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Design Rules for Steel Box Girder Bridges

Règles de dimensionnement pour les ponts en acier à poutres en caisson

Richtlinien für die Bemessung von Stahlbrücken mit Kastenträgern

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SUMMARY

Proposed American specifications for steel box girder bridges are comprehensive but simplified. The design rules reflect current state of the art and are based on ultimate load principles. Compression flanges are designed with consideration of effects of geometric imperfections and residual stresses implicitly included in the strength curves. Web design is based on elastic theory and partial utilization of tension field action. Also given are fabrication tolerances and erection guidelines.

RÉSUMÉ

Les règles de dimensionnement du projet de norme américaine sur les ponts en acier à poutres en caisson ont une portée générale, tout en étant simplifiées. Elles reflètent l'état actuel des connaissances en la matière et tiennent compte du calcul à la ruine. Le dimensionnement des membrures comprimées tient compte des imperfections géométriques et des contraintes résiduelles, dont les effets sont implicitement inclus dans les courbes de résistance. Le dimensionnement de l'âme est basé sur la théorie de l'élasticité et une participation partielle des bielles de traction. Les tolérances de fabrication et des directives de montage sont également indiquées.

ZUSAMMENFASSUNG

Die vorgeschlagenen vereinfachten amerikanischen Richtlinien für die Bemessung von Stahlbrücken mit Kastenträgern sind leicht verständlich. Sie entsprechen dem heutigen Stand der Kenntnisse und beruhen auf Traglastprinzipien. Die Bemessung der Druckgurte berücksichtigt geometrische Unregelmässigkeiten sowie Profileigenspannungen. Die Bemessung des Stegs baut auf der Elastizitätstheorie auf, unter teilweiser Benutzung von Zugspannungsfeldern. Herstellungstoleranzen sowie Ausführungsrichtlinien werden dargelegt.



1. INTRODUCTION

The design of highway bridges in the USA is governed by the "Standard Specifications for Highway Bridges" [1] issued by the American Association of State Highway and Transportation Officials (AASHTO). The rules for steel box girders are given only for the short to moderate span (about 15-60 m) multi-box composite bridges, with a typical box depth of about 1.3 to 2.5 m (Type (a), Fig. 1), however there is not much guidance in the specifications for the design of longer span steel box girder structures (Types (b) and (c)).

To fill the need for comprehensive but practical design rules for steel box girders, the Federal Highway Administration (FHWA) outlined the desired scope of the specifications and invited proposals for the execution of this project. The FHWA contract was awarded in 1977 to the firm of Wolchuk and Mayrbaurl. The work was carried out under the guidance of the American Society of Civil Engineers-Technical Council on Codes and Standards

(ASCE-TCCS) Committee on Steel Box Girder Bridges under the chairmanship of A. Lally. Professor A. Ostapenko (Lehigh University) was acting as consultant to Wolchuk and Mayrbaurl.

Final report on recommended specification provisions for design, fabrication and erection of steel box girder bridges was submitted to the FHWA in 1979, and was issued by the FHWA in January, 1980 [2]. The report also contains a detailed commentary giving the background of the proposed rules, and a survey of the state of the art, as well as a discussion of the current and proposed new specifications applicable to box girders in the USA, United Kingdom and Germany.

In preparation of the proposed rules extensive use was made of the recent work on box girders and steel plate structures done in the United Kingdom and in Germany. The writer is thankful to the specification committees and the many individual researchers in these countries for their cooperation in making this information available and for their helpful advice.

