web girder panels fail in fatigue. The effects of flange rigidity and vertical stiffener rigidity on fatigue life were determined and recommendations were made with regard to minimum desirable flange rigidities. Finally, it was shown that initial lateral web deflections have an effect on the fatigue behavior of thin web girders.
ACKNOWLEDGEMENT

The study reported herein is part of an investigation of the Flexural Fatigue Strength of All-Welded Beams and Girders which is being conducted at the University of Illinois under the sponsorship of the Department of Commerce, Bureau of Public Roads. This investigation is part of the structural research program of the Department of Civil Engineering.

The author wishes to express his indebtedness to Professor J. E. Stallmeyer, his advisor, for his encouragement and advice throughout the duration of the investigation. The writer wishes to express his thanks to Mr. G. E. Rymer of the laboratory shop for his helpful suggestions and care in the welding of the test specimens. The efforts of Mr. D. F. Lange and other members of the laboratory shop in preparing specimens and maintaining test equipment are greatly appreciated.
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<td>A</td>
<td>Area of stiffener.</td>
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<tr>
<td>a</td>
<td>Length of side of square plate.</td>
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<tr>
<td>b</td>
<td>Stiffener spacing.</td>
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<tr>
<td>C₁, C₂</td>
<td>Diagonal Tension Theory Constants derived by Wägner.</td>
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<td>D</td>
<td>Flexural rigidity of unit width of plate, ( \frac{E t^3}{12(1-\mu^2)} )</td>
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<tr>
<td>d</td>
<td>Clear depth of web between flanges.</td>
</tr>
<tr>
<td>E</td>
<td>Young's modulus of elasticity.</td>
</tr>
<tr>
<td>I</td>
<td>Moment of inertia of flange about its own horizontal centroidal axis.</td>
</tr>
<tr>
<td>Iₛ</td>
<td>Moment of inertia of a vertical stiffener or stiffeners about a horizontal axis in the central plane of the web.</td>
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<tr>
<td>kₛ</td>
<td>Theoretical shear buckling coefficient.</td>
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<td>t</td>
<td>Thickness of web plate.</td>
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<tr>
<td>Wₜ</td>
<td>In all cases, the theoretical buckling load of an ideal web panel with simply supported rigid boundaries.</td>
</tr>
<tr>
<td>W/Wₜ</td>
<td>Ratio of actual panel load to theoretical buckling load.</td>
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<tr>
<td>w</td>
<td>Lateral web deflection.</td>
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<tr>
<td>wₜ</td>
<td>Lateral web deflection at center of web panel.</td>
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<tr>
<td>wₘₜ</td>
<td>Maximum lateral web deflection.</td>
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<tr>
<td>1/bₜ</td>
<td>Flange rigidity parameter.</td>
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<td>$E_l s / Db$</td>
<td>Stiffener rigidity parameter.</td>
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<td>$\alpha$</td>
<td>$d/b$, the aspect ratio of a web panel.</td>
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<td>$\beta$</td>
<td>$d/t$, web depth to web thickness ratio.</td>
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<td>$\gamma$</td>
<td>$l/bt^3$, a parameter.</td>
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<td>$\mu$</td>
<td>Poisson's ratio.</td>
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1. INTRODUCTION

1.1 Background

Plate girders are structural members which are required to carry both bending moment and shearing forces. If the forces are resisted by pure beam action, the shear is resisted almost entirely by the web of the girder and the bending moment is resisted almost entirely by the flanges of the girder. In most civil engineering applications, plate girders are primarily moment resisting members. As a result, it has been general practice to place as much material as possible in the flanges of the girder and to locate the flanges as far apart as possible. Economical use of high-strength materials in the flanges makes this factor even more significant. If optimum utilization of material is to be realized, this procedure leads to the need for a very deep thin web. However, when the web thickness becomes small in relation to the web depth, the web plate buckles. It is this phenomenon of buckling which has limited the optimum utilization of material in girders designed for civil engineering structures.

It has been assumed by civil engineers that a plate girder reaches its limit of usefulness when any one of the individual web panels buckles. This assumption has led to extensive theoretical and experimental research into the problem of stiffened plates under combined bending and shearing stresses. The purpose of most of this research has been the determination of the magnitude of various combinations of boundary stresses at which perfectly plane plates become unstable and cease to be perfectly plane.

A design method has evolved in the United States in which the theoretically computed buckling stresses of ideal plates have been taken as the limiting stresses of the panels of plate girder webs. The girder is assumed to carry the imposed shears and moments by pure beam action and the resulting
stresses are computed by elementary beam formulae. The present specifications simply dictate a web geometry which provides an adequate factor of safety with respect to buckling for all of the panels of a plate girder web.

Aeronautical engineers have, on the other hand, taken a different approach to the design of plate girders (web spars) from that taken by civil engineers. Minimum weight design requirements have forced the aeronautical engineer to take advantage of the total strength that a girder possesses. Tests have shown that the load carrying capacity of a stiffened plate girder is not limited by the buckling of individual girder panels. After an individual panel begins to buckle, it no longer resists an increase in load by pure beam action alone, but gradually changes to a more efficient method of carrying the load. The girder begins to behave like a Pratt truss with the web panels acting as tension diagonals and the stiffeners acting as vertical compression posts. An attempt to theoretically evaluate the ultimate carrying capacity of plate girders with very thin webs was made by Wagner.\(^{(1)}\) By ignoring beam action completely and considering the thin webs as membranes that could resist tension only, Wagner formulated his pure diagonal tension theory for the load carrying capacity of girders with very thin webs. Later, Kuhn\(^{(2)}\) formulated a theory of incomplete diagonal tension in which he assumed that the girder resisted the load partly by beam action and partly by pure diagonal tension action. With this background of knowledge, the method of design in the aeronautical field became one of proportioning the girder so that it would have a required factor of safety with respect to its ultimate load.

\*Numbers in parentheses refer to entries in the list of references.
The experience of the aircraft industry and tests of individual plate girders has convinced civil engineers that more efficient use could be made of the material in plate girders if advantage was taken of the post-buckling strength of the girders. In general, girders in civil engineering structures are provided with webs that are thicker in relation to their depth and which are subjected to a higher ratio of moment to shear than are girders in aircraft structures. Consequently, theoretical and experimental work had to be carried out to adapt the theory of incomplete diagonal tension to civil engineering structures. The most comprehensive undertaking of this nature was conducted at Lehigh University from 1957 to 1961. The purpose of the investigation at Lehigh was to determine simple but general formulae to predict the ultimate capacity of girders subjected to pure shear, pure bending and combined shear and bending. This combined experimental and analytical program led to the conclusion that the stresses in plate girder web panels at working loads could exceed the theoretical buckling stresses of the panels without ill effects.

As a result of the investigation extensive revisions of the civil engineering design specifications for plate girders have been initiated. The new specifications allow larger allowable web depth to web thickness ratios than do the presently accepted specifications. Furthermore, the proposed specifications no longer base the allowable load on the buckling load of the individual web panels but on a computed ultimate capacity of the girder.

1.2 General Discussion

A new term has recently been added to the technical vocabulary of many civil engineers. This term is "thin web girder." For the purpose of this report, a thin web girder is defined as any plate girder in which the
web depth to web thickness ratio is sufficiently large so that buckling of any of the individual web panels of the girder occurs before the allowable working load of the girder is attained.

Buckling of the individual web panels of thin web girders results in lateral deflections of the web at loads which are less than the working load of the girder. If the girder is subjected to a fatigue type loading, the lateral deflections of the web result in the flexing of the web at the boundary members of the panel, i.e., the flanges and stiffeners. Pilot tests on small scale model girders have shown that this flexing action can lead to fatigue failures at a very small number of load applications. More information is needed concerning the fatigue behavior of thin web girders if they are to be used to resist cyclic loads.

In the last decade, there has been a marked increase in the use of welding for highway bridges, particularly welded girder bridges. This increase in application has resulted in a greater need for information concerning the fatigue behavior of such structures. If the proposed plate girder design specification for buildings are adopted, the next step will probably be the consideration of new specifications for welded thin web plate girders in bridges. Unfortunately, there has been no systematic study of the parameters which influence the fatigue behavior of this type of structural member.

1.3 Object and Scope

The objectives of the research program which is described herein were to determine the manner in which thin web girders fail when subjected to repeated loads, to determine what factors influence the fatigue strength of thin web girders, and to determine the manner in which these factors influence the fatigue strength.
The fatigue strength of thin web girders is affected by many parameters. It would be impossible to study the effect of all of these parameters in a program of limited scope. Consequently, the investigation reported herein has been directed toward the determination of the effect of flange rigidity and vertical stiffener rigidity. Attention has also been given to initial deflections and type of loading.

Fatigue tests of full size girders are prohibitively costly and time consuming. Consequently, it was decided to develop a method of scaling plate girders so that the desired information could be derived from tests of scale model girders.

Fifteen fatigue tests were conducted on scale model thin web all-welded girders in which the only geometrical variable was the rigidity of the flanges. Nine of these girders were subjected to a large shearing force and a very small bending moment. The test loads were chosen to approximate a condition of pure shear in the test girder. The remaining six girders were subjected to combined shearing forces and bending moments. Five additional tests were conducted on scale model all-welded girders in which the only variable was the rigidity of the vertical stiffeners. Girders in this latter series were subjected to combined shearing forces and bending moments. The results of the tests have been analyzed and the behavior of the girders compared with the behavior of a theoretical model.
2. REVIEW AND ANALYSIS OF LITERATURE

2.1 Initial Deflections

Initial lateral web deflections are expected to influence the fatigue behavior of thin web girders. The importance of this influence will most probably depend on the magnitude and pattern of the initial deflections. Initial deflection properties are a function of both the fabrication procedures and size of the individual component parts of the girder and will vary considerably from girder to girder. In particular, a welded thin web girder can be fabricated with any one of a large number of welded procedures. The residual welding stresses will cause distortion of the web panels of the girder. The magnitude and shape of the distortion will depend on the magnitude and distribution of the residual stresses. Since each welding procedure will result in a particular set of residual stresses, it follows that each procedure will also result in a particular set of initial deflections. Due to the existence of a variation in initial deflections, it becomes important to understand if this variation can result in a diversification of fatigue behavior of welded thin web girders.

Because of the small size of the individual component members, the control of the magnitude of initial deflections for scale model welded girders will be more difficult than for full size girders. Little data is available on the magnitude of initial deflections to be expected in full size girders but the data which is available indicates that initial deflections will certainly occur in this type of structural member. It is expected that relative to the over-all size of the girder, the initial web deflections in small scale model welded girders will be larger than the corresponding deflections in full size welded girders. If the fatigue behavior of full size girders is to be determined from the results of tests carried out on small scale model
girders, it is clear that an attempt must be made to determine the effects of initial web deflections on the fatigue behavior of thin web welded girders.

The first step in this determination was a literature study of plates with initial deflections. The object of this study was to define the factors which might affect the fatigue strength of thin web girders and to determine the effect of initial deflections on these factors.

Even at the very beginning of loading, the total lateral deflections of a web plate with initial lateral deflections are usually of an order of magnitude which is incompatible with the linear theory of plates. For this reason, it becomes necessary to use the non-linear theory of plates. The mathematical treatment of the buckling of plates by means of the non-linear theory is very complicated. This problem involves the solution of a system of two simultaneous partial differential equations of the fourth order. The equations take into consideration the initial deflections of the plate, the deflections of the plate due to load and an unknown stress function. Two methods have been devised for the solution of this system of equations. One method requires the assumption of a stress function which contains sufficient arbitrary constants to permit the prescribed boundary conditions to be satisfied. The other method is the energy approach due to Ritz. Both of these methods are exceedingly complicated and involve a great deal of computational work. Consequently, only a few solutions of the buckling behavior of plates with initial deflections are to be found in the literature. The solutions which are to be found are only for simple and special loading cases.

Some studies of the buckling behavior of plates with initial deflections have been reported by Marguerre,(3) Coan,(4) Yamaki(5) and Hu.(6) Marguerre(3) and Coan(4) presented the theory for the buckling of plates with
small initial curvatures. Coan\(^{(4)}\) also solved the problem of a simple supported plate with boundary conditions that are usually met in test practice, i.e., stress-free supported edges and uniformly displaced loaded edges. Curves are given for net center deflection, median fiber strain at the center of the plate, median fiber strain at mid-point of the loaded edge and extreme fiber strains at the center of the plate, each as a function of applied load. The plate was assumed to have a maximum initial deflection of one-tenth the thickness of the plate and the shape of the deflections was taken in the form of a product of two half sine waves.

In his treatment of the buckling behavior of rectangular plates with small initial curvatures, Yamaki\(^{(5)}\) considered eight different kinds of boundary conditions including two different kinds of loading conditions and four different kinds of support conditions. Concerning the loading conditions, the two opposite loaded edges were displaced uniformly in both cases. In one case, the unloaded edges were kept straight by a distribution of normal stresses the resultant of which was zero. In the other case, the unloaded edges were free of stress. Concerning the boundary conditions, the following four cases were treated: all edges simply supported; loaded edges simply supported, other edges clamped; loaded edges clamped, other edges simply supported and finally all edges clamped. The results of the calculations are presented in the form of tables of deflection coefficients for various magnitudes of load and for both initially plane plates and for plates with maximum initial deflections of 0.025h and 0.1h, where h is the thickness of the plate. Curves are also presented which relate maximum lateral web deflection, edge shortening and effective width to the imposed load.
Hu (6) treats the problem of the effect of small deviations from flatness on the effective width of buckled plates in pure compression. One of Hu's conclusions is as follows: "The effects of initial deviations from flatness upon buckle growth and effective width of plates are most marked at stresses around the theoretical flat plate critical stress; at stresses well above or below this theoretical stress, the behavior of a plate with initial deviations from flatness is very much the same as an initially perfectly flat plate."

The aforementioned theoretical solutions are concerned with plates which are subjected to loadings which differ markedly from the loadings which are imposed on the web panels of plate girders. Moreover, the solutions refer to either simply supported edges or clamped edges. In welded girders, the web plates are welded to the boundary members, i.e., the flanges and the stiffeners. In the neighborhood of the welds residual stresses are produced which are of such an order of magnitude that the yield limit can be locally exceeded even when stresses in other parts of the web are relatively small. In general, thin web plates welded to stiff boundary members are considered to have clamped boundaries. However, tests (7) on such plates have shown high rates of increase in lateral web deflection at loads which correspond to the theoretical critical load for an initially flat plate with simply supported edges. It is apparent that the boundary conditions along the edges of web plates are not strictly defined. It must be assumed that the boundary conditions vary from girder to girder and also that they may vary with the load on a given girder. Furthermore, the solutions referred to consider only plates with very small initial deflections. It is expected that actual plate girders will have larger initial deflections than are considered in
the theoretical solutions. Consequently, the foregoing theoretical solutions are only of limited value for providing information on the buckling behavior of thin web girder panels. The solutions do provide information on the effect of clamping at the boundaries on the rate of growth of lateral deflections. They also show the manner in which lateral deflections tend to increase with load for plates with small initial deflections.

