Preliminary Investigation of

STEEL GIRDER END PANEL SHEAR RESISTANCE

Charles Sikorsky
Office of Earthquake Engineering
Division of Engineering Services

Chia-Ming Uang
Department of Structural Engineering
University of California, San Diego

Prepared for
Lian Duan
Structure Design Services
Division of Engineering Services

PURPOSE

The Caltrans Division of Research and Innovation (DRI) evaluate numerous research problem statements for funding every year. In order to qualify for funding, a Preliminary Investigation (PI) must be completed for each problem statement to assess the state of the art as it relates to the particular problem, and to improve the scope of the researchable problem based on work available in the literature. The technical literature, both nationally and internationally, is reviewed. The views and conclusions in cited works, while generally peer reviewed or published by authoritative sources, may not be accepted without qualification by all experts in the field.

EXECUTIVE SUMMARY

BACKGROUND

Prior to 1973, steel bridges in California were designed based on Allowable Stress Design and the shear design of web and transverse stiffeners was based on the average shear stress in the web. The tension field action equation similar to the current tension field action was introduced in shear design for all locations for both working stress (WSD) and load factor design (LFD) in the AASHTO Standard Specifications, 11th Edition (AASHTO 1973). Later, the shear capacity was reduced for the first (or end) panel locations designed per WSD and LFD in AASHTO 12th Edition (AASHTO 1977), and AASHTO 1978 Interim Specifications (AASHTO 1978), respectively. The format of current shear design was introduced in 1984 - 1986 AASHTO Interim Specifications (AASHTO 1986). The current AASHTO LRFD (2009) and LRFR (2008) Specifications exclude the post-buckling tension-field action for the end web panel.

The AASHTO LRFD (AASHTO, 2009) and LRFR (AASHTO, 2008) Specifications both specify that (1) for stiffened interior web panels, the nominal shear resistance is developed from a combination of beam action and post-buckling tension-field action; and (2) for an end web panel adjacent to a simple support, the nominal shear resistance is limited to beam action in order to provide an anchor for the tension field in adjacent interior panels. Since the thickness of a steel girder web is usually constant in California and the maximum shear force effects occur at the support, the end web panel becomes the controlling component for steel girder design, evaluation and load rating. AASHTO LRFD and LRFR Specifications do not provide any provisions to allow the end web panel to be designed for tension-field action. Therefore, the only means available to increase shear resistance for an end web panel is to increase the web thickness, which is costly and sometimes impractical.

As a result, there are many steel girder bridges, where the end shear controls the overall bridge rating and the rating factor falls below 1.0. Rating factors less than 1.0 mean the Department must either strengthen the bridge or restrict vehicular loading on the bridge. Either option would be very costly and have a negative impact on the travelling public, particularly the extra-legal weight permit industry.

PROBLEM STATEMENT

There are no specific provisions in the AASHTO LRFD and LRFR Specifications to allow post-buckling tension-field action to be considered in the design of the steel plate girder end panels. Limited tests also show the composite slab effect would also increase the shear strength of the plate girder, which is ignored in the AASHTO Specifications. There is an urgent need to develop realistic and practical shear design provisions that include tension-field action for steel girder bridge design, evaluation and rating to avoid costly strengthening of girder webs or restriction of extra-legal permit traffic.

SUMMARY OF FINDINGS

The contribution of tension field action to the load carrying capacity of plate girders has long been recognized, beginning with the work by Basler (1961). Unfortunately, tension field action is limited to the interior panels of plate girders in the AASHTO (2009) and AISC (2005) codes as shown in the work by White and Barker (2008) and White et al. (2008). Recent work by Yoo and Lee (2006) demonstrates this assumption is too conservative and points out that tension field action is possible in the end panel of steel plate girders. More importantly, the recent edition of the Guide to Stability Design Criteria for Metal Structures (SSRC, 2010) provides one possible detail for end panels designed for tension-field action, however experimental evidence supporting this detail is limited. The contribution from the composite deck slab and girder (Shanmugam and Baskler, 2003) should also be explored.

RECOMMENDATION

Fund research to investigate post-buckling tension field action in the end panels of steel plate girders to form the basis for recommendation to change the current LRFD and LRFR Specifications that relate to the design, and evaluation and rating of shear in the end web panel of steel girders used in highway bridges.

