Research on plate girders at Lehigh University

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Research on Plate Girders at Lehigh University

Recherche sur les poutres de grandes dimensions à âme mince à l'Université de Lehigh

Forschung an hohen Blechträgern an der Lehigh University

ALEXIS OSTAPENKO BEN T. YEN LYNN S. BEEDLE

1. INTRODUCTION

The purpose of this discussion is to report on the most recent work which is currently in progress at Lehigh University and on some of the results obtained. Particular areas which will be discussed and which are in addition to and will supplement the presentation by Mr. Massonnet[1] are:

1. Edge loading - girders with loads applied to the compression flange between transverse stiffeners (Item 11 of the research problems proposed by M. Massonnet).
2. Ultimate strength of unsymmetrical plate girders.
3. Fatigue strength of plate girders (Item 2).

2. EDGE LOADING

Quite often the question arises in design as to whether a bearing stiffener is needed when the loads are relatively small. This problem becomes important not only for service loads, but also for erection loads, when for example the girder must be rolled out as a cantilever. Figure 1 shows a typical girder with a web panel under stresses due to bending moment M, shear V, and the edge loading q. Theoretical analysis of the buckling strength of a plate girder web subjected to an edge loading has been performed recently by Wilkesmann,[2] Klöppel and Wagemann,[3] Warkenthin,[4] and Kawano and Yamakoshi.[5] In all these solutions the

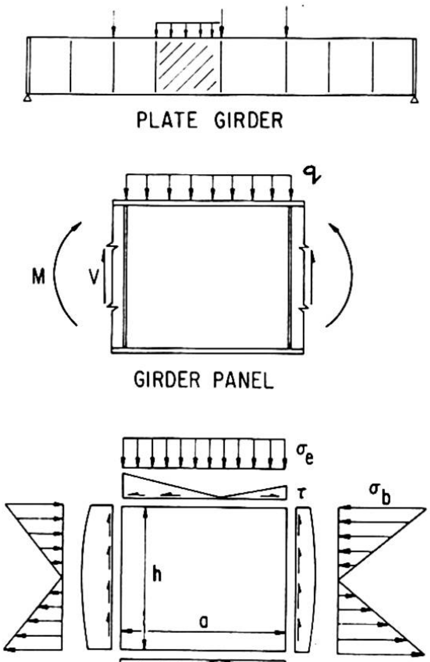

Figure 1

edges of the plate were assumed to be simply supported. Basler proposed a simplified buckling analysis considering the top edge of the web to be simply supported or fixed.(6) However, neither the effect of bending nor of shear was included. Basler's approach was later incorporated in the Specification of the American Institute of Steel Construction.(7)

Normally the ultimate strength of plates is substantially greater than the buckling strength; but, in general, there is no direct relationship between these two quantities. Since all previous work on edge loaded web has been concerned only with the elastic buckling strength, a series of tests was conducted at Lehigh University to investigate the ultimate strength and to arrive at a method of evaluating it.(8)

The theoretical buckling stress was computed by means of the finite difference method. The web plate was assumed to be fixed at the flanges and simply supported at the stiffeners. This assumption is based on the observations of the web behavior during tests on plate girders of ordinary proportions. Figure 2 gives the buckling coefficient K as a function of the aspect ratio, α = a/h, and the bending stress ratio σ_b/σ_e. The bending stress is seen to have a pronounced effect on the value of the buckling coefficient and thus on the critical edge stress.

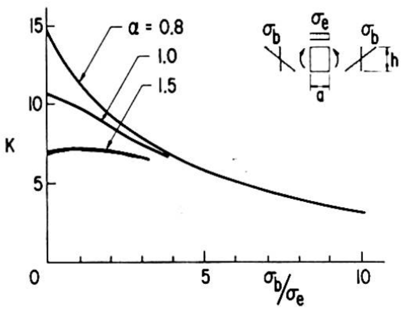

Figure 2

Ten tests were conducted on three plate girder specimens. The principal parameters were:

1. Aspect ratio a/h which varied from 0.8 to 1.6.
2. The ratio of the bending stress to the edge stress, σ_b/σ_e,

which varied from 0 to 5.0.

The edge loading consisted of two to four concentrated loads spread over the compression flange through 1.5 inch thick plates to simulate uniform distribution.

The ultimate strength from the tests is plotted non-dimensionally in Fig. 3. It is remarkable that the plotted points lie in a relatively narrow band seemingly independent of the bending stress ratio σ_b/σ_e. A horizontal line passing through the ordinate equal to 3.0 is seen to give a conservative estimate of the ultimate strength. It follows then that the ultimate strength can be approximated fairly well from the buckling strength. The resulting equation for the allowable intensity of the edge stress σ_a, with a factor of safety against the ultimate strength equal to 2.0, is shown in Fig. 3 by the light horizontal line.