The proposed specifications, based on ultimate load design principles, are applicable to all types of steel box girder bridges (except for horizontally curved structures), regardless of span. They are given in the format of the AASHTO specifications, and refer to other sections of that code for loading and load distribu-

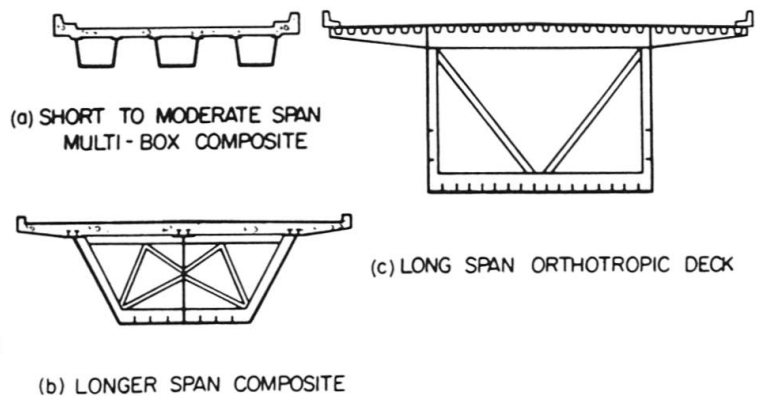


Fig. 1 Types of steel box girder bridges

tion, load factors, fatigue design and other general provisions. The proposed rules are now being reviewed for adoption by the AASHTO Subcommittee on Bridges and Structures.

This paper discusses the provisions for the principal components of box girder bridges that were the main object of these specifications: the bottom compression flanges and the webs.

2. BOTTOM FLANGES

2.1 Unstiffened Flanges

The proposed strength curve for an unstiffened plate in pure compression attempts to take into consideration the effects of geometric imperfections (out-of-flatness) and residual stresses in the plate. The reported range of strength of axially loaded plate with various degrees of imperfections and residual stresses, based on tests and computer-generated results in Britain [3, 4, 5, 6, 7], is shown by a hatched band in Fig. 2. Dimension "b" is the width of flange between the webs, F_u is the ultimate strength, F_y is the yield strength of steel, F_{cr} is the elastic buckling stress. The curve chosen for the proposed specification lies closer to the upper edge of the strength band, corresponding to a relatively low level of residual stress (about 15 MPa) in a "lightly welded" plate. It should be noted that the strength of very slender plates is greater than that predicted by elastic buckling theory.

The strength curve used in the current AASHTO specifications, with a transition curve for stocky panels based on analogy with column behavior, is also shown.

The effect of shear coincident with compression is ignored in the proposed specifications for the values of shear smaller than $0.175 F_y$. For larger values of shear the strength is adjusted by a modified von Mises formula. An average value of shear in the panel is used in the calculations.

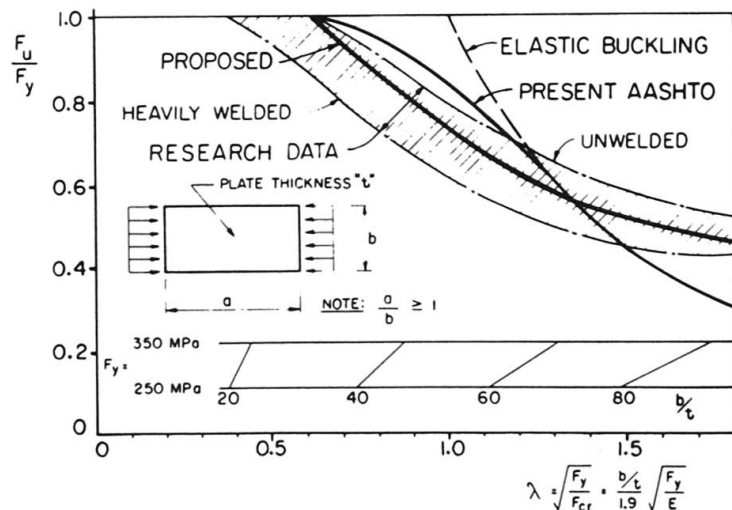


Fig. 2 Strength of unstiffened flange in compression



2.2 Stiffened Flanges

The ultimate compression strength of a bottom flange reinforced by longitudinal stiffeners is affected by the strength of the plate panels between the stiffeners and also by the residual stresses and the out-of-straightness of the longitudinal stiffeners themselves. This out-of-straightness, Δ , may be positive, with stiffeners bent towards the interior of the box, or negative, with stiffeners bent the opposite way.

