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STEEL RESEARCH for construction

TENTATIVE CRITERIA
for
LOAD FACTOR DESIGN OF STEEL HIGHWAY BRIDGES

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TENTATIVE CRITERIA

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LOAD FACTOR DESIGN OF STEEL HIGHWAY BRIDGES

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PREFACE

The primary objective of Load Factor Design is to obtain greater consistency in the maximum live load carrying capacities of steel bridges – long and short, heavy and light. The attainment of this objective will in itself result in more effective use of steel. Furthermore, the more sophisticated analyses required to predict maximum strengths will lead to more efficient structural assemblies. The load factors used in establishing the capacity requirements (Section 1.7) and the safety provisions applied to the calculated maximum strengths of members (e.g. ϕ in Table 3 of Article 2.5.1A) represent selective evaluations of the various uncertainties which are lumped into factors of safety in working stress design.

Load factor design is a method of proportioning structural members for multiples of the design loads. With properly selected multiples, it can assure a design allowing the expected number of passages of ordinary vehicles during the life of the structure, occasional passages of reasonable overload vehicles without damage and, for an extreme emergency, a few passages of exceptionally heavy overloads.

Since design specifications based on these criteria should eventually constitute a compatible component of the AASHO Standard Specifications for Highway Bridges, the requirements of the latter for load distribution, design of details, limiting ratios, fatigue, deflection, various basic assumptions, etc. have been incorporated by reference except where the desirability of alteration in some of these is indicated by recent research. Nomenclature follows that of the 1965 AASHO Specifications for Highway Bridges*, where applicable, and prevalent practice in other cases.

Regardless of the philosophy underlying such a design procedure as this, its prospective output must be compared with the only body of pertinent experience available, the service behavior of the bridges which have been built under past and present working stress design. The composite judgments reached on past experience and research cannot be disregarded, although they can be altered on the basis of new knowledge or a better understanding of the old.

These criteria are supplemented by a commentary containing detailed explanations, supporting evidence and references.

The material was prepared by several contributors, as follows:

Sections 1 and 3 and
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Beams and Girders....................Ben T. Yen and A. Ostapenko
Composite Beams...........James W. Baldwin, Jr.
Compression Members..............T.V. Galambos
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The Advisory Committee, contributors and sponsors note with sorrow the passing of George S. Vincent shortly after the completion of the work on this report. His supervision, overall coordination and guidance of the many contributors and his preparation of the text were indispensable to the development and completion of this document.

*"Standard Specifications for Highway Bridges." American Association of State Highway Officials, Ninth Edition, 1965.

Nomenclature

A =area of cross section (in.^{2})
A_f =area of one flange of beam or girder (in.^{2})
$(A F_y)_{bf}$ =product of area and yield point for bottom flange of steel section of composite beam (lb)
$(A F_y)_{tf}$ =product of area and yield point for top flange of steel section of composite beam (lb)
$(A F_y)_w$ =product of area and yield point for web of steel section of composite beam (lb)
A_r =total area of longitudinal reinforcing steel at interior support within effective flange width at composite beam (in.^{2})
A_s =total area of steel section including cover plates (in.^{2})
A_s =gross effective area of column cross section
$(A F_y)_c$ =product of area and yield point of that part of reinforcement which lies in compression zone of slab of composite beam (lb)
A_w =area of web of beam (in.^{2})
a =depth of equivalent rectangular stress block in concrete (in.)
a =distance from center of bolt to edge of plate (in.)
a =spacing of transverse stiffeners of box girder (in.)
B =a coefficient
b =distance center to center of box girder web plates (in.)
b =distance from center of bolt to center of fillet of connected part (in.)
b =effective width of slab of composite beam (in.)
b' =width of projecting flange element (in.)
b' =width of outstanding stiffener element (in.)
C =compressive force in slab of composite beam (lb)
C =a web buckling coefficient
C =equivalent moment factor for beam-column
C' =compressive force in top portion of steel section of composite beam (lb)
D =dead load
D =distance center to center of box girder flange plates (in.)
D_c =clear distance between the neutral axis and the compression flange of an unsymmetrical section (in.)
D_c =moment caused by dead load acting on composite section (in.-lb)
D_s =moment caused by dead load acting on steel section (in.-lb)
d =depth of member (in.)
d_b =depth of beam
d_c =depth of column
d_o =distance between transverse stiffeners (in.)
d_w =depth of steel web of a composite section (in.)
d_s =diameter of stud shear connector (in.)
E =modulus of elasticity (29,000,000 psi)
F =a stress (psi)
F_{cr} =buckling stress (psi)
F_e =Euler buckling stress in plane of bending (psi)
F_r =allowable fatigue stress
F_u =specified minimum tensile strength (psi)
F_v =maximum allowable basic shear stress on effective weld area (psi)
F_{vc} =maximum allowable shear stress for combined tension and shear on bolts and rivets in bearing type *connections*
F_{vy} =shear yield stress equal to $F_y/\sqrt{3}$ (psi)
F_y =specified minimum yield point or yield strength of the type of steel being used (psi)
F_{yf} =specified minimum yield strength of flange (psi)

F_{yr} =specified minimum yield point of reinforcing steel (psi)

F_{yw} =specified minimum yield strength of web (psi)

f'_c =specified 28-day compressive strength of concrete (psi)

f_r =range of stress due to live load plus impact in slab reinforcement over support (psi)

f_t =axial tensile stress in bolt due to applied load (psi)

f_v =maximum allowable shear stress on bolt under tension (psi)

G =modulus of elasticity in shear (11,200,000 psi)

G_{st} =strain-hardening modulus in shear (psi)

H_s =height of a stud (in.)

h =average thickness of flange of channel shear connector (in.)

I =impact

I =moment of inertia (in.^{4})

I_s =moment of inertia of a longitudinal stiffener (in.^{4})

I_t =moment of inertia of a transverse stiffener (in.^{4})

J =torsional constant (in.^{4})

K =effective length factor

k =buckling coefficient depending on boundary conditions

k =distance from outer face of flange to toe of web fillet of member to be stiffened

k_l =a buckling coefficient

L =length of a compression member (in.)

L =distance between points of bracing of compression flange (in.)

L =live load

M, M_1, M_2 =moment on a cross section (in.-lb)

M_c =moment in column

M_p =full plastic moment capacity (in.-lb)

M_u =maximum moment capacity (in.-lb)

N =number of shear connectors

N_c =number of additional shear connectors at point of contraflexure

N_w =number of live load lanes on bridge

n =number of longitudinal stiffeners on box girder

P =axial compression on the member (lb)

P_u =maximum axial compression capacity (lb)

Q =prying force per bolt (lb)

Q =statical moment of transformed compressive concrete area about the neutral axis of the composite section or the statical moment of the area of reinforcement imbedded in the concrete for negative moment (in.^{3})

Q_u =maximum load capacity of shear connector (lb)

R =number of live load lanes per box girder

R =reduction factor for maximum strength moment of hybrid beam (in.-lb)

R_s =vertical force on connection of transverse stiffener to longitudinal stiffener of box girder (lb)

R_w =vertical force on connection of transverse stiffener to web of box girder (lb)

r =radius of gyration (in.)

r_y =radius of gyration with respect to Y-Y axis (in.)

S =section modulus (in.^{3})

S_l =section modulus of longitudinal stiffener (in.^{3})

S_r =range of horizontal shear per linear inch of junction of slab and girder (lb/in.^{2}/in.)

S_t =section modulus of transverse stiffener (in.^{3})

T =direct tension per bolt due to external load (lb)

t =flange thickness (in.)

t_b =thickness of flange delivering concentrated force (in.)

t_c =thickness of flange to be connected (in.)

t =thickness of thinnest part connected by bolts (in.)

t_s =slab thickness in composite beam (in.)

t_{tf} =thickness of steel compression flange in composite section (in.)

t =web thickness of channel shear connector (in.)

t_w =web thickness (in.)

t' =thickness of outstanding stiffener element in box girders (in.)

V =shear force on the cross section (lb)

V_p =plastic shear capacity (lb)

V_r =range of shear due to live load plus impact (lb/in.)

V_u =maximum shear capacity (lb)

V_w =design shear on web of box girder

W_c =roadway width between curbs (ft)

W_L =fraction of a wheel load applied to one box girder

w =width of flange of box girder between longitudinal stiffeners (in.)

w =length of channel shear connector (in.)

Y = ratio $\frac{\text{Web plate yield strength}}{\text{Stiffener plate yield strength}}$

$\bar{y}$ =distance from top of steel section to neutral axis of composite beam (in.)

Z =Plastic Section Modulus (in.^{3})

Z_r =allowable design range of load on a shear connector (lb)

α =ratio of numerically smaller to larger end moment on a column

β =ratio, A_w/A_f (tension flange)

θ =angle of inclination of web plate to the vertical (box girder)

υ =Poisson's ratio (0.3)

ρ =ratio F_{yw}/F_{yf} (tension flange)

ϕ =angle between beam axis and column web stiffener of a rigid connection

ϕ =reduction factor

ψ =distance from outer edge of tension flange to neutral axis divided by depth of steel section of hybrid beam

$$\Sigma (A\,F_y) = (A\,F_y)_{bf} + (A\,F_y)_{tf} + (A\,F_y)_w$$

Contents

Section 1—General Provisions

1.1 SCOPE

These tentative criteria are intended for use in the design of simple and continuous beam and girder structures of moderate length. The provisions of Division 1, Design, of the Standard Specifications for Highway Bridges of the American Association of State Highway Officials, hereinafter referred to as AASHO Specifications, shall govern where applicable except as specifically modified by requirements set forth in these design criteria.

1.2 DEFINITION

Load factor design is a method of proportioning structural members for multiples of the design loads. To insure serviceability and durability, consideration is given to the control of permanent deformations under overloads, to the fatigue characteristics under service loadings and to the control of live load deflections under service loadings.

1.3 LOADS

Service live loads are vehicles which may operate on a highway legally without special load permit.

For design purposes, the service loads are taken as the dead, live and impact loadings described in Section 1.2 (except Art. 1.2.4) of AASHO Specifications.

Overloads are the live loads that can be allowed on a structure on infrequent occasions without causing permanent damage. For design purposes the maximum overload is taken as $5/3\,(L + I)$ as specified in Article 1.7 of these Criteria.

The maximum loads are the loadings specified in Article 1.7.

1.4 DESIGN THEORY

The moments, shears and other forces shall be determined by assuming elastic behavior of the structure except as modified in Article 2.1.1A3.

The members shall be proportioned by the methods specified in Section 2 so that their computed maximum strengths shall be at least equal to the total effects of design loads multiplied by their respective load factors specified in Groups I, II and III of Article 1.7.

Service behavior shall be investigated as specified in Section 3.

1.5 ASSUMPTIONS

a) Strain in flexural members shall be assumed directly proportional to the distance from the neutral axis.

b) Stress in steel below the yield strength, F_y, of the grade of steel used shall be taken as 29,000,000 psi times the steel strain. For strain greater than that corresponding to the yield strength, F_y, the stress shall be considered independent of strain and equal to the yield strength, F_y. This assumption shall apply also to the longitudinal reinforcement in the concrete floor slab in the region of negative moment, when shear developers are provided to secure composite action in this region.

c) At maximum strength the compressive stress in the concrete slab of a composite beam shall be assumed independent of strain and equal to $0.85f'_c$.

d) Tensile strength of concrete shall be neglected in flexural calculations.

1.6 DESIGN STRENGTH FOR STEEL

The design strength for steel shall be the specified minimum yield point or yield strength, F_y, of the steel used as set forth in Article 1.7.1, AASHO Specifications.

1.7 MAXIMUM DESIGN LOADS

The maximum moments, shears or forces to be sustained by a stress-carrying steel member shall be computed from formulas 1.7-1 through 1.7-4. Members subject to combinations of loads and forces shall be designed for the combined effects.

$$\textit{Group I} = 1.25\left[D + \frac{5}{3}(L + I)\right] \qquad (1.7\text{-}1)$$

For all loadings less than $H20$, provision shall be made for an infrequent heavy load by applying Group IA loading, with the live load assumed to occupy a single lane without concurrent loading in any other lane.

$$\textit{Group IA} = 1.25\,[D + 2.2\,(L + I)] \qquad (1.7\text{-}2)$$

$$\textit{Group II} = 1.25\,[D + W + F + SF + B + S + T] \qquad (1.7\text{-}3)$$

When earthquake loading is taken into account, Equation 1.7-3 shall be used substituting EQ for W. When ice pressure is taken into account, Equation 1.7-3 shall be used substituting ICE for SF.

$$\textit{Group III} = 1.25\,[D + L + I + CF + 0.3W + WL + F + LF] \qquad (1.7\text{-}4)$$

The symbols in Equations 1.7-1 through 1.7-4 represent the moments, shears or forces caused by the loads and effects listed as follows and described in Section 1.2, AASHO, Specifications:

D = Dead load
L = Live load
I = Live load impact
W = Wind load on structure
WL = Wind load on live load
CF = Centrifugal force
LF = Longitudinal force due to live load
F = Longitudinal force due to friction
S = Shrinkage
T = Temperature
SF = Streamflow pressure
B = Buoyancy
ICE = Ice pressure
EQ = Earthquake

Section 2—Computation of Maximum Strength

2.1 BEAMS AND GIRDERS

2.1.1 Symmetrical Beams and Girders

A. *Compact Sections*

Symmetrical I-shaped beams with high resistance to local buckling and proper bracing to resist lateral torsional buckling qualify as compact sections. Compact sections are able to form plastic hinges which rotate at near constant moment.

Rolled or fabricated I-shaped beams meeting the requirements of paragraph 1 below shall be considered compact sections and the maximum strength shall be as computed:

$$M_u = F_y Z \qquad (2.1.1\text{-}1)$$

where F_y is the specified yield point of the steel being used,

Z is the plastic section modulus.*

1. Beams designed as compact sections shall meet the following requirements: (for certain frequently used steels these requirements are tabulated in Article 2.1.1A2).

a) Projecting flange element

$$b'/t \leqslant \frac{1600}{\sqrt{F_y}} \qquad (2.1.1\text{-}2)$$

where b' is the width of the projecting flange element,

t is the flange thickness.

b) Web thickness

$$d/t_w \leqslant \frac{13{,}300}{\sqrt{F_y}} \qquad (2.1.1\text{-}3)$$

where d is the depth of the beam,

t_w is the web thickness.

*See Commentary for method of computing Z. Values for rolled sections are listed in the "Manual of Steel Construction," Sixth Edition, 1963, American Institute of Steel Construction.

c) Lateral bracing

$$L/r_y \leqslant \frac{7000}{\sqrt{F_y}} \text{ when } M_2 \geqslant 0.7M_1 \qquad (2.1.1\text{-}4)$$

or

$$L/r_y \leqslant \frac{12{,}000}{\sqrt{F_y}} \text{ when } M_2 < 0.7M_1 \qquad (2.1.1\text{-}5)$$

where L is the distance between points of bracing of the compression flange,

r_y is the radius of gyration with respect to the Y-Y axis,

M_1 and M_2 are the moments at the two adjacent braced points.

In no case shall L exceed the value given by Equation 2.1.1-11.

The required lateral bracing shall be provided by braces capable of preventing lateral displacement and twisting of the main members or by embedment of the top and sides of the compression flange in concrete.

d) Maximum axial compression

$$P \leqslant 0.15\, F_y\, A \qquad (2.1.1\text{-}6)$$

where A is the area of the cross section.

e) Maximum shear force.

$$V \leqslant 0.55\, F_y\, d\, t_w \qquad (2.1.1\text{-}7)$$

2. Equation 2.1.1-1 is applicable to steels with stress-strain diagrams which exhibit a yield plateau followed by a strain hardening range.

Steels such as ASTM A36, A242, A440, A441, A572 and A588 meet these requirements. The limitations set forth in Article 2.1.1A1 are given in Table 1.

TABLE 1

F_y, psi	36,000	42,000	46,000	50,000	55,000
b'/t	8.4	7.8	7.5	7.2	6.8
d/t_w	70	65	62	59	57
L/r_y (eq. 2.1.1-4)	37	34	33	31	30
L/r_y (eq. 2.1.1-5)	63	59	56	54	51

3. In the design of a continuous beam of compact section complying with the provisions of Article 2.1.1A1, negative moments over supports determined by elastic analysis may be reduced by a maximum of 10%. Such reduction shall be accompanied by an increase in maximum positive moment in the span equal to the average decrease of the negative moments in the span. The reduction shall not apply to negative moments produced by cantilever loading.

B. *Braced Non-compact Sections*

For rolled or fabricated I-shaped beams not meeting the requirements of Article 2.1.1A1 but meeting the requirements of paragraph 1 below, the maximum strength shall be computed as:

$$M_u = F_y S \qquad (2.1.1\text{-}8)$$

where S is the section modulus.

1. Equation 2.1.1-8 is applicable to beams meeting the following requirements:

a) Projecting flange element

$$b'/t \leqslant 2200/\sqrt{F_y} \qquad (2.1.1\text{-}9)$$

When $M < M_u$, b'/t may be increased in the ratio $\sqrt{M_u/M}$

b) Web thickness

$$D/t_w \leqslant 150 \qquad (2.1.1\text{-}10)$$

in which D is the clear unsupported distance between flange components.

c) Spacing of lateral bracing for compression flange

$$L \leqslant \frac{20{,}000{,}000\, A_f}{F_y\, d} \qquad (2.1.1\text{-}11)$$

where d is the depth of beam or girder, A_f is the flange area.

d) Maximum axial compression

Axial compression shall not exceed the value given by Equation 2.1.1-6.

e) Maximum shear force

$$V \leqslant \frac{3.5\, E t_w^{\,3}}{D} \qquad (2.1.1\text{-}12)$$

but not more than $0.58\, F_y D t_w$

2. The limitations set forth in paragraph 1 are given in Table 2.

TABLE 2

F_y psi	36,000	42,000	46,000	50,000	55,000	90,000	100,000
b'/t	11.6	10.7	10.3	9.8	9.4	7.3	7.0
$\frac{Ld}{A_f}$	556	476	435	400	364	222	200

C. *Transition*

The maximum strength of members with geometric properties falling between the limits of Articles 2.1.1A and 2.1.1B may be computed by straight line interpolation, except that the web thickness must always satisfy Equation 2.1.1-3.

D. *Unbraced Sections*

1. For members not meeting the lateral bracing requirement of Equation 2.1.1-11, the maximum strength shall be computed as:

$$M_u = F_y S \left[1 - \frac{3F_y}{4\pi^2 E}\left(\frac{L}{b'}\right)^2\right] \qquad (2.1.1\text{-}13)$$

When the ratio of applied moments at the two ends of the braced length, L, is less than 0.7, the maximum strength, M_u, as computed by the above formula may be increased 20% but not to exceed $F_y S$.

2. In members not meeting the requirements of Article 2.1.1B1e the web shall be provided with transverse stiffeners as specified in Article 2.1.1E.

3. Members with axial loads in excess of $0.15\, F_y A$ should be designed as beam-columns as specified in Article 2.4.

E. *Transversely Stiffened Girders*

1. For girders not meeting the shear requirements of Equations 2.1.1-7 and 2.1.1-12, transverse stiffeners are required for the web.