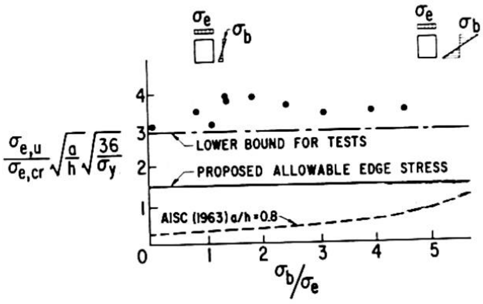

Figure 3

$$\sigma_a = \frac{\sigma_{e,u}}{\text{F.S.} = 2.0} = \frac{4000}{(d/t)^2\sqrt{a/h}} \sqrt{\frac{36}{\sigma_y}}\ K \quad \text{(ksi)}$$

where

h = girder depth
t = web plate thickness
a = distance between transverse stiffeners
σ_y = yield stress of the web plate in ksi
K = buckling coefficient

Pending results of an analytical study and additional tests, this formula is subjected to the following limitations: $34\ \text{ksi} < \sigma_y < 45\ \text{ksi}$; $240 < h/t < 300$; $\sigma_b/\sigma_e < 5$; $0.8 < a/h < 2.0$.

For comparison, the dashed curve in Fig. 3 gives the allowable edge stress according to the American Institute of Steel Construction Specification ($a/h = 0.8$, loaded edge fixed). Although quite conservative, this curve leads to a reduction of the margin of safety with an increasing bending stress.

3. UNSYMMETRICAL GIRDERS

Most prior research on the ultimate strength of plate girders has been concerned with symmetrical members, that is, girders with the centroidal axis at the mid-depth, such as the cross section in Fig. 4(a).[9,10,11,12] However, many plate girders are unsymmetrical in the sense that they have unequal flanges and their centroidal axis is close to the larger flange, for example, composite and orthotropic deck girders shown in Figs. 4(b) and 4(c). Designing such girders as symmetrical members would lead to results that are either overconservative or unconservative depending on whether the larger portion of the web is in tension or in compression. Currently, a study is being conducted at Lehigh University on the ultimate strength of unsymmetrical plate girders subjected to bending, shear, and a combination of bending and shear.

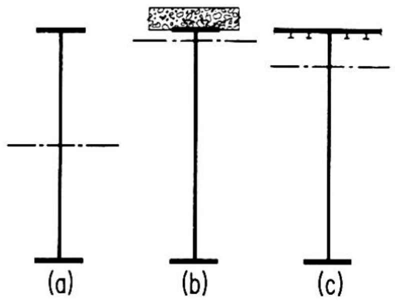

Figure 4

The experimental phase of the study consisted of fourteen tests conducted on four plate girders with transverse stiffeners.[13,14] One flange had approximately double the area of the other flange.

In the new method of analysis developed as part of the theoretical work, the ultimate strength of a plate girder panel is assumed to be controlled principally by the following three contributions shown schematically in Fig. 5:

a) Beam strength up to the point at which web buckling would theoretically occur,

b) Post-buckling strength of the web, and,

c) Frame action which is based on the formation of a panel mechanism with plastic hinges in the flanges.

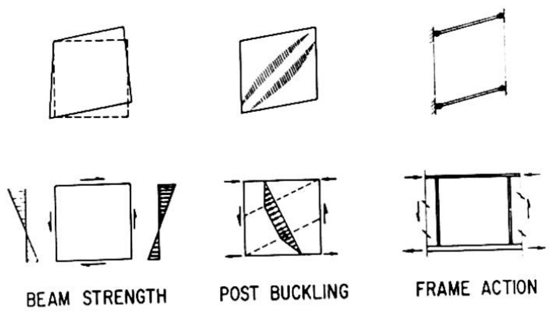

Figure 5

An interaction relationship between moment and shear for a typical plate girder panel is shown in Fig. 6. The non-dimensionalizing values are the ultimate capacity in shear V_u and the ultimate bending capaci-

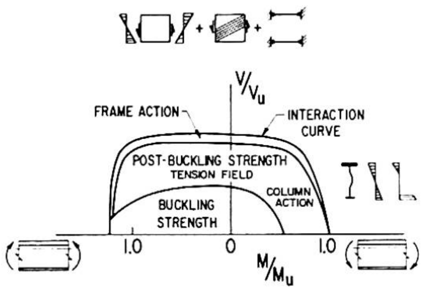

Figure 6

ty M_u, a moment which acts to produce compression in the larger portion of the web. To the right is the interaction plot for a loading condition when a larger portion of the web is in compression and to the left is the interaction plot for a loading condition when a larger portion of the web is in tension. The ultimate strength is seen to be different for these two cases.