Several methods of treatment of stiffened compression panels were studied [8, 9, 10, 11] and the numerical method of Little [8] was selected as a basis of determination of strength for the purpose of design. This is a second-order iterative numerical computer method for inelastic column analysis that considers the interaction between the local buckling of the plate between the stiffeners and the overall buckling of the plate-stiffener combination.

The original paper [8] considered only flange panels with positive out-of-straightness with bulb-flat stiffeners. For the purposes of this project subsequent studies and calculations were made also for other types of stiffeners (flat bars, T-stiffeners), with positive and with negative out-of-straightness, for two grades of steel with a yield stress of 250 and 350 MPa [12, 13, 14]. The effects of continuity of the longitudinal stiffeners were also considered, as suggested by Ostapenko ([2], Pg. 119).

For design purposes the following assumptions were made in the strength calculations:

- a) flange plate between the stiffeners: governing out-of-flatness ("ripple component") $\delta_o = 0.001$ times the stiffener spacing, residual stress in the plate 73 MPa.
- b) stiffeners: out-of-straightness $\Delta = L/500$, where L = spacing of transverse stiffeners. In calculations of strength plating with negative out-of-straightness, where the strength of the stiffener outstand may govern, the inherent self-equilibrating residual stresses in the stiffeners prior to welding (of the order of 60-70 MPa compression at the outstanding tips of T-stiffener flanges or flat bar stiffeners) were superimposed on the welding residual stresses.

Additional sample calculations were made with a residual stress in the flange plate of 25 MPa and stiffener out-of-straightness of $L/1000$ in order to assess the sensitivity of the calculated strength to the variation of these parameters. The results show that the most important single effect on strength is that of the assumed residual stress in the flange plate (which is also least predictable and may vary considerably with stiffener spacing), the type of stiffeners and the stiffener out-of-straightness being relatively less important. This led to the conclusion that undue refinement of the

strength curves for practical design purposes (such as differentiation between different types of stiffeners) would not be warranted.

Therefore the strength curves, based on the conservative assumptions of the residual plate stress of 73 MPa and $\Delta = L/500$, were obtained as averages between the values calculated for other variable parameters. The simplified end results are presented in a form of an "interaction diagram", Fig. 3, as a function of two geometric parameters: λ_{col} , the column slenderness, and λ_{pl} , the plate slenderness. Such presentation was first used for a similar application in the new German specifications for plates in compression [11].