REVIEW OF SELECT WORK

The techical literature was reviewed for work investigating tension field action in steel plate girders. After an extensive search, the following references were identified as significant to this problem and are summarized below. As a minimum, the abstract and summary from each paper are presented below.

STRENGTH IN SHEAR

Konrad Basler, J of the Structural Division, ASCE, October 1961

ABSTRACT

A study of the shear strength of plate girders is presented. In utilizing the post-buckling strength offered by the transverse stiffening of girders, new design rules are proposed. The new approach is checked with ultimate load tests carried out at Fritz Engineering Laboratory.

Summary

In perhaps one of the first papers on the subject, Basler (1961) presented new design rules for plate girders that utilize the post-buckling strength due to transverse stiffeners of plate girders. With the introduction of plate girders, also came the realization that shear could be carried by means other than beam action. At that time shearing stresses in the webs of plate girders were analyzed according to classical beam theory (Navier and St Venant). To satisfy the theory's condition of small deformations, transverse stiffeners must be spaced close enough so that instability due to shear is excluded. Basler (1961) went on to develop design methods to specify the size of stiffeners needed for efficient girder design to achieve the required tension field action.

STRENGTH OF PLATE GIRDER WEB PANELS UNDER PURE SHEAR

Sung C. Lee and Chai H. Yoo, J of Structural Engineering, ASCE, February 1998

ABSTRACT

Nonlinear analyses have been conducted on three-dimensional finite element models of transversely stiffened plate girder web panels (without longitudinal stiffeners) subjected to pure shear, including the effects of initial out-of-flatness. Currently, the design equations for shear in plate girder web panels in the American Association of State Highway and Transportation Officials (AASHTO) and the American Institute of Steel Construction (AISC) specifications account for beam action shear buckling strength and post-buckling strength separately and combine these resisting capacities based on the aspect ratio of the web panels. Although equations in these specifications predict the overall shear strength with reasonable accuracy, they often underestimate the beam action shear buckling strength, due to an underestimation of the rigidity at the flange-web juncture, and often overestimate the post-buckling strength of certain web panels, as a result of excluding the effect of out-of-plane bending stresses. Based on a parametric study of numerical results, new design equations are proposed for the determination of ultimate shear strengths of web panels. To validate these equations, ultimate shear strengths computed from the equations are compared with existing experimental data.

Introduction

The primary functions of the web plate in a plate girder are to maintain the relative distance between the top and bottom flanges and to resist the introduced shearing force. In most practical ranges of span lengths for which a plate girder is designed, the induced shearing force is relatively low as compared with the axial forces in the flanges resulting from flexure. As a result, the thickness of the web plate is generally much smaller than that of the flanges. Consequently, the web panel buckles at a relatively low value of the applied shear loading. The webs are often reinforced with transverse stiffeners to increase their buckling strength, and web design involves finding a combination of an optimum plate thickness and stiffener spacing that renders economy in terms of the material and fabrication cost. The design methods of plate girder webs are divided into two categories: (1) allowable stress design based on elastic buckling as a limiting condition; and (2) strength design based on ultimate strength, including post-buckling as a limit state.

Web buckling due to shear is essentially a local buckling phenomenon. Depending upon the geometry, the web plate is capable of carrying additional loads considerably in excess of that at which the web starts to buckle, due to post-buckling strength. Taking advantage of this reserve strength, a plate girder of high strength/weight ratio can be designed. Despite the fact that this postbuckling behavior was discovered as early as 1886 by Wilson and diagonal tension theory was developed by Wagner (1931), elastic buckling was used as a basis for the design of plate girder webs almost exclusively until the 1960s. This was due primarily to the fact that formulas for predicting the elastic shear buckling strength of web panels are relatively simple and had been known for many years, whereas a comprehensive and simple procedure to account for the observed postbuckling strength had not yet been developed.

In the late 1950s, however, extensive studies were made on the postbuckling behavior of web panels by Basler and Thurlimann (1959). As a result of these and other studies, the American Institute of Steel Construction (AISC) first adopted post-buckling strength into its specifications in 1963, and in 1973 the American Association of State Highway and Transportation Officials (AASHTO) followed suit.

Summary and Concluding Remarks

Although existing failure theories on web panels, including models suggested by Basler (1963) and Porter et at. (1975), can predict the ultimate shear strength of web panels adequately for a wide range of practical designs, the assumed failure mechanisms employed in these theories may not accurately represent the shear behavior of web panels with regard to the following points of view:

1. The assumption that the boundary condition at the flange-web juncture is simply supported gives much too conservative shear strengths for many plate girder web panels. The nonlinear finite element analyses show that the boundary condition at the juncture is much closer to a fixed support.

