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Strength of rectangular composite box girders: recommendations for design of composite box girders final report

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Strength of Rectangular Composite Box Girders

RECOMMENDATIONS FOR DESIGN OF COMPOSITE BOX GIRDERS FINAL REPORT

by

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16. Abstract

Failure behavior of steel-concrete composite box girders under static load is briefly reviewed, confirming the similarity between webs of plate girders and box girders. Rules are proposed for Allowable Stress Design procedures which are directly adopted from proposed rules for Load Factor Design of box girders.

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STRENGTH OF RECTANTULAR COMPOSITE BOX GIRDERS
Project 69-4

RECOMMENDATIONS FOR DESIGN OF COMPOSITE BOX GIRDERS

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LEHIGH UNIVERSITY
Office of Research
Bethlehem, Pennsylvania

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1. INTRODUCTION

Composite box girders are common structures in highway bridges. The U-shaped or trapezoidal-shaped steel section and a concrete deck are connected by shear connectors to form a unit. The sizes of the composite box girders are such that the deck width usually constitutes the full width of a roadway or a traffic lane of a highway bridge. The span length commonly is below 150 feet.

Because of their configuration, size and position in the cross section of a bridge, composite box girders are usually subjected to torsional loads as well as flexural loads. It is in part due to the torsional strength of the closed cross sections of boxes that composite box girders gained popularity.

The analysis of box girders in the elastic range of material properties has been studied extensively.[1,2,3,4.5] The primary concern has been the evaluation of stresses in the component parts of box sections, assuming no buckling of steel plates will occur and no general yielding of material takes place. From the examination of results of analyses, provisions have been established for design of composite box girders to ensure sufficient safety margin against buckling and yielding.[6]

It is well recognized that the "yield strength" of a structural member and the "buckling strength" of its components do not represent the load carrying capacity (or ultimate strength) of the structural member. This condition is true of plate girders as well as beams.[7,8,9,10] Since the webs of composite box girders are similar to those of plate girders, the load carrying capacity of composite box girders are also expected to be higher than the web buckling strength or the load at first yielding of a point in the box girder. Current design provisions [6] recognize this fact and permits the use of rules for plate girders for the design of webs of composite box girders (see Articles 1.7.49(A) and 1.7.64 of Ref. 6).

The work of this study included the development of an analytical procedure for the evaluation of stresses and deflections of composite box girders within the elastic range of behavior, and the examination of the load carrying capacity of such girders through testing of specimens. While results of these analyses and testing have been presented earlier[5.11] this report briefly summarizes the findings and indicates the basis of recommendations for design. The recommendations are given.

2. REVIEW OF BEHAVIOR OF COMPOSITE BOX GIRDERS

2.1 Elastic, Prebuckling Range of Behavior

Within the elastic property ranges of the materials and below the buckling stresses of the component plates of webs and steel flanges, the behavior of composite box girders can be evaluated and described by numerous classical procedures.[12,13,14,15] The composite concrete deck is usually converted into an equivalent steel plate to form a steel box girder.[5] By the thin-walled elastic beam theory,[14] the applied load on the equivalent steel box girder can be decomposed into flexural (bending), torsional and distortional components. (see Fig. 1). The box girder bends, twists and changes its cross-sectional shape according to the magnitude of the component loads. The summation of normal and shearing stresses due to each of these load components gives the total normal and shearing stresses at points of the composite box girder. As far as the total stresses are below yield stresses and buckling stresses, the analysis is valid and the behavior of a composite box girder is described.

Current design provisions in the AASHTO Specifications[6] define the geometry of the steel flange plates of composite box girders so that the resulting stresses in these plates are below the buckling stresses.

The results of analysis from this study confirmed the validity of the thin-walled elastic beam theory for composite box girders.[5] Figure 2 shows the comparison of torsion and shearing stresses from two test girders. Figure 3 compares the measured and computed normal stresses in the cross section of another test girder. Both show satisfactory prediction of stresses. Figure 4 compares the measured deflections with computed values along the span of two composite box girders. All computed values agree well with measured ones.

2.2 Elastic Post Web-Buckling Behavior

The elastic, post web-buckling behavior is signified by the on-set of the tension field action.[8] One web panel of a composite box girder section "buckles" under shear or combination of shear and normal stresses, and it is capable of carrying more load through the development of the diagonal band of tensile stresses.

Because web plates have initial out-of-flatness conditions, the "buckling" behavior is not a sudden occurrence but a gradual phenomenon. This behavior has been observed in numerous plate girders[16,17] and in the testing of composite box girders of this study.[11] The existing of a tension field in a web panel is usually not visible during testing of plate girders or composite box girders, until yielding has caused a trademark tension field band in the inelastic range of behavior (see, for example, Fig. 5).

The hypothetical upper limit of the elastic post web-buckling behavior is the first yielding of a point in the web along the inclined tensile stress field. This limit does not have any significant influence on the behavior of steel or composite box girders.[18,19,20] More important is the inelastic post web-buckling tension field strength of a composite box girder panel.

2.3 Tension Field Action of Web Panels

When yielding of the tension field band develops, it requires anchorage. The ability of web boundary elements to anchor the tension field determines the strength of the web. A large number of studies have been conducted in this area,[8,9,21 through 30] and the results have been incorporated into design provisions for plate and box girder webs.[4,6,31,32]

For a web panel of a composite box girder, the forces of the inclined tension field are anchored by the composite top flange, the steel bottom flange, the web plates in adjacent panels, and the transverse stiffeners which form the boundary of the web panel. When these anchoring components are not capable of sustaining additional forces, the strength of the tension field is reached. Figures 5 and 6 show two composite box girder web panels after testing to failure. The failure of the flanges accompany the failure of the webs. The corresponding load-deflection curves of the composite box girders are shown as Figs. 7 and 8. Failure of the webs and the composite box girders is not a sudden event.

Existing bridge design provisions for composite box girders recognize this post web-buckling tension field action and permits its utilization through the rules for plate girder webs.[6] The permission is not specifically stated that webs of composite plate girders should be or could be designed according to given formulas; only that the general rules of plate girders may be applied to composite box girders. With the results of this study[11,20] it is evident that the tension field action of composite box girder web panels is confirmed, and that more specific statements may be incorporated in the design provisions.

2.4 Strength of Composite Box Girder Segments

A composite box girder segment has four major components. the webs and the flanges. The development of tension field and failure of one web does not constitute the failure or attainment of the load carrying capacity of the box girder segment. Analysis and tests show that the capacity of a composite box girder segment is reached only when three or more of the four components have failed.[11,20] Depending on the loading and the relative strength of the component parts, the failure phenomenon or behavior is different.[11,18,20] Some of the possible sequences of strength development are listed in Table 1. Figures 7, 8 and 9 are examples of load-deflection curves of composite box girders, and Figs. 5, 6, 10, 11 and 12 are some photographs of failed components.

As background information for formulating design provisions, a number of conclusions from testing and analysis are important.

(a) Under negative bending moment, the buckling of the compressive steel flange causes failure of a composite box girder.

(b) Failure of composite box girders in positive bending and torsion is not a sudden occurrence. The load-deflection curves have a plateau.

(c) The attainment of tension field strength is accompanied by flange failure.

All these are analogous to the conditions for steel box girders.[32,33]

3. RECOMMENDATIONS

The primary concern of this study was the influence of web plates on the load carrying capacity of composite box girders. Consequently, emphasis is on proportioning of webs.

3.1 Existing Provisions

There are currently in the United States the following existing or proposed provisions for the design of web plates of bridge girders.

A. AASHTO Specifications(6)

1. 1.7.43(c) for Plate Girders, Allowable Stress Design
2. 1.7.59 for Symmetrical beams and Girders, Load Factor Design
3. 1.7.60 for Unsymmetrical Beams and Girders, Load Factor Design

B. Proposed Design Specifications for Steel Box Girder Bridges(32)

4. 1.7.210 for Unstiffened Webs, Load Factor Design
5. 1.7.211 for Transversely Stiffened Webs, Load Factor Design
6. 1.7.212 for Transversely and Longitudinally Stiffened Webs, Load Factor Design

The AASHTO Specifications for plate girders, as implied in the provisions, are adoptable for the design of composite box girders. The proposed specifications for steel box girders, as stated in the relevant commentary of Ref. 32, are also recommended for composite box girders and plate girder webs.

Since all these provisions are based on the same background information, and the study on composite box girders confirms the expected behavior of their webs, it is logical and rational to have the same design provisions for web plates of plate girders, composite box girders, and steel box girders. Two sets of provisions are needed for Allowable Stress Design and Load Factor Design, respectively.

Therefore, two tasks are to be carried out:

(a) Adoption of the Proposed Specifications of Ref. 32 (for Load Factor Design of Steel Box Girders) to Plate Girders, and specifying their adequacy for composite box girders.

(b) Formulating corresponding proposed specifications for Allowable Stress Design.

Task (a) is beyond the scope of this work. The proposed provisions of Task (b) are adopted directly from those of Ref. 32, and are given in the next section.

3.2 Proposed Design Specifications, Allowable Stress Design

3.2.1 The following are proposed articles for AASHTO Specifications for design of web plates of composite box girders.

1.7.XX Unstiffened Webs

(A) Scope

This article applies to box girder webs without stiffeners, except bearing stiffeners at supports.

(B) Allowable Shearing Stress

(1) General - The average calculated unit shearing stress in the gross section of the web plate, f_v shall be less than 0.55 F_{vcr}

$$F_{vcr} \leq 0.58 \sqrt{F_y^{\,2} - (\tfrac{2}{3} f_{av})^2}$$

where F_{vcr} is the critical shear buckling stress, as defined in Art. 1.7.XX (B)(2), and f_{av} is the average numerical value of the flexural axial stresses at the opposite longitudinal edges of the web, f_{1w} and f_{2w}, as defined in Art. 1.7.YY (B)(4), disregarding the sign of the stress.

The design shear force of an inclined web, V_W, is

$$V_W = V_V / \cos\theta$$

where V_V is vertical shear force and θ is the angle of inclination of the web to the vertical.

(2) Calculation of Critical Shear Buckling Stress, F_{vcr}

The value of F_{vcr} shall be determined in accordance with Article 1.7.YY (B)(2),(3) and (4), except for the following provision applying to the case of unstiffened web:

F^{o}_{vcr}, the critical web buckling stress under shear stress acting alone, shall be calculated as a function of the web plate slenderness parameter

$$\lambda_v = 0.30 \frac{D}{t_w} \sqrt{\frac{F_y}{E}}$$

where D = clear depth of web between the flanges, measured along the web.

t_w = web thickness

The values of F^{o}_{vcr} are found from equations in Table 1.7.YY(B)(2), or from Fig. 1.7.XX.