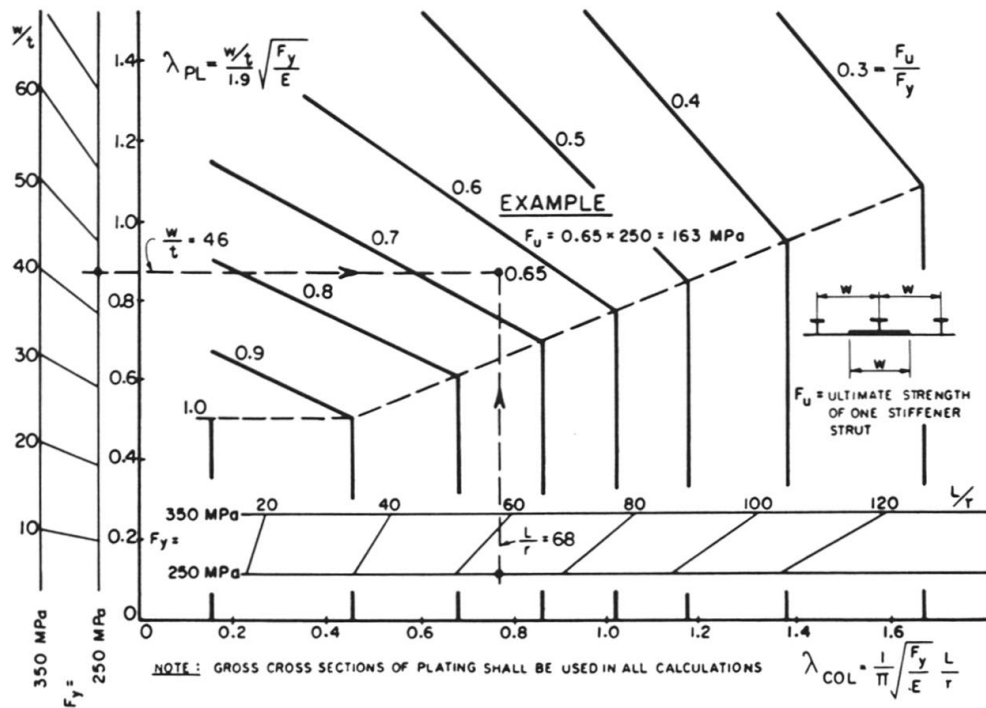


Fig. 3 Strength of stiffened flange in compression

For low w/t ratios the lower limit of strength was assumed to be given by Lehigh column curve 2 [15], applicable to light welded columns. This column strength curve is represented by the vertical lines in the diagram.

Thus, while the background of this design method is complex, its practical application is quite simple. The design, assuming "column behavior" of the flange panel, considers only one stiffener strut consisting of one stiffener with a corresponding width of flange plate. Gross cross sections of plating are used in all calculations. The strength of flange is obtained by multiplying the ultimate strength of the strut by the total area of the flange. Since the effect of the flange plate strength, depending on the w/t ratio, is accounted for in determining the strut strength, a separate check of the plate sub-panels between the longitudinal stiffen-



ers is not required.

The results obtained from this diagram have been found to be in line with strength predictions by other methods [9, 10, 11] and correlate well with available test data.

The specification also contains a formula for reduced effective length, L' , based on work by Rogers [16], to be used for "narrow stiffened flanges" where the effect of longitudinal support along the edges of the flange is considerable and "plate behavior" rather than "column behavior" prevails.

The design of stiffened flanges assumes that stiffeners are not endangered by local torsional buckling. To ensure local stiffener stability appropriate slenderness criteria are stipulated.

3. WEBS

3.1 Web Panels

The shear carrying capacity of a web panel is given as the sum the beam shear strength (the elastic buckling strength), V_B , and the tension field strength (the postbuckling strength), V_T . Both strength contributions are calculated separately.

In the first step of the calculation the critical elastic buckling stress of the web panel under consideration, F_{vcr} , is computed for combined action of shear and flexural stresses by interaction equations of the linear elastic theory of plate buckling given in the specification. The shear strength curve selected for use is based on the semi-empirical Chern-Ostapenko basic strength curve [17]. Design calculations are facilitated by graphs giving critical stresses for shear and for axial stresses acting alone, for the various depth-to-thickness ratios and panel aspect ratios.

In the calculation of the postbuckling strength the tension field capacity is only partially utilized, because the flanges of box girders are generally less rigid than those of plate girders and, therefore, cannot be counted upon to ensure sufficient anchorage of tension field forces. Furthermore, a fully developed tension field would endanger the stability of the compression flange by formation of a hinge. Therefore the specification utilizes only the lower limit of the tension field strength corresponding to the assumption of negligible flange rigidity, known in literature as the "true Basler" solution [18, 19], see Fig. 4.

Thus the tension field strength of a web panel is given as:

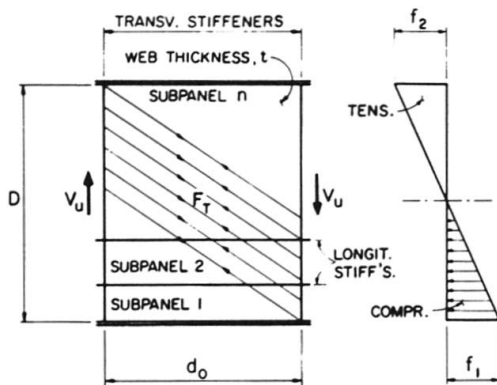
$$V_T = DtF_T/2 (\alpha + \sqrt{1 + \alpha^2})$$

where: D = depth of web between flanges
 $\alpha = d_o/D$, with d_o = distance between transv. stiffeners
 t = web thickness
 F_T = tension field band stress utilized for postbuckling shear capacity