A theoretical study which is more closely related to the buckling of webs in plate girders was undertaken by Bergman. Bergman solved the problem of an originally slightly curved plate with rigid supports subjected to shearing forces. The initial deflections of the plate were represented by the function

\[ w(x, y) = f_0 \sin \frac{\pi x}{a} \sin \frac{\pi y}{a} \]  

where

\[ a = \text{length of side of square plate} \]
\[ f_0 \text{ is a parameter} \]

and with the origin of coordinates taken at one of the corners of the plate. The additional deflection of the plate under load was taken as a Fourier series in three terms,

\[ w(x, y) = f_1 \sin \frac{\pi x}{a} \sin \frac{\pi y}{a} + f_2 \sin \frac{2\pi x}{a} \sin \frac{2\pi y}{b} + f_3 \sin \frac{3\pi x}{a} \sin \frac{3\pi y}{b} \]  

where \( f_1, f_2, \) and \( f_3 \) are parameters. By means of the energy method, Bergman computed values of \( f_1, f_2, \) and \( f_3 \) for the following values of the maximum initial deflection \( f_0: 0.05h, 0.50h, 1.00h, 2.00h \) and \( 3.00h, \) where \( h \) is the thickness of the plate. The results were presented in the form of curves which
show the relationship between load and maximum deflection at the center of the panel for the several values of the maximum initial web deflection \( f_0 \). These curves are reproduced in Fig. 1.

Bergman's curves show that in all cases when the plate has initial deflections, additional deflections occur as soon as the load is applied to the plate. If the initial deflections are very small, the additional deflections due to load at small loads are also quite small. When the load approaches the critical load of a plane plate, the deflections increase very rapidly. The load-deflection curves are similar in appearance to the curves of a plane plate which show a pronounced stability limit. However, if the initial deflections of the plate are relatively large, the load-deflection curves are almost linear and a sudden change in the rate of increase of the deflections corresponding to the critical load of a plane plate does not occur. Furthermore, the plates are deformed in such a manner that the total deflection surfaces, i.e., initial deflections plus additional deflections due to load, tend to assume the same shape as that obtained in the case of an originally plane plate at loads above the critical load.

It is known that when the lateral deflections of a plate become large with respect to the thickness of the plate, pure tensile stresses referred to as membrane stresses are set up in the middle surface of the plate. These stresses are caused by the restraint imposed by the boundary members to which the edges of the plate are joined. When a plate deflects laterally, its edges tend to move towards one another due to purely geometrical causes. However, if the edges are attached to fairly rigid boundary members, such as the flanges of a plate girder, they cannot deflect freely. The deflections of the edges of the plate are forced to be compatible with the deflections of the
supporting boundary members. In order to satisfy compatibility when the lateral deflections are no longer small in comparison with the thickness of the plate, elongations in some cross-sections of the plate will be produced. The membrane stresses correspond to the effect of the elongations. The magnitude of the membrane stresses, which are anchored at the edges of the plate, vary over the surface of the plate. The variation depends on the magnitude of the deflections, the shape of the deflection surface and the rigidity of the boundary supports.

The aforementioned information on the buckling of plates with initial deflections reveals two factors which should have a marked influence on the fatigue behavior of thin web girders. These factors are the lateral web deflections and the membrane stresses which result from the lateral web deflections. The lateral web deflections will cause a flexing action of the web about the boundary members of the panels with each repetition of load. This flexing action will develop localized stresses along the toes of the fillet welds which attach the boundary members to the web plates. It is expected that fatigue failures will initiate somewhere along the toe of the boundary fillet welds because of the stresses caused by the flexing action. The second factor is the tensile membrane stresses which are set up in the web when the lateral deflections become large relative to the thickness of the web. The membrane stresses are anchored at the boundaries of the web panel and are transmitted into the boundary members. In welded girders, the stresses are transferred to the boundary members through fillet welds. This stress transfer should have an effect which is analogous to the direct stressing of any fillet welded T-joint. It is expected that fatigue cracks may initiate as a result of the membrane stresses.
The effect that initial deflections will have on the fatigue behavior of thin web girders will most probably be determined by two factors. The one factor is the effect that the initial deflections have on the web flexing action at the panel boundaries. The other factor is the effect that the initial deflections will have on the membrane stresses which are anchored at the panel boundaries. The extent to which these factors will influence the fatigue behavior will depend on the magnitude of the initial deflections, the pattern of the initial deflections and the load cycle to which the girder is subjected.

In order to discuss the influence of the magnitude of initial deflections, let us consider two thin plates which are similar in all respects with the single exception of the magnitude of the initial deflections. Both plates are subjected to equal loads. Additional lateral deflections will occur in both plates. The plate with the larger initial deflections will develop greater membrane stresses than will the plate with the smaller initial deflections. Since membrane stresses have a stabilizing influence on lateral deflections, the deflections due to load of the former plate will be less than the corresponding deflections in the latter plate. Therefore, the range of lateral deflections between any two given loads is inversely proportioned to the magnitude of the initial web deflections. If the range of lateral web deflections is decreased, the flexing action of the web at the boundaries will also be decreased. Hence, this aspect of the fatigue behavior of girder panels is enhanced by large initial deflections. However, the effect of initial deflections on the magnitude of the membrane stresses must also be considered. The magnitude of the membrane stresses at any given load vary directly with the size of the initial deflections. Higher membrane stresses could offset the beneficial effect of the decrease in web flexing action.
The initial deflection pattern of a web panel may be significantly different from the theoretical shape into which the web panel would buckle if it were initially plane. If this is the case, the initial deflections will cause the web to take up a deflected configuration which is different from the theoretical configuration of an initially plane web. This alteration in the deflection configuration will result in a different distribution of membrane stresses in the web panel. The same alteration will also cause a difference in the intensity and distribution of web flexing action at the boundaries. Since web flexing and membrane stresses are expected to affect the fatigue behavior of thin web girders, the shape of the initial deflection pattern may also affect the fatigue strength by affecting both the web flexing action and the membrane stress distribution. It has already been pointed out that plates with small initial deflections of simple shape tend to deform so that the deflection surfaces assume the same shape as those obtained in the case of an originally plane plate at loads beyond the critical load. Consequently, small initial deflections of simple shape will have little or no effect on the fatigue behavior of thin web girders in which the critical loads of the individual panels are exceeded in each cycle of loading.

The effect of the initial deflections on the fatigue strength of thin web girders will also depend on the loading cycle to which the girders are subjected. For instance, it can be seen from Fig. 1 that if both the maximum and minimum loads of the loading cycle are well above the theoretical buckling load, web panels with a wide range of magnitudes of initial deflections behave similarly with respect to lateral web deflections. However, if the loading cycle is a zero-to-tension cycle with the maximum load greater than the theoretical critical load, web panels with different magnitudes of
initial deflections can behave quite differently with respect to lateral deflections.

In summary, it is evident that initial web deflections will have an effect on the fatigue behavior of thin web girders. Initial deflections are of importance because of their influence on the web flexing action and on the membrane stresses in the web. The effect is very complicated, not well defined, and varies from girder to girder with the fabrication procedures and type of load.

It is of importance to know the magnitude of initial web deflections which can be expected in full size welded girders. With this information, a comparison can be made between the size of the initial deflections of the model girders with the size of similar deflections that can be expected to exist in full size girders.

Two extensive series of tests on thin web welded girder panels have been reported in the literature. The details of the test panels and the maximum value of initial deflections are reported in Tables 1 and 2. One series of tests was conducted by Wästlund and Bergman\(^{(7)}\) in 1942 and 1943 at the Royal Institute of Technology in Stockholm. The other series of tests was conducted by Basler, Yen, Mueller and Thürlimann at Lehigh University\(^{(9)}\) during the period 1958 to 1961. The ratio of maximum initial deflection amplitude to web thickness for the various test panels varied from a minimum of 0.3 to a maximum of 3.2. The magnitude of the initial deflections reported in Table 1 vary directly with both the web depth to web thickness ratio and the aspect ratio of the web panel. The initial web deflections reported in Table 2 do not exhibit as good a correlation with \(\alpha\) and \(d/t\) as do those in Table 1.
It must be considered that the girder panels described in Tables 1 and 2 were carefully prepared for laboratory tests. Some girders fabricated with normal shop care will have larger initial web deflections than the specimens in Tables 1 and 2. It is estimated that girders fabricated with web depth to web thickness ratios in the range of 250 to 320 may have initial web deflections which are as large as 2.5 times the thickness of the web.

2.2 Effect of Flange Stiffness

The development of membrane stresses in buckled plates with rigid boundary members has been noted in section 2.1. These membrane stresses are anchored at the boundaries of the plate. If the supporting boundary members are not infinitely rigid, they will deflect toward one another when acted upon by the membrane stresses. This behavior will result in an increase in the magnitude of lateral deflections of the plate. It has been stated that the fatigue behavior of web panels will be influenced by the magnitude of the lateral plate deflections. Hence, it follows that the rigidity of the flanges of a web panel of a plate girder will influence the fatigue behavior of the girder.

In the theory of the pure diagonal tension field as developed by Wagner\(^{(1)}\) it is assumed that the plate which constitutes the web of a girder has no bending stiffness. Under the action of shearing forces, the web forms a large number of narrow and shallow folds inclined at an angle to the flanges. The state of stress in the sheet is pure tension in the direction of the folds. Under these conditions, Wagner showed that the tensile stress in the web decreases with an increase in rigidity of the flanges. The tensile stress in the web was given by Wagner as

\[
\sigma = 2 \frac{S}{d \cdot t \cdot c_2}
\]  

**(3)**
where $\sigma =$ tensile stress in web
$S =$ shear force on web
$d =$ depth of beam
$t =$ thickness of web

$C_2 =$ stress concentration factor which depends on the flexibility of the flanges.

Wagner's values of $C_2$ are presented in Fig. 2. It was also shown that the secondary bending moments in the flanges decrease as flange thickness is increased. The secondary moments are caused by the vertical component of the diagonal tension and are expressed as follows:

$$M = \frac{k_m C_1 S b^2}{d}$$

(4)

where $M =$ secondary flange bending moment

$k_m = 1/12$ over uprights and $1/24$ midway between uprights

$C_1 =$ factor which depends on flange flexibility

$b =$ stiffener spacing.

Values of $C_1$ from Wagner's paper are reproduced in Fig. 2.

Practical experience showed that, in many cases, the theory of pure diagonal tension was too conservative. In order to overcome this conservatism, Kuhn (2) formulated his theory of the incompletely developed plane diagonal tension field. Kuhn understood that as the shear force on a beam web is increased beyond the buckling load, the compressive stresses which correspond to the shear do not vanish suddenly and completely. Some compressive stresses continue to exist and this diagonal compression combines
with part of the diagonal tension into a shear stress. The imposed shear force is resisted partly by pure shear action and partly by diagonal tension action. As the shear force increases, the relative importance of the compressive stresses decreases and the state of pure diagonal tension is approached, but never reached. Even with this different understanding of the action of webs of shear girders, Kuhn retained Wagner's expression for the tension stress in the web. Furthermore, his expression for secondary bending moments in the flanges was very similar to the expression which was presented by Wagner. In two tests which Kuhn conducted, it was discovered that the secondary moments were higher between the uprights than at the uprights. This behavior was explained by the stiffening effect of the web on the flange in the corners of the panels. A single moment coefficient, \( k_m = 0.10 \), was chosen to represent the maximum secondary bending moments in the flanges. It was understood that these moments occur between the stiffeners but that the moments over the stiffeners may sometimes be practically as large as the maximum moments. Kuhn also substituted \( S_{DT} \), the portion of the shear that is carried by diagonal tension action, for \( S \) which appears in Wagner's equation for secondary flange bending moments.

Legett (10) noticed that the test results obtained by Kuhn suggested that the variation in maximum flange bending moment was small over a wide range of flange stiffnesses. He felt that this observation was important enough to warrant theoretical confirmation. It is very difficult to do this for the general case in which the shear in the web is not necessarily far removed from its buckling value. Therefore, Legett set out to obtain a theoretical check for the limiting case in which the web is highly buckled and is acting as a Wagner tension field. By means of the energy method, relationships between flange stiffness and maximum flange bending moment
were developed. Legett concluded that throughout the practical range of flange stiffnesses, the variation in the maximum flange bending moment is small.

The three foregoing investigators were working in the aircraft industry when they developed their theories of shear girder action. The theories were developed for girders with much thinner webs than those which are provided for civil engineering girders. Consequently, the results cannot be applied directly to civil engineering structures. However, it is reasonable to conclude that the flange stiffness may have an effect on the magnitude and distribution of membrane stresses in the girder web. In girders with very thin webs subjected to shear, an increase in flange stiffness results in a decrease in the magnitude of membrane stresses. It is felt that this may be true, but to a lesser degree, in civil engineering girders. Hence, following the reasoning of Section 2.1 of this report, the flange stiffness may influence the fatigue behavior of girders by influencing the magnitude of the membrane stresses in the webs.

It is not expected that the secondary bending moments in the flanges will be significant in civil engineering girders.

Bergman attempted to determine to some extent the effect of flexible boundary members. To this end, he solved the problem of an originally plane square plate with elastic supports and subjected to shear. In the problem considered, the boundary conditions are dependent not only on the change in angle between the lines connecting the corners of the plate, but also on the elastic deformations of the supports of the plate. These deformations occur in the plane of the plate and are caused by the membrane stresses. Bergman chose to express the deflections of the four elastic supports by a simple sine function. This function was zero at the corners of the plate and took on its maximum value mid-way between the corners.
Results are presented for the two limiting cases in which the ratios of the flexural rigidity of the supports, $E_l$, to the flexural rigidity of the plate, $D_b$, are zero and infinity respectively. The results are reproduced in Fig. 3. When interpreting these results, it must be considered that Bergman stipulated a compatibility condition for the edges of the plate so that the supports force the edges of the buckled plate to take the form of a half sine wave. This condition cannot hold true in the case of a rigorous solution for a plate with supports which have no flexural rigidity. Therefore, it is doubtful that Bergman's results can be applied to those cases when an actual plate has supports of very low stiffness in relation to the web stiffness.