2. In all existing failure mechanisms, the effects of through thickness bending stress on the ultimate shear strength are neglected. However, it has been found that considerable bending stresses develop at failure, resulting in substantial reduction of the ultimate shear strength for web panels with low slenderness.

3. The flange rigidity appears to have little effect on the post-buckling strength of web panels; however, it affects the elastic shear buckling strength of web panels as it affects the degree of restraint at the flange-web juncture.

From the parametric study of the data generated by the nonlinear finite element analyses presented, a set of new design equations has been formulated. Comparative studies demonstrate that the proposed design equation can accurately predict the ultimate shear strength of plate girder web panels. These design equations have been validated by a series of comparative studies, including the comparison with existing test data as presented in the paper.

It has also been found that, for web panels having low web slenderness ratios, the ultimate shear strength reduces significantly as the initial distortion increases. Based on analyses of a large number of web panels with large initial distortions close to the maximum allowed by the *ANSI / AASHTO /* AWS D1.5-96 Bridge Welding Code (1996), equations to determine the strength reduction have been formulated.

STEEL-CONCRETE COMPOSITE PLATE GIRDERS SUBJECTED TO SHEAR LOADING

N. E. Shanmugam and K. Baskar, J of Structural Engineering, ASCE, September 2003

ABSTRACT

The paper is concerned with an experimental investigation on simply supported steel-concrete composite plate girders subject to shear loading. Four composite and two bare steel plate girders were tested to failure in order to study their ultimate strength behavior. The effect of composite action with the concrete slab on tension field action in the web panels is the main focus of the study. Extensive strain measurements were made on the web panels in order to measure the extent of tension field action. It is observed from the tests that the ultimate shear capacity of composite plate girders increases significantly compared to bare steel girders. The test specimens were analyzed using finite element modeling and the predicted results compared with the corresponding experimental values. The comparison shows good agreement thus confirming the accuracy of the modeling.

Summary

Post-buckling behavior of web plates subjected to shear loading was first noticed by Wagner (1931) for plate girders with slender webs and stiffer flanges. Later in the 1960's, Basler (1961) made an attempt to determine the post buckling behavior of plate girders used in civil engineering structures. He assumed that flanges of the commonly used girders were flexible and could not withstand the lateral loading imposed by the inclined tension field. Based on this assumption, Basler used the off diagonal yield band in the web plate and developed the failure mechanism. Later, Cooper et al. (1964) conducted four tests on two welded constructional alloy steel plate girders to investigate the applicability of the shear strength theory to girders. Shear strength theory was used to calculate the shear capacity of plate girder webs based on elastic shear buckling and post-buckling shear capacity due to tension field action of web plate. It was concluded that the shear strength theory could be applied safely to plate girders made up of any structural steel used today.

In normal practice today, plate girders are used to support reinforced concrete deck slabs in bridge and industrial constructions. When the top flange of the girder is effectively connected to a reinforced concrete deck slab by means of sufficient number of shear connectors, the behavior will be different from that of plain steel plate girders because of the composite action between the steel girder and concrete slab. The ultimate strength analysis of plain steel plate girders includes the advantage of tension field action in the web plate. If the slab is assumed to be laid over the girder, the girder may be designed as an ordinary steel plate girder. When the concrete slab is connected effectively to the girder, there may be some variation in the tension field due to the composite action.

After reviewing the literature, Shanmugam and Basker (2003) concluded the available information on steel–concrete composite plate girders was not sufficient to understand the elastic and ultimate load behavior of such girders and that a detailed study was needed to understand the contribution of the concrete slab to tension field action. Studies were proposed in which two sets of steel–concrete composite plate girders were tested to failure. Each girder consisted of two rectangular panels with necessary vertical and horizontal stiffeners. Details of all the specimens are summarized in Table 1. The girders were simply supported at the ends and at the locations corresponding to interior transverse stiffeners; the specimens had an effective span of 2400 mm and an overall length of 3246 mm. All the girders were tested to failure under a concentrated load applied at the midspan.