The value of F_T is calculated conservatively, with consideration of the reduction of the useful tension in the yield band by the effect of equivalent tension present in the band due to flexural and shear stresses in the web, by the formula

$$F_T = F_Y - \sqrt{0.25f_2^2 - 3F_{vcr}^2}$$

where: F_Y = yield strength of web
 f_2 = maximum flexural tensile stress in the web panel
 F_{vcr} = critical elastic buckling shear stress under combined action of shear and flexure



$$\alpha = \frac{d_o}{D}$$

$$V_B = Dt F_{vcr \text{ min.}}$$

$$V_T = \frac{Dt F_T}{2(\alpha + \sqrt{1 + \alpha^2})}$$

$$V_u = V_B + V_T$$

In the design of web panels with longitudinal stiffeners (Fig. 4) the elastic buckling shear strength, F_{vcr} , is determined separately for each web subpanel. The buckling strength of the entire web is governed by the strength of the weakest sub-panel, $F_{vcr \text{ min.}}$

Fig. 4 Design of webs subject to combined shear and flexural stresses

Calculation of the post-buckling strength is based on the assumption of a tension field developing across the entire depth of the web panel between the flanges, regardless of the presence of the longitudinal stiffeners, see Fig. 4. Such approach is indicated by tests and theoretical work by Rockey, Evans and Porter [19, 20].

Design rules are also given for web panels under shear combined with tension, based on treatment by Scheer [21].

The capacity of the web to resist compressive axial flexural stress is limited to the stress at which the web buckles. Axial compression forces that would cause web stresses in excess of this limit must be resisted by the flanges. Such additional forces in flanges adjacent to webs designed with utilization of tension field action are calculated by formulas given in the proposed specification.



3.2 Web Stiffeners

The proposed design rules for web stiffeners are based on both rigidity and strength considerations.

In accordance with the assumptions made in the design of the web, the transverse and the longitudinal stiffeners must provide rigid supports to the web subpanels, that is, stiffener must remain straight and not deflect out of the plane of the web during the web buckling and postbuckling stages, and up to the web ultimate design capacity. Based on these requirements the specification stipulates conservatively that the relative rigidity coefficient of a stiffener should be at least equal to the "theoretical optimum rigidity", γ^* , multiplied by an empirical factor, m .