(C) Design Stresses in Web

The governing load-induced coincident shear and flexural or direct stresses to be used in the design of an unstiffened web shall be calculated at the following locations:

(a) at distance D/2 from support

(b) at location of maximum positive moment between the supports of box girder

(c) At distance D/2 from location of change of thickness or yield stress of web material, on side of smaller thickness or yield stress.

The shear stresses due to flexure or other effects shall be assumed uniformly distributed over the depth of the web panel. Direct stresses due to flexure or other effects shall be computed in accordance with elastic theory.

(D) Slenderness Limitations

The thickness of unstiffened webs shall meet the following requirements, but shall not be less than 3/8" (10 mm).

$$D_c \leq D/2: \quad \frac{D_c}{t_w} \leq \frac{3.4}{\sqrt{F_y/E}}$$

$$D_c > D/2: \quad \frac{D}{t_w} \leq \frac{6.8}{\sqrt{F_y/E}}$$

where D_c = distance between neutral axis and compression flange.

1.7.YY Transversely Stiffened Webs

(A) Scope

This article applies to box girder webs with transverse stiffeners but without longitudinal stiffeners.

(B) Allowable Shearing Stress

(1) General - The average calculated unit shearing stress in the gross section of the web plate, f_v

shall not exceed the allowable shearing stress,

F_v (in psi).

$$F_v = 0.55 \ (F_{vcr} + F_{VT})$$

where F_{vcr} = critical buckling shear stress, see Art. 1.7.YY (B)(4)

$$F_{VT} = \frac{F_T}{2(\sqrt{1 + \alpha^2} + \alpha)}$$

where D_w = depth of web between flanges measured along web

$\alpha = d_o/D$

d_o = distance between transverse stiffeners

F_T = tension field stress, see Art. 1.7.YY(B)(5)

The F_{VT} term in the equation for F_v may be disregarded if its utilization in accordance with the provisions of Articles 1.7.YY and 1.7.ZZ is not advantageous in the design.

The maximum value of F_v shall not exceed 0.55 of F_{vy} (yield shear strength of the web with consideration of coincident axial stress

$$F_{vy} \leq 0.58 \sqrt{F_y^2 - (\frac{2}{3} f_{av})^2}$$

where f_{av} = average numerical value of the flexural axial stresses at the opposite longitudinal edges of the web panel

$\left.\begin{array}{l} f_{1w} \\ f_{2w} \end{array}\right\}$ as defined in Article 1.7.YY (B)(4) disregarding the sign of the stress.

The design shear force of an inclined web, V_W, is

$$V_W = V_V/\cos\theta$$

where V_V is vertical shear force and θ is the angle of inclination of the web to the vertical.

(2) Critical Shear Buckling Stress

Critical buckling stress in the case of shear stress acting alone, F^o_{vcr}, shall be computed as a function of the plate slenderness parameter

$$\lambda_v = \frac{D}{t_w}\sqrt{\frac{10.92}{\pi^2\sqrt{3}\,k_v}\,\frac{F_y}{E}} = 0.8\,\frac{D}{t_w}\sqrt{\frac{F_y}{E\,k_v}}$$

where k_v is the plate buckling coefficient defined as:

$$k_v = 5 + \frac{5}{\alpha^2}$$

The values of F^o_{vcr} are given by the equations given in the table, or may be obtained from Fig. 1.7.YY(A)

TABLE 1.7.YY(B)(2)

Web Slenderness, λ_v	Critical Shear Stress, F^o_{vcr}
$\lambda_v \leq 0.58$	$F^o_{vcr} = 0.58\ F_y$
$0.58 \leq \lambda_v \leq 1.41$	$F^o_{vcr} = [0.58 - 0.357\ (\lambda_v - 0.58)^{1.18}]F_y$
$\lambda_v > 1.41$	$F^o_{vcr} = 0.58\ F_y/\lambda_v^2$

(3) Critical Flexural Buckling Stress

Since any unsymmetrical axial stress distribution in the web can be represented as a combination of pure compression (or tension) and pure bending, critical stresses for these basic cases only are needed in the computation of the critical buckling stress for combined shear and axial stress (see Art. 1.7.YY(B)(4).

The critical stresses F^o_{ccr} for Case (1), compression acting alone, and F^o_{bcr} for Case (2), bending acting alone, are given by the following equations:

$$0.65 \leq \lambda \leq 1.5: \quad F^o_{cr}/F_y = 0.072\ (\lambda - 5.65)^2 - 0.78$$

$$\lambda \geq 1.5: \quad F^o_{cr}/F_y = 1/\lambda^2$$

where

$$\lambda = \frac{D/t_w}{0.95}\sqrt{\frac{F_y}{E\ k}}$$

The value of k shall be taken as

Case (1): $\alpha > 1$, $k = 4$; $\alpha < 1$, $k = (\alpha + 1/\alpha)^2$

Case (2): $\alpha > 2/3$, $k = 24$; $\alpha < 2/3$, $k = 24 + 73\,(2/3 - \alpha)^2$

The values of critical stresses F^o_{ccr} for $\alpha > 1$ and F^o_{bcr} for $\alpha > 2/3$ may also be read from Fig. 1.7.YY(B).

(4) <u>Critical Buckling Stress for Combined Shear and Axial Stress</u>

The critical buckling stress of panels subject to simulaneous shear and axial stresses shall be computed from the interaction equation

$$\left(\frac{F_{vcr}}{F^o_{vcr}}\right)^2 + \left(\frac{F_{bcr}}{F^o_{bcr}}\right)^2 + \left(\frac{F_{ccr}}{F^o_{ccr}}\right) = 1$$

where F^o_{vcr} = critical shear buckling stress in the case of shear stress acting alone, obtained from equations in Table 1.7.YY (B)(2) or from Fig. 1.7.YY(A)

F^o_{bcr} = critical bending buckling stress in the case of bending acting alone, to be obtained from equations in Art. 1.7.YY (B)(3) or from Fig. 1.7.YY(B) Curve (2)

F^o_{ccr} = critical compressive buckling stress in the case of pure axial compression acting alone, to be obtained from equations in Art. 1.7.YY(B)(3) or from Fig. 1.7.YY(B) Curve (1)

F_{vcr}, F_{bcr}, and F_{ccr} are individual (shear, pure bending and pure compression) stress components which cause buckling of the web panel when acting simultaneously. These stress components are interdependent and may be expressed in terms of F_{vcr} by the following expressions:

$$F_{bcr} = \frac{1 - R}{2} \quad \mu F_{vcr}$$

$$F_{ccr} = \frac{1 + R}{2} \quad \mu F_{vcr}$$

where $R = \dfrac{f_{2w}}{f_{1w}}$

$\mu = \dfrac{f_{1w}}{f_v}$

with f_{1w} = governing axial compressive stress at longitudinal edge of web panel at location of the design stress (see Arts. 1.7.XX(C) or 1.7.YY (C)) due to moment, M_v, coincident with maximum design shear, V, used in design of web panel

f_{2w} = axial stress at opposite edge of panel coincident with f_{1w}. Compression is designated positive, tension negative.

f_v = governing shear stress = V/Dt_w

These stresses are illustrated in Fig. 1.7.YY(C).

The value of R may be positive or negative, depending on the signs of stresses f_{1w} and f_{2w}.

When the maximum tensile stress is numerically greater than the compressive stress, $(R < -1)$, the interaction equation reduces to the following form:

$$\left(\frac{F_{vcr}}{F^o_{vcr}}\right)^2 + \left(\frac{F_{bcr}}{F^o_{bcr}}\right)^2 = 1$$

where $F_{bcr} = \mu F_{vcr}$.

(5) Tension Field Stress

The tension field stress of a web panel, F_T, to be used for determination of F_{VT} in accordance with Art. 1.7.YY(B)(1) shall be found from the following formula:

$$F_T = F_y - \sqrt{0.25\ f^2_{2w} + 3F^2_{vcr}}$$

with the notation as given in Art. 1.7.YY(B)(4).

(6) Web Panels Adjacent to End Support of Girder

Web panels adjacent to end supports of the box girder may be designed with or without the utilization of the tension field strength.

If tension field strength is utilized, the end bearing stiffeners shall be designed in accordance with Art. 1.7.213(B)(3) of Ref. 32.

(C) Design Stresses in Web Panel

The coincident shear and flexural or direct stresses for web panel design shall be calculated at the cross section of the panel midway between transverse stiffeners.

Shear stresses due to flexure or other effects shall be assumed uniformly distributed over the web depth.

Direct stresses due to flexure or other effects shall be computed in accordance with elastic theory.

(D) Slenderness Limitations

(1) The thickness of transversely stiffened webs shall meet the requirements, but shall not be less than 3/8" (10 mm).

$$D_c \leq D/2: \qquad D/t_w \leq 6.8/\sqrt{F_y/E}$$

$$D_c > D/2: \qquad D_c/t_w \leq 3.4/\sqrt{F_y/E}$$

where D_c = distance between neutral axis and compression flange.

(2) Web stiffener sizes shall be governed by the requirements of Art. 1.7.213 of Ref. 32.

(E) Additional Average Stresses in Flanges due to Post-Buckling Behavior of Webs

Since the capacity of the web to carry compressive stresses is limited by compressive stress corresponding to web buckling, any additional axial stress assigned to web under the assumption of linear stress distribution must be carried by the flanges. Also, additional flange stresses due to assumptions used in formulating the tension field action must be considered.

The additional average flange stresses, Δf_b, to be added to the flange stresses computed in accordance with elastic analysis, shall be calculated in the web panel at the box girder cross section used for design of the flange panel under consideration by the following formulas

Compression flange:

$$\Delta f_{b1} = (1 - \frac{\Sigma F_{vcr}}{f_{VM}})[(f_{1R} - f_1) + \frac{1}{2}\frac{Dt_w}{A_{fc}} f_{VM} \cot (\frac{\theta_d}{2})]$$

Tension flange:

$$\Delta f_{b2} = (1 - \frac{\Sigma F_{vcr}}{f_{VM}})[(f_{2R} - f_2) - \frac{1}{2}\frac{Dt_w}{A_{ft}} f_{VM} \cot (\frac{\theta_d}{2})]$$

Notation is as follows:

f_{VM} = overall average shearing stress = total shear force acting on box girder coincident with maximum moment, M, at the same box girder cross section, divided by Dt_w

Σf_{vcr} = sum of buckling (beam action) shear stress of all webs at box girder cross section under consideration, determined for combined action of

f_{VM} and M in accordance with Art. 1.7.YY(B)(4)

f_1, f_2 = stress in compression or in tension flange, respectively, due to moment calculated by elastic theory, assuming fully participating webs

f_{1R}, f_{2R} = stress in compression or tension flange respectively, due to moment M calculated by elastic theory, assuming reduced moment of inertia, I_R, of box girder cross section

I_R = moment of inertia of box girder cross section obtained by removing those portions of web in compression. For purposes of calculation of Δf_b it may be assumed that this removal does not change position of the box girder neutral axis.