For girders with transverse stiffeners but without longitudinal stiffeners the thickness of the web shall meet the requirement:

$$D/t_w \leqslant \frac{36{,}500}{\sqrt{F_y}} \qquad (2.1.1\text{-}14)$$

For different grades of steel this limit is:

D/t_w	for F_y (psi)
192	36,000
178	42,000
170	46,000
163	50,000
156	55,000
122	90,000
115	100,000

2. The maximum bending strength of transversely stiffened girders meeting the requirement of Equation 2.1.1-14 shall be computed by Equation 2.1.1-8 or 2.1.1-13 as applicable subject to the requirement of Equation 2.1.1-18.

3. The shear capacity of beams and girders with webs fulfilling the requirements of Equation 2.1.1-14 shall be computed as:

$$V_u = V_p \left[C + \frac{0.87\,(1 - C)}{\sqrt{1 + (d_o/D)^2}} \right] \qquad (2.1.1\text{-}15)$$

where:

$$V_p = 0.58\, F_y\, Dt_w \qquad (2.1.1\text{-}16)$$

$$C = 18{,}000\,(t_w/D)\sqrt{\frac{1 + (D/d_o)^2}{F_y}} - 0.3 \leqslant 1.0 \qquad (2.1.1\text{-}17)$$

D = clear, unsupported distance between flange components,
d_o = distance between transverse stiffeners.

If a girder panel is subjected to simultaneous action of shear and bending moment with the magnitude of the shear higher than 0.6 V_u, then the moment shall be limited to not more than:

$$M/M_u = 1.375 - 0.625\, V/V_u \qquad (2.1.1\text{-}18)$$

4. Transverse stiffeners shall be spaced at a distance, d_o, according to shear capacity as specified in Article 2.1.1E3, but not more than 1.5D. Transverse stiffeners may be omitted in those portions of the girders where the maximum shear force is less than the value given by Equation 2.1.1-12.

The first stiffener space at the ends of girders with simple supports shall not be greater than D nor:

$$d_o = 14{,}500\sqrt{Dt_w^3/V} \qquad (2.1.1\text{-}19)$$

The width-to-thickness ratio of transverse stiffeners shall be such that

$$b'/t \leqslant \frac{2{,}600}{\sqrt{F_y}} \qquad (2.1.1\text{-}20)$$

where b' is the projecting width of the stiffener.

The gross cross-sectional area of intermediate transverse stiffeners shall not be less than:

$$A = [0.15\, BDt_w (1-C)\,(V/V_u) - 18t_w^2]\ Y \qquad (2.1.1\text{-}21)$$

where Y is the ratio of web plate yield strength to stiffener plate yield strength
B = 1.0 for stiffener pairs,
1.8 for single angles,
2.4 for single plates.
C is computed by Equation 2.1.1-17.

The moment of inertia of transverse stiffeners with reference to the mid-plane of the web shall be not less than:

$$I = d_o\, t_w^3\, J \qquad (2.1.1\text{-}22)$$

where:

$J = 2.5\,(D/d_o)^2 - 2$, but not less than 0.5.

Transverse stiffeners need not be in bearing with the tension flange. The maximum distance between the stiffener-to-web connection and the face of the tension flange shall not be more than $4t_w$. Stiffeners provided on only one side of the web must be in bearing against but need not be attached to the compression flange.

F. *Longitudinally Stiffened Girders*

1. Longitudinal stiffeners shall be required when the web thickness is less than that specified by Equation 2.1.1-14 and shall be placed at a distance $D/5$ from the inner surface of the compression flange.

The web thickness of plate girders with transverse stiffeners and one longitudinal stiffener shall meet the requirement:

$$D/t_w \leqslant \frac{73{,}000}{\sqrt{F_y}} \qquad (2.1.1\text{-}23)$$

For different grades of steel, this limit is:

D/t_w	for F_y (psi)
385	36,000
356	42,000
340	46,000
326	50,000
311	55,000
243	90,000
231	100,000

2. The maximum bending strength of longitudinally stiffened girders meeting the requirements of Equation 2.1.1-23 shall be computed by Equation 2.1.1-8 or 2.1.1-13 as applicable subject to the requirement of Equation 2.1.1-18.

3. The shear capacity of girders with one longitudinal stiffener shall be computed by Equation 2.1.1-15.

The dimensions of the longitudinal stiffener shall be such that:

a) the width-to-thickness ratio is not greater than that given by Equation 2.1.1-20.

b) the rigidity of the stiffener is not less than:

$$I \geqslant Dt_w^3\ [2.4\,(\frac{d_o}{D})^2 - 0.13] \qquad (2.1.1\text{-}24)$$

c) the radius of gyration of the stiffener is not less than:

$$r \geqslant \frac{d_o\ \sqrt{F_y}}{23{,}000} \qquad (2.1.1\text{-}25)$$

In computing I and r values above, a centrally located web strip not more than $18t_w$ in width shall be considered as a part of the longitudinal stiffener.

Transverse stiffeners for girder panels with longitudinal stiffeners shall be designed according to Article 2.1.1E4, except that the depth of subpanels shall be used instead of the total panel depth, D. In addition, the section modulus of the transverse stiffener shall be not less than:

$$S_t = \frac{1}{3}\,(D/d_o)\ \ S_l \qquad (2.1.1\text{-}26)$$

where D is the total panel depth (clear distance between flange components) and S_l is the section modulus of the longitudinal stiffener at $D/5$.

2.1.2 Unsymmetrical Beams and Girders

A. *General*

For beams and girders symmetrical about the vertical axis of the cross section but unsymmetrical with respect to the horizontal centroidal axis, the provisions of Articles 2.1.1A through 2.1.1D shall be applicable except that in computing the maximum strength by Equation 2.1.1-13 the term b' is replaced by $0.9b'$.

B. *Unsymmetrical Sections with Transverse Stiffeners*

Girders with transverse stiffeners shall be designed and evaluated by the provisions of Article 2.1.E except that when D_c, the clear distance between the neutral axis and the compression flange, exceeds $D/2$ the web thickness, t_w, shall meet the requirement:

$$\frac{D_c}{t_w} \leqslant \frac{18{,}250}{\sqrt{F_y}} \qquad (2.1.2\text{-}1)$$

C. *Longitudinally Stiffened Unsymmetrical Sections*

Longitudinal stiffeners shall be required on unsymmetrical sections when the web thickness is less than that specified by Equation 2.1.1-14 or 2.1.2-1.

For girders with one longitudinal stiffener and transverse stiffeners, the provisions of Article 2.1.1F for symmetrical sections shall be applicable provided that:

a) the longitudinal stiffener is placed $2D_c/5$ from the inner surface or the leg of the compression flange element.

b) When D_c exceeds $D/2$, the web thickness, t_w, shall meet the requirement:

$$\frac{D_c}{t_w} \leqslant \frac{36{,}500}{\sqrt{F_y}} \qquad (2.1.2\text{-}2)$$

2.2 COMPOSITE BEAMS AND GIRDERS

2.2.1 General

Composite beams shall be so proportioned that the following criteria are satisfied:

a) The maximum strength of any section shall not be less than the sum of the computed moments at that section multiplied by the appropriate load factors.

b) The web of the steel section shall be designed to carry the total external shear and must satisfy the applicable provisions of Articles 2.1.1 and 2.1.2. In such application the value of D_c shall be taken as the clear distance between the neutral axis of the composite section for live loads and the compression flange.

2.2.2 Positive Moment Sections

A. *Compact Sections*

When the steel section satisfies the compactness requirements of paragraph 2 below, the maximum strength shall be computed as the resultant moment of the fully plastic stress distribution acting on the section (Figure 1).

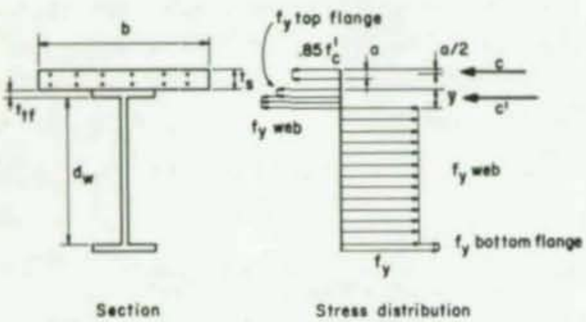


Fig. 1

1. The resultant moment of the fully plastic stress distribution may be computed as follows:

a) the compressive force in the slab is equal to the smallest of the values given by the Equations 2.2.2-1, 2.2.2-2 and 2.2.2-3.

$$C = 0.85\, f'_c\, bt_s + (A\, F_y)_c \qquad (2.2.2\text{-}1)$$

where b is the effective width of slab,

t_s is the slab thickness,

$(A\, F_y)_c$ is the product of the area and yield point of that part of reinforcement which lies in the compression zone of the slab.

$$C = (A\, F_y)_{bf} + (A\, F_y)_{tf} + (A\, F_y)_w \qquad (2.2.2\text{-}2)$$

where $(A\ F_y)_{bf}$ is the product of area and yield point for bottom flange of steel section (including cover plate if any),

$(A\, F_y)_{tf}$ is the product of area and yield point for top flange of steel section,

$(A\, F_y)_w$ is the product of area and yield point for web of steel section.

$$C = \Sigma Q_u \qquad (2.2.2\text{-}3)$$

where ΣQ_u is the sum of ultimate strengths of shear connectors between the section under consideration and the section of zero moment.

b) the depth of the stress block is computed from the compressive force in the slab.

$$a = \frac{C - (A\, F_y)_c}{0.85\, f'_c\, b} \qquad (2.2.2\text{-}4)$$

c) when the compressive force in the slab is less than the value given by Equation 2.2.2-2, the top portion of the steel section will be subjected to the following compressive force:

$$C' = \frac{\Sigma\,(A\, F_y) - C}{2} \qquad (2.2.2\text{-}5)$$

d) the location of the neutral axis within the steel section measured from the top of the steel section may be determined as follows:

for $C' < (A\, F_y)_{tf}$

$$\bar{y} = \frac{C'}{(A\, F_y)_{tf}}\, t_{tf} \qquad (2.2.2\text{-}6)$$

for $C' \geqslant (A\, F_y)_{tf}$

$$\bar{y} = t_{tf} + \frac{C' - (A\, F_y)_{tf}}{(A\, F_y)_w}\, d_w \qquad (2.2.2\text{-}7)$$

e) the maximum strength of the section in bending is the first moment of all forces about the neutral axis, taking all forces and moment arms as positive quantities.

2. Composite beams qualify as compact when their steel section meets the requirements of Equations 2.1.1-3 and 2.1.1-7, and the stress-strain diagram of the steel exhibits a yield plateau followed by a strain hardening range.

B. *Non-compact Sections*

When the steel section does not satisfy the compactness requirements of Article 2.2.2A2, the maximum strength of the section shall be taken as the moment at first yielding.

Maximum compressive and tensile stresses in girders which are not provided with temporary supports during the placing of dead loads shall be the sum of the stresses produced by 1.25 D_s acting on the steel girder alone and the stresses produced by $1.25\ [D_c + 5/3\ (L + I)]$ acting on the composite girder, where D_s and D_c are the moment caused by the dead load acting on the steel girder and composite girder, respectively.

When the girders are provided with effective intermediate supports which are kept in place until the concrete has attained 75% of its required 28-day strength, stresses are produced by the loading, $1.25\ [D + 5/3\ (L + I)]$, acting on the composite girder.

2.2.3 Negative Moment Sections

The maximum strength of beams and girders in the negative moment regions shall be computed in accordance with Articles 2.1.1 and 2.1.2, as applicable. It shall be assumed that the slab concrete does not carry tensile stresses. In cases where the slab reinforcement is continuous over interior supports, the reinforcement may be considered to act compositely with the steel section.

2.2.4 Box Girders

This section pertains to the design of simple and continuous bridges of moderate length supported by two or more single-cell composite box girders. It is applicable to box girders, having width center-to-center of top steel flanges approximately equal to the distance center-to-center of adjacent top steel flanges of adjacent box girders. The cantilever overhang of the deck slab, including curbs and parapet, shall be limited to 60 percent of the distance between the centers of adjacent top steel flanges of adjacent box girders, but in no case greater than 6 feet.

A. *Maximum Strength*

The maximum strength of box girders shall be determined according to the applicable provisions of Articles 2.2.1, 2.2.2 and 2.2.3. In addition, the maximum strength of the negative moment sections shall be limited by

$$M_u = F_{cr} S \qquad (2.2.4\text{-}1)$$

where F_{cr} is the buckling stress of the bottom flange plate as given in Article 2.2.4E.

B. *Lateral Distribution*

The live load bending moment for each box girder shall be determined by applying to the girder the fraction W_L of a wheel load (both front and rear) determined by the following equation:

$$W_L = 0.1 + 1.7R + \frac{0.85}{N_w} \qquad (2.2.4\text{-}2)$$

where $R = \dfrac{N_w}{\text{Number of Box Girders}}$

$N_w = W_c/12$, reduced to the nearest whole number,

W_c = roadway width between curbs or barriers (in feet).

R shall be not less than 0.5 nor greater than 1.5.

C. *Web Plates*

The design shear V_w for a web shall be calculated using the following equation:

$$V_w = V/\cos\theta \qquad (2.2.4\text{-}3)$$

where V = one half of the total vertical shear force on one box girder,
θ = angle of inclination of the web plate to the vertical.

The inclination of the web plates to the vertical shall not exceed 1 to 4.

D. *Tension Flanges*

In the case of simply supported spans, the bottom flange shall be considered fully effective in resisting bending if its width does not exceed one-fifth the span length. If the flange plate width exceeds one-fifth of the span, only an amount equal to one-fifth of the span shall be considered effective.

For continuous spans, the requirements above shall be applied to the distance between points of contraflexure.

E. Compression Flanges

1. Unstiffened compression flanges designed for the yield stress, F_y, shall have a width-to-thickness ratio equal to or less than the value obtained from the formula:

$$b/t = \frac{6140}{\sqrt{F_y}} \qquad (2.2.4\text{-}4)$$

where b = flange width between webs in inches,
t = flange thickness in inches.

For greater b/t ratios, but not exceeding $13{,}300/\sqrt{F_y}$, the buckling stress of an unstiffened bottom flange is given by the formula:

$$F_{cr} = 0.592F_y\ (1 + 0.687 \sin \frac{c\pi}{2}) \quad (2.2.4\text{-}5)$$

in which c shall be taken as

$$c = \frac{13{,}300 - \left(\frac{b}{t}\right)\sqrt{F_y}}{7160} \qquad (2.2.4\text{-}6)$$

For values of b/t exceeding $13{,}300/\sqrt{F_y}$, the buckling stress of the flange is given by the formula:

$$F_{cr} = 105\ (t/b)^2 \times 10^6 \qquad (2.2.4\text{-}7)$$

2. If longitudinal stiffeners are used, they shall be equally spaced across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener is at least equal to:

$$I_s = \phi t^3 w \qquad (2.2.4\text{-}8)$$

where $\phi = 0.07k^3n^4$ when n equals 2,3,4 or 5.
$\phi = 0.125k^3$ when $n = 1$.
w= width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener,
n = number of longitudinal stiffeners,
k = buckling coefficient which shall not exceed 4.

For a longitudinally stiffened flange designed for the yield stress, F_y, the ratio w/t shall not exceed the value given by the formula

$$w/t = \frac{3070\sqrt{k}}{\sqrt{F_y}} \qquad (2.2.4\text{-}9)$$

For greater values of w/t, but not exceeding $6650\sqrt{k}/\sqrt{F_y}$, the buckling stress of the flange, including stiffeners is given by formula 2.2.4-5 in which c shall be taken as

$$c = \frac{6650\sqrt{k} - (w/t)\sqrt{F_y}}{3580\sqrt{k}} \qquad (2.2.4\text{-}10)$$

For values of w/t exceeding $6650\sqrt{k}/\sqrt{F_y}$ the buckling stress of the flange, including stiffeners, is given by the formula:

$$F_{cr} = 26.2\,k\ (t/w)^2 \times 10^6 \qquad (2.2.4\text{-}11)$$

When longitudinal stiffeners are used, it is preferable to have at least one transverse stiffener placed near the point of dead load contraflexure. The stiffener should have a size equal to that of a longitudinal stiffener.

3. The width-to-thickness ratio of any outstanding element of the flange stiffeners shall not exceed the value determined by the formula:

$$b'/t' = \frac{2600}{\sqrt{F_y}} \qquad (2.2.4\text{-}12)$$

where b' = width of any outstanding stiffener element,
t' = thickness of outstanding stiffener element.

F. *Diaphragms*

Diaphragms, cross-frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement and distortion.

Intermediate diaphragms or cross-frames are not required for box girder bridges designed in accordance with this specification.

2.2.5 Shear Connectors

A. *General*

The horizontal shear at the interface between the concrete slab and the steel girder shall be provided for by mechanical shear connectors throughout the simple spans and the positive moment regions of continuous spans. In the negative moment regions shear connectors shall be provided when the reinforcement steel imbedded in the concrete is considered a part of the composite section. In case the reinforcement steel imbedded in the concrete is not considered in computing section properties of negative moment sections, shear connectors need not be provided in these portions of the span, but additional connectors shall be placed in the region of the points of dead load contraflexure as specified in Art. 3.2.2C.

B. *Design of Connectors*

The number of shear connectors required between the points of maximum positive moment and the end supports or dead load points of contraflexure, or between points of maximum negative moment and the dead load points of contraflexure, shall be equal to or exceed the number given by:

$$N = \frac{C}{0.85\, Q_u} \qquad (2.2.5\text{-}1)$$

where C is the force in the slab as defined below,
Q_u is the maximum strength of an individual shear connector in pounds.

1. At points of maximum positive moment, the force in the slab is taken as the smaller of the values given by formulas 2.2.2-1 or 2.2.2-2.

2. At points of maximum negative moment the force in the slab is taken as:

$$C = A_r\, F_{yr} \qquad (2.2.5\text{-}2)$$

where A_r is the area of longitudinal reinforcing steel at the interior support within the effective flange width,
F_{yr} is the specified yield strength of the reinforcing steel.

3. The maximum connector strengths are as follows:

Channels

$$Q_u = 550\,(h + t/2)\, w \sqrt{f'_c} \qquad (2.2.5\text{-}3)$$

Welded Studs $(H_s/d_s \geqslant 4)$

$$Q_u = 930\; d_s^2\; \sqrt{f'_c} \qquad (2.2.5\text{-}4)$$

where

h is the average thickness of the channel flange (in.),
t is the thickness of the channel web (in.),
w is the length of a channel shear connector (in.),
H_s is the height of a stud (in.),
d_s is the diameter of the stud (in.).

C. *Maximum Spacing*

The maximum pitch shall not exceed 24 inches except over the interior supports of continuous beams where wider spacing may be used to avoid placing connectors at locations of high stresses in the tension flange.

2.3 HYBRID BEAMS AND GIRDERS

2.3.1 General

This section pertains to the design of (1) noncomposite beams and girders that have flanges of the same minimum specified yield strength and a web with a lower minimum specified yield strength, and (2) composite girders that have a tension flange with a higher minimum specified yield strength than the web and a compression flange with a minimum specified yield strength not less than that of the web. It is applicable to both simple and continuous girders. In noncomposite girders and in the negative moment portion of continuous composite girders, the area of the compression-flange shall be equal to the area of the tension flange or larger than the area of the tension-flange by an amount not exceeding 25 percent. In composite girders, excluding the negative moment portion in continuous girders, the area of the compression flange shall be equal to or smaller than the area of the tension flange. The minimum specified yield strength of the web shall not be less than 35 percent of the minimum specified yield strength of the tension flange.