An experimental investigation conducted by K. C. Rockey(11) on light alloy girders and shear panels indicates that the effect of flange stiffness on lateral web deflections may be more pronounced than is indicated by Bergman's theoretical solution. Rockey found that the depth of the buckles which are formed in the web are dependent upon the load ratio $W/W_{cr}$ and the flange stiffness parameter $I/b^3t$ where

$$W = \text{actual load on web panel}$$

$$W_{cr} = \text{theoretical buckling load of web panel}$$

$$I = \text{moment of inertia of flange about its own centroidal axis, in.}^4$$

$$b = \text{stiffener spacing, in.}$$

$$t = \text{thickness of web, in.}$$

Rockey's experimentally determined relationship between these three variables is given in Fig. 4. The curves show that for a given load ratio, $W/W_{cr}$, a reduction in flange stiffness below a given value (marked by a circle on the curves) results in a rapid increase in the lateral deflection of the web plate.
In summary, theoretical and experimental investigations show that the stiffness of the flanges may have two important effects on the behavior of the panels of thin web girders. Flange stiffness can affect the magnitude and distribution of membrane stresses in the web panel. It can also affect the magnitude of lateral web deflections. Both membrane stresses and lateral web deflections are expected to influence the fatigue behavior of thin web girders. Therefore, flange stiffness is also expected to influence the fatigue behavior of thin web girders.

2.3 Effect of Vertical Stiffener Rigidity

Theoretical and experimental work has shown that the static behavior of webs of plate girders is dependent upon the rigidity of the intermediate vertical web stiffeners. For this reason, it seems reasonable to expect that the fatigue behavior of girders might also be dependent upon the rigidity of the intermediate vertical stiffeners. In order to better understand the manner in which the static behavior of girders is influenced by the stiffeners and how the fatigue behavior might be related to the static behavior, a review of previous theoretical and experimental work has been carried out.

Most of the theoretical investigations reported in the literature, which deal with the effect of vertical stiffeners, are concerned with the effect of stiffener size and spacing on the buckling coefficients of ideal plate-stiffener combinations. No solutions are available for plate-stiffener combinations in which there are initial imperfections. Two different types of plate-stiffener combinations have received a great deal of attention in the literature.

One type of plate-stiffener combination which has been considered is one in which the stiffeners are quite weak but closely spaced. It is
assumed that the stiffeners stiffen the plate in one direction but have no effect on the stiffness in the direction normal to the stiffeners. The problem thus reduces to the study of the buckling of an orthotropic plate. The stiffeners remain straight up to the buckling load and then buckle with the plate. Solutions to this type of problem have been provided by Schmeiden, \(12\) Seydel \(13\) and Wang \(14\). The relationship between buckling coefficient and stiffener stiffness which was obtained by Wang \(14\) is reproduced in Fig. 5. The above described behavior is not representative of the behavior of webs of civil engineering girders. In such girders, the stiffeners are usually quite rigid. Moreover, they are not necessarily closely spaced. Consequently, the results of the orthotropic plate type of solution are of no value to the investigation described herein.

In the other type of plate-stiffener combination that has received theoretical attention, the webs are stiffened by strong stiffeners which are not necessarily closely spaced. Concerning this problem, Timoshenko \(15\) and Way \(16\) used the energy method to derive an approximate solution for simply supported finite rectangular plates with one or two transverse stiffeners. The plates were subjected to shear. Because of the choice of deflection function, both investigators were able to show that by increasing the flexural rigidity of the stiffeners, a stage would be reached when the stiffener would remain straight after the plate had buckled, thus forming a nodal line. The minimum flexural rigidity which stiffeners should possess if they are to remain straight and supply panels with simple pinned boundary supports was calculated. These results are reproduced in Fig. 5. Wang \(14\) extended Timoshenko's theory to plates reinforced by any number of transverse stiffeners and for infinitely long plates.
Stein and Frahlich\(^{(17)}\) presented a theoretical solution for the critical shear stress of an infinitely long simply supported flat plate with identical, equally spaced transverse stiffeners. The stiffeners were considered to have zero torsional rigidity. In this paper, a study was made of the effect of the choice of deflection expression on the magnitude of the buckling coefficients that are derived from the energy solution. The study showed that an improper choice of deflection expression will lead to buckling loads well above the true minimum value. By taking the boundary condition such that plate and stiffener deflection must be equal and by choosing the proper deflection function, Stein and Frahlich\(^{(17)}\) arrived at accurate relationships between the buckling coefficient and stiffener rigidity for three different stiffener spacings. The relationship for a girder with panels of aspect ratio 2 is reproduced in Fig. 5. The points of discontinuity in the curve represent changes in the buckle pattern. Included in the same figure are values of buckling coefficients that were computed by Timoshenko\(^{(15)}\). The solution of Stein and Frahlich\(^{(17)}\) showed that the earlier theoretical work of Timoshenko\(^{(15)}\) and Way\(^{(16)}\) overestimated the value of the critical shear stress coefficient obtained for stiffeners of intermediate rigidity and underestimated this coefficient for fairly rigid stiffeners.

Kleeman\(^{(18)}\) and Cook and Rockey\(^{(19)}\) solved the problem of the buckling of both clamped and simply supported infinitely long rectangular plates with stiffeners of considerable torsional rigidity. The plates were subjected to shear. The solutions show that the effect of the torsional rigidity of open section stiffeners is small enough to be neglected. However, the torsional rigidity of closed section stiffeners results in a significant increase in the buckling coefficient. Cook and Rockey's curve for a plate with double sided
stiffeners of semi-circular cross-section and simply supported edges is given in Fig. 5. The effect of the torsional rigidity on the magnitude of the buckling coefficient is immediately evident from the comparison with the Stein and Frahlich\(^{(17)}\) results.

The conditions to which plate-stiffener combinations are subjected in welded plate girders are markedly different from those assumed in theoretical solutions. The boundaries of the individual web panels are neither clamped nor simply supported. The individual panels are subjected to bending as well as shear stress and buckle into patterns which are much different from those assumed in theoretical solutions. The web and stiffeners have deflections under zero load which are due to residual welding stresses and plate imperfection. Consequently, there are no pronounced stability limits but deflections increase slowly as the load is increased. It has been necessary to conduct experimental investigations in order to correlate the actual behavior of girders with the theoretically predicted behavior.

Only a few experimental investigations have been concerned with the effect of stiffener dimensions on the stability of web plates of girders. In 1942, Moore\(^{(20)}\) reported on tests of two aluminum alloy girders. He was unable to obtain values of the experimental buckling load and resorted to determining test loads corresponding to an arbitrary constant value of web or stiffener deflection. Moore produced the empirical design formula

\[
\frac{EI}{Db} = \frac{14}{(\frac{b}{d})^3}
\] (5)
where \( E \) = Young's modulus of elasticity

\[
1_s = \text{required flexural rigidity of stiffeners}
\]

\[
D = \text{flexural rigidity of unit width of web panel}
\]

\[
b = \text{stiffener spacing}
\]

\[
d = \text{clear depth between flanges}
\]

Stiffeners providing the value of \( 1_s \) required by Eq. (5) insure that at loads corresponding to the theoretical buckling loads for panels with partially (50 per cent) clamped edges, the web plate and stiffener deflections will be small. Scott and Weber\(^{(21)}\) reported on tests of stiffened panels which were subjected to combined compression and shear. They concluded that an effective stiffener requires a flexural rigidity greater than that predicted by the theoretical solution of Timoshenko.\(^{(15)}\) In an extensive program of research conducted on plate girders, Sparkes\(^{(22)}\) found that vertical stiffeners having a flexural rigidity equal to twice Timoshenko's theoretical value failed to divide the web plate into separate panels.

K. C. Rockey\(^{(23)}\) conducted a comprehensive series of tests on stiffened web plates subjected to shear. The tests were conducted on bolted carefully constructed girders of high strength aluminum alloy. Very well defined stability limits were realized. Altogether, 220 different plate stiffener combinations were tested of which 125 involved single sided stiffeners. It was shown that there is a certain limiting value of stiffener rigidity above which an increase in stiffener rigidity had little effect on the magnitude of the buckling stress of the web panel. Rockey derived empirical relationships between the value of the limiting stiffener rigidity and the aspect ratio of the panel. Different formulae were presented for single and double sided stiffeners. In Fig. 6, Rockey's formula for double
sided stiffeners is presented in graphical form along with Moore's empirical formula (Eq. 5), Timoshenko's (15) theoretical results and two results obtained by Stein and Frahlich (17). It can be seen in this figure that experimenters such as Rockey and Moore found that stiffener rigidities greater than the theoretical values of Timoshenko (15) are required for adequate web behavior in actual girders.

It can be stated that both theoretical and experimental investigators have felt that the main purpose of vertical stiffeners is to divide the web of a plate girder into a number of panels. In this way, the resistance to buckling of each of the individual panels can be made greater than the buckling resistance of the unstiffened web. This purpose is most effectively accomplished by stiffeners which remain absolutely straight and completely isolate adjacent panels. When the stiffeners remain straight and the web panels buckle, the web deflects laterally in the center of the panel but remains straight at the stiffened boundaries. This behavior causes the web to flex about the stiffener. However, in actual girders the stiffeners have initial deflections which cause the stiffeners to deflect with the web, but to a lesser extent than the web. The amount of web flexing action at the stiffened boundaries will be related to the amount that the web deflects relative to the stiffener.

When the stiffener deflects laterally, it also allows some of the membrane stress in one adjacent panel to be transferred to the other adjacent panel. Consequently, the amount of membrane stress that is transferred through the stiffener welds into the stiffener will be related to the magnitude of the lateral stiffener deflections.
The magnitude of the lateral deflections of a vertical stiffener is related to the relative flexural rigidity of the stiffener and web panel. Consequently, the intensity of web flexing action and the amount of membrane stress transferred through the stiffener welds is dependent upon the relative stiffener rigidity. Hence, the fatigue behavior should also be influenced by this same stiffener rigidity.

2.4 A Qualitative Analysis of the Fatigue of Webs Subjected to Shear

A qualitative analysis of the behavior under load of ideal shear web panels reveals the manner in which this type of structural element can be expected to fail in fatigue. At the beginning of loading, the web carries the load in pure shear. The principal stresses are oriented at an angle of 45 deg. with the horizontal and are equal throughout the middle surface of the web. As the shear load is increased past the theoretical buckling load of the panel, the compressive stresses cause the web to buckle. The longitudinal axes of the buckles are perpendicular to the compressive stresses. As the load is further increased, the web continues to buckle and the compressive stresses increase only a small amount. However, the principal tensile stresses in the neighborhood of the buckle increase rapidly with increase in load. The magnitude of the tensile stresses is a maximum at the crests of the buckles.

Concerning the pattern into which the web buckles, a web which is nearly square in shape will deflect into one large diagonal buckle which extends from one corner of the panel to the diagonally opposite corner. A contour plot of a fictitious shear web which has buckled in this manner is given in Fig. 7a. If the depth to width ratio of the panel is increased,
the buckling pattern will change from one consisting of one buckle to one consisting of two buckles. A buckle in the upper portion of the web will originate in one of the panel corners with its axis oriented at an angle of 45 deg. with the horizontal. A buckle in the lower portion of the web will originate in the diagonally opposite corner to that in which the upper buckle originates. The axis of this latter buckle will be parallel to the axis of the upper buckle. A contour plot of a fictitious shear web which has buckled in this manner is shown in Fig. 7b. As the depth to width ratio of the panel is increased, more and more buckles will form until the number of buckles becomes infinite in an infinitely long plate. In all cases, the diagonal tension stresses are oriented parallel to the longitudinal axis of the buckles.

Shear webs which have initial deflections of small magnitude and simple pattern will behave in a manner very similar to that described above. The one major difference will be the absence of a stability limit in the shear webs with initial deflections. It is impossible to predict the behavior of shear webs with initial deflections of large magnitude and complicated pattern. The behavior will be dependent upon the type of initial deflections.

With an understanding of the behavior of web panels subjected to shear, it can be predicted where fatigue failures may initiate. In panels which form only one diagonal buckle, the failures will most probably occur in the corners of the panel through which the longitudinal axis of the buckle passes. The magnitude of the membrane stresses which are anchored at the boundaries of the web panel will be a maximum in these corners. It has already been explained how membrane stresses such as the diagonal
tension stress will initiate fatigue failures along the toes of the boundary fillet welds.

It is not anticipated that the maximum flexing action of the web will occur at the aforementioned failure locations. The flanges and stiffeners should minimize the lateral web deflections in the corners of the web panel. Hence, it is felt that there will not be a great deal of flexing action at these locations.

In shear panels which deflect into two or more buckles, the failures will probably occur along the toe of the vertical stiffener fillet welds. The failures will most probably initiate at the points where the longitudinal axes of the web buckles intersect the toe of the fillet welds. The diagonal tension stresses which are anchored at the panel boundary should be a maximum at these locations. Furthermore, since maximum web deflections occur at the crests of the buckles, it is also expected that the aforementioned locations will also be the position of maximum web flexing action.

2.5 A Qualitative Analysis of the Fatigue of Web Panels Subjected to Shear and Moment

A qualitative analysis of ideal thin web girder panels which are subjected to shear and moment stresses reveals the manner in which this type of girder panel can be expected to fail in fatigue. At the beginning of loading, the web resists the load by pure beam action. Figure 8 has been prepared to show the magnitude and orientation of the principal compressive stresses in a particular web panel which resists the applied loads by pure beam action. The web panel, which has an aspect ratio of 2, is subjected to a constant flexural stress of 17 ksi and a constant shear stress of 10.7 ksi.
This panel is identical both in geometry and loading to the test panels of the series of girders in which the effect of stiffener rigidity is to be studied. The width of the heavy dark lines is proportional to the magnitude of the principal compressive stress. The direction of the lines gives the orientation of the same stress. As the load is increased, the theoretical buckling load is exceeded and the compressive stresses cause the web to buckle. The web buckles first where the compressive stresses are maximum, i.e., in the upper portion of the web. As the load is further increased, the web continues to deflect laterally and the shape of the buckle becomes elongated in a direction which is perpendicular to the principal compressive stresses. The principal tensile stresses in the region of the buckle increase rapidly as the load is increased and are a maximum at the crest of the buckle the orientation of which will depend on the ratio of flexural stress to shear stress. As this ratio is increased, the slope of the crest of the buckle will be decreased.

If the aspect ratio of the web panel is sufficiently large, another buckle will form in the lower portion of the web. The principal compressive stresses in this part of the web are diminished by the tensile bending stresses. Hence, the buckle which will form in the lower part of the web will be smaller than the buckle which develops in the upper part of the web. Moreover, the diagonal tensile stress which is associated with the lower buckle will be smaller than the corresponding stress which is associated with the upper buckle. A contour plot of the anticipated buckling pattern of the web panel of Fig. 8 is given in Fig. 9.