A_{fc}, A_{ft} = compression or tension flange area, respectively or equivalent steel area of a composite flange.

$\theta_d = \cot^{-1}(\alpha)$ = angle of inclination of web panel diagonal to the horizontal

1.7.ZZ Transversely and Longitudinally Stiffened Webs

(A) Scope

This article applies to box girder web panels with transverse and longitudinal stiffeners.

(B) Allowable Shearing Stress

(1) The average calculated unit shearing stress in the gross section of the web plate, f_v, shall not exceed the allowable shearing stress F_V, in accordance with the procedures given in Art. 1.7.YY(B) and (C) with modifications as given in (2), (3), (4) and (5) hereunder.

(2) The critical shear buckling stress, F_{vcr}, under combined shear and axial stresses shall be determined separately for each web subpanel between the flange and the longitudinal stiffener, or between two longitudinal stiffeners. Longitudinal stiffeners are treated as rigid supports. The minimum value of F_{vcr} of the critical subpanel, $F_{vcr\ min}$, shall govern the buckling stress of the web.

(3) In calculation of the shear buckling stress, F_{vcr}, of the subpanels under combined shear and flexural compression (such as subpanels 1 and 2 in Fig. 1.7.ZZ) or shear and flexural compression and tension (such as subpanel n in Fig. 1.7.ZZ) the following notation shall apply:

$$\alpha' = d_o/D'$$

$$R = f'_{2w}/f'_{1w}$$

$$\mu = f'_{1w}/f_v$$

where D' = depth of subpanel

f'_{1w} = governing axial compressive stress at longitudinal edge of subpanel, computed midway between transverse stiffeners, due to the moment, M_V, coincident with maximum design shear, V, used in design of web panel

f'_{2w} = axial stress coincident with f'_{1w} at opposite edge of subpanel. Compression is designated positive, tension negative

f_v = governing shear stress = V/Dt_w

(4) The shear buckling stress, F_{vcr}, of the subpanels under combined shear and flexural tension (no compression stress in the subpanel) is given by the following equations:

$$\text{for} \quad \lambda_v < 0.58: \quad F_{vcr} = 0.58\, F_y$$

$$\text{for } 0.58 \le \lambda_v \le 1.41: \quad F_{vcr} = [0.58 - 0.357\,(\lambda_v - 0.58)^{1.18}]\, F_y$$

$$\lambda_v > 1.41: \quad F_{vcr} = 0.58\, F_y/\lambda_v^{\,2}$$

where

$$\lambda_v = 0.8\,\frac{D'}{t_w}\sqrt{\frac{F_y}{E\,k_v^*}}$$

With K_v^*, the plate buckling coefficient for combined shear and tension, to be taken as:

$$\text{for } 0.5 \le \alpha' \le 1: \quad k_v^* = 5 + 5/\alpha'^2 + (4 + 3/\alpha'^2)[-f_t/f_v + (f_t/f_v)^2]$$

for $\alpha' > 1$: $k_v^* = 5 + 5/\alpha'^2 + (1.5 + 5.5/\alpha'^2)[-f_t/f_v + (f_t/f_v)^2]$

where f_t = the average value of the tension stresses, coincident with governing shear stress, at the two longitudinal edges of the subpanel, computed midway between transverse stiffeners. Tension stress is designated negative; therefore the ratio f_t/f_v is always negative.

$\left.\begin{matrix} f_v \\ \text{and} \\ \alpha' \end{matrix}\right\}$ are defined in Art. 1.7.ZZ(B)(3)

(5) The tension field stress, F_{V_T}, of the web shall be determined for the entire web panel between transverse stiffeners, with horizontal stiffeners disregarded.

(C) <u>Slenderness Limitations</u>

(1) <u>Webs with one line of Longitudinal Stiffeners</u>

The web thickness shall meet both of the following requirements:

$$\frac{D}{t_w} \leq \frac{13.6}{\sqrt{F_y/E}} \text{ and } \frac{D'}{t_w} \leq \frac{2.7}{\sqrt{F_y/E}}$$

where D = clear depth of web between the flanges

D' = the depth of subpanel adjacent to compression flange $\geq 2D_c/5$

where D_c = clear distance between neutral axis and compression flange.

The horizontal stiffener shall not be placed impractically close to the compression flange.

(2) Webs with two or more Lines of Longitudinal Stiffeners

The web thickness shall meet the following requirements for each subpanel in the compression zone

$$\frac{D'_n}{t_w} \leq \frac{8.1}{\sqrt{F_y/E}} \quad \frac{\eta_n D}{D_c}$$

where D'_n = depth of subpanel between compression flange and stiffener or between two stiffeners in the compression zone

$\eta_n D$ = distance between compression flange and stiffener n, see Fig. 1.7.ZZ.

The depth of subpanel between compression flange and the first stiffener shall meet the requirement

$$D'_1 \leq \frac{2D_c}{5}$$

(3) Minimum web thickness shall be 3/8" (10 mm)

(4) The sizes of stiffeners shall be governed by the requirements of Art. 1.7.213 of Ref. 32.

(D) Additional Average Stresses in Flanges due to Post-Buckling Behavior of Webs

(1) Additional axial stresses in the flanges due to load shedding and tension field action of the webs shall be determined by the formulas for Δf_b given in

Art. 1.7.YY(E), except that if longitudinal stiffeners are continuous, the reduced moment of inertia, I_R, of box girder cross section may incorporate the area of the longitudinal stiffeners including appropriate effective widths of the web plate.

3.2.2 Commentary

The proposed Articles 1.7.XX, 1.7.YY and 1.7.ZZ are direct adoptions of Articles 1.7.210, 1.7.211 and 1.7.212, respectively for Load Factor Design in the proposed Design Specifications for Steel Box Girders.[32]

The allowable stresses are derived by incorporating a factor of safety 1.83. All comments in the Commentary to these articles of Load Factor Design are applicable here, and are not repeated. Articles 1.7.213 (of Ref. 32) for the design of web stiffeners is also directly applicable with a direct conversion of forces to stresses; thus is also not repeated.

The existing provisions for design of flanges (Ref. 6) can also be replaced by the corresponding provisions in Ref. 32.

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The research project was conducted in Fritz Engineering Laboratory and Department of Civil Engineering, Lehigh University.

Thanks are due Mrs. Dorothy Fielding for handling and processing this report.

TABLE 1 SEQUENCE OF STRENGTH DEVELOPMENT

COMPOSITE BOX GIRDER SEGMENTS

Positive Bending and Torsion			Negative Bending and Torsion
Buckling of Web(s)	Buckling of Web(s)	Yielding of Bottom Flange	Buckling of of Web(s)
Tension Field Action	Yielding of Bottom Flange	Yielding of Web(s)	Buckling of Bottom Flange
Yielding of Bottom Flange	Tension Field Action	Failure of Concrete Deck	
Failure of Concrete Deck	Failure of Concrete Deck		