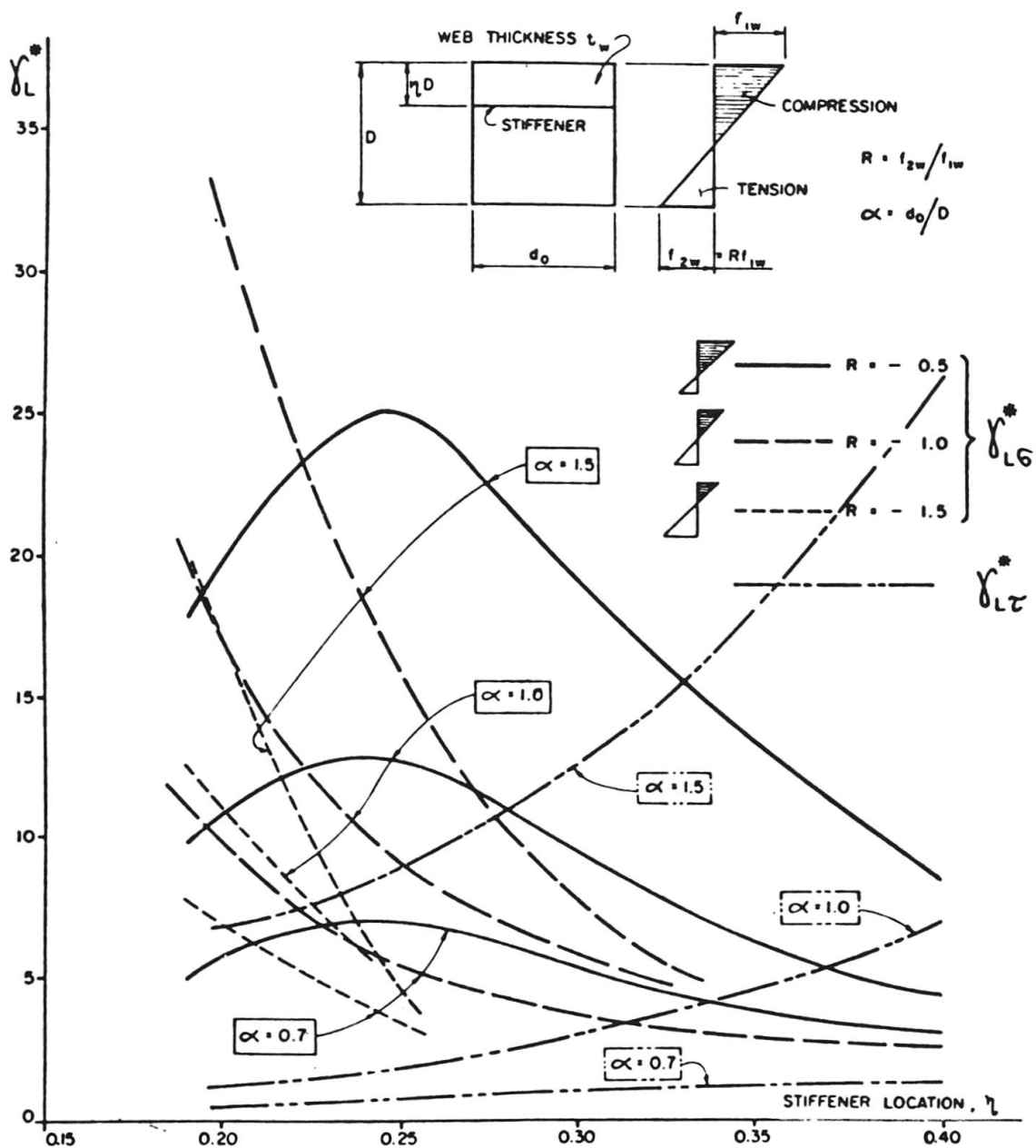


Fig. 5 Rigidity coefficients γ_L^* for longitudinal web stiffeners

The γ^* values have been adapted from the work by Kloeppe, Scheer and Moeller [22, 23], and condensed for the purposes of practical design into two simple diagrams. The values of γ_L^* , for longitudinal stiffeners, are given for flexural loading and for shear loading, as a function of the panel aspect ratio, stiffener location, and the ratio of the minimum to maximum flexural stress in the web panel (Fig. 5). For practical design purposes the needed value of γ_L^* for combined axial and shear stress is obtained by an interaction formula given in the proposed rules, similar to that suggested by Djubek and Skaloud [24, 25].

The values of minimum rigidities of the transverse stiffeners, γ_T^* , obtained with consideration of shear only, are given in Fig. 6.

The use of the multiplier m is indicated by the behavior of stiffeners in the web postbuckling range. However, because the postbuckling strength is not utilized to its full extent in the proposed web design rules, the use of the "m" factors equal to about one-half of the values recommended in literature [25, 26] is stipulated in the proposed rules. The recommended values of m are further reduced for webs with low depth-to-thickness ratios.