Even though an increase in the ultimate load-carrying capacity and effectiveness of the web plate due to composite action was observed in CPG1 and CPG2, the girders were subjected to a sudden shear failure immediately after the ultimate load. This behavior is catastrophic and cannot be allowed in any engineering structure. Therefore, there is a need to improve the behavior of such girders. Studies on shear enhancement of a high strength concrete plate carried out by Marzouk and Jiang (1996) show that shear links provided in the slab improve the shear behavior of structures. Based on this study, a new concept of providing shear links in the deck slab was attempted in CPG3 and CPG4 as shown in Figure 17. These girders were provided with additional shear links in the deck slab so that a shear failure of the slab could possibly be eliminated.

Table 1: Details of Test Specimen

Specimen			**Flanges**					
	Panel aspect ratio		Top		Bottom		Web	**Slab Properties**
	(a/d)	d/t	B_f (mm)	T_f (mm)	B_f (mm)	T_f (mm)	t (mm)	
SPG1	1.5	250	200	20	200	20	3	No slab
SPG2	1.5	150	260	20	260	20	5	
CPG1	1.5	250	200	20	200	20	3	Reinforced Concrete
CPG2	1.5	150	260	20	260	20	5	
CPG3	1.5	250	200	20	200	20	3	Reinforced concrete slab w/ shear links
CPG4	1.5	150	260	20	260	20	5	

Shanmugam and Baskar (2003).

Load-deflection curves for the steel girder and the corresponding composite girders with and without shear links are given Figures 26 & 27 below for *d/t* equal to 250 and 150, respectively. An increase of 77% and 123% in load-carrying capacity was observed for CPG 1 and CPG3, respectively, compared to SPG 1. This increase could be attributed to the enhancement in the web plate capacity due to composite action, and the additional strength provided by the deck slab. The corresponding increase for CPG2 and CPG4 was 40% and 68%, respectively, compared to SPG2. Larger increases in load carrying capacities were observed for girders with a slender web (i.e., *d/t=250).*

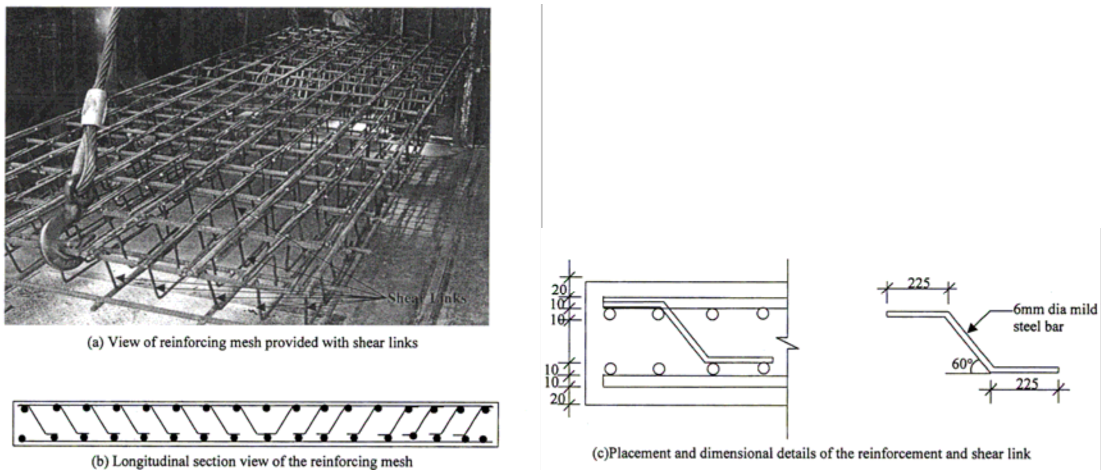


Figure 17 Details of shear links in CPG3 and CPG4
(Shanmugam and Baskar, 2003).