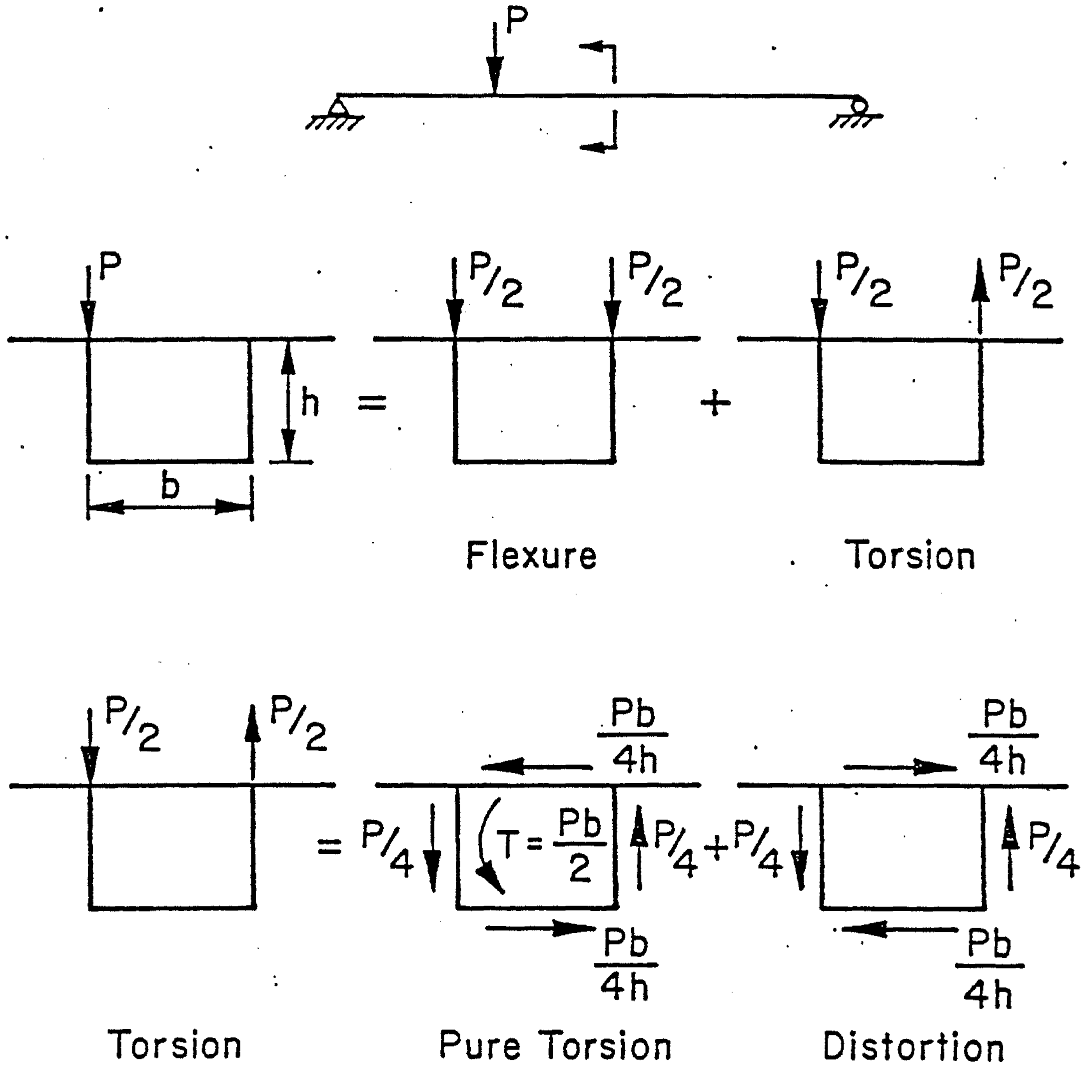


Fig. 1 Decomposition of Box Girder Loading

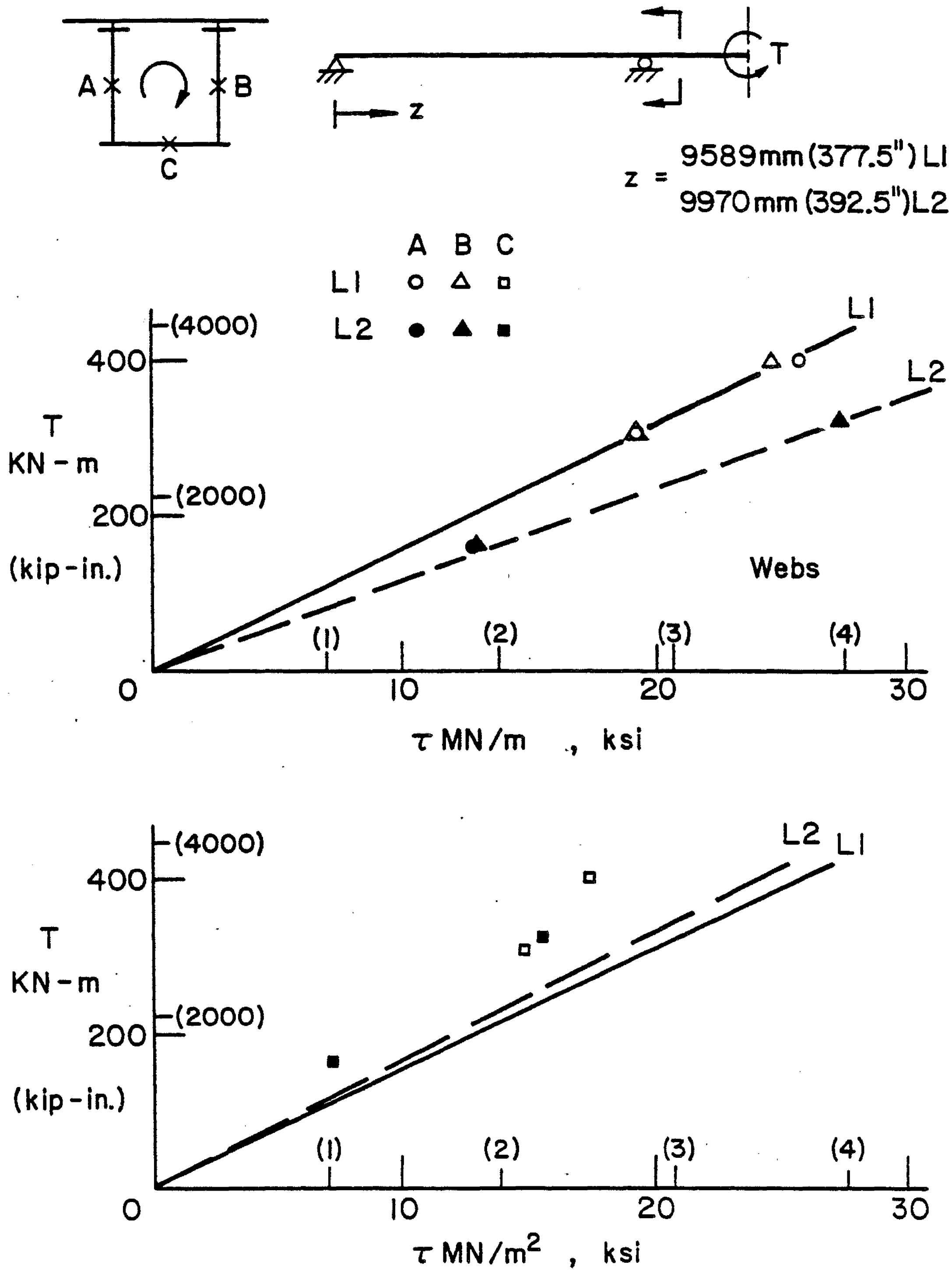


Fig. 2 Shearing Stresses in Specimen L1 and L2

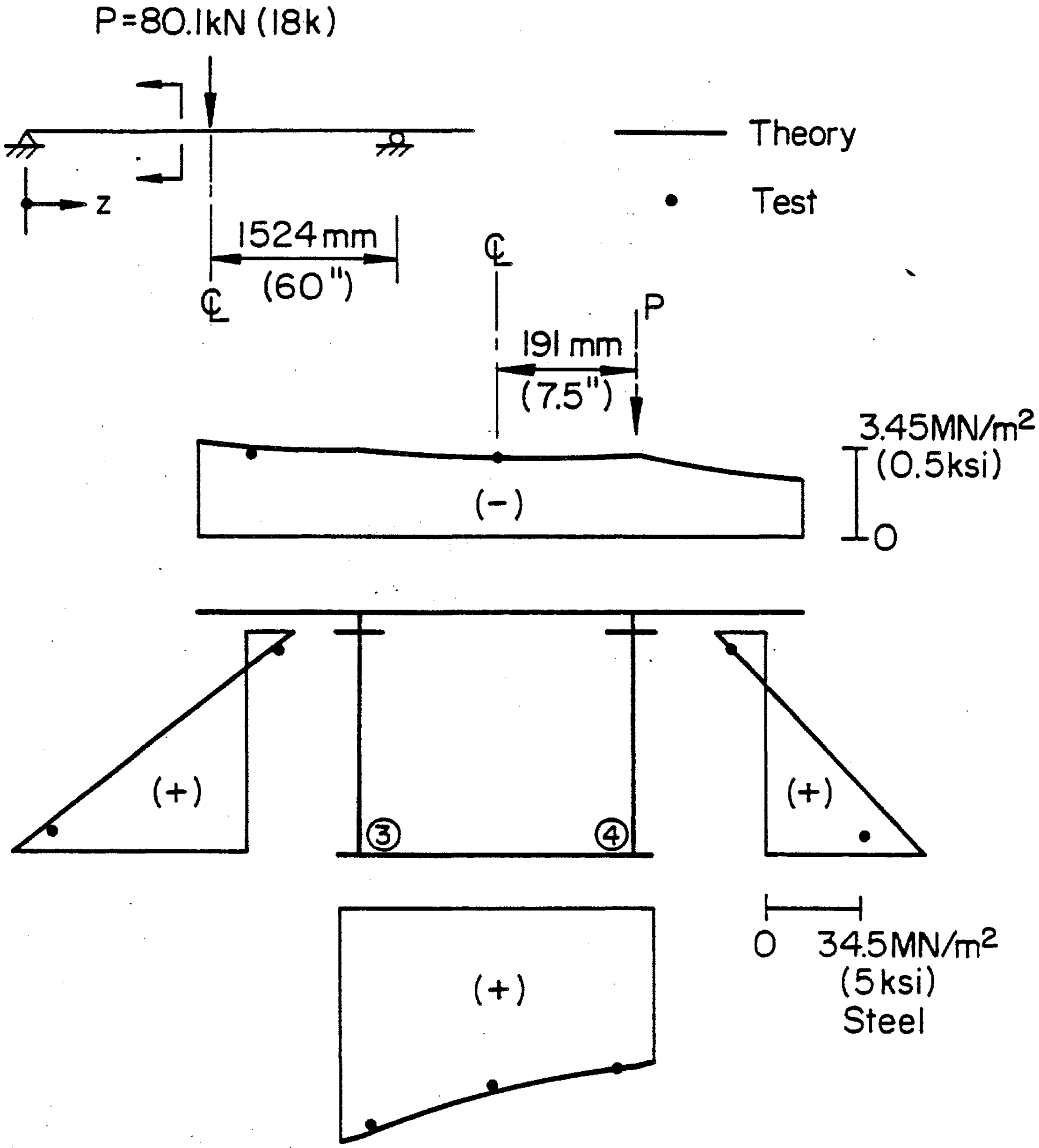


Fig. 3 Normal Stresses in the Cross Section of Girder D2 at Z = 1397 mm (55 in.)