The provisions of Articles 2.1 and 2.2 shall apply to hybrid beams and girders except as modified below.

In all equations of Articles 2.1 and 2.2, F_y shall be taken as the minimum specified yield strength of the steel of the element under consideration.

2.3.2 Noncomposite Girders

A. *Compact Sections*

Equation 2.1.1-1 for the maximum strength of compact sections shall be replaced by the expression

$$M_u = F_{yf} Z \qquad (2.3.2\text{-}1)$$

where F_{yf} is the specified minimum yield strength of the flange and Z is the plastic section modulus.

In computing Z, the web thickness shall be multiplied by the ratio of the minimum specified yield strength of the web, F_{yw}, to the minimum specified yield strength F_{yf}.

B. *Braced Non-compact Sections*

Equation 2.1.1-8 for the maximum strength shall be replaced by the expression

$$M_u = F_{yf} SR \qquad (2.3.2\text{-}2)$$

For symmetrical sections,

$$R = \frac{12 + \beta\,(3\rho - \rho^3)}{12 + \beta} \qquad (2.3.2\text{-}3)$$

where

$$\rho = F_{yw}/F_{yf}$$

$$\beta = A_w/A_f$$

For unsymmetrical sections,

$$R = 1 - \frac{\beta\psi\,(1-\rho)^2\,(3 - \psi + \rho\psi)}{6 + \beta\psi\,(3 - \psi)} \qquad (2.3.2\text{-}4)$$

where ψ is the distance from the outer fiber of the tension flange to the neutral axis divided by the depth of the steel section.

C. *Unbraced Noncompact Sections*

Equation 2.1.1-13 for the maximum strength of unbraced noncompact sections shall be replaced by the expression

$$M_u = F_{yf} S\left[1 - \frac{3F_{yf}}{4\pi^2 E}\left(\frac{L}{b'}\right)^2\right] R \qquad (2.3.2\text{-}5)$$

where R is given by Equation 2.3.2-3 or 2.3.2-4.

D. *Transversely Stiffened Girders*

Equation 2.1.1-15 for the shear capacity of transversely stiffened girders shall be replaced by the expression

$$V_u = V_p C \qquad (2.3.2\text{-}6)$$

Equation 2.1.1-21 is not applicable to hybrid girders.

2.3.3 Composite Girders

The maximum strength of the composite section shall be the moment at first yielding of the flanges times R from Equation 2.3.2-4, in which ψ is the distance from the outer fiber of the neutral axis of the transformed section divided by the depth of the steel section.

2.4 COMPRESSION MEMBERS

2.4.1 Axial Loading

A. *Maximum Capacity*

The maximum strength of concentrically loaded columns shall be computed as:

$$P_u = A_s F_{cr} \qquad (2.4.1\text{-}1)$$

where A_s is the gross effective area of the column cross section and F_{cr} is determined by one of the following two formulas:

$$F_{cr} = F_y \left[1 - \frac{F_y}{4\pi^2 E}\left(\frac{KL}{r}\right)^2\right] \qquad (2.4.1\text{-}2)$$

for $\frac{KL}{r}$ less than or equal to $\sqrt{\frac{2\pi^2 E}{F_y}}$

$$F_{cr} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad (2.4.1\text{-}3)$$

for $\frac{KL}{r}$ more than $\sqrt{\frac{2\pi^2 E}{F_y}}$

where
- K is effective length factor in the plane of buckling,
- L is length of the member between points of support, in inches,
- r is radius of gyration in the plane of buckling, in inches,
- F_y is yield stress of the steel, in psi,
- E is 29,000 psi,
- F_{cr} is buckling stress, in psi.

B. *Effective Length*

The effective length factor K shall be determined as follows:

1. For members having lateral support in both directions at its ends:
   $K = 0.75$ for riveted, bolted or welded end connections,
   $K = 0.875$ for pinned ends.

2. For members having ends not fully supported laterally by diagonal bracing or an attachment to an adjacent structure, the effective length factor shall be determined by a rational procedure.*

2.4.2 Combined Axial Load and Bending

A. *Maximum Capacity*

The combined maximum axial force P and the maximum bending moment M acting on a beam-column subjected to eccentric loading shall satisfy the following equations:

$$\frac{P}{A_s F_{cr}} + \frac{MC}{M_u\left(1 - \frac{P}{A_s F_e}\right)} \leqslant 1.0 \qquad (2.4.2\text{-}1)$$

$$\frac{P}{A_s F_y} + \frac{M}{M_p} \leqslant 1.0 \qquad (2.4.2\text{-}2)$$

where
- F_{cr} is buckling stress as determined by Equations 2.4.1-2 or 2.4.1-3,
- M_u is the maximum strength as determined by Equations 2.1.1-1, 2.1.1-8 or 2.1.1-13,
- $F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$, the Euler buckling stress in the plane of bending,
- C is the equivalent moment factor,
- $M_p = F_y Z$, the full plastic moment of the section,
- Z is the plastic section modulus,
- $\frac{KL}{r}$ is the effective slenderness ratio in the plane of bending.

B. *Equivalent Moment Factor*

If the ends of the beam-column are restrained from sidesway in the plane of bending by diagonal bracing or attachment to an adjacent laterally braced structure, then the value of equivalent moment factor, C, may be computed by the formula:

$C = 0.6 + 0.4\,a$ but not less than 0.4

*B.G. Johnston, "Guide to Design Criteria for Metal Compression Members," John Wiley and Sons, Inc., New York, 1966.

where α is the ratio of the numerically smaller to the larger end moment. The ratio α is positive when the two end moments act in an opposing sense (i.e., one acts clockwise and the other acts counterclockwise) and negative when they act in the same sense. In all cases, factor C may be taken conservatively as unity.

2.5 SPLICES, CONNECTIONS & DETAILS

2.5.1 Connectors

A. *General*

Connectors shall be proportioned so that their maximum strength multiplied by the reduction factor, ϕ, shall be at least equal to the effects of design loads multiplied by their respective load factors specified in Article 1.7. The maximum strengths multiplied by the reduction factors are listed in Table 3.

B. *Welds*

The ultimate strength of weld metal in groove welds shall be equal to or greater than that of the base metal. The ultimate strength of the weld metal in fillet welds need not match the strength of the base metal. However, the welding procedure and weld metal shall be selected to insure sound welds. The effective weld area shall be taken as defined in Article 1.7.29, AASHO Specifications.

C. *Bolts and Rivets*

In proportioning fasteners, the nominal diameter shall be used except when a shear plane intersects the threads.

High-strength bolts preferably shall be used for fasteners subject to tension or combined shear and tension.

For combined tensions and shear in bearing type connections, bolts and rivets shall be proportioned so that the shear stress does not exceed:

$$F_{vc} \leqslant \sqrt{F_v^{\,2} - (0.6f_t)^2} \qquad (2.5.1\text{-}1)$$

where F_v = shear strength of the fastener, ϕF, as given in Table 3,

f_t = tensile stress due to the applied load.

TABLE 3

Type of Fastener	Strength (ϕF)
Groove Weld(1)	$1.00\,F_y$
Fillet Weld(2)	$0.45\,F_u$
Low-Carbon Steel Bolts	
ASTM A307	
Tension	27 ksi
Shear(3)	25 ksi
Power-Driven Rivets	
ASTM A502	
Shear – Grade 1	25 ksi
Shear – Grade 2	30 ksi
High-Strength Bolts	
ASTM A325	
Tension(5)	76 ksi
Shear (Bearing-Type)(3)(4)(5)	54 ksi
ASTM A490	
Tension	84 ksi
Shear (Bearing-Type)(3)(4)	67 ksi

(1) – F_y = yield point of connected material.

(2) – F_u = minimum strength of the welding rod metal but not greater than the tensile strength of the connected parts.

(3) – When a shear plane intersects the bolt threads, the root area shall be used.

(4) – Bearing stresses in bearing-type connections shall not exceed the tensile strength of the connected material.

(5) – For A325 bolts the tensile strength decreases for diameters greater than 7/8 in. The design value listed is for bolts up to 7/8 in. diameter. For diameters greater than 7/8 in. diameter the design value shall be computed as $0.56\,F_u$ for tension and $0.45\,F_u$ for shear where F_u is the ASTM minimum tensile strength of the bolt.

D. *Friction Joints*

Friction joints shall be designed to prevent slip at the overload in accordance with Article 3.1.3. Maximum strength of the bolts need not be considered in the design of such joints.

2.5.2 Connections

A. *Splices*

Splices may be made with rivets, with high-strength bolts or by the use of welding. Splices, whether in tensions, compression, bending or shear, shall be designed for not less than the average of the calculated stress resultant at the point of the splice and the strength of the member at the same point but in any event not less than 75% of the maximum strength of the member. Where a section changes at a splice, the maximum strength of the splice shall be at least 75% of the smaller section spliced.

The maximum strength of the member shall be determined by the gross section for compression members. For members primarily in bending, the gross section shall be used except that if more than 15% of each flange area is removed, that amount removed in excess of 15% shall be deducted. For tension members and splice material, the gross section shall be used unless the net section area is less than 85% of the corresponding gross area, in which case that amount removed in excess of 15% shall be deducted.

B. *Bolts Subjected to Prying Action by Connected Parts*

Bolts required to support applied load by means of direct tension shall be proportioned for the sum of the external load and tension resulting from prying action produced by deformation of the connected parts. The total tension should not exceed the values given in Table 3 of Article 2.5.1A.

The tension due to prying action shall be computed as:

$$Q = \left[\frac{3b}{8a} - \frac{t^3}{20}\right] T \qquad (2.5.2\text{-}1)$$

where

- Q = the prying force per bolt (taken as zero when negative),
- T = the direct tension per bolt due to external load,
- a = distance from center of bolt to edge of plate,
- b = distance from center of bolt to toe of fillet of connected part,
- t = thickness of thinnest part connected, in.

C. *Rigid Connections*

All rigid frame connections, the rigidity of which is essential to the continuity assumed as the basis of design, shall be capable of resisting the moments, shears, and axial loads to which they are subjected by maximum loads.

The beam web shall equal or exceed the thickness given by:

$$t_w \geqslant \sqrt{3}\left(\frac{M_c}{F_y\, d_b\, d_c}\right) \qquad (2.5.2\text{-}2)$$

where

M is the column moment,
d_b the beam depth,
d_c the column depth.

When the thickness of the connection web is less than that given by the above formula, the web shall be strengthened by diagonal stiffeners or by a reinforcing plate in contact with the web over the connection area.

At joints where the flanges of one member are rigidly framed into one flange of another member, the thickness of the web supporting the latter flange shall be checked by formula 2.5.2-3 and the thickness of the latter flange shall be checked by formula 2.5.2-4. Stiffeners are required on the web of the second member opposite the compression flange of the first member when

$$t_w < \frac{A_f}{t_b + 5k} \qquad (2.5.2\text{-}3)$$

and opposite the tension flange of the first member when

$$t_c < 0.4\sqrt{A_f} \qquad (2.5.2\text{-}4)$$

where

- t_w = thickness of web to be stiffened,
- k = distance from outer face of flange to toe of web fillet of member to be stiffened,
- t_b = thickness of flange delivering concentrated force,
- t_c = thickness of flange of member to be stiffened,
- A_f = area of flange delivering concentrated load.

Section 3—Service Behavior

3.1 OVERLOAD

3.1.1 Noncomposite Beams

For noncomposite beams designed under the provisions of Article 2.1.1, the moment caused by $D + \frac{5}{3}(L+I)$ shall not exceed 0.8 F_yS. For such beams designed for Group IA loading, the moment caused by D+2.2 $(L+I)$ shall not exceed $0.8F_yS$. In the case of moment redistribution under the provisions of Article 2.1.1A3, the above limitation shall apply to the modified moments but not to the original moments.

3.1.2 Composite Beams

For composite beams designed under the provisions of Article 2.2.2A, the moment caused by $D + \frac{5}{3}(L+I)$ shall not exceed 95% of the moment at first yielding in the section. For such beams designed for Group IA loading, the moment caused by D+2.2 $(L+I)$ shall not exceed 95% of the moment at first yielding in the section. In computing dead load stresses the presence or absence of temporary supports during the construction shall be considered.

3.1.3 Friction Joints

The shear caused by the loading, $D + \frac{5}{3}(L+I)$, in friction-type high-strength bolted joints shall not exceed 21,000 psi for ASTM 325 bolts nor 28,000 psi for ASTM A490 bolts.

For combined shear and tension in friction-type joints where applied forces reduce the total clamping force on the friction plane, the maximum shear stress shall not exceed the values obtained from the following equations:

For A325

$$f_v = 21{,}000\,[1 - f_t/0.53F_u)] \qquad (3.1.3\text{-}1)$$

For A490

$$f_v = 28{,}000\,[1 - f_t/(0.53\,F_u)] \qquad (3.1.3\text{-}2)$$

where F_u is the tensile strength of the bolt,
f_t is the applied tensile stress.

3.2 FATIGUE

3.2.1 General

The analysis of the probability of fatigue of steel members or connections under working loads and the allowable fatigue stresses, F_r, shall conform to Article 1.7.3, AASHO Specifications except that the limitation imposed by the basic allowable stresses given in Articles 1.7.1 and 1.7.2, AASHO Specifications, shall not apply.

3.2.2 Composite Construction

A. *Slab Reinforcement*

When composite action is provided in the negative moment region, the range of stress in slab reinforcement shall be limited to 20,000 psi.

B. *Shear Connectors*

The shear connectors shall be designed for fatigue* as follows:

1.
$$S_r = \frac{V_r Q}{I} \qquad (3.2.2\text{-}1)$$

where

S_r = the range of horizontal shear per linear inch at the junction of the slab and girder at the point in the span under consideration.

V_r = the range of shear due to live loads and impact. At any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads),

*"Fatigue Strength of Shear Connectors," by R.G. Slutter and J.W. Fisher, American Iron and Steel Institute, Bulletin No. 5, October, 1967.

Q = the statical moment of the transformed compressive concrete area about the neutral axis of the composite section or the statical moment of the area of reinforcement embedded in the concrete for negative moment,

I = the moment of inertia of the transformed composite girder in positive moment regions and the moment of inertia provided by the steel beam and the area of reinforcement embedded in the concrete in the negative moment regions.

2. Allowable design range of load Z_r in pounds on individual shear connectors is as follows:

Channels

$$Z_r = Bw \tag{3.2.2-2}$$

Welded studs (for ratios, H_s/d_s equal or greater than 4)

$$Z_r = \alpha d_s^2 \tag{3.2.2-3}$$

In the above, the following notations apply:

w = the length of a channel shear connector in inches measured in a transverse direction on the flange of a girder,
d_s = diameter of studs
α = 13,000 for 100,000 cycles,
10,600 for 500,000 cycles,
7,850 for 2,000,000 cycles,

B = 4,000 for 100,000 cycles,
3,000 for 500,000 cycles,
2,400 for 2,000,000 cycles,

H = height of stud, in inches.

3. The required pitch of shear connectors is determined by dividing the resistance of all connectors at one transverse girder cross section (Z_r) by the horizontal range of shear S_r per linear inch. Over the interior supports of continuous beams the pitch may be modified to avoid placing the connectors at locations of high stresses in the tension flange provided that the total number of connectors remains unchanged.

C. *Anchorage*

When reinforcement steel embedded in the concrete is not used in computing composite section properties for negative moments, the number of additional connectors required at points of contraflexure shall be computed by the formula:

$$N_c = \frac{A_r f_r}{Z_r} \tag{3.2.2-4}$$

in which

N_c = number of additional connectors at the point of contraflexure,
A_r = area of longitudinal reinforcing steel at the interior support within the effective flange width,
f_r = range of stress due to live load plus impact, in the slab reinforcement over the support (in lieu of more accurate computations, f_r may be taken as equal to 10,000 psi),
Z_r = the allowable design range of load on an individual shear connector.

The additional connectors, N_c, shall be placed adjacent to the point of contraflexure within a distance equal to 1/3 the effective slab width.

3.2.3 Hybrid Beams and Girders

Hybrid girders shall be designed for fatigue as if they were homogeneous girders of the flange steel, except that the allowable fatigue stresses for web splices and for attachments to the web shall be based on the web steel.

3.3 DEFLECTION

The control of deflection of steel or of composite steel and concrete structures shall conform to the provisions of Article 1.7.13, AASHO Specifications.

COMMENTARY

Contents of Commentary

PREFACE

The purpose of a commentary on a specification or set of criteria is to clarify, amplify and support those provisions which may appear to require such supporting data. It may cite references to research or analysis from which the provisions were developed, show the need for and the suitability of the provisions anticipated and clear up questions as to their application and assist the designer generally in understanding the provisions.

Such support is helpful to a new designer using a well-established specification and to an experienced designer faced with an unusual application of a familiar specification. For both it can assist in forming a balanced and sound judgment leading to an adequate and economical design.

A commentary is especially needed when new design criteria are set up involving unfamiliar concepts and procedures. In such cases, the designer has not developed, through experience, an appreciation of the range of application of the provision and its influence on the form and strength of the structure. This situation characterizes many of the provisions of these Criteria for Load Factor Design of Steel Highway Bridges, and the intent has been to supply this expository support.

Nomenclature

A =area of cross section (in.^{2})
A_b =nominal bolt area (in.^{2})
A_f =area of one flange of beam or girder (in.^{2})
A_g =gross section area (in.^{2})
A_n =net section area (in.^{2})
A_w =area of web of beam (in.^{2})
a =distance from center of bolt to edge of plate (in.)
a =a distance between centroids of areas
a =a distance between box girders
B =a coefficient
b =distance from center of bolt to center of fillet of connected part (in.)
b' =width of projecting flange element (in.)
C =buckling coefficient
c =cantilever overhand of slot in box girders
D =clear unsupported distance between flange components (in.)
D =dead load
D_c =clear distance between the neutral axis and the compression flange of an unsymmetrical section (in.)
d =depth of member (in.)
d_f =depth of Section centerline flanges
d_o =distance between transverse stiffeners (in.)
E =modulus of elasticity (29,000,000 psi)
E_{st} =strain hardening modulus
F =a stress (psi)
F_{cr} =critical buckling stress
F_s =vertical component of tension field force
F_u =specified minimum tensile strength (psi)
F_v =maximum allowable shear stress, bearing-type bolts (psi)
F_y =specified minimum yield point or yield strength of the type of steel being used (psi)
f_t =axial tensile stress in bolt due to applied load (psi)
G =modulus of elasticity in shear (11,200,000 psi)
G_{st} =strain-hardening modulus in shear (psi)
g =gage of the fasteners (in.)
I =impact
I =moment of inertia (in.^{4})
I_f =moment of inertia of effective compression flange column
I_s =moment of inertia of a longitudinal stiffener (in.^{4})
I_t =moment of inertia of a transverse stiffener (in.^{4})
K =effective length factor
k =buckling coefficient depending on boundary conditions
L =distance between points of bracing of compression flange (in.)
L =live load
M, M_1, M_2 =moment on a cross section (in.-lb)
M_p =full plastic moment capacity (in.-lb)
M_{pn} =plastic moment capacity of net section (in.-lb)
M_u =maximum moment capacity (in.-lb)
N_w =number of live load lanes on bridge
P_u =maximum axial compression capacity (lb)
Q =prying force per bolt (lb)
Q =statical moment of transformed compressive concrete area about the neutral axis of the composite section or the statical moment of the area of reinforcement imbedded in the concrete for negative moment (in.^{3})
R =number of live load lanes per box girder
R_s =ratio of section modulus required by load factor design to that required by working stress design
r =radius of gyration (in.)
r_y =radius of gyration with respect to Y-Y axis (in.)
r' =radius of gyration of compression flange about its vertical axis (in.)
S =section modulus (in.^{3})

T =direct tension per bolt due to external load (lb)

T_i =initial bolt tension

t =flange thickness (in.)

t_w =web thickness (in.)