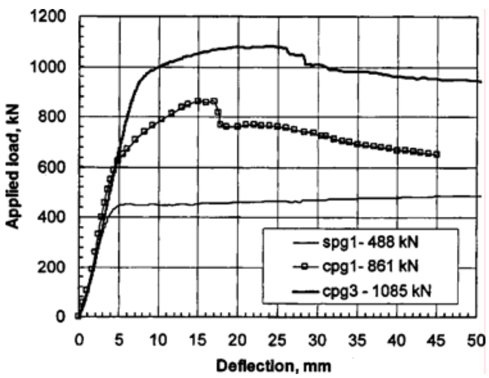


Figure 26: Comparison of load versus deflection for girders with web d/t ratio=250
(Shanmugam and Baskar, 2003)

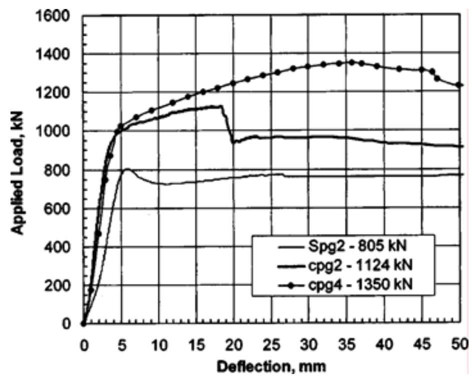


Figure 27: Comparison of load versus deflection for girders with web d/t ratio=150
(Shanmugam and Baskar, 2003)

Conclusions

A detailed experimental study was carried out on composite plate girders subject primarily to shear loading to study the variation in the tension field action of the web plate due to composite action with the deck slab. Conclusions based on the experimental work include:

a) composite action enhances the effectiveness of web plate to resist larger shear;
b) the width of the yielded tension band increased due to composite action of the deck slab;
c) composite action is more significant in the case of plate girders with a larger d/t ratio;
d) significant enhancement was also observed in the load-carrying capacity of composite plate girders with a smaller d/t ratio;
e) the shear failure mode observed in the deck slab of CPG 1 and CPG2 leads to a sudden failure after reaching the ultimate load;
f) provision of shear studs over the end support is an important factor that enhances the composite action;
g) shear links provided in the deck slab of CPG3 and CPG4 significantly increased the load-carrying capacity;
h) shear links also altered the mode of failure from shear to flexure and leads further to more or less ductile failure after the ultimate load.

It is necessary to carry out more detailed studies in order to ascertain the effectiveness of shear links and their distribution within the slab so that recommendations could be made for their applications in the design.

MECHANICS OF WEB PANEL POST-BUCKLING BEHAVIOR IN SHEAR

Chai H. Yoo and Sung C. Lee, J of Structural Engineering, ASCE October 2006

ABSTRACT

This paper revisits a fundamental assumption used in most classical failure theories for post-buckled web plates under shear, namely that the compressive stresses that develop in the direction perpendicular to the tension diagonal do not increase any further once elastic buckling has taken place. This assumption naturally led to a well-known theory that tension field action in plate girders with transverse stiffeners must be anchored by flanges and stiffeners in order for the webs to develop their full postbuckling strength. However, a careful examination of the results of the nonlinear finite-element analyses carried out for this study reveals that the diagonal compression continuously increases in close proximity to the edges after buckling, thereby producing in the web panel a self-equilibrating force system that does not depend on the flanges and stiffeners. These findings provide a fuller understanding of the actual mechanics of tension field action.

Summary and Conclusions

Wilson (1886) is credited with the first study of the post-buckling strength of plate girder web panels. As Wagner (1931) first presented a diagonal tension theory for aircraft structures with very thin web panels, many researchers have studied the tension field action for plate girders, as summarized in SSRC (2010). Although these classical failure theories assumed different yield zones, the following fundamental assumption was implicit in all the theories: compressive stresses that develop in the direction perpendicular to the tension diagonal do not increase any further once elastic buckling has taken place. The application of this fundamental assumption to the whole web panel leads naturally to the well-known theory that the tension field action in plate girders with transverse stiffeners needs to be anchored by flanges and stiffeners in order for the webs to develop their full post-buckling strength.

Recent findings strongly suggest that it is time to revisit this assumption. The objective of the present study is to examine the validity of the fundamental assumption using nonlinear finite-element analysis and thus shed light on the mechanics of the postbuckling behavior of shear web panels. Yoo and Lee (2006) found that existing theories predict shear capacity sufficient for design, but unable to explain the behavior of the girder seen in experimental results. Their findings are listed below.

1) If out-of-plane deflections are restrained along the edges of a rectangular panel by means of simple supports, an external anchor system is not necessary for the development of practically meaningful postbuckling strength. This is possible only because the diagonal compression increases near the edges after elastic shear buckling.
2) Diagonal compression continuously increases near the edges of panels after buckling, which is contrary to the fundamental assumption adopted previously. Due to this increase in the diagonal compression, the normal stress perpendicular to the edge is not necessary for equilibrium. Hence, a simply supported panel is able to develop postbuckling strength with no external anchor system.