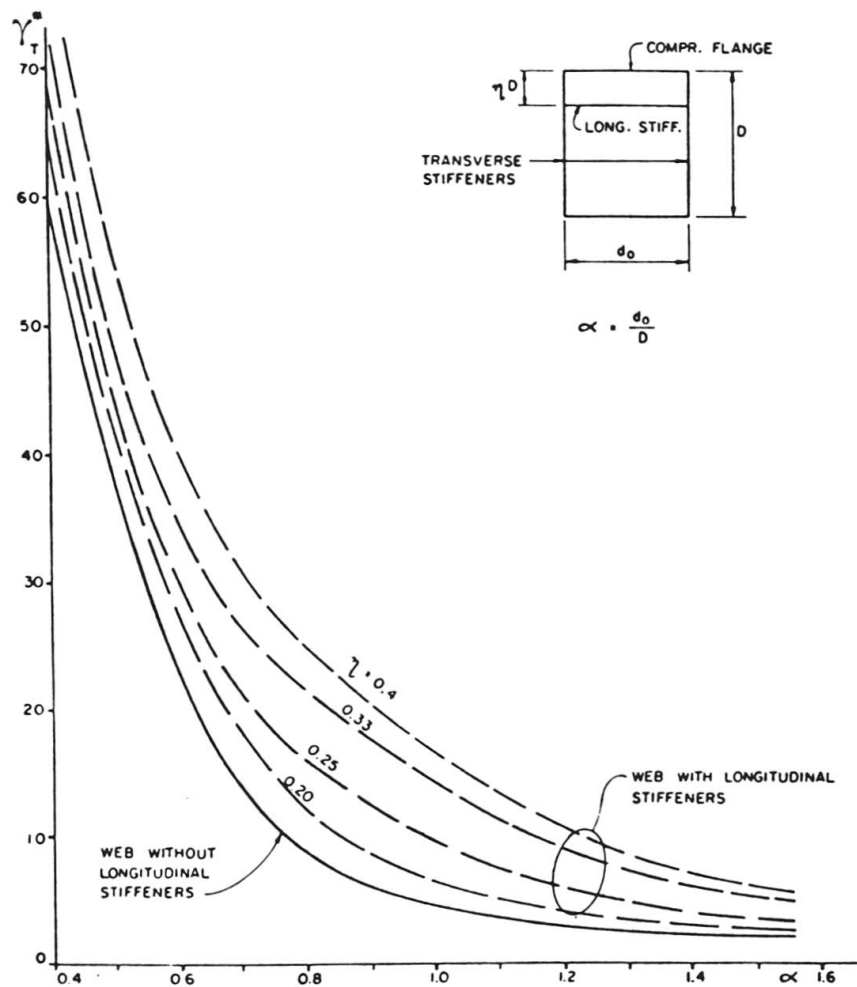


Fig. 6 Rigidity coefficients γ_T^* for transverse web stiffeners

The web stiffeners shall also satisfy the strength requirements, and must be designed as eccentrically loaded struts to resist the compression forces to which they are subjected. These include vertical forces due to tension field action and direct loads on the deck acting on the transverse stiffeners, and compression forces



due to flexural stresses in the web acting on the longitudinal stiffeners.

It should be noted that many questions regarding the design of stiffened webs are still insufficiently clarified and remain to be solved by further continuing theoretical and experimental research. Therefore, the proposed provisions for webs, and especially for the web stiffeners, are necessarily conservative, and should be regarded tentative.

4. OTHER PROVISIONS

The proposed box girder design specifications also include provisions for effective width of flanges, to be considered in calculation of stresses under working loads for the purpose of fatigue design, criteria for tension flanges, intermediate cross frames, transverse flange stiffeners, diaphragms at box girder supports and miscellaneous detail provisions.

Construction provisions of the proposed rules include fabrication tolerances for out-of-flatness of plate panels and out-of-straightness of stiffeners in compression. These tolerances are not very restrictive and are expected to be easily achieved in normal fabrication practice. Tolerance provisions for the webs, flanges in tension and orthotropic decks are more liberal than for the bottom flanges in compression. Also given are general guidelines for box girder erection emphasizing importance of thorough consideration of stability of box girders during construction.

5. CONCLUSIONS

The proposed specifications for steel box girder bridges cover comprehensively the design of all principal components. The methods of determining the design strengths, based on the current state of the knowledge, were selected with the aim of simplicity in practical engineering application. The rules are conservative. Their liberalization and improvement should be undertaken as the questions not yet sufficiently clarified are more fully answered by continuing research.

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