V =shear force on the cross section (lb)

V_u =maximum shear capacity (lb)

V_w =design shear on web of box girder

W_L =fraction of a wheel load applied to one box girder

W_m =max. load per girder

Y =ratio $\frac{\text{web plate yield strength}}{\text{stiff. plate yield strength}}$

Z =Plastic Section Modulus (in.^{3})

θ =angle of inclination of web plate to the vertical (box girder)

υ =Poisson's ratio (0.3)

γ =coefficients – Max. Design Loads

α =coefficients – Max. Design Loads

β =coefficients – Max. Design Loads

λ =buckling coefficient

Section 1—General Provisions

1.4 DESIGN THEORY

Depending on their proportions, the maximum strength of steel structural members is limited by elastic buckling,

$$M_u = FS$$

inelastic buckling at nominal first yielding,

$$M_u = F_y S$$

or fully plastic state of stress

$$M_u = F_y Z.$$

In the first two cases the stress distribution throughout the structure is nominally elastic so that the internal moments, shears and other forces are accurately determined by assuming elastic behavior.

When a fully plastic state of stress governs, the structure undergoes an internal redistribution of forces before reaching its maximum strength so that the analysis of the structure based on elastic behavior is no longer valid. As long as all members are compact, the structure is transformed into a mechanism and the corresponding internal forces can be computed by the techniques of plastic design.

Unfortunately, as of the writing of these criteria a number of questions remains unanswered regarding the application of plastic design to bridges. For example, the load distribution factors in current use apply only to elastic stress conditions; the knowledge of the lateral load distribution after yielding is incomplete.

Accordingly, Section 1.4 stipulates that the internal forces shall be computed on the basis of elastic behavior in all cases but recognizes some of the higher strength indicated by plastic design studies by permitting a moderate reduction in negative moments (Article 2.1.1A3) when compact sections are used.

The dimensioning is accomplished on the basis of maximum strength. However, this does not always assure a satisfactory performance at service loads. Thus, a number of separate checks at service loads are prescribed in Section 3.

1.7 MAXIMUM DESIGN LOADS

In load factor design the maximum strength of a member is equated to the strength required to resist the various forces to which the member will be subjected. The maximum strength is decreased and the load effects are increased by suitable factors intended to offset uncertainties in their magnitude and application. This can be expressed by the following equation:

$$\phi \times \text{maximum strength} = \gamma\,[\alpha D + \beta\,(L + I)]$$

in which ϕ allows for uncertainty as to the strength of a section,
γ allows for uncertainty concerning the load analysis and other overall effects,
α allows for possible increase in dead load
β allows for overload.

Uncertainties covered by factor ϕ may be listed:

1) uncertainty as to the analysis and in the calculation of the strength of a section,
2) variation in the strength of the material,
3) variation in the size of the section,
4) natural spread in test results,
5) applicability of test results to the actual structure,
6) consequence of failure of an element.

In these criteria α is taken as 1.0 on the assumption that the designer will anticipate and allow for future additions to the dead load on the structural members such as sidewalks, surfacing, barriers, utilities, etc. as provided for in the AASHO Specification.

Factor β is taken as 5/3 to represent overloads, whether authorized, unauthorized or accidental. This is approximately equivalent to a double live load in one lane of a multilane bridge with no other vehicle on the structure.

Inherent in the recognition of uncertainties is the inability to estimate their magnitudes, especially as they may be involved in various combinations. A consideration of the uncertainties covered by the strength factor, ϕ, suggests no basis for using different values for the principal elements of design-flexure, shear and direct stress. For those analyses, then, a uniform value of ϕ may be selected and shifted to the denominator of the right side of the above equation. This has been done in Equation 1.7-1 using 1.25 as the value of γ/ϕ.

There remain special uncertainties concerning the strength of connections, and appropriate values of ϕ have been applied to their evaluation in Articles 2.2.5 and 2.5.1.

The objective has been to choose the load factor formula so as to provide the same section as now provided in the working stress design in the short-span range. This has been done as illustrated in Figure A-2.

If it is assumed for simplicity that the maximum moment capacity of a noncomposite simple beam will by $F_y S$ while the working stress design moment is $0.55 F_y S$, then the ratio of the section modulus required by the load factor design to that required by the working stress design will be:

$$R_s = 0.55 \frac{\gamma}{\phi} \frac{R+\beta}{R+1} \qquad \text{(A)}$$

in which R is the ratio of the dead load moment to that produced by live load plus impact. Values of R as a function of span are shown in Figure A-1 for simple-span standard designs prepared by the Bureau of Public Roads in 1960.

Values of R_s are plotted in Figure A-2, based on $R = 0.0132L$, $\gamma/\phi = 1.25$ and $\beta = 5/3$. It will be noted that the curve crosses the ordinate 1.0 at about 40'. For longer spans, load factor design requires lighter sections than the working stress design unless the serviceability rather than the strength governs.

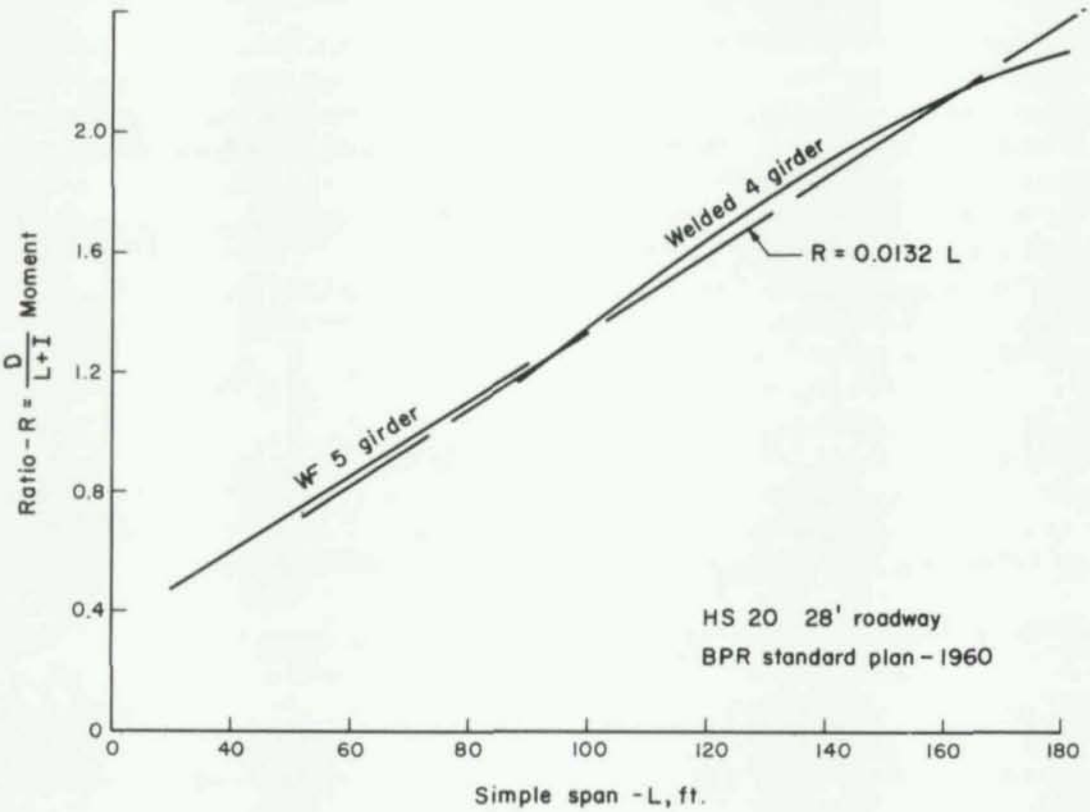


Fig. A-1

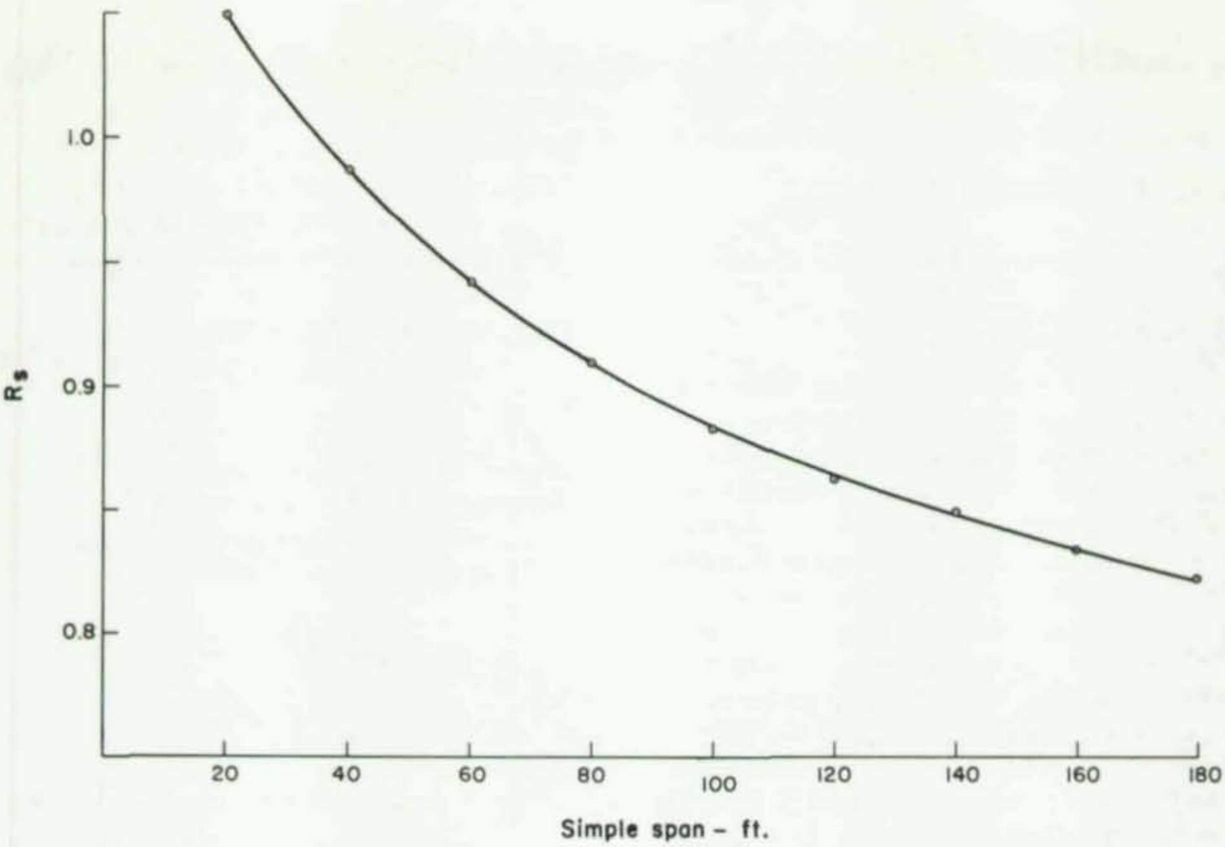


Fig. A-2

Curves such as the one illustrated in Figure A-2 were useful in arriving at a reasonable value of the overall factor γ/ϕ but they are not suitable for judging the adequacy of the load factor design. A meaningful comparison between the load factor design and the working stress design requires consideration of both the maximum strength and the serviceability. Such a study, made as a part of the development of load factor design, is summarized in the section on comparative designs.

Group I is the basic loading for dead and live load plus impact. For bridges designed for less than H20 loading, Group IA replaces Group I. Group II is intended to include the combinations of loading which may affect the structure in the absence of live loading. Group III includes the forces which may act on the structure in combination with a full live load with impact.

Section 2—Computation of Maximum Strength

2.1 BEAMS AND GIRDERS

2.1.1 Symmetrical Beams and Girders

A. *Compact Sections*

Symmetrical I-shaped beams with compact cross sections and adequate bracing are able to form plastic hinges which rotate at constant moment. The ability to plastify and thereby redistribute moments assures that prior to any local or lateral buckling the member will be able to reach a bending moment greater than the yield moment by the ratio Z/S.

Conditions for this behavior applicable to ASTM A36 steels are set forth in Reference 1. Newer work extending the use of the concept to steels up to 50,000 psi yield strength (ASTM A441) has shed additional light on the interpretation of results for the structural carbon steels as well. This work is summarized, references are cited, and design recommendations are presented in Reference 2. The provisions recommended here draw on the latter work.

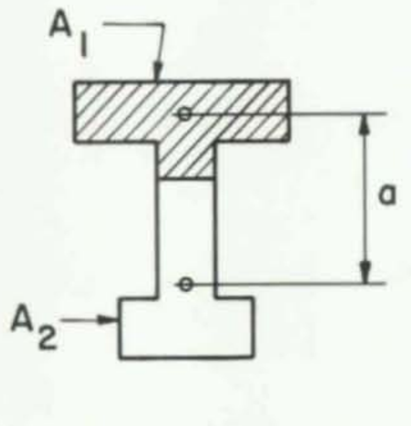
Computation of Plastic Modulus Z

The plastic modulus Z is the statical first moment of one half-area of the cross section about an axis through the centroid of the other half area.

A_1 (shaded) $= A_2$ (clear) $= A/2$

a = distance between centroid of A_1 & A_2

$Z = aA_1 = aA_2$



When a section is built up from plates or shapes of more than one yield point, the plastic moment should be computed on the basis of equilibrium on the cross section with all fibers stressed to the appropriate yield point in either tension or compression.

1.a) *Projecting flange element*

To delay local flange buckling until strain hardening is developed the compression flange of a beam must satisfy Equation 2.1.1-2.

Based on Reference 2, Section 3.4,

$$\frac{b'}{t} \leqslant \sqrt{\frac{G_{st}}{F_y}\left(\frac{4}{3+\frac{F_u}{F_y}}\right)}$$

$$\text{where } G_{st} = G\left(\frac{2}{1+\dfrac{E}{4E_{st}\,(1+\nu)}}\right)$$

$$G = \frac{E}{2\,(1+\nu)}$$

The above equation may be rewritten as follows:

$$b'/t \leqslant \frac{\sqrt{\dfrac{4\,G_{st}}{3+F_u/F_y}}}{\sqrt{F_y}}$$

The numerator is equal to 1640 for A36 steel and 1500 for A441 steel with $F_y = 50{,}000$. This narrow range was replaced by the numerical value of 1600.

Flanges not meeting the limitation of Equation 2.1.1-2 will buckle locally before strain hardening is reached. This buckling will not be prevented by partial embedment in the concrete slab but can be prevented by anchoring the flange into the slab with shear connectors as discussed in the Commentary on Section 2.2.

b) Web thickness

Local web buckling prior to strain hardening is avoided when the web of the beam satisfies Equation 2.1.1-3.

It has been shown in Article 6.2 of Reference 1 that for A36 steel 70 is a conservative limiting depth-to-thickness ratio. Research currently in progress at Lehigh University showed that a conservative limiting ratio for other steels exhibiting a yield plateau is inversely proportional to the square root of F_y. The analysis in Reference 1 is based on d_f, the distance center-to-center of flanges, but Equation 2.1.1-3 is referred to the over-all depth, d, in order to make use of the convenient d/w values in the "Plastic Section Modulus Table" in the AISC Manual of Steel Construction, Sixth Edition, 1963, in which w is the web thickness.

Equation 2.1.1-3 gives essentially the same values of those listed in Section 1.7.72 of AASHO Specifications for an unstiffened web.

c) *Lateral bracing*

To avoid lateral displacement and twisting of the main member until strain hardening is developed in the compression flange, lateral bracing must be spaced according to Equation 2.1.1-4 or 2.1.1-5.

Solutions of the problem of spacing the lateral bracing are assembled in Section 3.5 of Reference 2.

The maximum spacing for a beam under uniform moment is given as:

$$\frac{L}{r_y} = \frac{\pi\sqrt{E}}{k\sqrt{F_y\left(1 + 0.56\ \dfrac{E}{E_{st}}\right)}}$$

in which E_{st} is the strain hardening modulus.

The term $0.56\ E/E_{st}$ approaches 30 and therefore may be used alone in the parentheses. The term, k, accounts for the end restraints and is equal to 0.54 when the strains are elastic. Thus:

$$\frac{L}{r_y} = 7.8\sqrt{\frac{E_{st}}{F_y}}$$

E_{st} is about 900,000 psi for A36 steel and 700,000 psi for the higher-strength steels. Using the average value, 800,000, under the radical changes the value of L/r_y a maximum of 6% which may be neglected. Making this substitution leads to Equation 2.1.1-4. If there is a moment gradient the stress condition is less critical. The maximum spacing for a beam under moment gradient is given by:

$$\frac{L}{r_y} = 0.7\pi\sqrt{\frac{E}{F_y}}$$

Substituting 29,000,000 for E, this reduces to $\dfrac{12{,}000}{\sqrt{F_y}}$

This second expression should be used when the ratio of end moments in the braced length is less than 0.7.

d) *Axial compression*

Reduction in the plastic moment of a wide flange beam bent about its major principal axis has been shown to be negligible when axial thrust is less than 15 percent of the calculated plastic axial load (Article 7.2, Reference 1).

e) *Shear force*

Ability of a wide flange section bent about its major principal axis to resist bending has been shown to be relatively unaffected by shear when the shear force is less than the amount calculated by formula 2.1.1-7 (Article 6.1, Reference 1). Equation 6.13 of Reference 1 yields $V \leqslant 0.54\ F_y\ dt_w$ when the over-all depth is used in place of the clear, unsupported depth, assuming as in Reference 1 that $d/D = 1.07$. In the Criteria the numerical coefficient is changed to 0.55 to agree with the AISC Specifications.

2. The theoretical and experimental studies on which the design of compact sections is based were made for steels characterized by the stress-strain diagram shown in Figure A-3. Thus the design provisions for compact beams should not be used with steels having substantially different stress-strain characteristics.

3. This provision takes account of the demonstrated ability of compact sections to redistribute moment (Reference 1, Chapters 1 and 5).