It is now possible to predict where fatigue failures will most likely occur in web panels which are subjected to combined shear and bending. It is felt that fatigue cracks will originate along the toe of the vertical
stiffener fillet welds at the location where the longitudinal axis of the upper web buckle intersects the toe of the fillet weld. Failures will initiate at this location for exactly the same reasons as were given for shear webs which develop more than one web buckle. The probable failure location is illustrated graphically in Fig. 9.
3. SCALING PARAMETERS

A welded plate girder consists of three basic components: flange, web and stiffeners. The web is considered and designed as the primary shear resisting element while the flange is considered and designed as the primary moment resisting element. The stiffeners divide the girder into a series of web panels for which the two flanges and two adjacent stiffeners act as supporting boundary members. The behavior of a girder, when subjected to load, is determined by the behavior of the individual panels of the girder. The behavior of the individual panel is determined by the relative rigidities of the three girder components, and these rigidities are evaluated in terms of the geometrical properties of the individual components. In order for the buckling behavior to be comparable the same relations between the properties must be maintained in both the model and prototype. Hence, the behavior of girders should be related to a set of non-dimensional parameters which take into account the properties of the various girder components.

Theoretical buckling solutions of the problem of a shear panel with elastic edge members have shown that the behavior of the panels depends on two dimensionless parameters. The formulation of these parameters has been different in the various solutions which have been presented in the literature. However, as will be shown in the following discussion, the different formulation used are all essentially the same. Bergman\(^7\) used the parameters \((t/a)^2\) and \(l/bt^3\) in his solution of a single shear panel where

\[
\begin{align*}
t & = \text{thickness of web, in.} \\
a & = \text{clear depth of web between flanges, in.} \\
b & = \text{stiffener spacing, in.} \\
l & = \text{moment of inertia of flange about its horizontal centroidal axis, ins}^4
\end{align*}
\]
Legett(10) used the parameters $A/tb$, $b/a$ and $1/b^2A$ in his solution of a web spar subjected to shear where $t$, $a$, $b$, $l$ are as defined above and

$$A = \text{area of stiffener, in.}^2$$

In the tests to determine the effect of flange thickness that are reported herein, the stiffener size and spacing are held constant. Consequently, the area of the stiffener can be expressed in the form of a proportion of the area of the web on a horizontal plane between stiffeners, or $A = Ctb$ where $C$ is the appropriate constant. This form reduces Legett's parameters to $C$, $b/a$ and $1/Cb^3t$ or, since $C$ is a constant, to $b/a$ and $1/b^3t$. If Bergman's parameters are multiplied by the dimensionless ratio $(t/b)^2$ the same parameters $b/a$ and $1/b^3t$ result.

Tests(8) have shown that the in-plane web stresses cause the flanges of a thin web plate girder to move together in the postbuckling range of loading. It is anticipated that this movement will be significant in determining the fatigue behavior of thin web girders. The portion of the flange between two adjacent stiffeners must act as a beam with unknown end restraints loaded by an unknown distributed load. This load arises from the in-plane web stresses. By elementary beam theory, the deflection of such a beam is proportional to $Wb^3/EI$ where $W$ is the total load on the beam. Since $E$ is the same for all steels and $W$ is a function of the web thickness and the membrane stresses the lateral deflection of the flanges is a function of the parameter $b^3t/l$ or the inverse $1/b^3t$.

In view of the foregoing it was concluded that the effect of flange size on the fatigue strength of thin web girders would be related to the parameter $1/b^3t$. In order to test this hypothesis, a series of
fatigue tests was conducted in which the value of $1/b^3 t$ was varied from $2.58 \times 10^{-5}$ to $159 \times 10^{-5}$. The lower value represents a much more flexible flange than would normally be used in plate girder design. The upper value represents a much stiffer flange than would normally be used in plate girder design. By conducting tests over a wide range of $1/b^3 t$ values, it was intended to show the relationship between $1/b^3 t$ and fatigue behavior over the whole practical range of relative flange rigidities.

The static requirements of vertical stiffeners are twofold. First, it is necessary that there be sufficient rigidity so that the stiffeners remain straight up to the ultimate load of the girder. Second, there must be sufficient strength so that the stiffeners act as vertical compression members in an incompletely developed diagonal tension field. Insofar as the stiffeners are concerned the fatigue strength of thin web girders will be influenced only by the rigidity of the vertical stiffeners since it is anticipated that the axial stresses in the vertical stiffeners will be small at working loads.

Theoretical solutions\(^{14,15,16,17}\) have shown that the effect of stiffeners on the behavior of girders can be related to the ratio of the rigidity of the vertical stiffeners to the flexural rigidity of the web panel. This ratio is described by the parameter

$$\frac{E_{ls}}{D_b}$$

where

- $E$ = Young's modulus of elasticity
- $I_s$ = Moment of inertia of the vertical stiffener or stiffeners about a horizontal axis in the central plane of the web
- $D = \text{Flexural rigidity of unit width of web plate, } \frac{E t^3}{12(1-\mu^2)}$
- $b = \text{Stiffener spacing.}$
When the value of this parameter exceeds a certain minimum value, girders which are similar except for vertical stiffening will behave in a similar manner up to the theoretical buckling load of its web panels. As an extrapolation of this result, it seems logical that there should be a minimum value of vertical stiffener rigidity which should cause the same girders to act in a similar manner up to the ultimate load. The above considerations led to the conclusion that if vertical stiffeners were to have any effect on the fatigue strength of thin web girders, this effect would most probably be related to the ratio of $E_s/Db$. It is this parameter which has been chosen to scale the stiffener size of thin web plate girders.

The final girder element which must be scaled is the web. The webs of plate girders are described by the parameters $\alpha$ and $\beta$ where

$$\alpha = b/a$$

the aspect ratio of a web panel.

$$\beta = d/t$$

the ratio of web depth to web thickness.

These parameters are dimensionless ratios which provide a basis for the scaling of the web panels.
4. TEST SPECIMENS

4.1 Girders with Various Flange Sizes Subjected to Shear

The shear girders were fabricated with a single central test panel having a d/t ratio of 267 and an aspect ratio of 1.33. The d/t value of 267 was also used in two fatigue tests of full size girders which were conducted at Lehigh University. With the available web material having a thickness of .075 in., the depth and width of the test panel were set at 20 in. and 15 in. respectively. The two panels which flanked the test panel were fabricated with aspect ratios of only 0.3 in order to be stronger than the test panel (See Fig. 10). The double sided stiffeners were cut from 2 in. by 1/8 in. bar stock. It was felt that these stiffeners would provide a stiffness well in excess of any minimum requirement and would not unduly influence the fatigue lives of the girders. The flange width was set at 5 in. and the flange thickness was varied from 1/4 in. to 1 in. in increments of 1/8 in., except that no tests were conducted on specimens with a flange thickness of 7/8 in.

4.2 Girders with Various Flange Sizes Subjected to Shear and Moment

The girders with varying flange sizes which were subjected to both shear and bending were fabricated with two adjacent test panels (See Fig. 11). A d/t ratio of 267 and an aspect ratio of 0.75 were chosen since these were the values that were used in the flange thickness tests on the shear girders. With a web thickness of 1/16 in., the web depth was set at 16 5/8 in. and the stiffener spacing at 12 1/2 in. It was desired to vary the flange stiffness from specimen to specimen without changing the flexural stress. Since the shear stress requirements dictated the same test load for all specimens, it was necessary to decrease the flange width as the flange
thickness was increased. In accordance with the above requirements, the following flange sizes were used on the test specimens: 4 3/8 in. by 1/4 in., 2 7/8 in. by 3/8 in., 2 1/8 in. by 1/2 in., 1 5/8 in. by 5/8 in., 1 3/8 in. by 3/4 in., 1 in. by 1 in. The stiffeners were chosen to comply to the AISC building specification requirements for thin web girders.

4.3 Girders with Various Stiffener Sizes Subjected to Shear and Moment

The stiffener test girders were fabricated with three adjacent test panels which were stiffened by two sets of double sided test stiffeners. A d/t ratio of 320 was chosen for the test panels since 320 is the highest value that the specifications allow for mild steel girders. This decision fixed the depth of the web at 20 in. The results of the tests on the first series of shear girders indicated that it was desirable to use an $\frac{1}{b^3}t$ value which was at least as great as, and preferably greater than, the value proposed by K. C. Rockey as derived from his static tests on shear panels. It was also desirable to maintain a reasonable flange width in view of fabrication considerations. Once a flange thickness was chosen, its width was governed by the required bending stress. In order to subject the panels to the maximum allowable stress conditions, it was decided to linearly reduce the width of the flange in the direction of decreasing moment in order that both the bending and shear stress would be constant over the length of the test panels. The above considerations led to the use of a 1/2 in. thick flange which was tapered in width from 4.5 in. to 2 in. over the length of the test section. After the flange thickness had been chosen, the desired value of $\frac{1}{b^3}t$ put an upper limit on the allowable stiffener spacing. A stiffener spacing of 10 in. was chosen. Consequently, the test section was 30 in. long and consisted of three adjacent panels with a d/t ratio of 320 and an aspect ratio of 0.5 (See Fig. 12).
5. MATERIALS, FABRICATION AND TEST PROCEDURE

5.1 Materials

Three different types of materials were used in the fabrication of the test girders including two different web materials and one flange material. The webs of the shear girders with varying flange thickness were taken from 6 by 10 ft. plates of .0747 in. thick A366-58T commercial quality cold rolled carbon sheet steel. The webs of the remaining girders were taken from 2 by 12 ft. plates of .0625 in. thick A245-61T flat rolled carbon steel sheets of structural quality. The flanges of the girders were flame cut from 6 by 12 ft. plates of A373-54T steel plates. The physical properties of the web materials are given in Table 3.

5.2 Fabrication Procedures

The test specimens were formed by bolting sturdy girder sections to each end of a short thin web test specimen. The test loads were applied to the end sections and were transmitted to the test section through the bolted connections. This method of fabrication eliminated the possibility of failures due to concentrated stresses at the load and reaction points and also eliminated the need for a complicated lateral support system.

Three different fabrication procedures were used for the three different series of tests. In all cases, fabrication was begun by clamping the flanges and web in their proper relative positions in positioning stands. The stands allow the specimen to be rotated freely so that all welding can be done in the downhand position. The girder components are then tack welded to hold them in place during the welding of the specimen.

The welding procedure for the three test series is shown in Fig. 13. The procedure is a back-stepping type with short passes used to balance the
heat input as much as possible. As soon as sufficient weld passes are deposited to attach the flange to the web past the point where a vertical stiffener is to be attached, that vertical stiffener is welded into place. This stiffener then provides additional support for the web while fabrication is continued.

The shear girders with the varying flange thicknesses were fabricated longer than the required test length. Stiffeners were welded to the web near the ends of the beam to prevent excessive buckling of the ends of the web which results from residual welding stresses. The girders were cut to length by removing the end portions. This procedure resulted in a flat end web section in which holes could be drilled to effect a web splice.

The girders with the varying flange size, which were subjected to shear and bending, were fabricated to the correct test length. This procedure did not result in as satisfactory a specimen as did the procedure used in fabricating the shear girders.

Two 1 in. thick cold rolled plates were clamped between the rotating stands to form a solid base to which the stiffener test girders could be clamped during welding. The girders were placed between the cold rolled plates with each flange clamped to the plate along its entire length. Double sided heavy stiffeners were welded to the flanges and web at both ends of the girder. The girder was then welded using the welding sequence shown in Fig. 13. The purpose of the cold rolled bars and heavy end stiffeners was to provide solid boundary members to help keep the web plane during welding. The specimens were finally cut to length and the holes were drilled to receive the splice bolts.
5.3 **Test Procedures**

After the specimens were placed in the 250,000 lb. Wilson lever-type fatigue machines, web deflection measurements were taken before the specimen was subjected to any load. In this report, these deflections are called initial web deflections. These deflection measurements were made by means of a jig which could be attached to the girder and placed in a vertical plane. This jig allowed a dial gauge to be slid over the surface of the web in order to measure the distance from the plane of the jig to the web over a specified grid. After the specimen had undergone a few thousand cycles of loading, the deflections of the web were measured when the web was under load.

The test load for the shear girders was determined by computing the allowable shear stress from the new AISC building specifications and using this value as the maximum test shear. A stress ratio of one-quarter was chosen for the loading cycle. The nominally computed shear stress ranged from 2.7 ksi to 10.8 ksi during each loading cycle.

In choosing the loading cycle for the flange thickness tests for which the web panels were subjected to both shear and bending, it was anticipated that bridge specifications for thin web girders might be very similar to the present AISC building specifications with the exception that the allowable stresses would be reduced by 2/13 for shear stresses and 2/20 for flexural stresses. Furthermore, it was decided that a reasonable factor of safety against failure by fatigue is 1.33. In view of this reasoning, it was decided to subject the test specimen to 1.33 times the anticipated allowable thin web girder stresses of future bridge specifications. As in the case of the shear girders, the loading cycle was chosen as one-quarter load to full load. The shear stress was constant across the two test panels at a
nominal maximum value of 10.9 ksi. The flexural stress varied from zero at the outside edge of one test panel to a nominal value of 14 ksi at the outside edge of the adjacent test panel.

Since the fatigue lives measured in the above flange size shear and bending tests were very short, it was decided to use a reduced test load in the stiffener tests. As in the case of the shear girders, the allowable stresses of the new AISC building code were chosen to dictate the maximum test load. Again, a stress ratio of one-quarter was used to determine the loading cycle. The test section consisted of three adjacent panels which were subjected to a constant shear stress and a constant flexural stress. The nominal maximum shear and bending stresses were 10.7 ksi and 16.2 ksi, respectively.

Measurements of strain were made on some of the girders. In the shear girders, a strain rosette was placed on either side of the test panel at the geometric center of the panel. Measurements of strain were taken at equal increments of load from zero to the maximum test load. In the stiffener tests, measurements of stiffener strain were made at mid-height of the stiffener at loads up to the test load. After the specimen failed in fatigue, the crack was welded and the girder was tested to its ultimate load. In some specimens, the stiffener strains were obtained up to the ultimate load. In addition, strain rosettes were placed at two locations on the upper part of the web of one of the specimens in an attempt to determine the approximate magnitude and direction of the diagonal tensile stress.
6. TEST RESULTS

6.1 Girders with Various Flange Sizes Subjected to Shear

The results pertinent to the static behavior of shear panels with varying flange sizes are given in Table 4. Two different patterns of initial web deflection were observed. The two patterns are illustrated in Fig. 16 where the initial web deflection contour plots for girders FT-3 and FT-4 are presented. Of the girders listed in Table 4, girders FT-3 and FT-9 were the only specimens to exhibit an initial web deflection pattern similar to that shown in Fig. 16 for FT-3. The other girders, FT-1, FT-2, FT-4, FT-6 and FT-10 all exhibited initial web deflection patterns similar to that of FT-4. The locations of the maximum values of initial web deflections reported in Table 4 occurred at the apexes of the buckles in the central portion of the web.