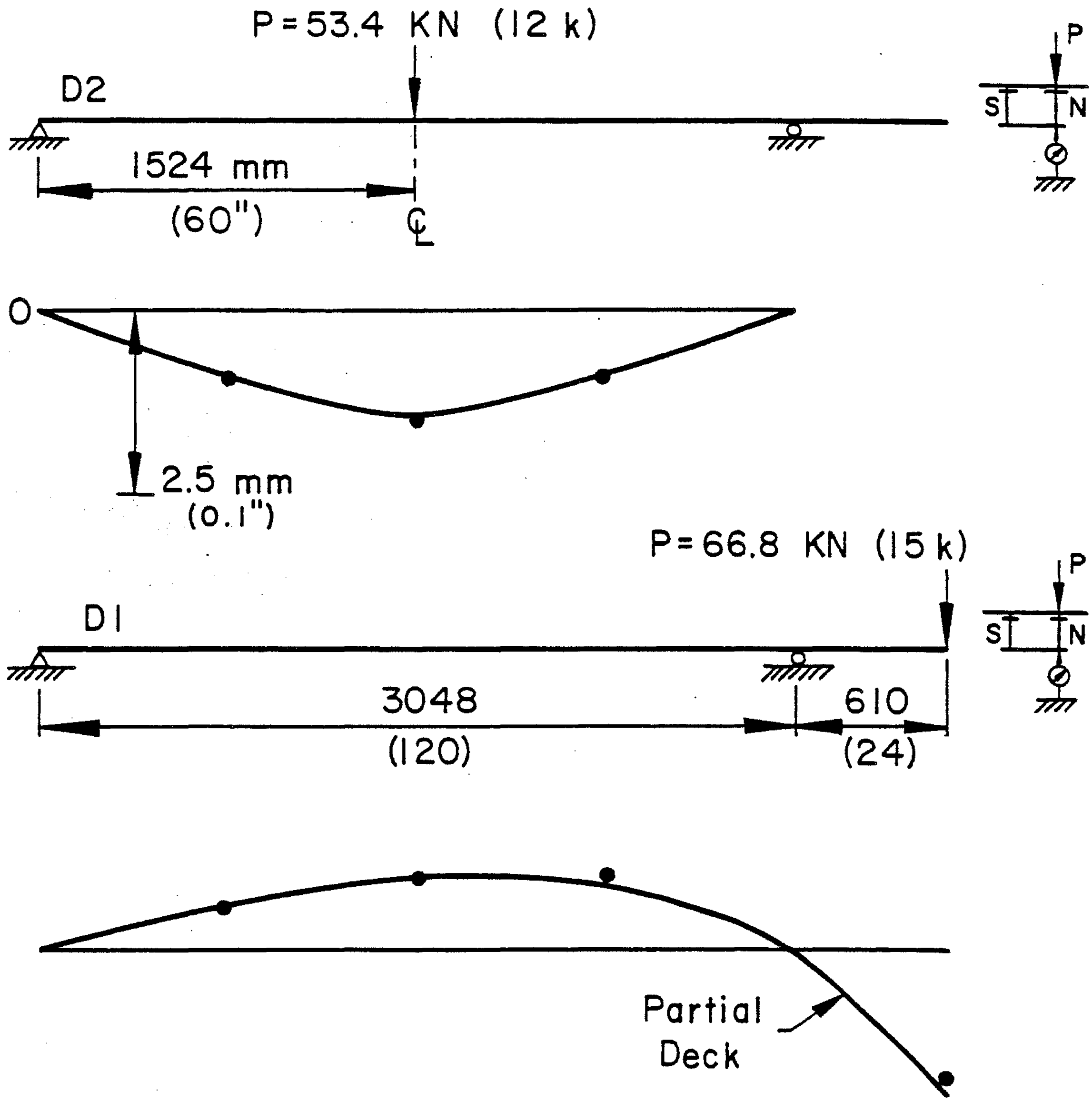


Fig. 4 Deflections along the Span of Box Girders D1 and D2

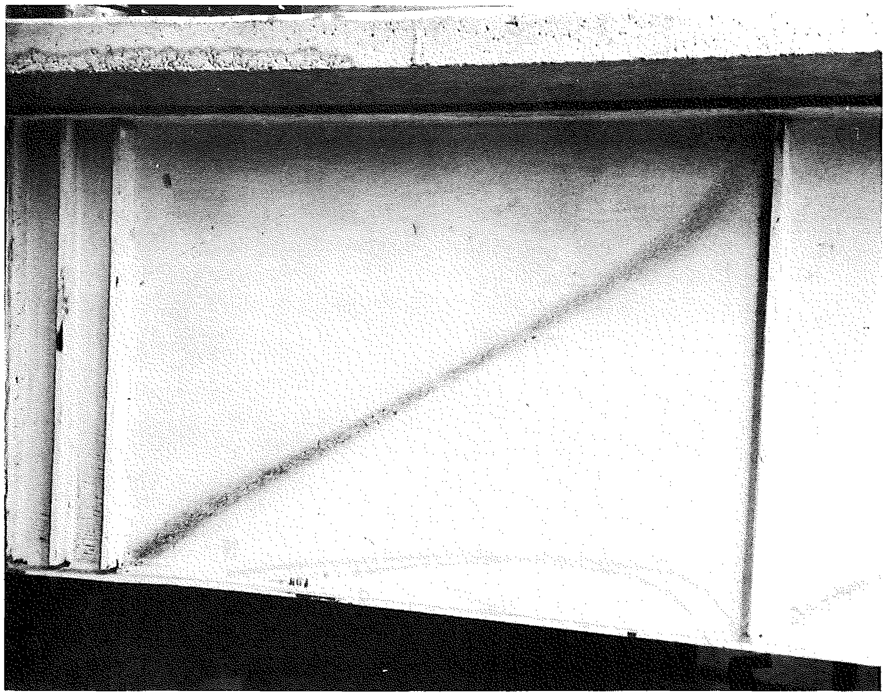


Fig. 5 Tension Field in North Web of End Panel L2-CD

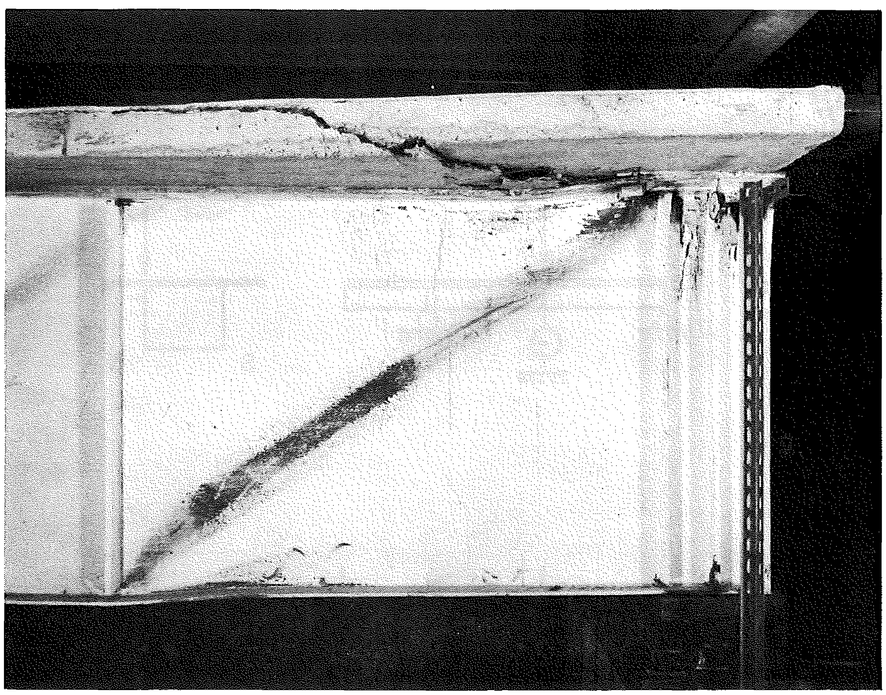


Fig. 6 Failed North Web and Flanges of West End Panel L2-CB

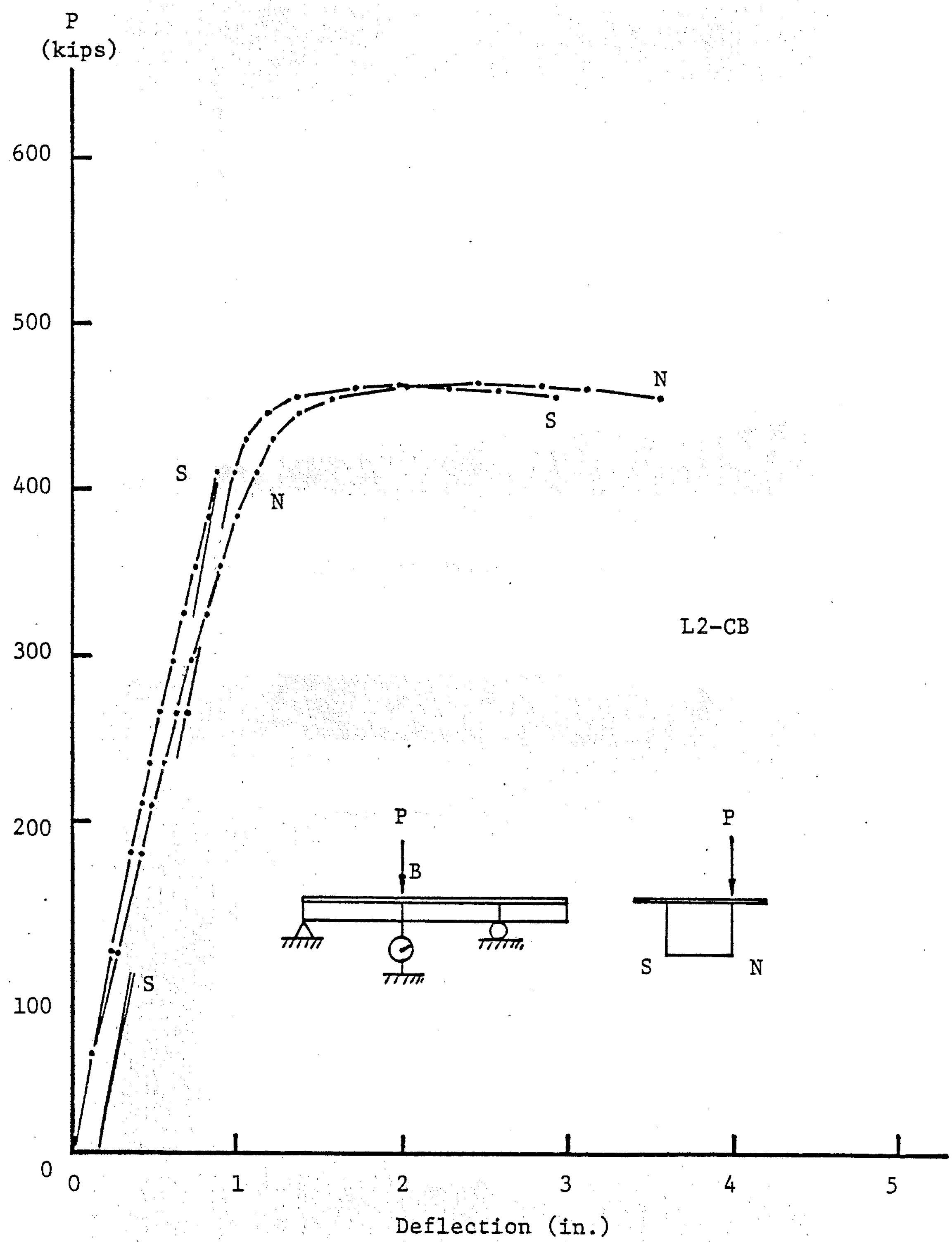


Fig. 7 Load-Deflection Curves, Composite Box Girder Test L2-CB

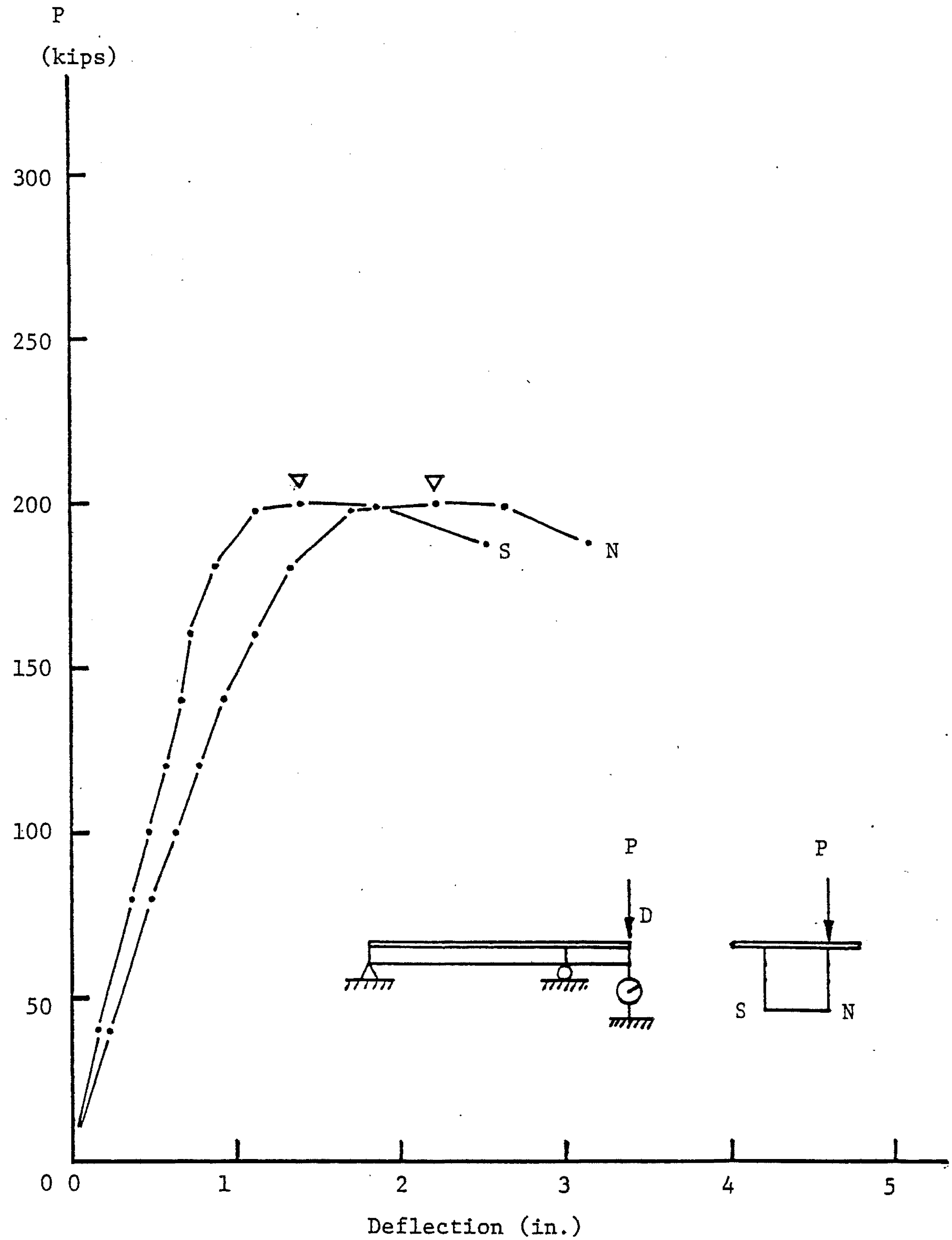