B. *Braced Non-compact Sections*

Sections which are not stocky or well

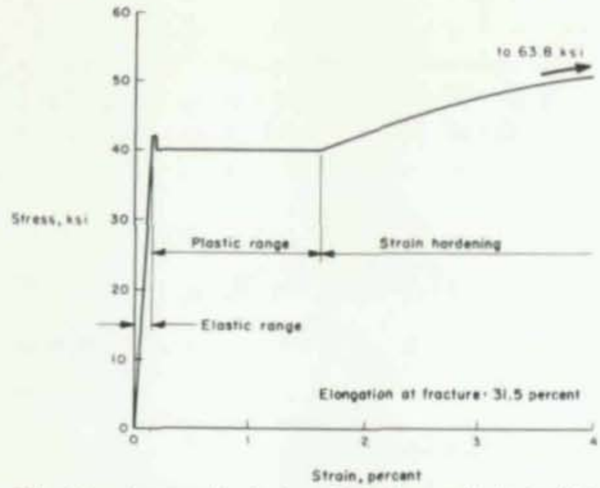


Fig. A-3 – An example of a stress-strain diagram for hot rolled carbon steel.

braced may buckle elastically before yielding or formation of a plastic hinge occurs. This article defines the limit where elastic buckling occurs at the same bending moment that causes yielding of the extreme fiber.

a) The value for b'/t is based on the classical plate buckling solution (Reference 5, Article 3.3). It is equivalent to the formula:

$$\frac{b}{t} = \frac{3250}{\sqrt{f_b}}$$

in Article 1.7.70 (A) of AASHO Specifications.

b) The value of D/t_w is the limiting ratio given in Article 1.7.72 of AASHO Specifications.

c) The spacing of bracing is based on the commonly used DeVries simplification of the classic solution for lateral-torsional buckling of an I-shaped member (Reference 4).

d) The maximum shear force V is based on the classical plate buckling solution and is equivalent to the formula:

$$t = \frac{D\sqrt{f_v}}{7500}$$

in Article 1.7.72 of AASHO Specifications.

C. *Transition*

To avoid discontinuity, a straight line interpolation between the requirements for compact and braced noncompact sections is used to determine the maximum strength for flanges and for spacing of lateral bracing. The interpolation for flanges in the range

$$\frac{1600}{\sqrt{F_y}} < b'/t < \frac{2200}{\sqrt{F_y}}$$

is illustrated in Figure A-4. The interpolation for spacing of lateral bracing in the range

$$\frac{7000\, r_y}{\sqrt{F_y}} < L < \frac{20{,}000{,}000}{F_y \dfrac{d}{A_f}}$$

is illustrated in Figure A-5.

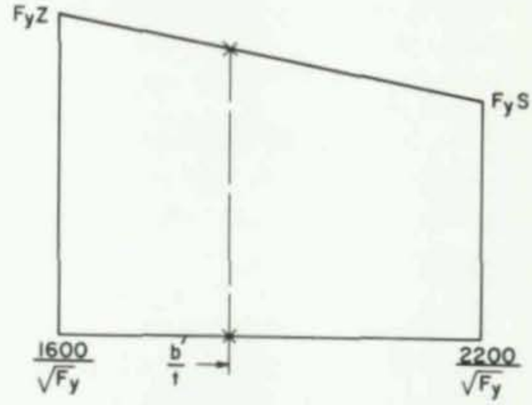


Fig. A-4

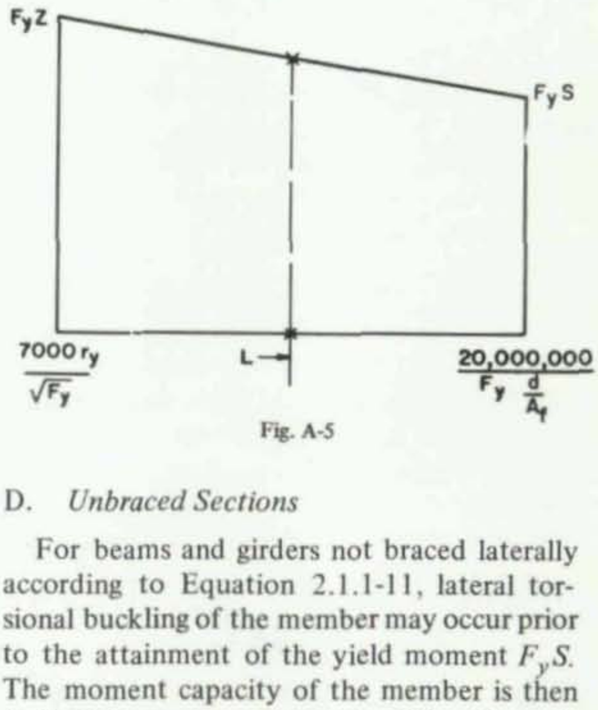


Fig. A-5

D. *Unbraced Sections*

For beams and girders not braced laterally according to Equation 2.1.1-11, lateral torsional buckling of the member may occur prior to the attainment of the yield moment F_yS. The moment capacity of the member is then derived from the column buckling formula (Reference 1, Article 5.4):

$$\frac{F_{cr}}{F_y} = \frac{M_u}{F_yS} = 1 - \frac{\lambda^2}{4}$$

where $\lambda^2 = \dfrac{(L/r')^2 F_y}{\pi^2 E}$

which is the characteristic value for lateral buckling of the compression flange. Assuming that:

$$r' = \sqrt{\frac{I_f}{A_f}} = \frac{b'}{\sqrt{3}}$$

the maximum strength Equation 2.1.1-13 is obtained. This is equivalent to the equation used in Section 1.7.1 of AASHO Specifications.

The maximum strength for lateral buckling is determined by the spacing of bracing. Formula 2.1.1-13 is derived for the condition of uniform stress over the length of the flange between adjacent braced points. If there is a stress gradient between braced points, lateral buckling is less critical. It has been shown that for the ratio of moments at the braced points equal to 0.7, the critical stress is increased by about 20% with an increase in this percentage as the moment ratio decreases (Reference 5, Article 4.4). The actual strength is computed by the equation:

$$\frac{F_{cr}}{F_y} = 1 - \frac{\lambda^2}{4C}$$

in which

$$C = 1.75 - 1.05\,\frac{M_2}{M_1} + 0.3\left(\frac{M_2}{M_1}\right)^2$$

The 20% increase in strength corresponds to that currently allowed in AASHO Specifications, Article 1.7.1, Footnote (1).

Equation 2.1.1-13 is applicable only to members with

$$\frac{L}{b'} < \sqrt{\frac{2}{3}\,\frac{\pi^2 E}{F_y}}$$

Beyond this slenderness, the strength of the member is governed by Euler buckling. This limit is not included in these criteria because it is beyond the practical range of values.

E. *Transversely Stiffened Girders*

1. Extensive theoretical and experimental studies completed recently and others now in progress have improved the understanding of the behavior of stiffened plate girders both under static and under fatigue loading. They demonstrated that the maximum static strength is determined by yielding rather than by buckling and that the web deflects laterally in a gradual manner as the load increases. This lateral deflection determines the fatigue strength of the plate girder.

Some results of fatigue tests of plate girders with transverse stiffeners [6,6A] are summarized in Figure A-6. It can be seen that all plate girders with $D/t_w < 200$ had relatively flat web plate and survived 2,000,000 or more cycles of stress without failure. On the other hand, several of the girders with $D/t_w > 200$ failed in fatigue at less than 2,000,000 cycles due to excessive lateral web deflection [6B]. The yield point of the web varied between 33,000 and 45,000 psi. The equation:

$$D/t_w = \frac{36{,}500}{\sqrt{F_y}}$$

defines for mild structural carbon steel (33,000 psi yield point) a web slenderness value of 200 below which fatigue is not a consideration. It is used in these Criteria as an upper limit for transversely stiffened plate girders (Equation 2.1.1-14).

It is of interest that the current upper limit for transversely stiffened plate girders, given in Article 1.7.71 (A) of AASHO Specifications, is about 15 percent more conservative than Equation 2.1.1-14. The AASHO limitation is based on an elastic buckling formula for a plate simply supported along all four edges. Recent studies* have shown that the deformation of the web of a plate girder is restrained by the flanges and is considerably smaller than would be expected for a simply supported plate. It is estimated conservatively that the restraint increases the theoretical buckling load by about 30 percent.

It also should be noted that these criteria require a reduction in the shear strength for the case of combined shear and bending (Equation 2.1.1-18) while no such reduction is required by the current AASHO Specification.

*Lehigh University current research.

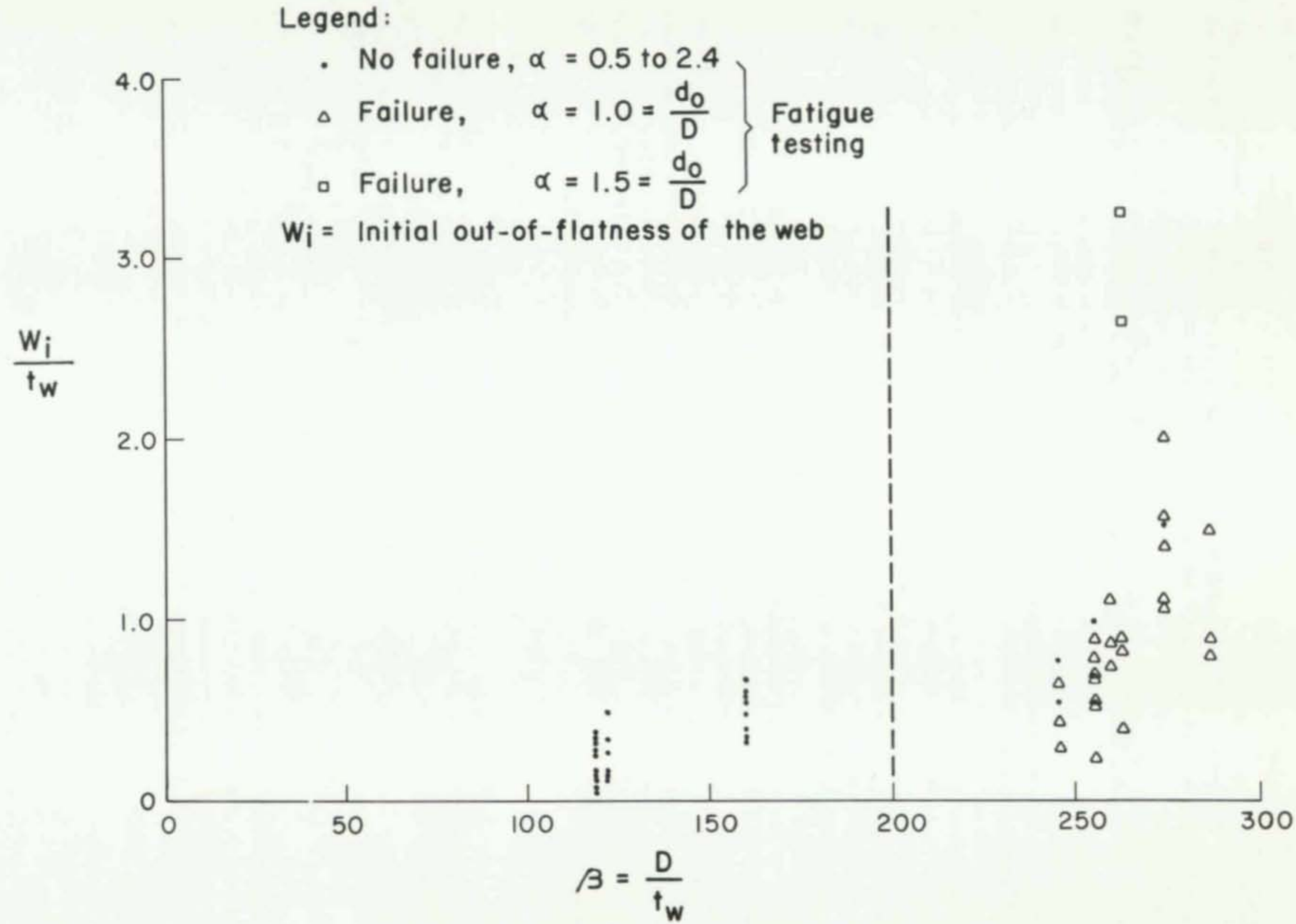

Legend:
. No failure, α = 0.5 to 2.4
△ Failure, α = 1.0 = $\frac{d_o}{D}$
□ Failure, α = 1.5 = $\frac{d_o}{D}$
Fatigue testing
W_i = Initial out-of-flatness of the web
$\frac{W_i}{t_w}$
4.0
3.0
2.0
1.0
0
0
50
100
150
200
250
300
$\beta = \frac{D}{t_w}$

Fig. A-6

2. It has been shown [7] that, as the web deflects laterally, the distribution of bending stress in the web becomes nonlinear; that is, the web carries a smaller proportion of the moment than would be predicted on the basis of the usual straight line stress distribution. The more slender the web, the larger will be the reduction in the contribution of the web to the maximum strength of the girder. Up to the slenderness ratio of $31{,}000 / \sqrt{F_y}$, the AASHO limiting value, it was shown that the maximum bending strength of a plate girder may be computed as $M_u = F_y S$. For more slender webs:

$$M_u = FS$$

where

$$F = F_y \left[1 - 0.0005 \frac{A_w}{A_f} \left(\frac{D}{t_w} - \frac{31{,}000}{\sqrt{F_y}} \right) \right]$$

Plate girders with transverse stiffeners are permitted up to web slenderness ratios of $36{,}500 / \sqrt{F_y}$ based on conditions of fatigue. Using this limit and assuming that $A_w = 2A_f$ and $F_y = 36{,}000$ psi, the above equation gives:

$$F = 0.975\, F_y$$

This represents the maximum reduction in the maximum strength due to lateral web deflection. It is considered satisfactory to neglect this reduction in the design of transversely stiffened plate girders.

3. The shear carrying capacity of girder panels depends upon the ability of the web to sustain applied loads. Stocky webs sustain loads in the familiar manner of "beam action" for which the shear is computed by VQ/I_t or simply by V/A_w. For slender webs which may buckle under shear force, "tension field action" develops in the web panels. [8] The action is analogous to that of the tension diagonals of Pratt trusses. It has been shown satisfactory [8] to compute the shear capacity of web panels by summing up the contributions of beam action and of post-buckling tension field action. The resulting expression is Equation 2.1.1-15, where the first term in the bracket relates to the limit of web buckling under shear and the second term relates to the post-buckling strength.

While Equation 2.1.1-15 is the same as Equation 14 in Reference 8, Equation 2.1.1-17 is only an approximation of the ratio of the web buckling shear stress to the shear yield. According to Basler [8] this ratio is given by the equation

$$C = \frac{F_{cr}}{F_y / \sqrt{3}} = k \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t_w}{D} \right)^2 \frac{\sqrt{3}}{F_y}$$

where $F_y / \sqrt{3}$ is the yield stress in shear and k is the buckling coefficient. By substituting numerical values for π, E and ν, the equation may be expressed as

$$C = \frac{45{,}000{,}000 k}{F_y} \left(\frac{t_w}{D} \right)^2 \qquad \text{(A)}$$

According to Reference 8, Equation A is applicable only for C not exceeding 0.8. For larger values

$$C = 6000 \frac{t_w}{D} \sqrt{\frac{k}{F_y}} \qquad \text{(B)}$$

should be used. Equations A and B are plotted in Figure A-7a as a dashed line; it can be seen that they can be approximated quite closely by

$$C = 8000 \frac{t_w}{D} \sqrt{\frac{k}{F_y}} - 0.3 \qquad \text{(C)}$$

shown as the full line in Figure A-7a.

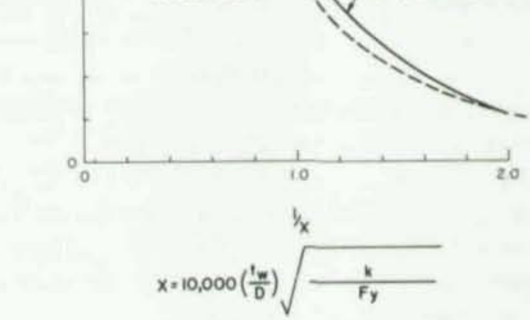


Fig. A-7a

Assuming that the web panel is simply supported on all four edges, the buckling coefficient k is given by the equations

$$k = 5.34 + 4.00\left(\frac{D}{d_o}\right)^2 \quad \text{for } \frac{d_o}{D} \geq 1$$

$$k = 4.00 + 5.34\left(\frac{D}{d_o}\right)^2 \quad \text{for } \frac{d_o}{D} \leq 1$$

The design procedure was simplified further by replacing the two equations for the buckling coefficient k with the expression

$$k = 5 + 5\left(\frac{D}{d_o}\right)^2 \qquad \text{(D)}$$

as shown in Figure A-7b, Equation 2.1.1-17 was obtained by substituting D into C.

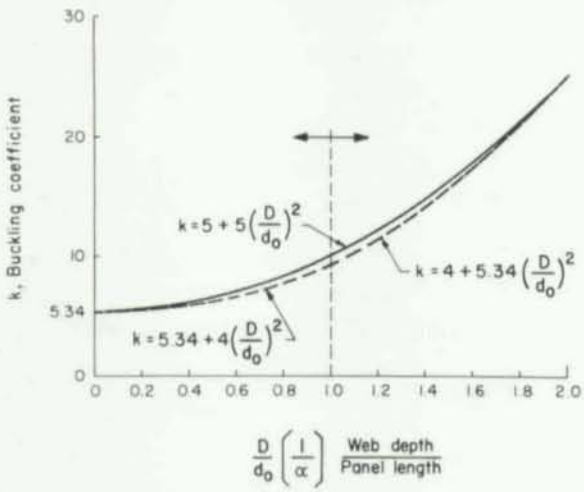


Fig. A-7b

When both shear and bending moment are high in a girder panel, the web plate must be strong enough to resist the shear and to participate in resisting moment. It has been shown [9] that web plates designed according to provisions herein given are capable of doing so as long as the shear is less than 0.6 V_u or the moment is less than 0.75 M_u. Above these two values, a straight-line interpolation (Figure A-8a) gives conservative limits. This straight line is expressed as Equation 2.1.1-18.

In case Equation 2.1.1-18 governs and an increase of web thicknesses or the flange size is not desirable, the simplest way to avoid interaction of shear and bending moment is to decrease the stiffener spacing.

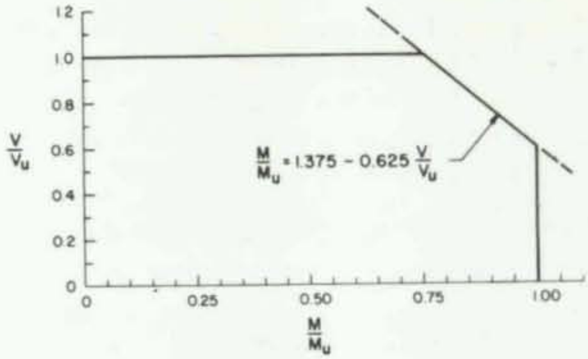


Fig. A-8a

4. Since the shear capacity of girder panels is influenced by the length of the panel, transverse stiffeners must be spaced according to expected shear capacity. Theoretically, if required shear capacity is low, stiffeners can be spaced at great distance. However, a maximum distance of $1.5D$ is arbitrarily imposed.

At the ends of girders where there is no neighboring panel for the last panel to anchor its tension field, shear capacity is contributed by beam action alone. Therefore, the stiffener spacing is to prevent theoretical buckling of the webs under shear. Equation 2.1.1-19 specifies this limit and gives the same results as the current AASHO requirement of $d = 11{,}000\, t_w / \sqrt{f_v}$.