Several interesting aspects of the web deflection patterns under maximum load were observed. In general, the pattern was dominated by a long thin central buckle as shown in Fig. 17. The crest of the buckle formed an angle of 45 deg. with the horizontal. The amplitude of the central buckle decreased as the flange thickness was increased. The central buckle was flanked by two buckles which were 180 deg. out of phase with the central buckle. In the girders with the low values of $l/b^3t$, the flanking buckles were also long and thin. As the value of $l/b^3t$ was increased, the flanking buckles tended to become less deep in magnitude and more rounded in shape. This change in web deflections was a gradual one with the exception of the girders with the 1/2 in. thick flanges. In girder FT-9, the central and flanking buckles were of equal amplitude with the amplitude of the central
buckle being somewhat smaller than would be expected on the basis of the results of the other girders. The web of girder FT-3 exhibited only two buckles which were of equal magnitude but opposite phase. The web contour plots of girders FT-9 and FT-3 are shown in Fig. 18. In the web of girder FT-6, five buckles were formed as shown in Fig. 19.

The stresses given in Table 4 were computed from strains measured at the centers of the web panels with a strain gage rosette. The manner in which the maximum principal tensile stress varied with load is shown in Fig. 20.

The permanent deformation recorded in Table 4 is the amount of permanent vertical set that one end of the panel experienced with respect to the other end. This permanent set was a result of the yielding of the flanges at the corners of the web panel in which the central diagonal buckle was anchored.

The results pertinent to the fatigue behavior of shear panels with varying flange thickness are given in Table 5 and Fig. 14. The locations of failure in the various girders are shown in Fig. 15. The maximum range of web deflection never occurred along the boundary members, but always in the central portion of the web panel. These ranges are provided to give an indication of the amount of breathing that occurred during the cyclic fatigue loading.

6.2 Girders with Various Flange Sizes Subjected to Shear and Moment

Test results for girders with various flange sizes which were subjected to both shear and moment are given in Table 6. The life of girder FTSB-1 is not reported since it is not known when failure occurred. This girder was first checked at 100,000 cycles and had completely failed by that time. Furthermore, the lives of girders FTSB-2 and FTSB-3 had to be estimated. For purposes of comparison, failure was defined as the number of repetitions
of load at which the fatigue crack had reached a length of 3 in. The cracks in girders FTSB-2 and FTSB-3 were 5 in. long when they were first discovered. At that time, girder FTSB-2 had been subjected to 60,000 repetitions of load and girder FTSB-3 had undergone 52,000 repetitions of load. Crack growth data from specimens subsequently tested made it possible to estimate the number of cycles at which the cracks would have been 3 in. long. Fatigue lives are related to the flange thickness parameter in Fig. 21.

Maximum values of initial lateral web deflection, web deflections at maximum test load and the range of deflections during loading are given in Table 6. The patterns of initial web deflection are shown in Fig. 22. It can be seen that the residual welding stresses caused the webs to deflect into either two or three distinct buckles. In the cases in which two buckles were formed, (e.g., panel 1 of FTSB-4) the buckles were located nearly symmetrically with respect to the horizontal centerline of the panel and the amplitudes of the two buckles were approximately equal. In cases where three buckles formed, the central buckle was very large and dominated the whole deflection pattern. This central buckle was flanked by two small buckles which were located near the flanges of the girder. The one notable exception to this pattern is panel 2 of girder FTSB-3. The web deflection patterns at maximum test load are presented in Fig. 23. In both Figs. 22 and 23, the values of the ratios of buckle amplitude to web thickness are given for the individual buckles. Locations of the fatigue failures in the various specimens are also shown in Fig. 23.

The approximate relative magnitudes of web flexing action along the stiffeners at which failure occurred are given in Fig. 24. This figure was laid out in the following manner. The lateral deflections of the stiffeners
at horizontal grid lines were subtracted from the corresponding lateral deflections of the web one inch away from the centerline of the stiffener. These differences in deflection were plotted horizontally, in the proper relative locations, from a vertical base line. The base line represents the length of the stiffener. The horizontal distance from the curved line to the base line is approximately proportional to the angle through which the web flexed with each repetition of load. The areas in which the fatigue failures initiated and propagated are darkened in Fig. 24.

6.3 Girders With Various Stiffener Sizes

To study the effect of variations in stiffener size five specimens were fabricated so that a range of values of $E I_s/Db$ was covered. All girders were subjected to identical loading conditions so that the effect of stiffener size can be evaluated on the basis of the fatigue lives. Results of tests on girders with various stiffener sizes are given in Tables 7 and 8. Table 7 contains the fatigue lives and the static ultimate loads obtained by tests for the several specimens. The fatigue lives are presented graphically as a function of the stiffener rigidity in Fig. 25. The ultimate capacity of the girders was limited by the lateral buckling of the upper flange. In the static tests no yielded areas were visible in the web when the ultimate load was reached.

Table 8 contains maximum values of lateral web deflections. The maximum initial deflection in the fabricated girders prior to any load application, the maximum deflection at full fatigue test load and the maximum range of deflection during the fatigue test are reported. The individual values presented occurred at different locations within the panel. The patterns of deflection corresponding to the maximum values given in Table 8 are
presented in Figs. 26 and 27. Actual failure locations are also shown in that figure. These sketches give a general indication of the location of some of the maximum web deflection values which appear in Table 8.

The effect of the relative magnitude of web flexing action along the stiffeners and the relation between this effect and the failure location was studied in a manner identical to that discussed in Section 6.2. Figure 28, which shows the magnitude of the web flexing and the location of failure, is comparable to Fig. 24. It will be noted that there is good agreement between the failure location and the point of maximum web flexing action.

Strain measurements at mid-height of the stiffeners showed that in all cases, the stiffeners carried a compressive force at the gage locations. In girders VST8-10 and VST8-16, strain measurements indicated a nearly uniform value of compressive strain across the width of the stiffeners. In girders VST16-8, VST8-4 and VST8-6, the width of the stiffeners permitted the application of only one gage on each side of each stiffener. Consequently, the average compressive strains at any load were computed by taking the average value of all of the strain readings taken from a particular pair of stiffeners at the particular load. Figure 29 shows the average stiffener strains as a function of the applied load. In order to compute a stiffener load, a uniaxial state of stress was assumed to exist in the stiffener and the average compressive strains were multiplied by Young's modulus of elasticity, E, and the area of the stiffener. Figure 30 gives the computed stiffener forces as a function of applied load.

The stiffener strain measurements also indicated a variable amount of bending in the stiffeners at mid-height of the girder in a plane perpendicular to the plane of the web. In order to compare the amount of bending
in the different sized stiffeners, the average compressive strain was computed for one of the stiffeners of a particular pair of stiffeners and this average strain was subtracted from the average compressive strain of the pair of stiffeners, as presented in Fig. 29. This procedure is based on the assumption that the neutral axis of a stiffener pair was located in the central plane of the web. The computed bending strains are plotted as a function of applied load in Fig. 31.

Stiffener deflections were measured at equal increments of load from zero to load values near the ultimate load. Values of stiffener deflection at the ultimate loads could not be measured because the excessive deflection of the girder caused the deflection measuring jig to bind. The maximum values of stiffener deflection are given in Fig. 32. These maximum values always occurred at, or near, the mid-height of the stiffener.

Strain measurements were also taken from two pairs of strain gage rosettes located on the web of girder VST8-6. Figure 33 shows the location of the gages with respect to the crest of the dominant web buckle which developed during the ultimate load test of the girder. The stresses shown in Fig. 33 were computed from strain measurements which were taken during two separate tests. The stresses which correspond to loads equal to or less than the maximum fatigue test load were computed from strain measurements which were taken after the girder had undergone 19,000 cycles of loading. The stresses which correspond to loads greater than the maximum fatigue test load were computed from strain measurements taken during the ultimate load test of the girder. The actual stresses are compared with theoretical stresses which were computed with the assumptions that the web was initially plane and resisted the applied load by pure beam action.
7. DISCUSSION OF TEST RESULTS

7.1 Girders with Various Flange Sizes Subjected to Shear

7.1.1 Flange Rigidity Effect on Static Behavior

7.1.1.1 Flange Effect on Web Deflections. The results of these tests indicate that the maximum value of web deflection that a shear panel will experience under a given load is dependent upon, among other things, the rigidity of the girder flange. The measured maximum web deflection to web thickness ratios are plotted as a function of flange rigidity in Fig. 34. Also included in this figure are results of the same nature obtained by K. C. Rockey from tests on small bolted aluminum alloy girders with initially plane webs.

The measured maximum web deflection of the tests described herein bear a consistent relationship with flange rigidity except for the girders with the 1/2 in. thick flanges. It has already been noted that these two girders contained initial web deflection patterns which were significantly different from the other girders. It is felt that the web deflections under maximum load for girders FT-3 and FT-9 would have been larger if the initial web deflection pattern for these girders had been similar to the other girders. It does seem, then, that initial web deflections do have an effect on the web behavior of thin web shear girders under static load. In view of this, the results for the girders with the 1/2 in. thick flanges have been ignored in the drawing of the curve of Fig. 34. It is thought that the curve that has been drawn represents the worst behavior to be expected from shear panels with initial web deflections. Figure 34 shows that for small flange rigidities, very deep web buckles were formed in the shear girders when
loaded above the theoretical critical load. However, as the flange rigidity was increased, the depth of the web buckles decreased until at a flange thickness of about 3/4 in., the buckle depth reached a minimum value which was not affected by further increase in flange thickness. Evidently, a 3/4 in. thickness provides a critical flange stiffness for the shear girder tests described in this report.

The results of the shear girder tests reported herein agree in many ways with the previously mentioned test results obtained by Rockey. The relationship between the maximum web deflections and $1/b^3 t$ closely parallels that obtained by Rockey. The effect of the initial web deflections was to displace the curve upward from Rockey's curve for shear girders with initially plane webs. The difference between the horizontal portions of the two curves of Fig. 34 is about 1.4 times the web thickness. On the basis of Bergman's theoretical work on the effect of initial curvatures on the buckling of shear webs with rigid boundaries, a difference of about 1.3 times the web thickness would be expected between the horizontal portions of the two curves in Fig. 34. Also, Rockey was able to carry out sufficient tests to establish a relationship between critical flange thickness and $W/W_{cr}$. This relationship is presented in Fig. 4. It can be seen that the critical flange rigidity obtained from the present tests agrees very closely with that obtained by Rockey. On this basis, it seems reasonable to apply Rockey's relationship between critical flange rigidity and $W/W_{cr}$ to the welded shear girder tests of this report.

7.1.1.2 Flange Effect on Stresses at Center of Web. The flange rigidity did not have any significant effect on the stress condition at the center of the shear panels. The principal stresses computed from the strain
measurements are listed in Table 4. It can be seen that there is no significant trend in principal tensile stress magnitude as the flange thickness was increased. However, there was a two-fold increase in the computed principal compressive stresses as the flange thickness was increased from 3/16 in. to 1-in. This increase would indicate that more of the load was carried by "pure shear" in the girders with thick flanges than in the girders with thin flanges.

The strain measurements which were made in the upper corner of FT-2 indicated the presence of a diagonal tension stress which was slightly greater than the principal tensile stress measured at the center of the web. These measurements showed that there is a large diagonal tension stress anchored in the two diagonally opposite corners of the girder panel.

Without a very large number of strain measurements, it is impossible to draw any firm conclusions as to the stress distribution in the web of a thin web girder. The location of the strain rosettes in these tests placed the gauge on or very near the crest of the central diagonal buckle with the diagonal rosette gauge very closely aligned with the longitudinal axis of the buckle. For this reason, it is felt that the gages give a good measure of the degree of development of the diagonal tension fields in the girder. If the girder loads had been carried by pure shear action alone, the theoretical principal tensile and compressive stresses at the center of the web would have been ±11,000 psi. If the girder loads had been carried by pure diagonal tension, the tensile stress at the center of the web would have been about 22,000 psi. The measured principal tensile stresses were of the order of 20,000 psi and the measured principal compressive stresses were of the order of -5,000 psi. These stress measurements indicate that the diagonal tension field was quite highly developed.
7.1.2 Flange Rigidity Effect on Fatigue Behavior

7.1.2.1 Flange Effect on Flexing Action of Web. The flexing action of the web about the boundary members of the test panels showed no significant trend as the flange rigidity was changed. The flexing action was most intense at the ends of the central and flanking buckles and did not significantly differ in magnitude between the various locations. Differences in the fatigue lives of the shear girders cannot be explained in terms of the intensity and pattern of the flexing action of the web. It can only be said that flexing action will occur in shear girders when loaded cyclically in the postbuckling range and that this flexing action will undoubtedly manifest itself in a reduction of fatigue life.

7.1.2.2 Flange Effect on Fatigue Life. The fatigue life of the shear girders showed a definite increase with increase in flange rigidity. This conclusion is most effectively illustrated by the plot in Fig. 14 which relates fatigue life to \( \frac{1}{b^3} \). There was no abrupt change in fatigue behavior of the girders at any given flange rigidity. This observation leads to the conclusion that the results are best represented by a smooth curve. Since there were no indications that a curve of any particular shape would best represent the results, the most simple curve has been chosen and the test results are represented in Fig. 14 by a straight line. It is felt that in these tests, a flange thickness of 1-in. provided, for all practical purposes, an infinitely stiff boundary member and that an increase in flange thickness above 1-in. would not have significantly increased fatigue life. This treatment is analogous to that of breaking the S-N curves for structural joints at 2,000,000 cycles. Of course, further tests should be conducted to verify this procedure but until such tests are completed, it is felt that the
application of these test results to full size structures would be on the safe side if such an upper limit of fatigue life is accepted.

The scatter of the results is comparable to that obtained from many fatigue testing programs which have been reported in the literature. However, the fatigue life of girder FT-9 does represent a significant difference from the other results. Since the difference was in the form of a very long fatigue life, it has been chosen to ignore this result in drawing the curve of Fig. 14 since this constitutes an error on the safe side.

The measurements which were made in the shear girder tests described herein do not make it possible to attribute the increase in fatigue life with increase in flange rigidity to any specific reason. However, since the flexing action of the web about the stiffeners did not vary significantly from girder to girder and since the welding and fabrication techniques were the same for all girders, it is felt that the difference in stress conditions along the stiffener welds played a major part in imparting the different fatigue lives to the girders. It has already been stated that the theory of partial diagonal tension leads to the conclusion that the magnitude of the diagonal tension stress in shear panels decreases with an increase in flange rigidity. If this is the case in these girders, the decrease in diagonal tension stress with increase in flange rigidity would serve to explain the increase in fatigue lives. However, it is difficult to make measurements to substantiate this theory.