The buckling strength (called beam strength in Fig. 5) is obtained for a combination of shearing and bending stresses assuming the web plate to be fixed at the flanges and simply supported at the stiffeners. It represents the upper limit of ordinary beam action.

The post-buckling strength of the web is developed differently in shear and in bending. In shear, it means a tension field assumed to be of the pattern shown in Fig. 5. In bending it means redistribution of the additional web stresses toward the compression flange.

The frame action contributes to the shear capacity by developing a panel mechanism with plastic hinges in the flanges. A portion of the web equal to 20 times the thickness is considered to be part of the flange and the axial flange force is included in the evaluation of the plastic moments.

The ultimate strength in shear is given as a sum of all three contributions as shown in the top sketch of Fig. 6.

The ultimate strength on pure bending is limited by the buckling capacity of the compression flange or the yielding of the tension flange with a portion of the web. Shear reduces the bending strength by contributing to the flange forces through the development of a full or partial tension field and frame mechanism.

The effect of the location of the centroidal axis on the mode of failure of a girder panel under pure bending is depicted in Fig. 7. Fixed values for this plot are the yield stress, the bracing spacing for the compression flange, and the ratio of the compression flange area to the web area. Compression flange failure is to be expected in the region

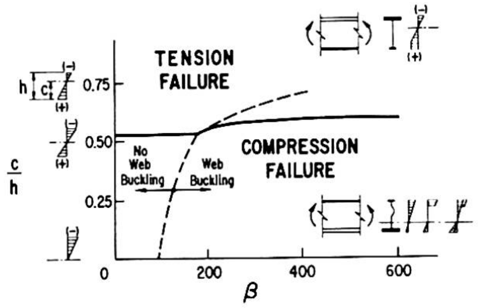

Figure 7

below the heavy solid line whereas tension flange yielding would be expected above that line. The dashed line separates the regions in which the web will buckle or will not buckle before the ultimate panel strength is reached.

The method is compared with the available test results in Fig. 8. Symmetrical and unsymmetrical girders are grouped separately for each of the loading cases: shear, combined bending and shear, and bending. The average agreement is within 5 percent with the maximum deviation being approximately 12 percent.

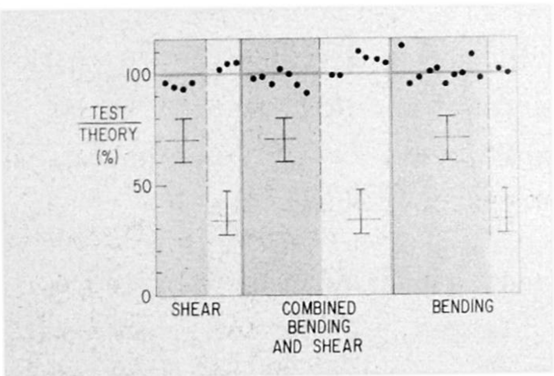

Figure 8

The method has also been extended to girders with one longitudinal stiffener and the correlation with test results is within the same range.

4. FATIGUE

A special problem arises in "thin-walled" plate girders, because of web deflection. Figure 9 shows exaggerated deformation in a panel as accentuated by painted black lines. Under repeated loading, a web plate moves back and forth, and fatigue cracks occur at the boundaries.(15,16,17)

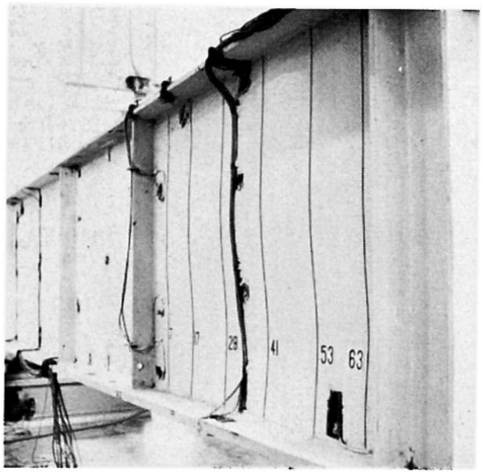

Figure 9

The most probable locations of the fatigue cracks are compared with the contours of the web deflection in Fig. 10. Crowding of controus indicates high curvature, hence high plate bending stress and thus the possibility of a fatigue crack.