Where tension field action is developed, transverse stiffeners are stressed as vertical struts in a Pratt truss [8]. The vertical component of tension field force, corresponding to the second term in the bracket of Equation 2.1.1-15, is assumed to be carried by the stiffeners.

When the tension field is fully developed, the magnitude of this vertical component is

$$F_s = \frac{1}{2} F_{yw}\, t_w D \left[\frac{d_o}{D} - \frac{(d_o/D)^2}{\sqrt{1 + (d_o/D)^2}}\right](1 - C)$$

where F_{yw} is the yield point of the web. For the practical range of d_o/D, i.e., between 1/3 and 1.5, the expression in the brackets varies between 0.21 and 0.3. Furthermore, when the tension field is not fully developed, the force F_s is reduced by the ratio V/V_u. Thus, the force to be resisted by one stiffener may be given conservatively as

$$F_s = 0.15\, F_{yw}\, D t_w\, (1 - C)\frac{V}{V_u}$$

Assuming that the vertical force F_s is resisted by the stiffener and a portion of the web, and that all material is stressed to its yield point as shown in Figure A-8b,

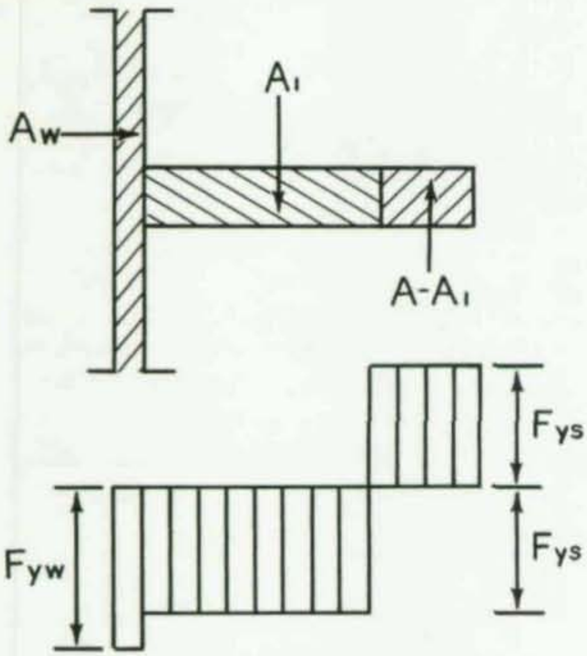


Fig. A-8b – Plastic Stress Distribution in a Stiffener

the equilibrium of forces requires that

$$F_s = (2A_1 - A) F_{ys} + A_w F_{yw}$$

The area of the stiffener can then be expressed as

$$A = [0.15Dt_w \ (1 - C) \frac{V}{V_u} - A_w] \ YB$$

where $B = \dfrac{1}{2\dfrac{A_1}{A} - 1}$

$$Y = F_{yw}/F_{ys}$$

With two symmetrical stiffeners, there is no bending; thus, $A_1 = A$ and $B = 1.0$. For a one-sided plate stiffener, $A_1 = A\sqrt{2}$ and B is equal approximately to 2.4; and for a single angle stiffener B is equal approximately to 1.8.

To obtain equation 2.1.1-21, the assumption was made that

$$A_w = \frac{18t_w^2}{B}$$

While very little information is available on the effective width of the portion of the web working with the stiffener, the test data in Table A-1 show that stiffeners alone cannot account for the full force F_s. For symmetrical bearing stiffeners AASHO Specifications Article 1.7.74 (A) assumes $A_w = 18t_w^2$. The contribution of the web is thought to be less for a one-sided stiffener. A decrease inversely proportional to B was assumed.

For the web to develop the buckling shear strength calculated by Formula 2.1.1-15, it is necessary for the transverse stiffener to have sufficient rigidity to cause a node to form along the line of the stiffener. Equation 2.1.1-22, based on an earlier investigation [10] and almost identical to AASHO Specification Article 1.7.72, is used for this purpose.

One of the common practices has been to fit transverse stiffeners snugly between the compression and tension flange. Unless it is necessary for the rigidity of the beam or girder in order to facilitate handling and erection, transverse stiffeners in bearing with compression flange alone provide as much support to the compression flange as those stiffeners which are snugly fitted. The unsupported distance of the web between the tension flange and the transverse stiffener shall not be too great lest crippling may occur. The maximum allowable distance of $4t_w$ was derived theoretically [11] and has been proven satisfactory [12].

Past specifications have required single stiffeners to be attached to the compression flange. Provisions in these criteria limit the flange b'/t ratio to prevent local buckling, limit the flange stress between laterally braced points to prevent lateral buckling and require stiffeners on both sides of the web at points of load concentrations on the flange to prevent flange tilting (AASHO Specifications Article 1.7.72). Thus, the attachment of a single stiffener to the compression flange is not necessary.

F. *Longitudinally Stiffened Girders*

When the web thickness is less than that permitted by Equation 2.1.1-14 for transversely stiffened girders, the web must be stiffened with one longitudinal stiffener. When longi-

TABLE A-1 – FORCES ON TRANSVERSE STIFFENERS

Girder	Aspect Ratio $a = d_o/D$	Stiffener Area, A (in^2)	F_s^* (kip)	σ_s (ksi)	$\sigma_s A$ (kip)	$\frac{\sigma_s A}{F_s^*}$
G 6	1.50	2.0	–36.8	–10.2$^\triangle$	–20.4	0.555
			–36.8	– 8.3$^\triangle$	–16.6	0.451
G 7	1.00	2.0	–42.8	– 6.2$^\triangle$	–12.4	0.290
G 8	1.50	2.0	–40.0	– 6.5$^\triangle$	–13.0	0.325
			–40.0	– 5.4$^\triangle$	–10.8	0.270
G 9	1.50	2.0	–33.8	– 7.5$^\triangle$	–15.0	0.444
E 1	1.50	2.0	–51.2	– 6.6$^\triangle$	–13.2	0.258
E 2	1.50	2.0	– 7.3	0$^\triangle$	0	0
E 4	1.50	2.0	–48.5	– 3.6	– 7.2	0.148
E 5	0.75	2.0	–20.0	0	0	0
			–26.2	0	0	0

*Theoretical value by tension field action.

△Measured stress on stiffener adjacent to failed panel.

tudinal stiffeners are properly positioned and proportioned, as described in this article, the stiffeners are rigid and strong enough to effectively reduce lateral web deflection and to maintain a linear distribution of bending (Mc/I) stresses in the transverse cross section of the girder [13]. Hence, there is no danger of fatigue failure and the maximum bending strength is governed by Equation 2.1.1-8 or 2.1.1-13.

Recent static tests of large-size plate girders of A36 steel with D/t_w ratios higher than 400 [13] have demonstrated the effectiveness of the longitudinal stiffeners in minimizing lateral web deflections. Fatigue tests of hybrid girders [14] have confirmed the effectiveness of a longitudinal stiffener in preventing fatigue cracking of the web. Accordingly, the web thickness of Equation 2.1.1-23 is used as the upper limit for girders with transverse and one longitudinal stiffener. This limiting thickness is twice that for girders with transverse stiffeners only.

A longitudinal stiffener divides a panel into two subpanels. If the longitudinal stiffener as well as the transverse stiffeners are properly designed, each subpanel develops its shear carrying capacity as described in Article 2.1.1E3. The shear capacity of the entire panel with the longitudinal stiffener is then the sum of those of the subpanels [13].

The contribution of the longitudinal stiffener at a distance $D/5$ from the compression flange is relatively small as illustrated in Figure A-9. Thusit is recommended that the influence of the longitudinal stiffener be neglected in computing the shear strength of the plate girder.

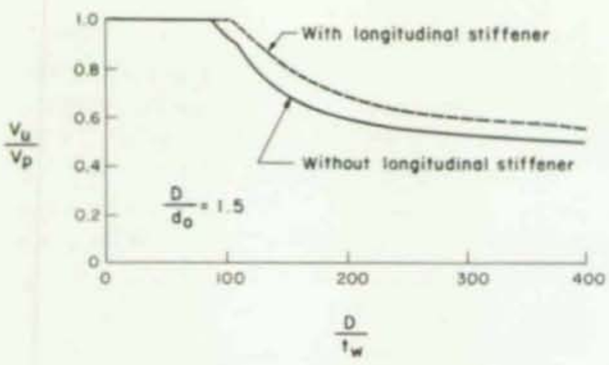


Fig. A-9–Effect of longitudinal stiffeners on shear strength.

The primary purpose of the longitudinal stiffener is to prevent lateral deflection of the web. Theoretical and experimental studies have indicated [5] that the optimum location of one longitudinal stiffener is $D/5$ for bending and $D/2$ for shear. Recent tests [13] also showed that longitudinal stiffeners located at $D/5$ can control effectively lateral web deflections when under bending. Distance $D/5$ is recommended by these criteria because shear is always accompanied by bending moment, a properly proportioned longitudinal stiffener at any location reduces lateral web deflection caused by shear, and $D/5$ has been accepted in bridge design practice.

Longitudinal stiffeners must fulfill a number of requirements to be considered properly proportioned. These are: a) the maximum width-to-thickness ratio to avoid local buckling of the stiffener, b) the maximum rigidity to ensure a nodal line at the stiffener when the web buckles, c) the minimum radius of gyration to avoid lateral buckling of the stiffener and d) the minimum area to anchor the tension field force.

The requirement a) is expressed by Equation 2.1.1-20. The yield point F_y should be that of the stiffener. Equation 2.1.1-20 is more liberal than Equation 2.1.1-9, but the difference arises from the fact that welding to the web provides more effective fixity to the edge of the thin stiffener than it does to the heavier flange (Reference 3, Article 17.1). Equation 2.1.1-20 is more conservative than Article 1.7.73 of the AASHO Specification.

The rigidity requirement of longitudinal stiffeners is expressed as Equation 2.1.1-24, which is the same as Article 1.7.73 in current AASHO Specifications.

When the longitudinal stiffener is properly proportioned, the distribution of bending stresses along a cross section of the girder is close to linear. The longitudinal stiffener at $D/5$ is then subjected to a compressive stress 60% of that of the compression flange, with a max. of 0.6 F_y (Figure A-10). The stiffener column must be rigid enough to withstand this stress without lateral buckling. By assuming that the eccentricity of the load and initial out-of-straightness cause a 20% increase in stress at the stiffener and using a partially restrained end condition for the stiffener, the required rigidity is evaluated by using the basic column formula given in Article 2.1.1D of this Commentary. Thus:

$$\left(\frac{0.6\,F_y}{F_y}\right) 120\% = 1 - \frac{F_y}{4\,\pi^2 E}\left(\frac{0.7 d_o}{r}\right)^2$$

and $$r = \frac{d_o \sqrt{F_y}}{23{,}000}$$

To anchor the tension field force, the longitudinal stiffener must have sufficient area. With the requirements a), b) and c) above, sufficient stiffener area is provided for; thus no additional rule is needed.

The longitudinal stiffener serves as a column, just as the compression flange does. The portion of the web which acts as a part of the stiffener column is between 20 t_w and 30 t_w [5]. To conform to the current rules for bridge design, a centrally located web strip of 18 t_w is permitted.

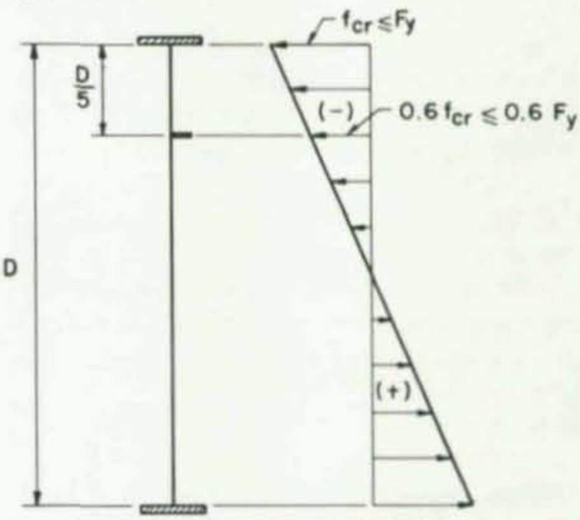


Fig. A-10 – Stress at Longitudinal Stiffener

Each subpanel of a subdivided panel behaves as a separate panel. The transverse stiffeners, therefore, must fulfill all requirements of Article 2.1.1E4 with D taken as equal to the depth of the subpanel. Lateral loads along the length of the longitudinal stiffener are transferred to the adjacent transverse stiffeners as concentrated reactions [13]. A relationship between the section moduli of the longitudinal and transverse stiffeners can be derived to make sure that the latter does not fail under the concentrated reactions. This relationship is Equation 2.1.1-26.

2.1.2 Unsymmetrical Beams and Girders

A. *General*

Beams and girders symmetrical about the vertical axis of the cross section, but unsymmetrical with respect to the horizontal centroidal axis, differ from doubly symmetrical cross sections in flexure in that the neutral axis for bending of unsymmetrical cross sections is not located at the mid-depth of the member or of the web. Consequently the section moduli, S, are different for the flanges and so are the flexural stresses.

Detailed discussions of beams and girders with singly symmetrical cross section can be found in Ref. 5, 7 and 16. Among the formulas for the computation of bending strength, the formula expressed as Equation 2.1.1-13 is the simplest and is quite accurate if proper recognition is given to the term b'.

In the derivation of Equation 2.1.1-13 it is considered that an "effective compression flange column" governs the strength of the member in bending, regardless of the (single or double) symmetry of the cross section [7]. The term b' in Equation 2.1.1-13, as discussed in Commentary Article 2.1.1D, is an expression for r', the radius of gyration of the compression flange column. Approximation has been adopted for simplicity by using

$$r' = \frac{b'}{\sqrt{3}}.$$

When applied to unsymmetrical sections, such an approximation may not be warranted. In the case where the compression flange is larger and the neutral axis is near this flange, the web contributes little to carry compression. The compression flange alone acts as the "effective compression flange column" so that the approximation $r' = b'/\sqrt{3}$ may be used. However, in case the neutral axis is near the tension flange, a relatively large portion of the web participates in carrying compression. In such case the radius of gyration may be approximated as:

$$r' = \sqrt{\frac{I_f}{A_f + \frac{1.5}{6}A_f}}$$

$$= (0.9)\sqrt{\frac{I_f}{A_f}} = \frac{(0.9b')}{\sqrt{3}}$$

which indicates a replacement of b' by $0.9b'$ in Equation 2.1.1-13.

B. *With Transverse Stiffeners*

Since shear capacity is not affected by the unsymmetrical nature of the cross section [8] and moment capacity in compression has been adjusted by modifying b' in Article 2.1.2A, all provisions of Article 2.1.1E are applicable to unsymmetrical sections.

Equation 2.1.2-1 specifies the distance D_c between the neutral axis and the compression flange must not be greater than:

$$\frac{18{,}250t_w}{\sqrt{F_y}} = \frac{1}{2}\ \frac{36{,}500t_w}{\sqrt{F_y}}$$

which is one-half the value defined by Equation 2.1.1-14. In other words, the maximum permissible web slenderness ratio as defined by Equation 2.1.1-14 is checked here proportionally with respect to the compression part of the web. If the web slenderness ratio, or the distance D_c, exceeds the limit, either longitudinal stiffeners should be used or the web thickness should be increased to reduce the possibility of large lateral deflection of the web.

C. *With Longitudinal Stiffeners*

Since moment capacity in compression has been adjusted by modifying b' in Article 2.1.2A, provisions for symmetrical sections are applicable to unsymmetrical sections if the longitudinal stiffener is located properly.

a) It has been specified that the longitudinal stiffeners be placed at $D/5$ from the compression flange for symmetrical girders. This is the optimum distance [5] and is 2/5 of the distance between the compression flange and the neutral axis. To maintain this effective control of lateral web deflection, $2D_c/5$ is specified.

b) Analogous to the situation for transversely stiffened sections, the value given by Equation 2.1.2-2, limits D_c to

$$\frac{36{,}500\, t_w}{\sqrt{F_y}} = \frac{1}{2}\ \frac{73{,}000\, t_w}{\sqrt{F_y}}$$

which is one-half the value defined by Equation 2.1.1-23. That is, the compression part of the web also fulfills the limit of slenderness ratio.

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2.2. COMPOSITE BEAMS AND GIRDERS

2.2.1 General

Criterion a) is intended to insure an adequate strength at every section of the beam. Unlike prismatic steel sections, the maximum moment capacity of a composite beam varies along the length of the span because only at specific locations is the shear connection sufficient to fully develop the section. At other locations, the maximum moment capacity may be reduced because of an inadequate shear connection. The shear connection should be checked at points of maximum moment and at each location of a change of the cross section, and the connectors distributed so as to insure that the required moment capacity can be developed at each of those sections.

Criterion b) refers the designer to Articles 2.1.1 and 2.1.2 on the design of steel beams. The basic differences between the web of a composite beam and that of a steel beam are: (1) Different proportions of the shear are carried by the web. (2) A composite beam is nearly always unsymmetrical and the neutral axis does not lie at mid-depth. (3) The fact that the top flange of the composite beam is firmly anchored to the slab provides greater restraint against buckling of the web than does the flange of a steel beam. There is little doubt that the slab in fact does carry part of the shear, but at the time of this writing there is insufficient information to determine just what percentage of the shear is carried in the slab. The assumption that the web of the steel section carries all the shear is conservative.

2.2.2 Positive Moment Sections

Compactness requirements to assure sufficient rotation to fully develop the section are somewhat more difficult to define for composite beams than for prismatic steel sections. The slab restrains local buckling of the top flange in two ways. Anchorage of the shear connectors in the slab restrains buckling of the flange directly, and in the event there is a tendency for local buckling between the shear connectors, the slab may pick up a larger portion of the compression and thus indirectly restrain the buckling.

Except in cases where the slab makes only a very small contribution to the strength of the composite section, the entire web is in tension at sections which are fully developed and compactness requirements for the web are of little significance. However, near the ends of the span where the ultimate moment capacity is limited by the shear connection, a substantial portion of the web may be in compression. Thus, in these cases some provision must be made to insure against local buckling of the web.

2.2.3 Negative Moment Sections

In the negative moment region, there is little difference between a composite beam and a steel beam except that the composite beam may be unsymmetrical. Thus, the designer is referred to Articles 2.1.1 – 2.1.2 for steel sections.