Fig. 8 Load-Deflection Diagram, Test L2-CD

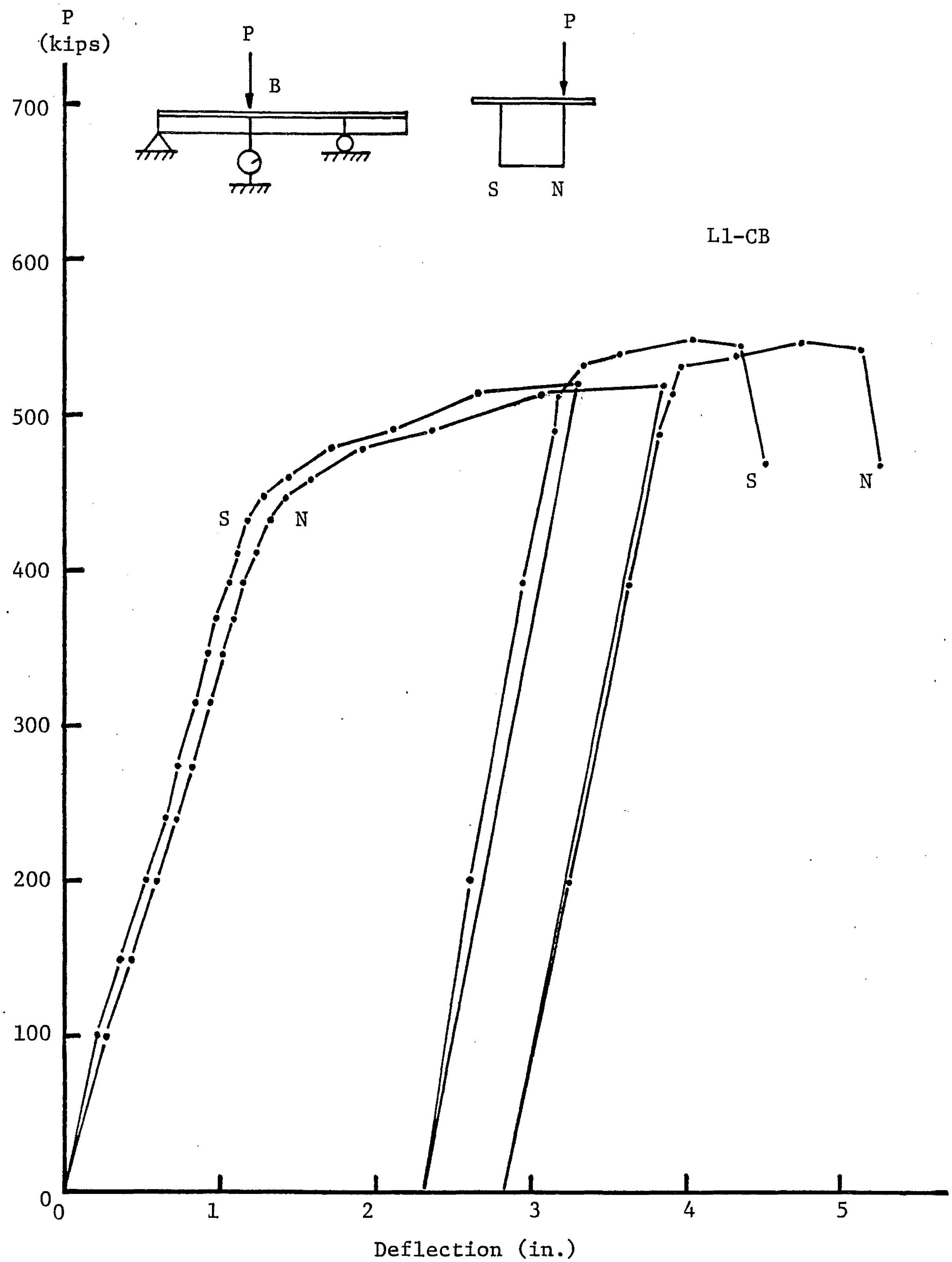


Fig. 9 Load-Deflection Curves, Composite Box Girder Test L1-CB

Fig. 10 Failure of Composite Box Girder L1-CB

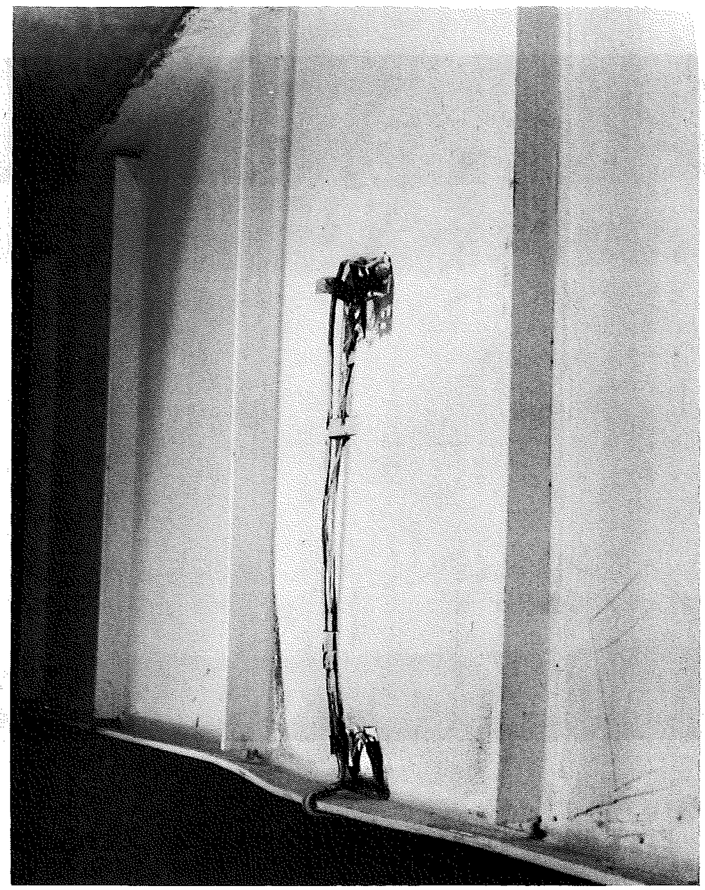


Fig. 11 Deflected Bottom Flange and South Web of Panel 11 L1-CD

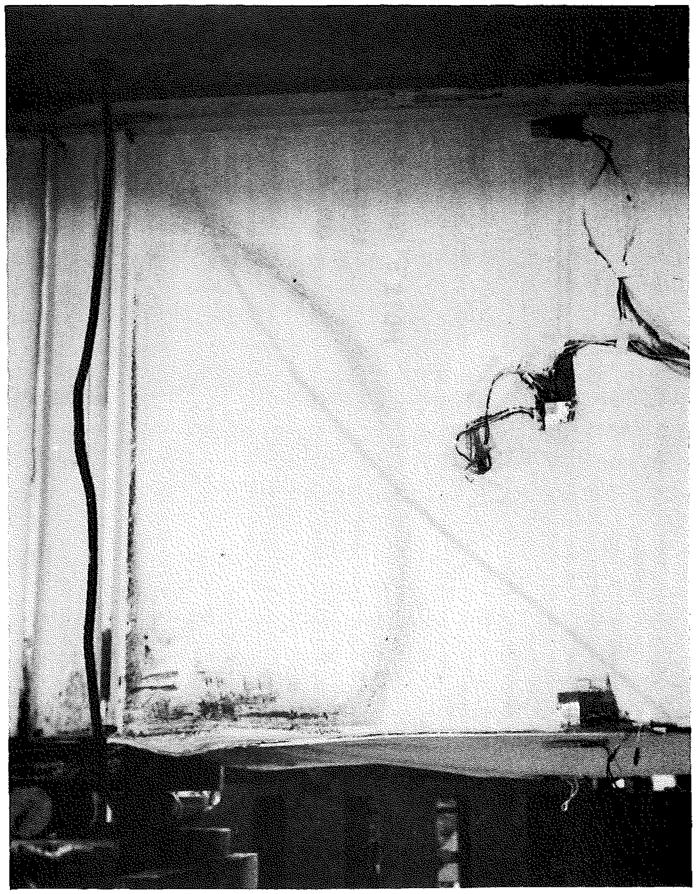


Fig. 12 Failure at Bottom Flange and South Web, L2-CD

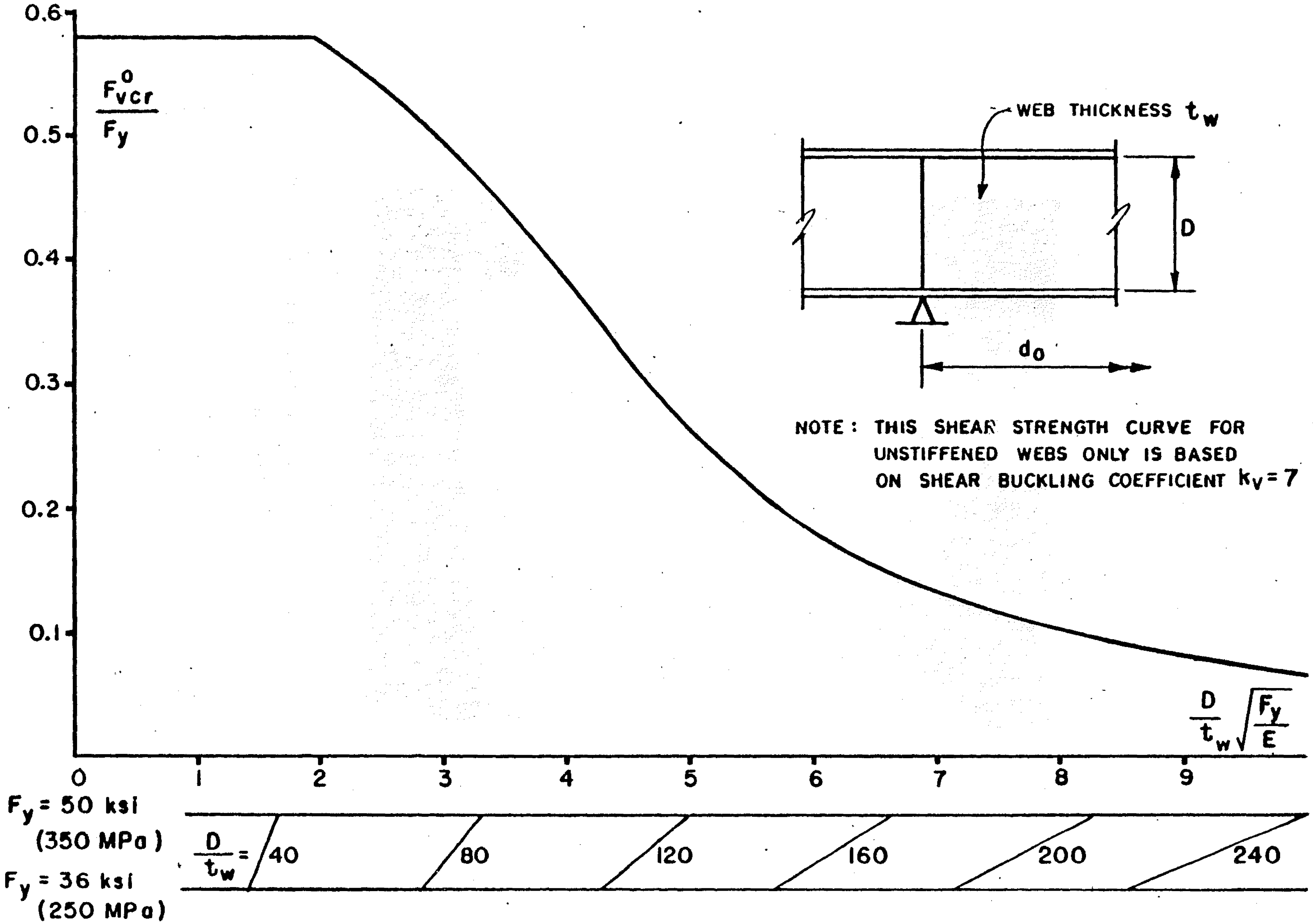