A preliminary analysis has been made, and Fig. 11 gives an example of the results.(18) The web panel of a test girder

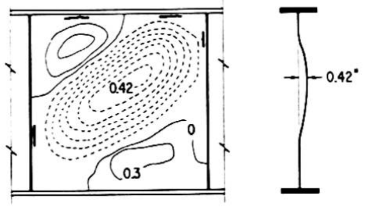

Figure 10

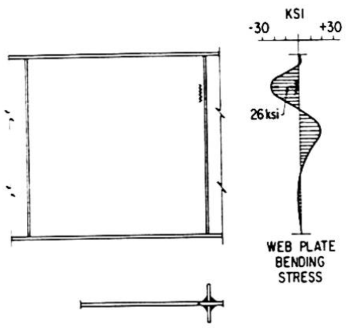

Figure 11

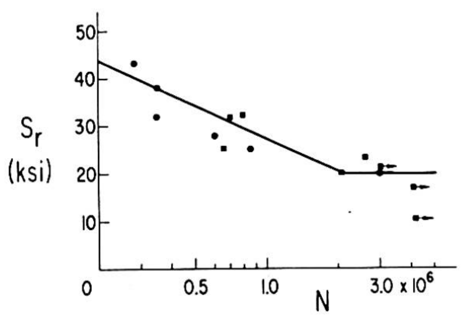

Figure 12

is shown with a fatigue crack at the toe of the web fillet weld. The web plate bending stresses have been estimated from web deflections measured under load, and Fig. 11 shows that the crack occurred at the point of highest stress.

The results of all fatigue tests for which deflection data are available are shown in Fig. 12. The relationship is a typical S-N curve between the range of variation of web plate bending stress and the number of load applications that were needed to develop a crack.

Generally speaking, large web deflections generate large stresses and thus lead to early cracks. If the web deflections are small, the corresponding stresses are small; and there is a run-out.

Based on these results, limits of web plate bending stresses and web deflections are being established. Two "proof" girders, designed according to preliminary rules, have endured at least 2,000,000 cycles without cracks.

5. CONCLUDING REMARKS

It is the eventual objective of the Lehigh research to develop design recommendations for symmetrical and unsymmetrical girders, both transversely and longitudinally stiffened, taking advantage of the most recent findings both in the United States and from abroad.

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SUMMARY

Current topics of research on plate girders and some of the findings are discussed. A formula is proposed for the ultimate capacity of a girder panel subjected to edge loading and bending moment; it is based on an elastic buckling analysis and ultimate strength tests. Ultimate strength of unsymmetrical plate girders is given by the sum of the beam action, post-buckling strength, and frame action; the theory is in good agreement with test results. Occurrence of fatigue cracks in plate girder web is dependent on the lateral web deflections and can be controlled by limiting the web slenderness ratio.

RÉSUMÉ

Cet article présente les recherches en cours ainsi que quelques résultats obtenus sur les poutres de grandes dimensions à âme mince. Une formule est proposée pour la charge de ruine d'un panneau d'âme de poutre situé entre deux raidisseurs assujetti à une charge répartie agissant sur la semelle supérieure. Cette formule est basée sur l'analyse élastique du voilement, et sur la charge de ruine obtenue lors des essais. La charge de ruine des grandes poutres à âmes minces est donnée comme superposition d'une action de flexion, de l'effet du voilement postcritique, et d'un mécanisme de panneau; la théorie concorde bien avec les résultats d'essais. L'apparition de fissures dans l'âme des poutres dues à la fatigue dépend de la flexion latérale de l'âme et peut être anticipée en limitant l'élancement de l'âme.

ZUSAMMENFASSUNG

Die Themen und einige Ergebnisse der gegenwärtigen Forschung auf dem Gebiet der Blechträger werden erörtert. Eine Formel für die Traglast von Stegblechfeldern unter Längsrandbelastung und Biegung ist vorgeschlagen. Diese Formel basiert auf der elastischen Beultheorie sowie auf Ergebnissen von Traglastversuchen. Die Traglast von unsymmetrischen Blechträgern wird aus der Balkenwirkung, der überkritischen Tragfähigkeit und der Rahmenwirkung zusammengesetzt angenommen; diese Theorie stimmt mit Versuchsergebnissen gut überein. Das Auftreten von Ermüdungsrissen im Steg ist von der Stegdurchbiegung abhängig und kann durch eine Begrenzung der Stegschlankheit verhindert werden.