7.1.3 Comparison of Actual Behavior with Predicted Behavior. A comparison of actual behavior with the behavior predicted from qualitative considerations shows good agreement. The web did tend to buckle in one large diagonal buckle from corner to corner of the test panel. A large diagonal
tension stress was measured. Furthermore, the girders actually failed at the predicted locations.

7.2 Girders With Various Flange Sizes Subjected to Shear and Moment

7.2.1 Initial Deflections. Relative to the thickness of the web, the initial deflections in the scale model girders were larger than the initial deflections that would be expected to occur in full size girders. This problem arises from the welding of the model girders. Due to the small size of the component parts, there is insufficient material to dissipate the heat generated by the welding. Hence, the residual welding stresses cause more extensive buckling in members of small size. This problem can be overcome only by reducing and localizing the heat input due to welding in small girders. For the tests described herein, equipment was not available to accomplish this reduction and localization. The result was relatively large initial web deflections.

The initial deflections influenced both the magnitude and pattern of web deflections that were caused by the test loads. A comparison of the initial deflection patterns in Fig. 22 with the corresponding deflection patterns at full test load in Fig. 23 reveals that the effect of the load was simply to distort the buckles which were initially in the web. This distortion changed the slope of the longitudinal axes of the buckles but did not greatly affect the magnitude of the amplitudes of most of the buckles.

The initial deflections also had an important influence on the determination of the location of maximum web flexing action at the stiffeners. This position was always situated between the web buckles rather than at the ends of these buckles. It has been pointed out in Section 2.1 that the magnitude of deflections due to load are inversely proportional to the
magnitude of the initial deflections. Since the amplitude of the initial buckles was large, and since the load simply distorted these buckles rather than producing new buckles, the deflections due to load at the crests of the web buckles were small. In this way, the initial deflections forced the locations of maximum web flexing action to occur between the buckles rather than at the ends of the buckles.

7.2.2 Web Flexing Action. A consistent relationship between web flexing action and failure location can be noted in Fig. 24. The failures initiated at the location of the maximum intensity of web flexing action. There is a strong correlation between the maximum measured intensity of web flexing action and fatigue life for girders FTSB-1, FTSB-2, FTSB-4 and FTSB-5. For these girders, the fatigue life was proportional to the maximum intensity of web flexing action. However, girder FTSB-6 does not fit into the foregoing correlation. In this girder, the web flexing intensity was greater and the fatigue life was longer than would have been predicted from the trend established by the other girders.

The foregoing observations led to the conclusion that web flexing action does play a significant role in the development of fatigue failures of thin web girders which are subjected to shear and bending. Further justification of this conclusion is provided by the types of failure noticed in girders FTSB-5 and FTSB-6. The cracks initiated on the side of the web in which tension stresses were caused by the flexing action. The cracks grew to over an inch in length before they propagated through the thickness of the web and became visible on the compression side. However, the results of girder FTSB-6 indicate that there are factors other than web flexing action (e.g. membrane stresses) which have an effect on the fatigue lives of the girders.
7.2.3 Flange Rigidity Effect on Fatigue Life. There was a significant increase in fatigue life as the flange rigidity of the test girders was increased. This conclusion is illustrated in Fig. 21. No conclusion can be drawn as to what the effect of flange rigidity would be outside of the range of rigidities that were tested.

In complicated specimens of the type that were tested, it is impossible to determine exactly the causes of failure and the extent to which each cause contributes to failure. It has been shown that web flexing action was one on the factors contributing to the failure. It is thought that membrane stress is another factor which contributed to failure. Although no measurements of membrane stress were carried out, the failure of girder FTSB-3 provides evidence that membrane stresses did exist and did influence the failure of the girders. This particular failure followed the toe of the stiffener fillet weld until it reached the end of the central web buckle. It then branched out into the web and propagated in a direction that was approximately perpendicular to the longitudinal axis of the buckle (see Fig. 23). Previous tests \(^{(24)}\) have shown that cracks of this nature tend to propagate perpendicularly to the principal tensile stress in the web. Hence, tensile membrane stresses which were oriented approximately parallel to the crests of the web buckles must have existed in the webs of the test panels.

No data were obtained in these tests which clearly show the mechanism by which the flange rigidity affected the fatigue lives of the girders. In Chapter 2, it was stated that the flange rigidity would affect fatigue life by affecting both the intensity of web flexing action and the magnitude of the membrane stress. In this series of tests, it was
impossible to separate entirely the effects of flange rigidity and initial deflections on the flexing action of the web at the stiffeners. Furthermore, no measurements of web stress were made. Therefore, the exact details of the individual effects which added up to the total flange rigidity effect is not known. However, it has been shown that the fatigue lives were dependent on flange rigidity and that the fatigue lives were related to web flexing action and membrane stress. It is concluded that the flange rigidity affected the fatigue lives of this test series by influencing both the web flexing action at the stiffeners and the membrane stresses in the web.

7.2.4 **Comparison of Actual and Predicted Behavior.** There were several differences between the actual behavior of this series of girders and the behavior that was predicted in Chapter 2. Notwithstanding these differences, the failures occurred in much the same manner as was predicted from the consideration of ideal web panels. Figure 35 illustrates the principal compressive stresses in an ideal web panel (initially plane panel which does not buckle). The panel is subjected to the maximum test stresses to which the test girders were subjected. It can be predicted from this figure (with the same reasoning as was used in Chapter 2) that the compressive stresses in the upper part of the web will cause a buckle to form. The longitudinal axis of the buckle should lie in the neighborhood of the dashed line of Fig. 35. A secondary smaller buckle will probably form in the bottom part of the web. The resulting buckling pattern would be similar to that shown in Fig. 9. The maximum web flexing effect and maximum membrane stress effect with respect to fatigue behavior should occur along the stiffener at the end of the buckle in the upper part of the web.
Because of the initial deflections, many differences were noted between actual and predicted behavior. In general, the web did not buckle into the predicted pattern because of the large initial buckles in the web. However, with the exception that the buckles in the bottom part of the web were much larger than predicted, the deflection pattern under full load of panel 1 of FTSB-2, panel 2 of FTSB-4 and both panels of FTSB-5 were quite similar to the predicted buckling pattern. Because the web buckles were preformed, the maximum web flexing action did not occur at the end of the buckles but between two adjacent buckles. One indication of the existence of a membrane stress (the failure of FTSB-3) led to the conclusion that a diagonal tension stress which was similar to the predicted diagonal tension stresses actually existed in the web panels.

7.2.5 Analysis of Flange Rigidity Requirements. The major behavioral difference between this series of tests and the tests of shear girders with various flange sizes was in the effect of flange rigidity on the magnitude of lateral web deflections.

The results of the shear girder tests revealed a critical flange rigidity. For rigidities above this critical value, the magnitudes of lateral web deflections at any particular load were independent of flange stiffness. However, if flange rigidity is reduced below the critical value, the lateral web deflections of the web plate start to increase rapidly at any particular load. The value of critical flange rigidity that was determined in this investigation agreed quite closely with a similar value obtained by K. C. Rockey from tests on shear panels.

The tests of girders with various flange sizes which were subjected to shear and bending did not reveal a critical flange rigidity. This was so
even though the shearing stress to which the test panels were subjected was equal to the shearing stress in the test panels of the shear girders. If a critical flange rigidity does exist for girders subjected to shear and moment, it falls outside the range of flange rigidities that were studied. Apparently the phenomenon of critical flange rigidity for girders subjected to shear is different from that for girders subjected to shear and bending.

The secondary bending moments in the flanges of the shear girders were large. It has been stated in the literature\(^\text{(2,10)}\) that the magnitudes of secondary flange bending moments in post buckled shear panels with extremely thin webs are small enough to be neglected. Neither the results of this investigation nor Rockey's investigation\(^\text{(11)}\) support this statement in the case of civil engineering type thin web girders. The secondary bending moments in the shear girders with flange rigidities below the critical value (with the single exception of girder FT-9) were sufficiently large to cause yielding of the flanges over the stiffeners. The yielding of the flanges allowed large shearing deformations to occur and resulted in large lateral web deflections. It has been noted that there was no evidence of flange yielding in girder FT-9 in which the web buckling pattern was similar to the girders which were subjected to shear and moment. Furthermore, there was no evidence of flange yielding in the girders which were subjected to shear and bending. Hence, static flange rigidity requirements depend on the type of load to which the girder is subjected and the manner in which the web buckles and resists the load. For girder panels which are loaded such that they buckle into one large diagonal buckle, the secondary flange bending moments will be large and rigid flanges will be required. For girder panels which are subjected to combined shear and bending or which
are forced to buckle into a pattern which is similar to the buckling pattern for girders under combined shear and bending, the flange rigidity requirements are less severe.

On the basis of the results of this investigation and the results of Rocky's work, (11) it is recommended that girders which are primarily shear resistant members be provided with a value of the flange rigidity parameter, $1/b^3 t$, which is equal to or greater than minimum values proposed by Rocky. These minimum values are given by the formula

$$\left(\frac{1}{b^3 t}\right)_{\text{min.}} = 0.00035 \left[\frac{W}{W_{cr.}} - 1\right]$$ (6)

valid over the range $1 \leq \frac{W}{W_{cr.}} \leq 4$

From the point of view of fatigue life, a higher minimum value for $1/b^3 t$ than that given by Eq. (6) might be considered. However, even the minimum value above requires $1/b^3 t$ values which are greater than those which have been provided for many welded girders designed in the past. Furthermore, girders with $1/b^3 t$ values greater than the values of Eq. (6) will probably behave in a similar manner so that reasonably constant values of fatigue life will be provided.

From the point of view of static behavior, this investigation indicates that no limitation need be placed on flange rigidities for girders which are primarily moment resistant members. However, for girders which are subjected to cyclic loads, the fatigue lives can be increased by increasing flange rigidity. It is recommended that the value of the flange rigidity parameter be kept above the minimum values prescribed by Eq. (6)
for all thin web girders, be they subjected to shear alone or to shear combined with bending.

It should be noted that the most effective way to increase the value of the flange rigidity parameter is to decrease the stiffener spacing b. Consequently, the requirements for minimum values of $1/b^3 t$ can be regarded as a limit on vertical stiffener spacing as well as a limit on flange size.

7.3 **Girders with Various Stiffener Sizes**

7.3.1 **Web Behavior**

7.3.1.1 **Initial Deflections.** Relative to the size of the girder, the magnitudes of initial web deflections in the model girders with the more rigid stiffeners (VST8-6, VST8-10, VST8-16) were comparable to the magnitudes of initial deflections that might be expected to occur in full size girders. It has been stated in Chapter 2 that the initial deflections in full size girders can be expected to be as large as 2.5 times the thickness of the web. The magnitudes of the initial deflections in VST8-6, VST8-10 and VST8-16 were less than the above stated value. However, the initial deflections became larger as the stiffener rigidity was decreased and reached a maximum value of 3.4 times the thickness of the web in girder VST16-8.

The initial web deflection patterns influenced the lateral deflection of the web that occurred when the girder was loaded. A comparison of Figs. 26 and 27 shows that the effect of adding load to the girders was to distort and change the amplitude of the buckles which were initially in the web. The distortion took the form of a change in slope of the longitudinal axis of the initial web buckles. The change in amplitude occurred mainly in the compression zone of the web where the amplitudes of the initial buckles were increased by the addition of load.
The initial deflection pattern had very little effect on the web configuration as the ultimate load of the girder was approached. The web configuration at this point was completely dominated by a large buckle in the upper part of the web. The crests of the buckles formed an angle of approximately 40 deg. with the horizontal and sloped from one of the upper corners of the panel. In view of the above described behavior, it is concluded that the effect of initial deflections on the deflection pattern of the web at any particular load is dependent upon the magnitude of the load.

7.3.1.2 Web Strain Measurements. The web strain measurements which were made at the center of girder VST8-6 show that the girders resisted the applied load by partial diagonal tension action. Even at the lowest loads, Fig. 33 shows that the experimental principal tensile stress both at the center of the web and near the stiffeners was larger than the theoretical principal tensile stress. The experimental principal compressive stress at the center of the web was always smaller than the theoretical principal compressive stress. These observations are both characteristic of partial diagonal tension action.

It is felt that the principal compressive stresses which were measured adjacent to the stiffener were strongly influenced by the compressive force in the stiffener (see Fig. 30). The vertical compressive load in the stiffeners set up vertical compressive stresses in the strips of web adjacent to the stiffeners. Hence, the principal compressive stresses adjacent to the stiffeners were larger than the theoretically computed compressive stresses at the same location.

The principal tensile stress which was measured at the stiffener was not as large as the principal tensile stress which was measured at the center
of the web panel. It is concluded that this behavior is due to the effect of the stiffeners. The stiffeners tend to minimize the lateral web deflections in the adjacent areas of the web. As a result, tensile membrane stresses do not tend to build up in one location as they do at the crests of the web buckles in the center of the web. Hence, the maximum value of membrane stress is lower at the stiffened boundaries of the panel. The more rigid the stiffener, the more pronounced will be this effect. The foregoing effect combined with the lower maximum intensity of web flexing action was probably sufficient to cause the increase in fatigue life that was noted in girder VST8-16.

The theoretical orientation of the principal tensile stress was 58 deg. with respect to the horizontal. The crest of the dominant web buckle in the upper part of the web was oriented at an angle of 41 deg. with respect to the horizontal. The direction of the principal tensile stress varied gradually from an angle of 55 deg. below the horizontal at a load of 20 per cent of the maximum test load to an angle of 46 deg. below the horizontal at a load of 90 per cent of the ultimate load. It can be seen that the orientation of the experimental principal tensile stress was very nearly the same as the theoretical orientation at low loads but gradually approached the orientation of the dominant web buckle as the load was increased.

7.3.1.3 Effect of Stiffener Size on Web Deflections. The stiffener size had a distinct effect on the magnitude of the initial web deflections in the various girders. The data in Table 6 shows that the maximum magnitudes of initial web deflections were inversely proportional to stiffener size. It is felt that this type of behavior will be typical for all girders.
The stiffener size had very little effect on the web deflections at maximum test load. There was only a 10 per cent decrease in the maximum magnitude of web deflection at full test load as the stiffener size was increased from 1/8 in. by 1/2 in. to 1/8 in. by 2 in. in the other girder VST16-8, the stiffeners buckled before the maximum test load was reached.