FIG. 1.7. XX CRITICAL BUCKLING STRESS OF UNSTIFFENED WEB DUE TO SHEAR ACTING ALONE

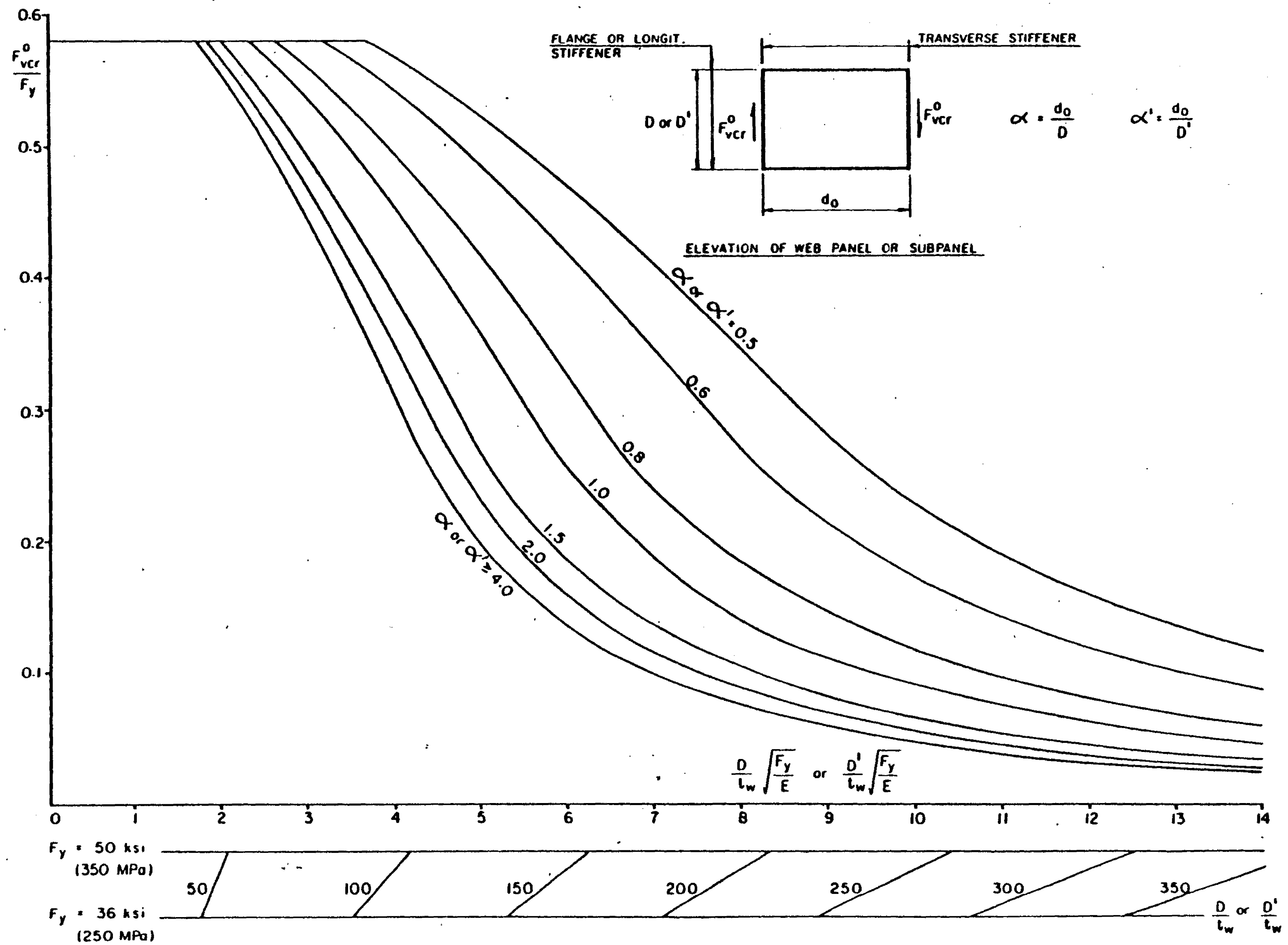


FIG. 1.7.yy (A) CRITICAL BUCKLING STRESS DUE TO SHEAR ACTING ALONE

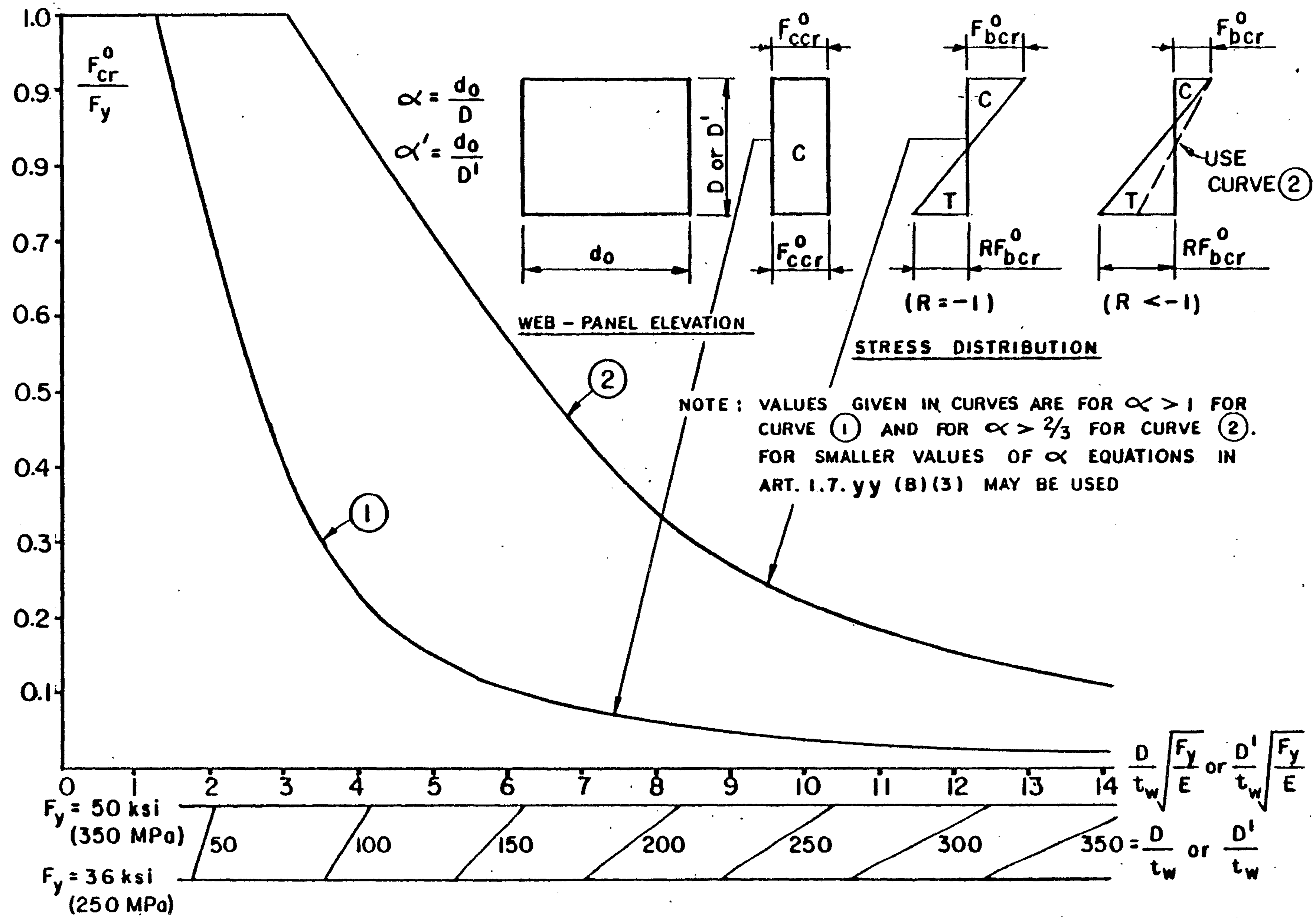
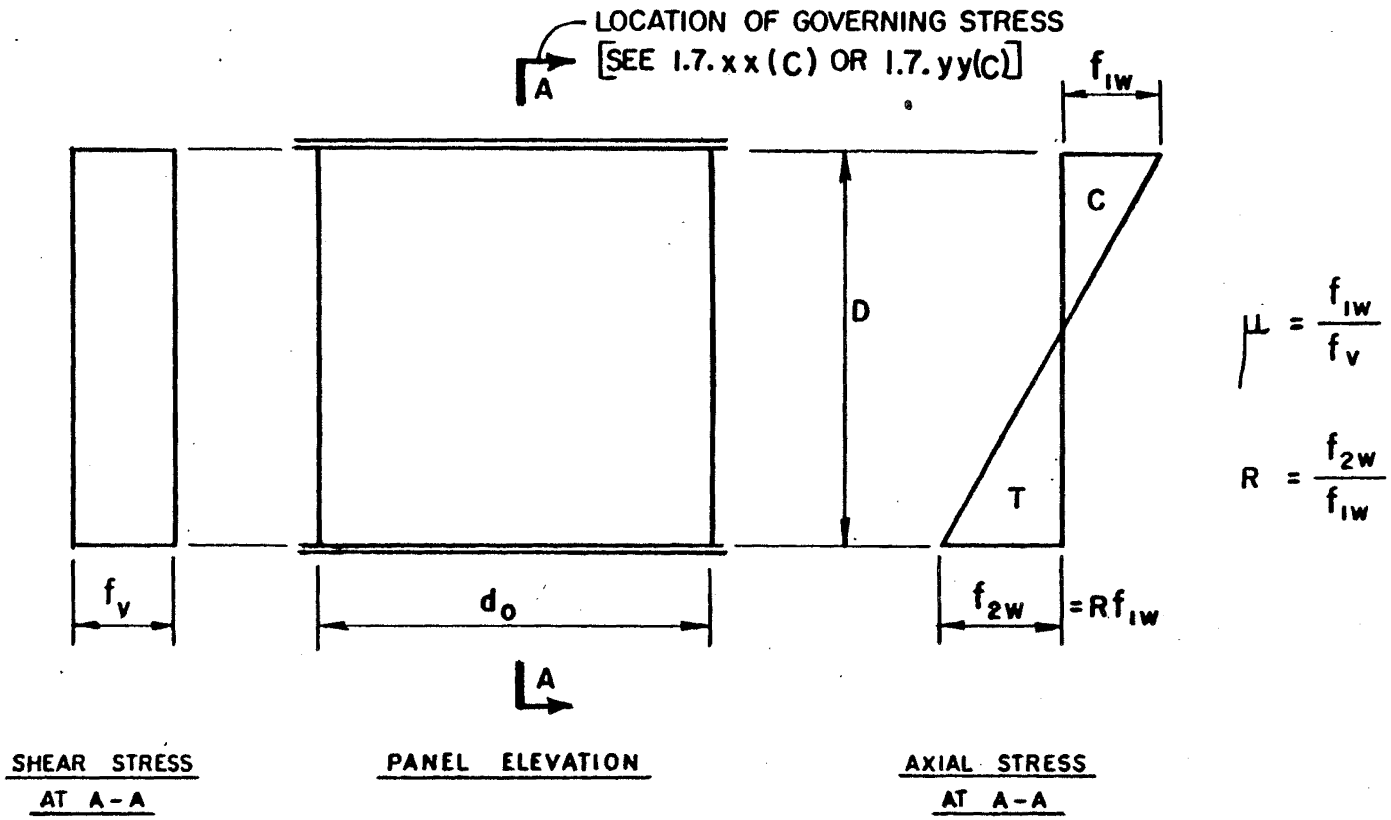
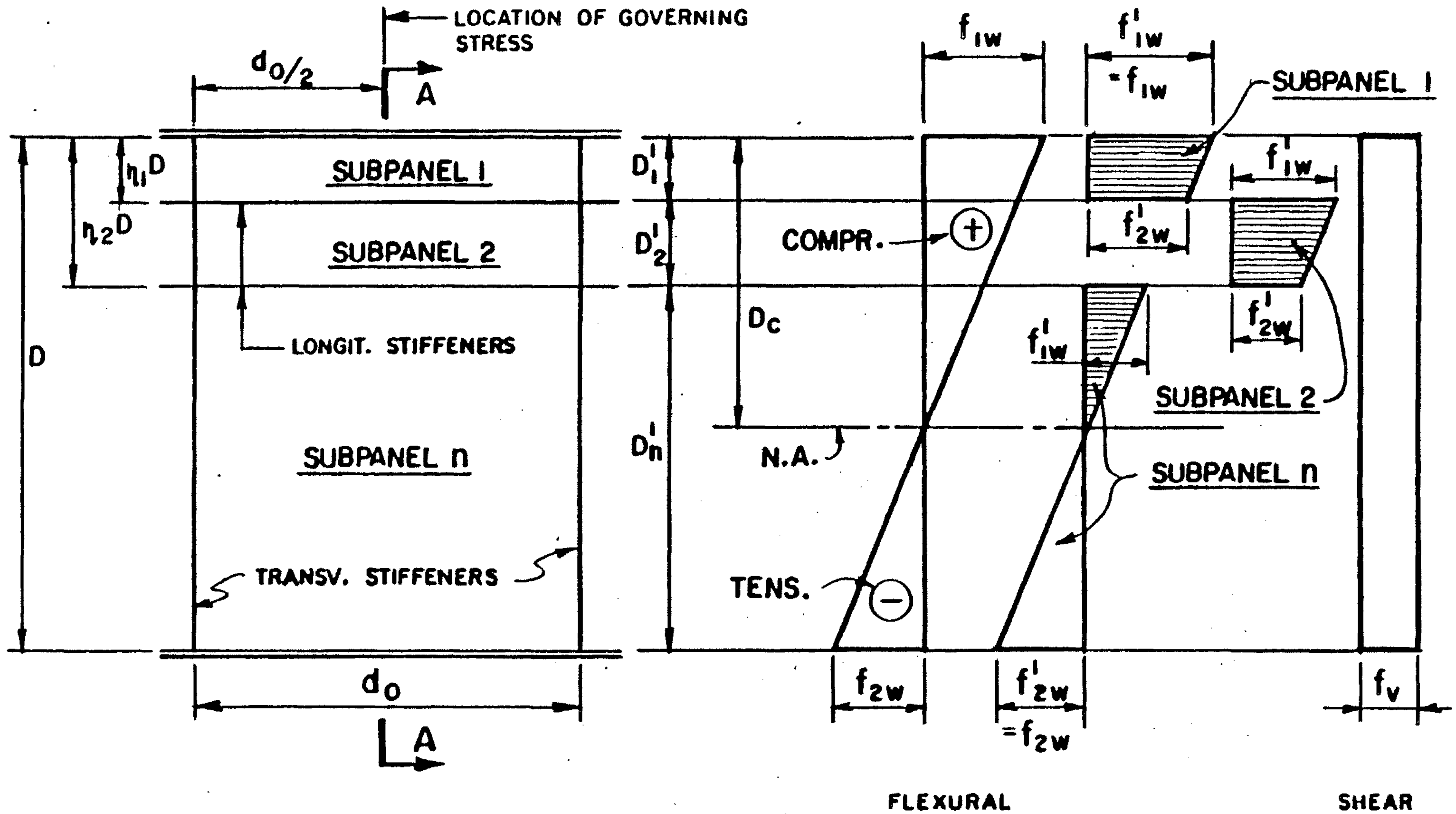


FIG. 1.7.yy (B) CRITICAL BUCKLING STRESS DUE TO AXIAL STRESSES ACTING ALONE



FOR DEFINITIONS OF f_{1w}, f_{2w}, f_v SEE ART. 1.7.yy (B)(4)

FIG. 1.7.yy(C) DEFINITION OF μ AND R FOR UNSTIFFENED AND TRANSVERSELY STIFFENED WEBS



PANEL ELEVATION

STRESSES AT A - A

$$\alpha = \frac{d_0}{D} \qquad \alpha'_n = \frac{d_0}{D'_n} \qquad \mu = \frac{f'_{1w}}{f_v} \qquad R = \frac{f'_{2w}}{f'_{1w}}$$

FOR DEFINITIONS OF f_{1w} , f_{2w}, f_v SEE ART. 1.7.yy (B)(4)

FIG. 1.7.zz TRANSVERSELY AND LONGITUDINALLY STIFFENED WEBS

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