2.2.4 Box Girders

This section is limited to the design of bridges supported by two or more symmetrical section single-cell box girders, arranged so that the distance center-to-center of adjacent top flanges of adjacent girders, *a*, is approximately equal to the width of the girders, *w*, measured between the centers of the top flanges, Fig. A-11a. Further, the cantilever overhang of the deck slab beyond the exterior web, *c*, is limited to 60 percent of distance *a*, measured at midspan, but not more than six feet. These limitations are necessary because the provisions of the Criteria concerning lateral distribution of loads, secondary bending stresses, and the effectiveness of the bottom flange plate are based on an extensive study of box girder bridges the proportions of which conform to these limitations. The extent to which conclusions drawn from this study are valid for box girder bridges not conforming to the specified limitations is uncertain. Hence bridges which do not conform should be studied using a more general method of structural analysis. [11]

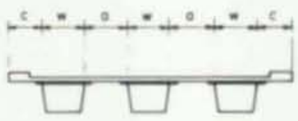

Fig. A-11(a) – Typical Bridge Cross Section

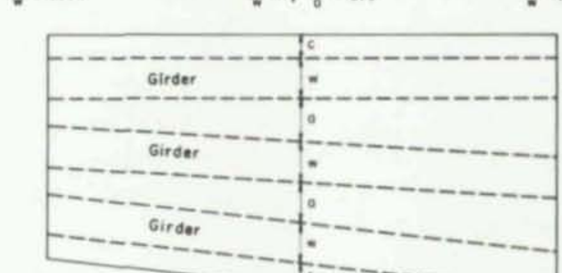

Fig. A-11(b) – Example of Possible Variation in Bridge Width

The width of the box girders may be limited by hauling restrictions or for other reasons. Also it may sometimes be necessary to splay the girders in plan to accommodate a roadway to varying width. Due to these considerations it is not always possible or convenient to make the distance center-to-center of adjacent top flanges of adjacent girders, a, exactly equal to the width of the girders, w, measured between the centers of the top flanges (Fig. A-11a). However, some limitations must be placed on the variation of distance a, with respect to distance w, since the studies on which some of the provisions of the Criteria are based were made on bridges in which w and a were equal. The following reasonable limitations will allow some flexibility of layout in design while maintaining the validity of those parts of the Criteria deriving from the study referred to above.

a) At midspan, distance a should not be less than 0.80 w nor greater than 1.20 w.

b) At the supports, distance a should be not less than $0.65w$ nor greater than $1.35w$.

An illustration of how a roadway of varying width may be accommodated while complying with these limitations is shown in Fig. A-11b.

Lateral Distribution

The equation for W_L, the fraction of a wheel load to be applied to each box girder in order to calculate the design live load bending moment, is based on analytical and model studies of simple-span composite box girder bridges. [3, 4] The results obtained in the study showed that folded plate theory can be used to analyze the behavior of bridges of this type. It was used to obtain the maximum load per girder produced by various critical combinations of loading on thirty-one bridges having various spans, numbers of box girders, and numbers of traffic lanes.

Section 1.2.9, Reduction in Load Intensity, of AASHO Specifications, allows a reduction of the maximum stress produced in any member by simultaneous loading of several traffic lanes. This is equivalent to using in design the most critical of the following loadings: 100 percent of the H or HS loading on one or two lanes, 90 percent of three lanes, or 75 percent on four or more lanes. The maximum load per girder caused by each of these loadings was calculated, and the maximum design load per girder, W_M, was thus obtained for each bridge. The values of W_M are listed in Table A-2, together with the values of W_L calculated using the equation:

$$W_L = 0.1 + 1.7R + 0.85/N_w$$

It can be seen that the equation predicts closely the maximum load per girder which should be used in design. The average value of W_L/W_M for all thirty-one bridges investigated is 1.01.

To stay within the range of bridge types studied, the value of R used in this equation must not be less than 0.5 nor greater than 1.5.

When the spacing of box girders varies along the length of the bridge, the value of N_w to be used in the equation for W_L should be that corresponding to the width of the bridge at midspan.

The bridges considered in the development of the equation for W_L were provided with diaphragms only at the supports. If diaphragms are provided within the span, the transverse load distribution characteristics of the bridge will be improved to some degree. If, in a particular case, it is desired to use the load distribution characteristics which result from the inclusion of diaphragms, then an additional study should be made using a suitable method of structural analysis.

For the distribution of dead load to each girder, it is considered that the provisions of Section 1.3.1 (B), Bending Moments in Stringers and Longitudinal Beams, of AASHO Specifications, are applicable to this type of bridge.

Web Plates

In the case of web plates inclined to the vertical, the shear V_w in the plane of the web plate will be greater than the vertical shear V.

In making the design calculations the shear to be resisted will be the inplane shear V_w, and the depth of the web plate D used in the calculations will be the depth measured on the slope.

Tension Flanges

The elementary theory of bending assumes

TABLE A-2 – MAXIMUM LIVE LOAD PER BOX GIRDER

Bridge No.	N_w Number of Lanes	G Number of Box Girders	Span, Ft.	W_M (1) Wheel Loads	W_L (2) Wheel Loads	$\frac{W_L}{W_M}$
1	6	4	50	2.87	2.79	0.97
2	6	4	75	2.85	2.79	0.98
3	6	4	100	2.79	2.79	1.00
4	6	4	150	2.79	2.79	1.00
5	5	4	75	2.41	2.40	1.00
6	5	4	150	2.39	2.40	1.00
7	4	4	50	2.08	2.01	0.97
8	4	4	75	2.02	2.01	1.00
9	4	4	100	1.96	2.01	1.03
10	4	4	150	2.07	2.01	0.97
11	4	5	50	1.69	1.67	0.99
12	4	5	75	1.64	1.67	1.02
13	4	5	100	1.62	1.67	1.03
14	4	5	150	1.53	1.67	1.09
15	3	3	50	2.18	2.08	0.96
16	3	3	75	2.12	2.08	0.98
17	3	3	100	2.12	2.08	0.98
18	3	3	150	2.03	2.08	1.03
19	3	4	50	1.69	1.66	0.98
20	3	4	75	1.64	1.66	1.01
21	3	4	100	1.60	1.66	1.04
22	3	4	150	1.57	1.66	1.06
23	3	2	75	3.04	2.93	0.96
24	2	2	50	2.18	2.23	1.02
25	2	2	75	2.16	2.23	1.03
26	2	2	100	2.12	2.23	1.05
27	2	2	150	2.11	2.23	1.06
28	2	3	50	1.70	1.66	0.98
29	2	3	75	1.63	1.66	1.02
30	2	3	100	1.58	1.66	1.05
31	2	3	150	1.55	1.66	1.07

(1) W_M = Max. load per girder calculated using folded plate theory. Average $\frac{W_L}{W_M} = 1.01$

(2) $W_L = 0.1 + 1.7R + 0.85/N_w$

that stress is proportional to the distance from the neutral axis, that is, that the stresses are constant over the width of the flange. If the flange is very wide the stress distribution across the flange is not uniform because of the shear lag. In such cases, the correct maximum bending strength can be computed by using the concept of "effective width."

Investigations of the "effective width" have been reported by several authors for both I-beams (5, 6, 7, 8) and box beams (9, 10). Stress analyses of actual box girder bridge designs, carried out using a computer program developed by Scordelis and Lo, have also been used to evaluate the effective width. This program uses the Goldberg-Leve [11] equations to evaluate plate edge forces, stiffnesses, and final internal forces, moments and displacements. Bridges for which the span-to-flange-width ratio varied from 5.65 to 35.3 were included in the study.

The effective flange width as a ratio of the total flange width covered a range of from 0.89 for the bridge with the smallest span-to-width ratio, to 0.99 for the bridge with the largest span-to-width ratio. On this basis it is reasonable to permit the flange plate to be considered fully effective provided its width does not exceed one-fifth of the span of the bridge. Although the results above were obtained for simply supported bridges, they apply equally to continuous bridges, using the equivalent span, i.e., the distance between points of contraflexure.

Compression Flanges

In heavy steel construction, local buckling is generally not a controlling factor in the design of the compression elements. However, in thin-walled structures such as box girder bridges, consideration must be given to the possibility of local buckling of the compression elements.

Plate elements having a width-thickness ratio no greater than $6140/\sqrt{F_y}$ when stiffened on both edges, or $2600/\sqrt{F_y}$ when stiffened on one edge, can be expected to develop yield point stresses without premature local elastic buckling. These limiting ratios correspond to a value of $\lambda = 0.6$ in the non-dimensional plate buckling curve, where $\lambda = \sqrt{F_y/F_{cr}}$; and to values of the plate buckling coefficient k of 4.0 and 0.72 respectively for the two edge conditions. Both values of k are conservative; the value of $\lambda = 0.6$ is reported by Beedle *et. al.* [12]

When λ is less than 0.6 failure will occur by yield of the steel.

For values of λ between 0.6 and 1.3, failure will occur by buckling at stresses below both the yield point of the steel and the elastic critical buckling stress. For width-thickness ratios less than $6650\sqrt{k}/\sqrt{F_y}$ the equations for F_{cr} are the equations of a transition curve joining the point $\lambda = 0.6$ at $F_{cr} = F_y$, and the point $\lambda = 1.30$ on the curve representing elastic buckling. This is shown in Fig. A-12.

For width-thickness ratios greater than $6650\sqrt{k}/\sqrt{F_y}$, the equation for F_{cr} is the equation of the line in Fig. A-12 which represents elastic buckling.

The provisions for compression flanges with longitudinal stiffeners are based on the theory of elastic stability [11]. They are formulated in such a way that the necessary stiffener stiffness can be calculated directly.

The equation for the required longitudinal stiffener stiffness, I_s, is an approximate expression which, within its range of applicability, yields values close to those obtained by use of the exact but cumbersome equations of elastic stability. In Table A-3 values of the plate buckling coefficient k obtained from the equations of elastic stability using $I_s = \phi t^3 w$ are compared with the initially assumed values of k used to compute the coefficient ϕ. It can be seen that the actual values of k are very close to the initially assumed values. The variation in the stress F_{cr} resulting from variation in the actual value of k as compared to the assumed value of k is considerably less than the difference between the assumed and actual values of k. These values of k are the minimum that can occur in a long compression flange where the buckling wave length is free to assume its most unfavorable value. For short compression flanges the buckling wave length will be less than the most unfavorable value, and the actual value of k will be greater than the assumed value used to calculate the coefficient ϕ. The proposed procedure is therefore conservative. An upper limit of 4 is placed on the value that may be assumed for k, since this $k = 4$ corresponds to buckling of the plate panels between stiffeners.

No provisions are given in these criteria for compression flanges stiffened by longitudinal stiffeners combined with transverse stiffeners. A working stress design procedure for this case may be found in the 1966-67 Interim AASHO Specifications.

No provisions are included for the design of

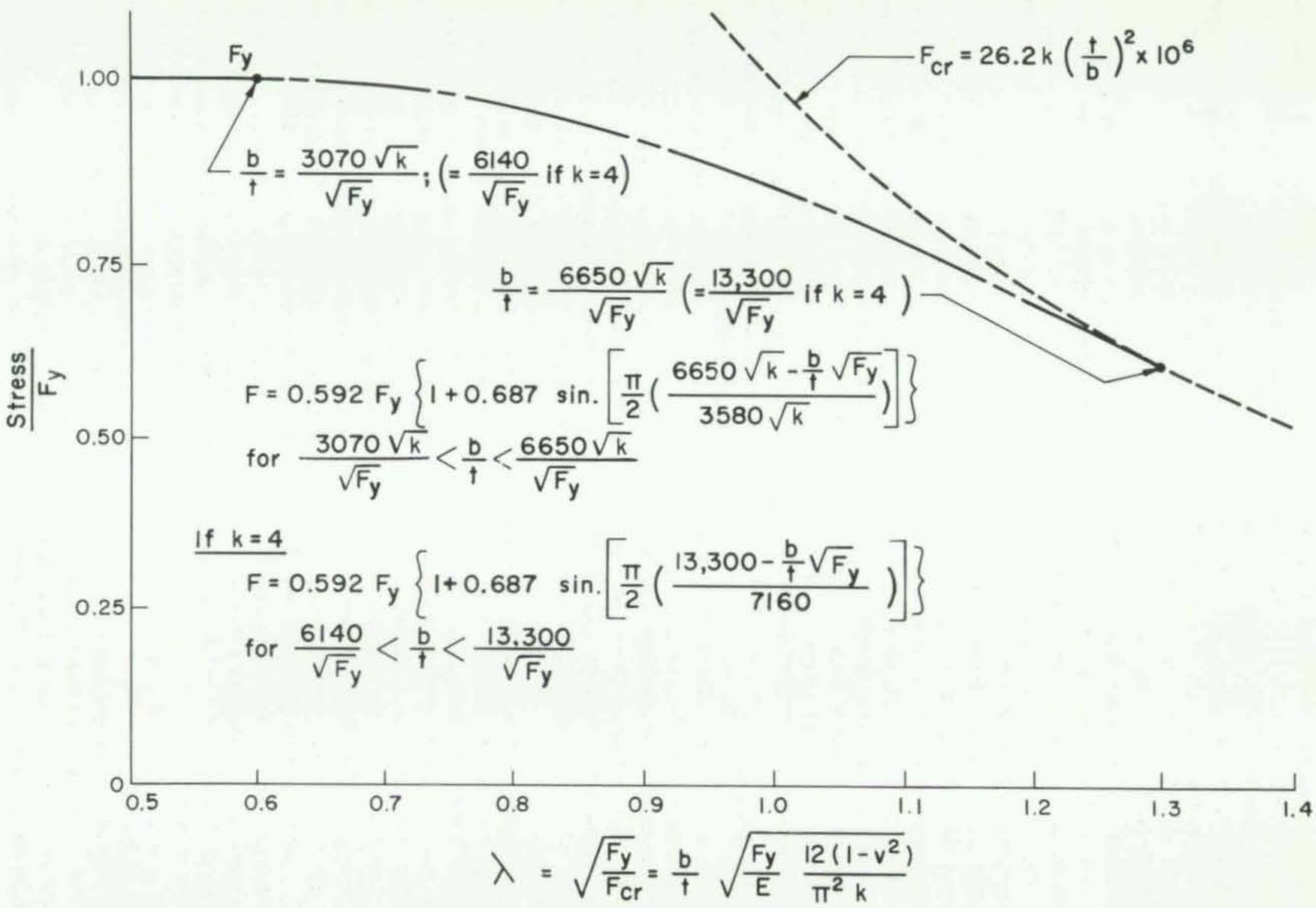

Fig. A-12–Plate Buckling Curve for Design

TABLE A-3

Number of Stiffeners	Assumed Value of "k"	$\phi = .07k^3n^4$	Value of "k"* Calculated by Elastic Theory for $I_s = \phi t^3 w$
2	1	1.1	0.91
2	2	8.9	1.91
2	3	30.3	3.03
2	4	71.8	4.15
3	1	5.7	0.90
3	2	45.5	1.98
3	3	153	3.05
3	4	362	4.12
4	1	17.9	0.89
4	2	143	1.92
4	3	482	2.89
4	4	1140	3.75
5	1	43.8	0.87
5	2	350	1.82
5	3	1180	2.65
5	4	2800	3.63

*Tabulated values correspond to $W/t = 36$. k will increase slightly for larger values of W/t.

the bottom flange plates for a combination of compression and of shear due to torsion of the girders. It was found by analytical studies that when the bridges were loaded so as to produce maximum moment in a particular girder, and hence maximum compression in the flange plate near an intermediate support, then the amount of twist in that girder was negligible. It therefore appears reasonable that, for bridges conforming to the limitations, shear due to torsion need not be considered in the design of the bottom flange plates for maximum compression loads.

For bridges whose proportions do not conform to the specified limitations, further study of the state of stress in the bottom flange should be made using one of the available methods of structural analysis. A comprehensive analysis is given in Ref. (16). A general discussion of this problem may also be found in Ref. (17).

Diaphragms

Bridges of this type can resist the applied loads effectively only if the geometry of the bridge cross section is maintained at the supports. This is the function of the diaphragms or cross-frames located at the supports; it is essential that they are within the box girders. Diaphragms or cross-frames between the box girders may be omitted if movement of the box girders, both translational and rotational, is prevented by some other means.

Intermediate diaphragms or cross-frames are not required. This is because the design loads per girder, Equation 2.2.4-2, are based on a study of the behavior of bridges without intermediate diaphragms. If intermediate diaphragms or cross-frames are provided, the conservatism of the design will be increased unnecessarily.

In order to maintain the geometry of the box girder section during fabrication, hauling, erection and placement of the deck, it may be necessary to provide removable or construction bracing until the deck is completed.

Supplementary Information

Secondary Bending Stresses. – When box girders of the type under consideration are

TABLE A-4 – SUMMARY OF BRIDGES ANALYZED FOR SECONDARY STRESSES

Loading	Truck in Lane 1 Only				Trucks in Lanes 1 and 2			
Span (Ft.)	50	75	100	150	50	75	100	150
1. 3-lane, 2-girder bridges		X						
2. 6-lane, 4-girder bridges	X		X	X	X		X	X
3. 2-lane, 2-girder bridges		X		X				
4. 3-lane, 3-girder bridges		X		X		X		X
5. 4-lane, 4-girder bridges		X		X		X		X
6. 4-lane, 5-girder bridges	X		X	X	X		X	X

subjected to eccentric loads, their cross section becomes distorted. This distortion gives rise to secondary bending stresses, which are at maximum at the corners of the section. An analytical study was made of the secondary bending stresses due to distortion in bridges of this type. It was found that for bridges having proportions conforming to the limitations of Articles 2.2.4, the stresses due to the secondary bending moments were within reasonable limits and need not be considered in design. Bridges having proportions which do not conform to these limitations should be further analyzed and the maximum range of secondary stress due to distortion should be calculated.

The bridges considered in the study referred to above are listed in Table A-4. All these bridges were analyzed for the loading conditions that produce the greatest distortions of the various girders, namely, one or two lanes loaded at the same positions along the span and in extreme eccentric locations in the width of the lanes. For these cases the maximum distortional stresses in the webs, the thinnest and most highly stressed members, ranged from approximately 3,000 psi to 6,000 psi for the various bridges. Loading the opposite side of the bridge produces some reversal of stress, and therefore the range of stress is of interest for evaluation of possible fatigue effects. The total range of stress, considering the worst possible sequence of loading, varies from approximately 3,000 psi to 11,000 psi. The maximum stresses and the maximum range of stresses occur in the center girder of those bridges with an odd number of girders. These stresses are within acceptable limits providing transverse bending stresses due to supplementary loadings, such as utilities, are restricted as specified.

In designs not meeting the limitations of these Criteria, cross-section distortion may be a problem. The resulting secondary stresses can be reduced by the introduction of intermediate diaphragms or cross-frames within the girders.

Box girder bridges experience vibration and impact with resulting dynamic stresses due to the passage of moving vehicles in much the same way as other types of bridges of comparable span. The usual AASHO impact formula is applicable. However, if wide horizontal plate elements are used in the bridge section, local plate vibrations may be excited by the overall motion of the bridge. An analytical study was made of the stresses in the plates caused by these vibrations, in bridges whose proportions conform to the specified limitations. The bridges chosen for study were those in which vibration would be most severe. The maximum stresses due to vibration were found to be moderate, being of the order of 3,000 psi in tension or compression. When the limitations of Article 2.2.4 are met, secondary stresses due to vibration need not be taken into account explicitly in the design because of the following considerations:

1) These stresses are at maximum at the centerline of the bottom flange and at the web-bottom flange connection, while the maximum secondary stresses due to distortion occur at the web-top flange connection.

2) Maximum vibration stresses occur in the edge box girder which has the largest vertical deflections, while maximum distortional stresses occur in an interior girder.

3) The largest distortional stresses occur when vehicles are in the outer lane and the lane adjacent to it; under such loading it is unlikely that the two vehicles could be exactly in phase and hence produce a critical dynamic effect. Therefore, maximum dynamic stresses are not likely to coincide with maximum distortional stresses.

4) The amplification factors used in the study are based on a steady-state response and are, therefore, conservative.

If the proportions of the bridge do not conform to the specified limitations, then an analysis of the dynamic behavior of the bridge may be desirable.