7.3.2 Stiffener Behavior

7.3.2.1 Lateral Deflections of Stiffeners. The lateral deflections of the stiffeners in a plane perpendicular to the plane of the web were greatly influenced by stiffener size. In all cases, the location of maximum lateral deflection was near the mid-depth of the girder. The flexible stiffeners of VST16-8 did very little to isolate the three girder panels and simply buckled with the web. This behavior resulted in large lateral deflections even at the maximum test load (see Fig. 32). The three girders with stiffener rigidities in the neighborhood of those required by current design practice, i.e., VST8-4, VST8-6 and VST8-10, behaved similarly with respect to growth of maximum lateral deflections. The lateral deflections of the very rigid stiffeners were practically zero and the stiffeners behaved as if they were infinitely rigid.

Even though the stiffeners of VST8-4, VST8-6, and VST8-10 all exhibited about the same magnitude of lateral deflection, Fig. 31 shows that there was a significant difference in the amount of bending for the different stiffener sizes. It can be seen that as stiffener size was increased, the bending of the stiffener was markedly reduced. This means that although the lateral deflections of the stiffeners were similar, the less rigid stiffeners tended to deflect into the buckling pattern of the web with resulting sharper radii of curvature. The more rigid stiffeners tended to resist the
influences of web buckling and remained straight to more effectively isolate adjacent girder panels.

7.3.2.2 Compressive Load in Stiffeners. The computed compressive force in the stiffeners varied directly with stiffener size. If the panels are effectively isolated from one another, the diagonal tension stress is all transmitted to the stiffeners and the stiffener load is large. If the panels are not effectively isolated, some of the diagonal tension stress is transferred into the adjacent panel and the stiffener loads are correspondingly less. Hence, the compressive forces are a measure of the amount of stress that is transferred through the vertical stiffener fillet welds.

It had been anticipated that the intensity of stress transfer through the stiffener fillet welds would have an effect on the fatigue lives of the girders. However, if stiffener load is taken as a measure of the intensity of stress transfer, there was no correlation between stress transfer and fatigue life for this series of tests.

7.3.3 Fatigue Behavior. The distribution of the intensity of web flexing action along the stiffeners had a significant effect on the determination of the location of failure. Every failure initiated along the toe of the stiffener fillet welds at the location of maximum web flexing intensity (Fig. 28).

Furthermore, there was some correlation between the fatigue lives of the girders and the intensity of web flexing at the point of initiation of failure. Girders VST16-8 and VST8-16 exhibited both the smallest intensity of web flexing action and the longest lives. Girders VST8-4 and VST8-10 exhibited equal intensities of web flexing action and equal fatigue lives.

The foregoing observations lead to the conclusion that the intensity of web flexing action at the vertical stiffeners is an important factor in the
determination of the fatigue lives of the various girders. Further justification of this conclusion is provided by the type of failure that was noticed in the various girders. The cracks always initiated on the side of the web in which tension stresses resulted from the flexing action. The cracks propagated over an inch or more on the tension side before they traversed the thickness of the web and became visible on the compression side.

The strain measurements which were made in girder VST8-6 revealed the existence of diagonal tensile membrane stresses in the webs of the girders. These stresses were anchored at the stiffeners and were cyclic in nature. Since the stresses were anchored at the locations at which failure occurred, they must have had an effect on the lives of the girders. It is impossible to assess the relative importance of membrane stress and web flexing action on the fatigue lives of the girders. These effects are superimposed and cannot be separated.

The results from girder VST16-8 show that girders with weak stiffeners which buckle with the web can have a higher resistance to fatigue than girders with more rigid stiffeners. This observation had been anticipated in Chapter 2 where it was stated that girders with weak stiffeners would exhibit less web flexing action at the boundaries and a smaller stress transfer through the stiffener fillet welds than girders with more rigid stiffeners. It can be seen in Fig. 28 that the flexing action was indeed less for VST16-8 than for girders VST8-4, VST8-6 and VST8-10 all of which were provided with more rigid stiffeners.

In summary, it can be said that the combined effect of web flexing action and membrane stresses caused fatigue failures to initiate at the boundaries of the test panels. Furthermore, the stiffener rigidity did have
an effect on fatigue life. Girders with very weak stiffeners and with very rigid stiffeners both had fatigue lives greater than girders with stiffeners whose rigidities were in the range of the rigidities that are required by current design practice.

7.3.4 Comparison of Actual Behavior With Predicted Behavior. In general, the actual behavior of the girders corresponded quite closely to the predicted behavior. Notwithstanding the initial deflections, the web deflection patterns at full load of girders VST8-4, VST8-10 and VST8-16 were similar to the anticipated deflection patterns (Fig. 9). The initial buckles which occurred in the lower portion of the web were unimportant. Girder VST8-6 differed in that it had two buckles in the upper part of the web. The stiffeners of girder VST16-8 buckled with the web and the behavior of the girder was unique among all the girders that were tested.

The web flexing action at the boundaries and the membrane stresses in the web behaved as had been predicted. The maximum intensity of web flexing action occurred at the ends of the buckles in the upper part of the web. The tensile membrane stresses along the crest of the upper buckle were much larger than the theoretical pure beam action stresses. Moreover, the orientation of the tensile membrane stress was approximately parallel to the crest of the dominant web buckle in the upper part of the web.

7.3.5 Analysis of Stiffener Rigidity Requirements. This investigation has shown that the fatigue lives of thin web girders can be influenced by stiffener rigidity. It appears that the fatigue lives of girders with vertical stiffeners which meet the presently accepted design requirements can be improved in one of two ways. The vertical stiffeners can be replaced with either very weak stiffeners or very rigid stiffeners.
Girders with very weak vertical stiffeners are unacceptable both from the point of view of strength requirements and from the point of view of aesthetic requirements. The stiffeners buckle before the ultimate load of the weakest web panel is reached or before the compression flange buckles. Hence, inefficient use is made of the web and flange materials. Furthermore, very large lateral web deflections occur with each repetition of load. Deflections of this type in an actual structure would be aesthetically unacceptable.

If the fatigue resistance is to be improved by the use of very rigid stiffeners, the size of vertical stiffeners will necessarily become very large. For example, if a full size girder with a 10 foot deep by 3/8 in. thick web were to be stiffened in a manner similar to the model girder VST8-16 (1/8 in. by 2 in. stiffeners), the vertical stiffeners would have to consist of two 1-in. by 5 in. plates.

Certainly, more tests must be completed before definite requirements can be made with regard to increasing the required rigidities of vertical stiffeners. If more tests are completed and verify the results obtained in this investigation, a choice will have to be made between medium sized vertical stiffeners and lower fatigue lives or very large vertical stiffeners and longer fatigue lives.
8. SUMMARY AND CONCLUSIONS

8.1 Summary

The objectives of this investigation were to determine the manner in which thin web girders fail when subjected to repeated loads, to determine what factors influence the fatigue strength of thin web girders and to determine the manner in which these factors influence the fatigue strength. In particular, the investigation has been directed toward the determination of the effect of flange rigidity and vertical stiffener rigidity on the fatigue behavior of thin web girders. Consideration was also given to the effect of initial lateral web deflection in girder panels and to the effect of the type of load.

A qualitative analysis was made of the fatigue behavior of girder panels with initial lateral web deflections. Two different loading conditions were considered: pure shear and combined shear and bending. The factors which influence the fatigue behavior of thin web girders were ascertained and the effect of both flange rigidity and vertical stiffener rigidity on these factors was qualitatively analyzed.

In the experimental part of the investigation, twenty fatigue tests were conducted on scale model thin web, all-welded girders. Nine girders with variations in flange size were subjected to a large shearing force and a very small bending moment. The test loads were chosen to approximate a condition of pure shear. The flange size was varied from 5 in. by 1/4 in. to 5 in. by 1-in. on girders with a web depth of 20 in. and a web thickness of .075 in. The girder contained only one test panel with an aspect ratio of 0.75. Six girders with varying flange sizes were subjected to combined shearing forces and bending moments. The flange size was varied from 4 3/8 in. by 1/4 in. to 1-in. by 1-in. on girders with a web depth of 16 5/8 in.
and a web thickness of 1/16 in. Two adjacent test panels with aspect ratios of 0.75 were provided. Finally, five tests were conducted on girders with various stiffener sizes which were subjected to combined shearing forces and bending moment. The stiffener size was varied from 1/16 in. by 1/2 in. to 1/8 in. by 2 in. on girders with a web depth of 20 in. and a web thickness of 1/16 in. The girders were fabricated with three adjacent test panels with aspect ratios of 0.5.

Results obtained from the experimental investigation were compared with the results of the qualitative investigation.

8.2 Conclusions

The conclusions which have been drawn from the results of the investigation reported herein may be summarized as follows:

(1) Fatigue resistance presents a more complex and serious problem in thin web girder design than in conventional civil engineering girder design. The fatigue resistance is not only a function of the applied stresses but also of the geometric properties of the girder. A systematic evaluation of the effect of the various geometric properties on the fatigue behavior of thin web girders is a necessary prerequisite to the determination of actual fatigue strength for such girders.

(2) The rigidity of the boundary members (flanges and stiffeners) of the individual web panels of plate girders has a significant effect on the fatigue behavior of thin web girders. In the investigation reported herein, the relationships between fatigue life and boundary rigidity that were determined for wide ranges of rigidities gave no indication of what the effect would be of boundary rigidities which fall outside of the test range.
(3) It is recommended that the flange rigidity parameter \(1/b^3t\) be maintained above a certain minimum value in the design of thin web girders which are to be subjected to fatigue type loadings. The recommended minimum value for this parameter is expressed by a formula that was derived by K. C. Rockey\(^{(11)}\)

\[
\frac{1}{b^3t} \text{ min.} = 0.00035 \left[ \frac{W}{W_{cr}} - 1 \right]
\]

valid over the range \(1 \leq \frac{W}{W_{cr}} \leq 4\).

(4) If more tests on girders with various stiffener sizes are completed which verify the results obtained in this investigation, a choice will have to be made. The choice will be between either leaving the present vertical stiffener design requirements unchanged and accepting the resulting fatigue lives or markedly increasing the required vertical stiffener sizes with resulting increases in fatigue life. The present investigation provided sufficient data to suggest the above choice but not sufficient data with which to make the choice.

(5) Two effects cause fatigue failures in thin web girders. The one effect is that of fluctuating membrane stresses which are anchored at the panel boundaries. The other effect is that of the web flexing action at the panel boundaries. Both of these effects are a result of the buckling action of the individual web panels.

(6) Initial lateral web deflections can have an effect on the fatigue behavior of thin web girders. It can be stated that the individual effects of initial deflections are both beneficial (a reduction in web flexing
action) and detrimental (an increase in membrane stresses) but it is impossible
to assess their absolute effect on fatigue behavior.

(7) In girders with square or nearly square web panels, the addition
of even small flexural stresses to a shear stress will reduce the fatigue life
of the girder from that which the girder would have exhibited were it subjected
to the shear alone. This result occurs because of a change in the location of
failure and more severe superposition of the maximum effects of web flexing
action and membrane stress.

(8) The testing of model girders can serve a very useful purpose.
The effect of a given parameter on the fatigue lives of thin web girders can
be more cheaply and conveniently evaluated from model girders than from full
size girders. A series of model girder tests could be used to provide the
relationship between fatigue life and the parameter under consideration. A
very few full size fatigue tests could then be conducted to establish the
relationship between the magnitudes of the fatigue lives of the models and
of the full size girders.

(9) An improvement could be made in the method of fabrication of
model girders that was used in this investigation. A welding technique which
reduces and localizes heat input would result in a decrease in the magnitude
of lateral web deflections and make the welding of the small girders more
representative of the welding of full size girders. Special welding
equipment which is needed to meet these requirements was not available for
the investigation reported herein.
LIST OF REFERENCES


**TABLE 1**

**INITIAL WEB DEFLECTIONS IN GIRDER PANELS**

TESTED BY BASLER ET AL.*

<table>
<thead>
<tr>
<th>Thickness of Web Plate in.</th>
<th>Face Dimensions of Web Plate in.</th>
<th>Aspect Ratio $\alpha$</th>
<th>Web Depth Web Thickness d/t</th>
<th>Initial Deflection in.</th>
<th>Initial Deflection Web Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.270</td>
<td>37 1/2 x 50</td>
<td>0.75</td>
<td>185</td>
<td>0.08</td>
<td>0.3</td>
</tr>
<tr>
<td>0.270</td>
<td>&quot;</td>
<td>&quot;</td>
<td>185</td>
<td>0.08</td>
<td>0.3</td>
</tr>
<tr>
<td>0.270</td>
<td>&quot;</td>
<td>&quot;</td>
<td>185</td>
<td>0.10</td>
<td>0.4</td>
</tr>
<tr>
<td>0.129</td>
<td>&quot;</td>
<td>&quot;</td>
<td>388</td>
<td>0.10</td>
<td>0.8</td>
</tr>
<tr>
<td>0.129</td>
<td>&quot;</td>
<td>&quot;</td>
<td>388</td>
<td>0.23</td>
<td>1.8</td>
</tr>
<tr>
<td>0.188</td>
<td>50 x 50</td>
<td>1.0</td>
<td>255</td>
<td>0.34</td>
<td>1.8</td>
</tr>
<tr>
<td>0.270</td>
<td>50 x 75</td>
<td>1.5</td>
<td>185</td>
<td>0.15</td>
<td>0.6</td>
</tr>
<tr>
<td>0.270</td>
<td>&quot;</td>
<td>&quot;</td>
<td>185</td>
<td>0.17</td>
<td>0.6</td>
</tr>
<tr>
<td>0.270</td>
<td>&quot;</td>
<td>&quot;</td>
<td>185</td>
<td>0.17</td>
<td>0.6</td>
</tr>
<tr>
<td>0.188</td>
<td>&quot;</td>
<td>&quot;</td>
<td>259</td>
<td>0.20</td>
<td>1.1</td>
</tr>
<tr>
<td>0.129</td>
<td>&quot;</td>
<td>&quot;</td>
<td>388</td>
<td>0.29</td>
<td>2.2</td>
</tr>
<tr>
<td>0.129</td>
<td>&quot;</td>
<td>&quot;</td>
<td>388</td>
<td>0.42</td>
<td>3.2</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Thickness of Web Plate in.</th>
<th>Face Dimensions of Web Plate in.</th>
<th>No. of Web Stiffeners</th>
<th>Aspect Ratio $\alpha$</th>
<th>Web Depth $d/t$</th>
<th>Initial Deflection in.</th>
<th>Initial Deflection Web Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.200</td>
<td>39.4 x 39.4</td>
<td>None</td>
<td>1.00</td>
<td>200</td>
<td>.079</td>
<td>0.4</td>
</tr>
<tr>
<td>0.200</td>
<td>39.4 x 39.4</td>
<td>None</td>
<td>1.00</td>
<td>200</td>
<td>.197</td>
<td>1.0</td>
</tr>
<tr>
<td>0.137</td>
<td>39.4 x 78.8</td>
<td>One</td>
<td>1.00</td>
<td>285</td>
<td>.158</td>
<td>1.1</td>
</tr>
<tr>
<td>0.150</td>
<td>50.5 x 31.4</td>
<td>None</td>
<td>1.61</td>
<td>210</td>
<td>.197</td>
<td>1.3</td>
</tr>
<tr>
<td>0.150</td>
<td>50.5 x 31.4</td>
<td>None</td>
<td>1.61</td>
<td>210</td>
<td>.158</td>
<td>1.1</td>
</tr>
<tr>
<td>0.150</td>
<td>50.5 x 31.4</td>
<td>None</td>
<td>1.61</td>
<td>210</td>
<td>.158</td>
<td>1.1</td>
</tr>
<tr>
<td>0.137</td>
<td>39.4 x 78.8</td>
<td>Two</td>
<td>2.00</td>
<td>144</td>
<td>.158</td>
<td>1.1</td>
</tr>
<tr>
<td>0.137</td>
<td>39.4 x 78.8</td>
<td>None</td>
<td>2.00</td>
<td>286</td>
<td>.118</td>
<td>0.9</td>
</tr>
<tr>
<td>0.157</td>
<td>27.6 x 94.5</td>
<td>None</td>
<td>3.43</td>
<td>175</td>
<td>.079</td>
<td>0.5</td>
</tr>
<tr>
<td>0.126</td>
<td>27.6 x 94.5</td>
<td>None</td>
<td>3.43</td>
<td>218</td>
<td>.236</td>
<td>1.9</td>
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</tbody>
</table>