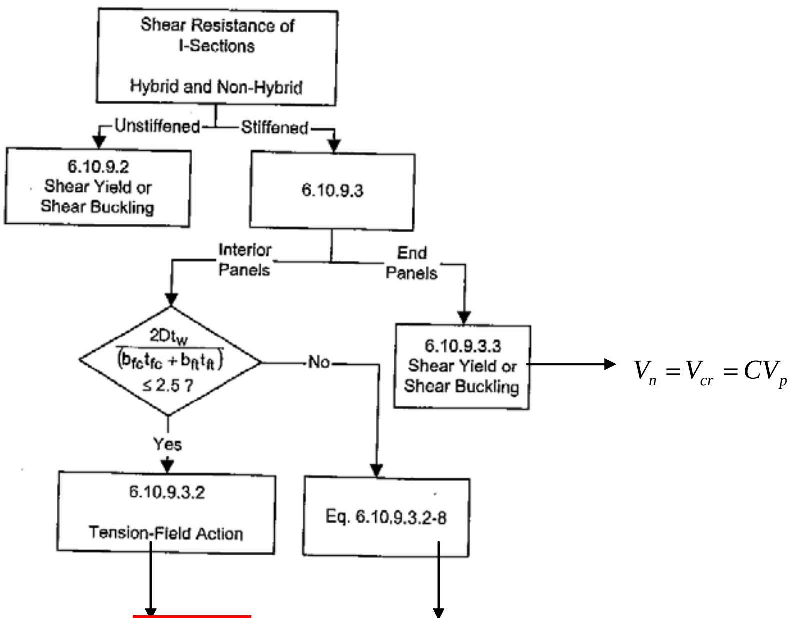
3) In the postbuckling stage, an increase in the diagonal compression is possible near the simply supported edges because the axial rigidity in the direction of the compression diagonal is not rapidly reduced, as in the center of the web.

4) As the intermediate transverse stiffeners are not subjected to the large axial compressive force predicted by the Basler model although they are subjected to some compression by virtue of their continuity with the web, the requirement for the area of the transverse stiffener developed by Basler (1961) is irrelevant.

5) There is no need to distinguish the end panel from the interior panel. *Tension field action can take place in the end panel.* All forces developed during postbuckling are self-equilibrated within the web panel. This means that even end panels can develop postbuckling strengths. As bearing stiffeners are designed to carry a large compressive force as columns, they have more than an adequate bending rigidity to function as simple supports needed for a self-equilibrated end panel to develop postbuckling. *The restriction of ignoring any tension field action in the end panels, therefore, needs to be revisited.*

AASHTO LRFD Bridge Design Specifications (2009)

Commentary C6.10.9.3.2 in 2009 AASHTO Specifications



$V_n = V_p[C +$ [redacted] $]$ $\quad V_n = V_p\left[C + \dfrac{0.87(1-C)}{\sqrt{1+\left(\frac{d_0}{D}\right)^2} + \frac{d_0}{D}}\right]$

(Postbuckling Tension Field Force)

where

$V_p = 0.58F_{yw}Dt_w$

Figure C6.10.9.1-1 Flow Chart for Shear Design of I-Sections

Stiffened interior web panels of non-hybrid and hybrid members satisfying Eq. 1 are capable of developing post-buckling shear resistance due to tension-field action (Basler, 1961 and White et al., 2004). This action is analogous to that of the tension diagonals of a Pratt truss. The nominal shear resistance of these panels can be computed by summing the contributions of beam action and post-buckling tension-field action. The resulting expression is given in Eq. 2, where the first term in the bracket relates to either the shear yield or shear-buckling force and the second term relates to the post-buckling tension-field force. If Eq. 1 is not satisfied, the total area of the flanges within the panel is small relative to the area of the web and the full post-buckling resistance generally cannot be developed (White et al., 2004). However, it is conservative in these cases to use the postbuckling resistance given by Eq. 8. Eq. 8 gives the solution neglecting the increase in stress within the wedges of the web panel outside of the tension band implicitly included within the Basler model (Gaylord, 1963; Salmon and Johnson, 1996).