### TABLE 3

**PHYSICAL PROPERTIES OF WEB MATERIALS**

<table>
<thead>
<tr>
<th>ASTM Designation</th>
<th>Yield Strength psi</th>
<th>Ultimate Strength psi</th>
<th>Elongation in 8 inches percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>A366-58T</td>
<td>40,800</td>
<td>48,500</td>
<td>21.7</td>
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<tr>
<td>A245-61T</td>
<td>52,100</td>
<td>66,100</td>
<td>22.3</td>
</tr>
</tbody>
</table>
TABLE 4

RESULTS PERTAINING TO STATIC BEHAVIOR OF GIRDERS WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR

<table>
<thead>
<tr>
<th>Girder</th>
<th>Flange Thickness in.</th>
<th>Initial $\delta/T$</th>
<th>Maximum $\delta/T$</th>
<th>Maximum Load $\sigma_1$ psi</th>
<th>Maximum Load $\sigma_2$ psi</th>
<th>Permanent Deformation in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>FT-1</td>
<td>3/16</td>
<td>3.2</td>
<td>6.8</td>
<td></td>
<td></td>
<td>9/32</td>
</tr>
<tr>
<td>FT-2</td>
<td>3/8</td>
<td>2.8</td>
<td>5.3</td>
<td>19,800</td>
<td>-2,700</td>
<td>3/32</td>
</tr>
<tr>
<td>FT-3</td>
<td>1/2</td>
<td>2.1</td>
<td>2.9</td>
<td>22,100</td>
<td>-4,700</td>
<td>3/32</td>
</tr>
<tr>
<td>FT-9</td>
<td>1/2</td>
<td>2.1</td>
<td>2.2</td>
<td></td>
<td></td>
<td>0</td>
</tr>
<tr>
<td>FT-4</td>
<td>5/8</td>
<td>3.3</td>
<td>4.5</td>
<td>23,100</td>
<td>-5,700</td>
<td>4/32</td>
</tr>
<tr>
<td>FT-5</td>
<td>3/4</td>
<td>2.8</td>
<td>3.4</td>
<td>19,600</td>
<td>-5,200</td>
<td>0</td>
</tr>
<tr>
<td>FT-6</td>
<td>1</td>
<td>2.1</td>
<td>3.5</td>
<td>21,700</td>
<td>-5,600</td>
<td>0</td>
</tr>
<tr>
<td>FT-10</td>
<td>1</td>
<td>3.4</td>
<td>3.2</td>
<td></td>
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<td>0</td>
</tr>
<tr>
<td>Girder</td>
<td>Cycles to Failure</td>
<td>Range $\delta/T$</td>
<td>Range, $\sigma_1$ psi</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>--------</td>
<td>------------------</td>
<td>-----------------</td>
<td>----------------------</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>FT-1</td>
<td>251,900</td>
<td>0.60</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>FT-2</td>
<td>293,900</td>
<td>0.91</td>
<td>15,400</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FT-3</td>
<td>915,600</td>
<td>0.66</td>
<td>17,400</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FT-9</td>
<td>508,600</td>
<td>0.77</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FT-4</td>
<td>476,900</td>
<td>0.37</td>
<td>17,900</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FT-5</td>
<td>408,300</td>
<td>0.91</td>
<td>15,100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FT-6</td>
<td>863,400</td>
<td>0.53</td>
<td>17,000</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FT-10</td>
<td>621,400</td>
<td>0.77</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>
### TABLE 6

**RESULTS OF FLANGE RIGIDITY TESTS ON GIRDERs SUBJECTED TO SHEAR AND BENDING**

<table>
<thead>
<tr>
<th>Girder</th>
<th>Flange Size in.</th>
<th>Initial δ/t Panel 1</th>
<th>Initial δ/t Panel 2</th>
<th>Max. Load δ/t Panel 1</th>
<th>Max. Load δ/t Panel 2</th>
<th>Range δ/t Panel 1</th>
<th>Range δ/t Panel 2</th>
<th>Cycles to Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>FTSB-1</td>
<td>4 3/8 x 1/4</td>
<td>3.64</td>
<td>4.79</td>
<td>3.72</td>
<td>4.70</td>
<td>1.38</td>
<td>1.25</td>
<td>-</td>
</tr>
<tr>
<td>FTSB-2</td>
<td>2 7/8 x 3/8</td>
<td>3.86</td>
<td>3.47</td>
<td>4.21</td>
<td>3.50</td>
<td>1.32</td>
<td>1.06</td>
<td>40,000</td>
</tr>
<tr>
<td>FTSB-3</td>
<td>2 1/8 x 1/2</td>
<td>3.34</td>
<td>3.96</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>32,000</td>
</tr>
<tr>
<td>FTSB-4</td>
<td>1 5/8 x 5/8</td>
<td>3.18</td>
<td>3.03</td>
<td>3.89</td>
<td>3.38</td>
<td>1.11</td>
<td>0.88</td>
<td>49,000</td>
</tr>
<tr>
<td>FTSB-5</td>
<td>1 3/8 x 3/4</td>
<td>3.76</td>
<td>1.94</td>
<td>3.78</td>
<td>2.85</td>
<td>1.04</td>
<td>0.72</td>
<td>84,000</td>
</tr>
<tr>
<td>FTSB-6</td>
<td>1 x 1</td>
<td>4.20</td>
<td>3.50</td>
<td>4.32</td>
<td>4.52</td>
<td>1.21</td>
<td>1.23</td>
<td>101,000</td>
</tr>
<tr>
<td>Specimen</td>
<td>Nominal Stress Cycle psi</td>
<td>Stiffener Size in.</td>
<td>Cycles to Failure</td>
<td>Ultimate Load lbs.</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>----------</td>
<td>--------------------------</td>
<td>--------------------</td>
<td>-------------------</td>
<td>-------------------</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VST 16-8</td>
<td>2.7 to 10.7</td>
<td>4.3 to 17.0</td>
<td>1/16 x 1/2</td>
<td>239,000</td>
<td>76,500</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VST 8-4</td>
<td>&quot;</td>
<td>&quot;</td>
<td>1/8 x 1/2</td>
<td>55,000</td>
<td>82,600</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VST 8-6</td>
<td>&quot;</td>
<td>&quot;</td>
<td>1/8 x 3/4</td>
<td>52,000</td>
<td>78,000</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VST 8-10</td>
<td>&quot;</td>
<td>&quot;</td>
<td>1/8 x 1 1/4</td>
<td>52,000</td>
<td>74,000</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>VST 8-16</td>
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<td>&quot;</td>
<td>1/8 x 2</td>
<td>250,000</td>
<td>84,000</td>
<td></td>
<td></td>
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</table>
TABLE 8

MAXIMUM WEB DEFLECTIONS IN GIRDERS WITH VARIOUS STIFFENER SIZES

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Initial $\delta/T$</th>
<th>Maximum Load $\delta/T$</th>
<th>Maximum Range $\delta/T$</th>
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</thead>
<tbody>
<tr>
<td>VST 16-8</td>
<td>3.4</td>
<td>3.5</td>
<td>1.2</td>
</tr>
<tr>
<td>VST 8-4</td>
<td>2.7</td>
<td>2.8</td>
<td>0.5</td>
</tr>
<tr>
<td>VST 8-6</td>
<td>2.4</td>
<td>2.8</td>
<td>0.7</td>
</tr>
<tr>
<td>VST 8-10</td>
<td>2.0</td>
<td>2.2</td>
<td>0.7</td>
</tr>
<tr>
<td>VST 8-16</td>
<td>1.7</td>
<td>2.5</td>
<td>0.6</td>
</tr>
</tbody>
</table>
FIG. 1. MAXIMUM DEFLECTIONS AT THE CENTERS OF SQUARE SHEAR PANELS WITH INITIAL DEFLECTIONS AND RIGID BOUNDARIES.

FIG. 3. DEFLECTIONS AT THE CENTERS OF SQUARE SHEAR PANELS WITH RIGID BOUNDARIES, $\gamma = \infty$, AND WITH BOUNDARIES HAVING NO FLEXURAL RIGIDITY, $\gamma = 0$. 
FIG. 2  DIAGONAL TENSION THEORY COEFFICIENTS AS DERIVED BY WAGNER.
FIG. 4  RELATIONSHIP BETWEEN LATERAL WEB DEFLECTION, FLANGE RIGIDITY AND LOAD FOR PLATE GIRDER WEB PANELS.
FIG. 5 THEORETICAL BUCKLING COEFFICIENTS FOR SHEAR WEBS vs. RIGIDITY OF TRANSVERSE STIFFENERS.
FIG. 6 LIMITING VALUES OF STIFFENER RIGIDITY AS A FUNCTION OF ASPECT RATIO.
FIG. 7 PREDICTED BUCKLING PATTERNS OF IDEAL SHEAR PANELS.
FIG. 8  PRINCIPAL COMPRESSIVE STRESSES IN AN IDEAL PANEL SUBJECTED TO SHEAR AND BENDING.
FIG. 9 PREDICTED BUCKLING PATTERN OF AN IDEAL WEB PANEL SUBJECTED TO SHEAR AND BENDING.
FIG. 10  GIRDER WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR.
FIG. 11 GIRDERS WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR AND MOMENT.
Stiffener Sequence

FIG. 13

WELDING SEQUENCE FOR TEST SPECIMENS.

Numbers indicate sequence
E 6013 Electrode
4 inch passes except as shown
FIG. 14. FATIGUE LIFE OF SHEAR GIRDERS vs FLANGE RIGIDITY PARAMETER
FIG. 15. FAILURES IN GIRDERS WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR.
FIG. 16. INITIAL WEB DEFORMATIONS IN GIRDER FT-3 AND FT-4.

Note: Web deflections are given in inches.
Note: Web deflections given in inches.

FIG. 17 WEB DEFLECTIONS AT MAXIMUM TEST LOAD IN GIRDER FT-2.
FIG. 18. WEB DEFLECTIONS AT MAXIMUM TEST LOAD IN GIRDERS FT-3 AND FT-9.

Note: Web deflections are given in inches.
Note: Web deflections are given in inches.

FIG. 19. WEB DEFLECTIONS AT MAXIMUM TEST LOAD IN GIRDER FT-6.
FIG. 20. PRINCIPAL TENSILE STRESS vs LOAD AT CENTER OF TEST PANELS OF GIRDERS WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR.
FIG. 21  FATIGUE LIFE vs FLANGE RIGIDITY PARAMETER FOR GIRDER WITH VARIOUS
FLANGE SIZES SUBJECTED TO SHEAR AND MOMENT.
FIG. 22. INITIAL WEB DEFLECTION PATTERNS FOR GIRDERS WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR AND MOMENT.

Note: Numbers give values of $w_{\text{max}}/t$ for individual buckles.
FIG. 23 WEB DEFLECTION PATTERNS AT MAXIMUM LOAD WITH FAILURE LOCATIONS FOR GIRDER WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR AND MOMENT.

Note: Numbers give value of $w_{\text{max}}/t$ for individual buckles.
FIG. 24 RELATIVE MAGNITUDES OF WEB FLEXING ACTION ALONG STIFFENERS IN GIRDERs WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR AND MOMENT.

Note: Darkened areas indicate locations of failure.
FIG. 25. FATIGUE LIFE vs STIFFENER RIGIDITY PARAMETER FOR GIRDERS WITH VARIOUS STIFFENER SIZES.
NOTE: Numbers give values of $\psi_{max}/t$ for individual buckles.

FIG. 26. INITIAL WEB DEFLECTION PATTERNS FOR GIRDERs WITH VARIOUS STIFFENER SIZES.
NOTE: Numbers give value of $w_{max./t}$ for individual buckles.

FIG. 27. WEB DEFLECTION PATTERNS AT MAXIMUM FATIGUE TEST LOAD AND FAILURE LOCATIONS FOR GIRDERs WITH VARIOUS STIFFENER SIZES.
NOTE: Darkened areas indicate locations of failure.

FIG. 28. RELATIVE MAGNITUDES OF WEB FLEXING ACTION ALONG STIFFENERS IN GIRDERS WITH VARIOUS STIFFENER SIZES.
FIG. 29. AVERAGE STIFFENER STRAINS AT MID-HEIGHT OF STIFFENER vs APPLIED LOAD.

FIG. 30. STIFFENER FORCE AT MID-HEIGHT OF STIFFENER vs APPLIED LOAD.
FIG. 31. BENDING STRAIN AT MID-HEIGHT OF STIFFENERS vs APPLIED LOAD.
FIG. 32. MAXIMUM LATERAL STIFFENER DEFLECTION vs APPLIED LOAD.
FIG. 33. PRINCIPAL WEB STRESSES in GIRDER VST8-6.
FIGURE 34. MAXIMUM LATERAL WEB DEFLECTION vs FLANGE RIGIDITY PARAMETER FOR GIRDERS WITH VARIOUS FLANGE SIZES SUBJECTED TO SHEAR.
FIG. 35. PRINCIPAL COMPRESSION STRESSES IN AN IDEAL WEB PANEL SUBJECTED TO THE GIVEN STRESSES.