With the restrictions specified by Eqs. 1 and 6.10.2.2-2 in general, and Article

6.10.9.3.1 for longitudinally-stiffened I -girders in particular, and provided that the maximum moment within the panel is utilized in checking the flexural resistance, White et al. (2004) shows that the equations of these Specifications sufficiently capture the resistance of a reasonably comprehensive body of experimental test results without the need to consider moment-shear interaction. In addition, ***the additional shear resistance and anchorage of tension field action provided by a composite deck are neglected within the shear resistance provisions of these Specifications***. Also, the maximum moment and shear envelope values are typically used for design, whereas the maximum concurrent moment and shear values tend to be less critical. These factors provide some additional margin of conservatism beyond the sufficient level of safety obtained if these factors do not exist.

SHEAR RESISTANCE OF TRANSVERSELY STIFFENED STEEL I-GIRDERS

Donald W. White, and Michael G. Barker, J of Structural Engineering, ASCE, September 2008

ABSTRACT

This paper evaluates the accuracy and ease of use of 12 of the most promising models for the shear resistance of transversely stiffened steel I-girders. Several models that are well established in civil engineering practice as well as a number of other recently proposed models are considered. As the model developed in Basler's seminal research is the method of choice in current American practice, the paper focuses on the merits and limitations of the alternative models relative to Basler's. Statistical analyses are conducted on the predictions by the various models using an updated data set from 129 experimental shear tests, including 30 hybrid and 11 horizontally curved I-girders. The results support the conclusion that the form of Basler's model implemented in *2004 AASHTO LRFD Bridge Design Specifications* and *2005 AISC Specification for Structural Steel Buildings* gives the best combination of accuracy and simplicity for calculation of the shear resistance of transversely stiffened steel I-girders.

Conclusions

The results presented in this paper support the conclusion that the Basler (k_{Vincent}) model implemented in the AASHTO (2004) and AISC (2005) specifications provides the best combination of accuracy and simplicity of the models considered for the calculation of the shear resistance of transversely stiffened I-girders. The Cardiff (k_{Lee}) model is the most accurate of these models, but requires substantially more calculation. The more accurate shear buckling coefficient, (k_{Lee}), tends to give a minor improvement in the dispersion of V_{test} / V_n for both the Basler and the Cardiff models. Further, the use of k_{Lee} results in slightly larger design strengths ϕV_n for large d_o / D and slightly smaller design strengths for small d_o / D.

SHEAR STRENGTH AND MOMENT-SHEAR INTERACTION IN TRANSVERSELY STIFFENED STEEL I-GIRDERS

Donald W. White, Michael G. Barker, and Atorod Azizinamini, J of Structural Engineering, ASCE, September 2008

ABSTRACT

With the advent of HPS495W steel, hybrid I-girders have again become advantageous in bridge design. Unfortunately, the use of tension field action is not permitted in determining the shear resistance of hybrid girders in prior AISC and AASHTO specifications. This is a significant penalty on the strength of these member types. Also, the checking of moment–shear ($M-V$) strength interaction is a significant complicating factor in the design and capacity rating of I-girders that use tension field action. The requirements for the shear strength design and the equations for $M-V$ strength interaction in the 1999 AISC and 1998 AASHTO specifications were developed originally without the benefit of a large body of experimental tests and refined finite-element simulations. This paper presents the results from the collection and analysis of data from a total of 186 high-shear low-moment, high-shear high-moment, and high-moment high shear experimental I-girder tests. References to corroborating refined finite element studies are provided. Particular emphasis is placed on the extent to which web shear tension-field strength is developed in hybrid I-girders, as well as on the interaction between the flexural and shear resistances in hybrid and non-hybrid I-section members. The results of the study indicate that, within certain constraints that address the influence of small flange size, Basler's shear resistance model can be used with the flexural resistance provisions of the 2004 AASHTO and 2005 AISC specifications without the need for consideration of $M-V$ strength interaction. Also, the research shows that a form of the Cardiff model can be used with these flexural resistance provisions without the need to consider $M-V$ strength interaction. These conclusions apply to both non-hybrid and hybrid I-girder designs.

Conclusions

This paper presents the results from the collection and analysis of the data from a total of 186 high-shear low-moment, high shear high moment, and high moment high-shear experimental I-girder test. Particular emphasis is placed on the extent to which web shear postbuckling (tension-field) strength is developed in hybrid I-girders, as well as on the M-V strength interaction in hybrid and nonhybrid I-section members. The results of this study indicate that either of the following shear strength models may be used with the unified AASHTO-AISC flexural resistance equations (White 2008) for design of I-section members, without the need for consideration of M-V strength interaction:

1. The shear strength model originally developed by Basler (1961) using a simplified expression for the shear buckling coefficient proposed by Vincent (1969), for I-girders with $D/b_{fmin} \leq 6$ and $A_w/A_{favg} \leq 2.5$, along with the true Basler shear strength model (Porter et al. 1975; AASHTO 2004) for girders with unusually small and/or narrow flanges characterized by $D/b_{fmin} > 6$ or $A_w/A_{favg} > 2.5$, or

2. A version of the more refined Cardiff shear strength model (Porter et al. 1975; White and Barker 2008) using the shear buckling coefficient equations from Lee et al. (1996). The equations from Lee et al. (1996) account for the shear buckling restraint provided by the flanges.

These conclusions apply to both non-hybrid and hybrid I-girder designs. Although no trends are evident in the governing V_{test}/V_n or M_{test}/M_n values as a function of F_{yw}/f_n, only two high-shear high-moment and high-moment high-shear tests have F_{yw}/f_n smaller than 0.61. Therefore, further tests with extreme differences between the web and flange yield strengths would be prudent. The high-shear high-moment and high-moment high-shear girders considered in this paper are predominantly tests in which the web yield strength is smaller than the yield strength of one or both flanges by one steel grade.

Within the above contexts, no penalty is incurred for high shear low-moment or high-moment low-shear if the resistance factors for shear and flexure are determined including the high shear high-moment or high-moment high-shear tests, respectively. The resulting implementation of these recommendations in AASHTO (2004) and in AISC (2005) results in substantial simplifications particularly in the design and capacity rating of bridge I-girders, and leads to significant economies in the shear design of thin-web hybrid I-section members. In addition, some incidental conservatism typically exists that is not considered in the development of these recommendations. This incidental conservatism comes from the fact that engineers usually base their calculations conservatively on separate non-concurrent maximum moment and maximum shear envelope values in bridge design. Also, the slab in composite I-girders generally provides some additional contribution to the shear strength that is neglected or predicted conservatively by the suggested shear strength equations. These factors tend to increase the net margin of safety beyond the levels associated with the test data compiled in this paper.

GUIDE TO STABILITY DESIGN CRITERIA FOR METAL STRUCTURES, Chapter 6 (Sixth Edition)

SSRC (2010)

The tension field in a plate girder panel is resisted by the flanges and by the adjacent panels and transverse stiffeners. Because the panels adjacent to an interior panel are tension-field designed, they can be counted on to furnish the necessary support. An end panel, however, does not have such support and must be designed as a beam-shear panel unless the end stiffeners are designed to resist the bending effect of a tributary tension field. Basler (1963a) assumed that an end panel designed for beam shear can support a tension field in the adjacent interior panel, and this assumption has been generally accepted. This means that the end panel stiffener spacing can be based on the shear buckling stress as discussed by Skaloud (1962). ***If the end panel is designed for tension-field action, an end post must be provided.*** A possible end post consists of the bearing stiffener and an end plate (Fig. 6.4a). According to Basler such an end post can be designed as a flexural member consisting of the stiffener, the end plate, and the portion of the web between, supported at the top and bottom flanges and subjected to the horizontal component of the tension field distributed uniformly over the depth. The required area A_e of the end plate, based on ultimate load consideration, is given by

$$A_e = \frac{(\tau - \tau_{cr})hA_w}{8e\sigma_y}$$

where e is the distance between the bearing stiffener and end plate. The bearing stiffener itself is designed to support the end reaction. According to tests reported by Schueller and Ostapenko (1970), web shear may control the design of the end post.

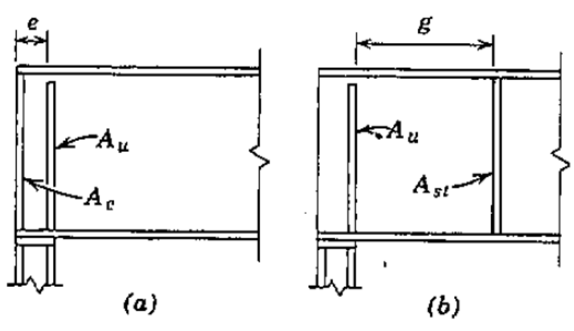


FIGURE 6.4 End stiffeners.

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