Flange to Web Welds. – Because of the possibility of secondary bending stresses developing in the box girders as a result of vibrations and/or distortions, it is essential that the web-flange welds be of sufficient size to develop the full web section. The maximum specified transverse bending stresses will then result in a section with adequate fatigue resistance, even though fillet welds are employed for the web-flange welds [14].

2.2.5 Shear Connector Strength

The design of shear connectors follows the 1966-67 Interim AASHO Specifications. The basis for these provisions is given in Reference 2.

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2.3 HYBRID BEAMS AND GIRDERS

Many theoretical and experimental studies, summarized in Reference 1, showed that the bending behavior of a hybrid girder differs very little from that of a homogeneous girder of the flange steel. This behavior is illustrated in Figure A-13, which shows the theoretical load-deflection curves for two girders of equal dimensions; both girders have A514 steel (yield strength: 100 ksi) flanges but one has an A514 steel web while the other has an A36 steel (yield strength: 36 ksi) web. Although web yielding occurs at a low load in the hybrid beam, this web yielding is controlled by the unyielded flanges and has little effect on the load-deflection curve. The load-deflection curve of either girder deviates significantly from a straight line only when yielding starts in the flanges.

Therefore, the design of noncompact hybrid sections can be based on the moment causing initial yielding of the flanges rather than of the web. This can be conveniently accomplished by applying a small reduction factor, which accounts for the effect of web yielding, to the yield moment of a homogeneous section of the flange steel.

Since the maximum bending strength for a compact section is based on the plastic moment, early web yielding in a hybrid section has no effect on the maximum bending strength and the conventional formula for the plastic moment can be applied to hybrid sections.

The studies summarized in Reference 1 also showed that web yielding in a hybrid girder

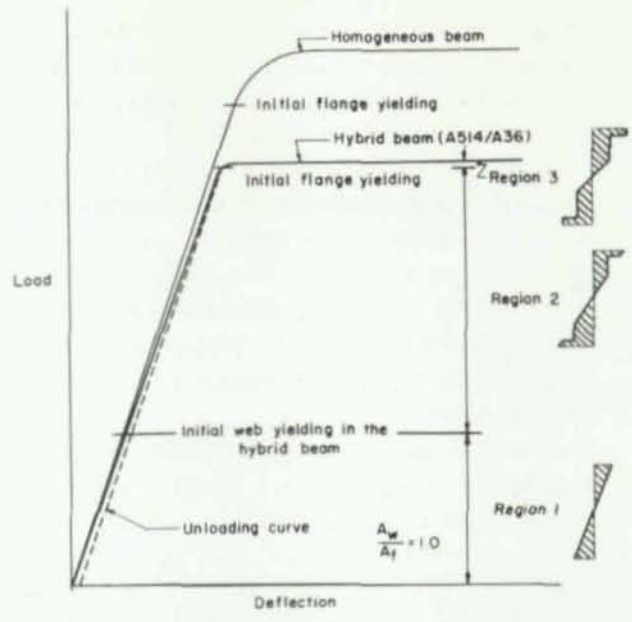

Fig. A-13 – Typical Load-Deflection Curve for Hybrid Beams.

has little effect on width-to-thickness limitations or stiffener requirements. However, sufficient information is not available to allow the preparation of design rules on the basis of postbuckling strength of the web. Therefore, the design of stiffened girder webs is based on elastic buckling.

REFERENCES

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2.4 COMPRESSION MEMBERS

2.4.1 Axial Loading

A. *Maximum Capacity*

The first of the column equations represents the case of column buckling after some portions of the column have already begun to yield and is the "CRC column strength equation" [1]. It applies to short and moderately long columns. The second equation is the well known Euler formula for elastic buckling used for long columns.

The CRC formula is of the same form as the column equations in the AASHO Specifications, Article 1.7.1. The advantage of the CRC formula is that it can account for any end restraint.

The determination of the buckling stress is influenced by many factors, such as residual stress, initial crookedness, etc., and many procedures are available for a more precise determination of the buckling stress [1] [2] [3]. The CRC formula represents a simple average curve which has been shown to give a fair representation of the strength of many different types of steel columns (see Figs. 9.23, 9.24 and 9.25 in Ref. 2). The CRC formula is in several other specifications [4] [5].

B. *Effective Length*

The slenderness ratio KL/r is an artifice which reduces the calculation of the buckling stress of columns in a framed or truss structure to the calculation of the buckling stress of an equivalent pin-ended member. For a member with ends prevented from translation (such as truss compression member) the effective length may vary from one-half to the full length of the column [1]. In such a case the traditional AASHO effective length factor of $K = 0.75$ for riveted, bolted or welded end connections and $K = 0.875$ for pinned ends have been retained. In the case of columns with ends which may translate with respect to each other, the effective length exceeds the actual length, and thus the use of the AASHO effective length factors of 0.75 and 0.875 is unconservative. There are many methods available for determining this factor [1], but the simple nomograph recommended by the CRC is conservative and easy to use (see Fig. 2.21 in Ref. 1).

2.4.2 Combined Axial Load and Bending

A. *Maximum Capacity*

The interaction equations for checking the adequacy of the beam-column represent a major departure from the procedures in the AASHO Specifications. The AASHO beam-column procedure is based on the limiting condition of reaching the yield stress in the most stressed fiber of the member. The ultimate load is, however, not reached unless some portions of the beam-column have yielded [6]. The following objections to the elastic approach (also called the secant formula method) are listed in Ref. 1:

1. It cannot be applied rationally to beam-columns with a non-linear stress-strain curve.
2. The effect of residual stress cannot rationally be taken into account.
3. Design on the basis of initial yield may be over-conservative in certain cases, for example, for an I-shaped member having large end eccentricities and subject to bending about the minor axis.
4. For the I-shaped column that is bent about the major axis and is laterally unsupported in the weak direction, a separate lateral buckling check must be made.

The interaction equations overcome all of these objections [1]. In the range, however, in which the secant formula method applies, both procedures will give about the same result [1] [7]. The interaction equations are thus more versatile and have a broader scope of application. They are not, on the other hand, rational expressions, but they provide an empirical transition between the two extreme conditions of zero axial force (beam) and zero moment (column).

The validity of the interaction equations for steel beam-columns has been amply substantiated by comparing them to more complex exact procedures and to test results. Some of these comparisons are documented, discussed and further referenced in Refs. [1], [6], [7] and [8].

The first of the interaction equations represents a measure of the stability of the member, and the second equation insures that the plastic moment of the section is not exceeded. This latter equation is a conservative form of the more exact equation [6]:

$$M_u = 1.18 M_p \left[1 - \frac{P_u}{A_s F_y} \right]$$

It should be noted that F_e in the first equation is computed for the effective slenderness ratio in the plane of the applied moments.

B. *Equivalent Moment Factor*

The equivalent moment factor accounts for the less severe cases of loading when the moments are not equal. More refinement is possible, but has been omitted in the interest of simplicity [1]. The rules given here are conservative.

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2.5 SPLICES, CONNECTIONS & DETAILS

2.5.1 Connectors

A. *General*

To assure that the maximum strength of the bridge is limited by the strength of members rather than by the strength of connections and to account for the greater variability in the strength of connections, a reduction factor, ϕ, is introduced into the design of connectors. The strength of connectors used in the design is obtained as the product of the reduction factor ϕ and the experimentally determined maximum strength F.

The ϕ factors are listed in Table A-5. A uniform value of 0.75 was selected for mechanical fasteners under all loading conditions. Only the low-carbon steel bolt and fillet welds were assigned lower values in recognition of the greater variability of the test data. Tensile loading of mild steel bolts may often result in thread stripping before development of the tensile strength. Fillet welds are subject to greater variability during fabrication and placement. When the yield point of the connected material governs the maximum strength, the reduction factor ϕ was taken as 1.0.

B. *Welds*

In groove welds, the maximum forces are usually tension or compression. Tests have shown that groove welds of the same thickness as the connected parts are adequate to develop the full capacity of the connected parts [1].

The ultimate strength of fillet welds subjected to shear alone is dependent upon the strength of the weld metal and the direction of applied load which may be parallel or transverse to the weld. In both cases the weld fails in shear, but the plane of rupture is not the same. Tests have shown that the ultimate strength of fillet welds based on the minimum throat area is 70 - 75% of the tensile strength of the deposited metal [1] [2] [3].

It was early recognized that shear yielding was not critical in welds, as the material strain hardened without large overall deformations occurring. Therefore the suggested unit stress for fillet welds, $F_y = 0.45\ F_u$, is based on the shear strength of the weld material and the application of a suitable factor ($\phi = 0.64$) to insure that the connected part will develop its full strength without premature failure of the weldment.

The minimum strength of the welding rod metal, F_u, indicated in Table A5, can be conservatively taken as the classification number (EXX). The letters XX stand for the various minimum strength levels (60, 70, 80, 90, etc.) of electrodes in ksi.

TABLE A-5

Type of Fastener	Maximum Strength, F	ϕ	ϕF
Groove Weld	1.00 X Yield Point[1]	1.00	$1.0 F_y$
Fillet Weld	0.7 X Tensile Strength[2]	0.64	$0.45 F_u$
Low-Carbon Steel Bolts, ASTM A307			
Tension	0.75 X Tensile Strength[3]	0.67	$0.50 F_u$
Shear[4]	0.60 X Tensile Strength[3]	0.75	$0.45 F_u$
Power-Driven Rivets			
Structural Steel Rivet ASTM A502 Gr. 1			
Shear	0.60 X Tensile Strength[5]	0.75	$0.45 F_u$
Structural Steel Rivet ASTM A502 Gr. 2			
Shear	0.60 X Tensile Strength[6]	0.75	$0.45 F_u$
High-Strength Bolts ASTM A325			
Tension	0.75 X Tensile Strength[3]	0.75	$0.56 F_u$
Shear (Bearing-Type)[4,7]	0.60 X Tensile Strength[3]	0.75	$0.45 F_u$
ASTM A490 (When adopted by AASHO)			
Tension	0.75 X Tensile Strength[3]	0.75	$0.56 F_u$
Shear (Bearing-Type)[4,7]	0.60 X Tensile Strength[3]	0.75	$0.45 F_u$

1 – Of connected material.
2 – Minimum strength of the welding rod metal but not greater than the tensile strength of the connected parts.
3 – ASTM minimum tensile strength.
4 – When a shear plane intersects the bolt threads, the root area shall be used.
5 – F_u = 55,000 psi.
6 – F_u = 67,000 psi.
7 – Bearing stresses in bearing-type connections shall not exceed the tensile strength of the connected material.

If fillet welds are subjected to eccentric loads that produce a combination of bending and shearing stresses [13] [14], they must be proportioned on the basis of a direct vector addition of the stresses.

The results of tests on vertical weld groups (E60 electrodes) subjected to combined bending and shear are plotted in Fig. A-14. These tests show clearly that the suggested weld stresses provide an ample margin of safety against premature weld failure. Also, it is readily apparent that the direct vector addition of the shear forces on the weld is a conservative approach to the design of eccentrically loaded fillet welds.

C. *Bolts and Rivets*

For greater convenience in the proportioning of bolted connections, the maximum unit tension stresses are given in terms of the nominal diameter of the bolt. The shear strength of bearing-type high-strength bolts is determined by the location of the shear planes. If a shear plane intersects the bolt threads, only the root area is effective in resisting the shear.

For low-carbon steel bolts in tension, the maximum stress is limited to the tensile strength of the steel applied to the stress area of the threaded portion. The ratio of the stress

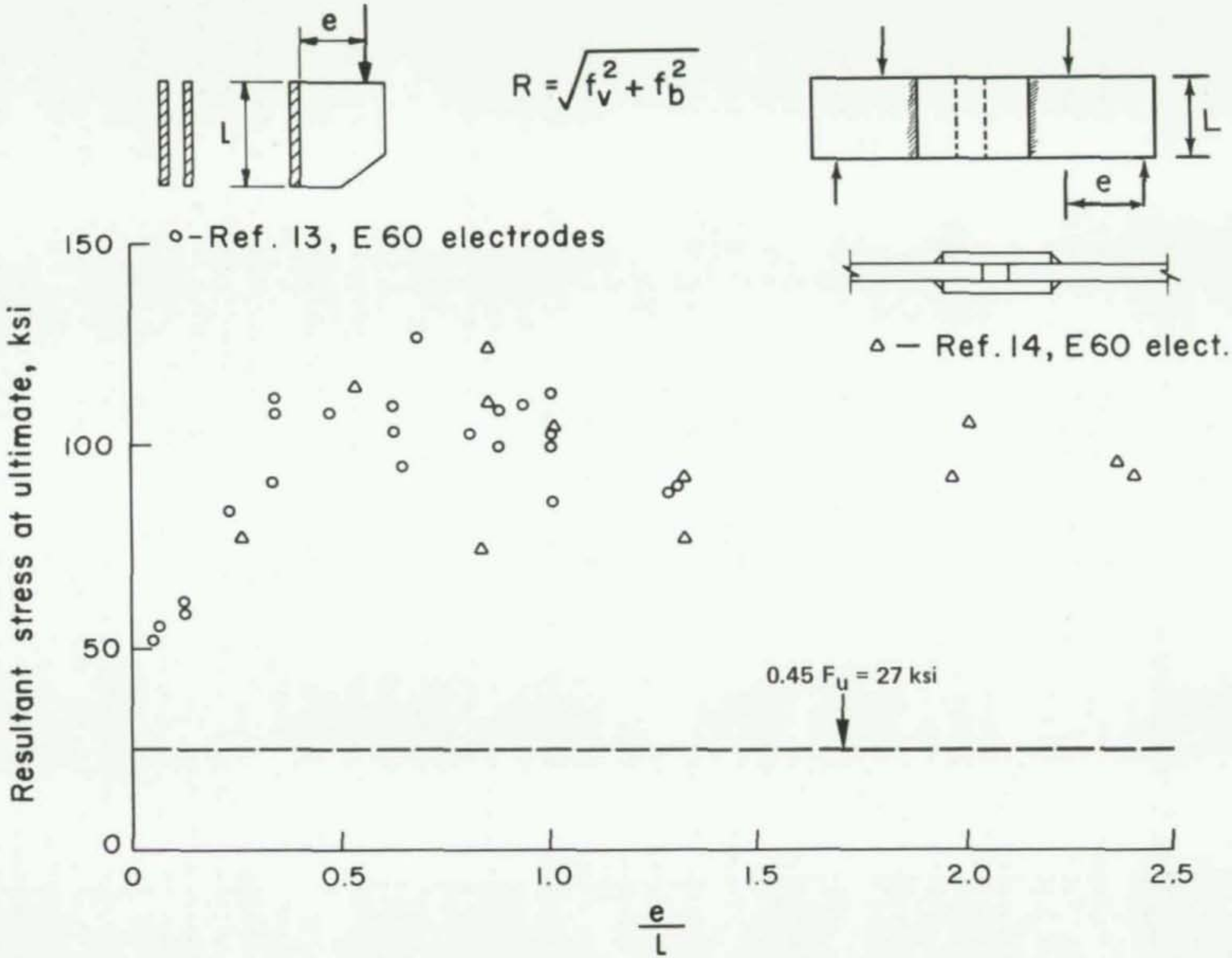

Fig. A-14 – Fillet Welds Subjected to Bending and Shear area to the nominal bolt area for ½ to 1 in. bolts varies from 0.725 to 0.773. Using an average value of 0.75, the maximum strength of low-carbon bolts may be expressed as 0.75 F_u.

For high-strength bolts in tension, the maximum stress is limited to the ultimate tensile strength of the steel applied to the stress area, i.e., 0.75 F_u. The ϕ factor, selected as 0.75, provides a 25% overstrength [5] and, at the same time, assures that the actual bolt preload will not be exceeded even under the maximum design loads specified in Article 1.7.

It is of interest that the product of the current allowable tensile stress for high-strength bolts (Article 1.7.5, AASHO Specifications) and the usual factor of safety, 1.83, is 0.54 F_u while the corresponding value in Table A-5 is 0.56 F_u.

The maximum shear strength for bearing-type connections fastened with low-carbon steel bolts, power-driven rivets and high-strength bolts was obtained by setting the average ultimate shear strength at 60% of the tensile strength of the bolt.

A ϕ value of 0.75 yielded shear stresses for the low-carbon bolts and power-driven rivets comparable to those obtained by factoring the currently used allowable shear values by 1.83. The same value was selected for high-strength bolts. The studies reported in Ref. 7 have shown that the resulting stresses are directly comparable to those used for the lower strength fasteners with ample reserve strength provided. These studies have been confirmed by an extensive test program.

Figure A-15 summarizes the shear strength of A325 bolts. The average shear strength of a bolt is shown for joints of varying lengths made of A325 bolts and of plates of A36 or A440 steels. The ratio of the net area of the plate, A_n, to the total shear area of the bolts, A_s, was slightly different for the two grades of steel. The two heavy lines show clearly that the average strength of a bolt is the highest in a joint with a single bolt and decreases as the length of the joint increases. It is readily apparent that adequate reserve strength is available at the maximum design level (.45 F_u) to insure the development of the strength of the connected parts.

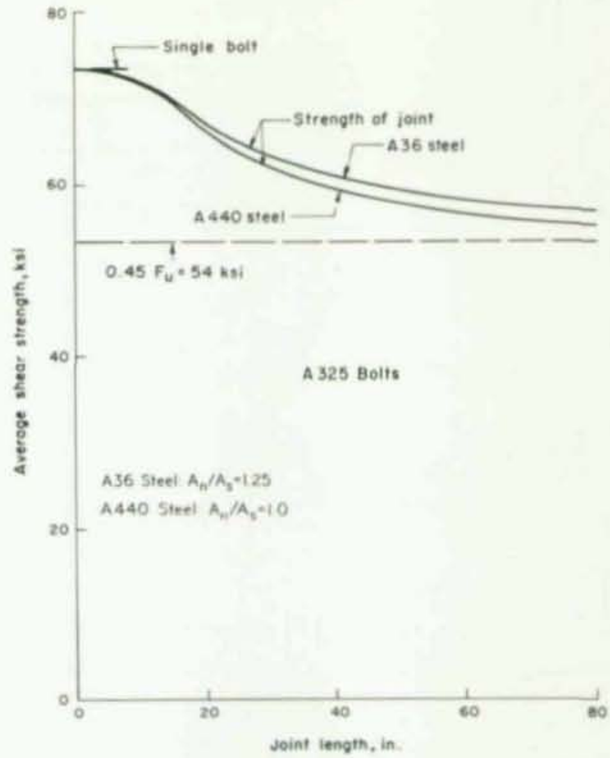

Fig. A-15–Effect of Joint Length on Strength of A325 Bolts

A comparison of the design values permitted for rivets with the shear strength of riveted joints as computed in Ref. 7 shows the reserve shear strength of the high-strength bolted joints is directly comparable to the strength of the riveted joints.

For rivets and high-strength bolts in bearing-type connections that are subjected to tension and shear, studies reported in Ref. 8 showed that the ultimate strength of rivets and bolts can be represented by an ellipse. The test results for rivets, bolts with threads excluded from the shear plane, and bolts with threads in the shear plane are compared in Fig. A-16 with the interaction curve suggested for design. It is apparent that good agreement exists. For bolts with threads in the shear plane, the applied stresses were computed on the root area; it can be seen that the root area provides an adequate measure of resistance when a shear plane intersects the bolt threads.

2.5.2 Connections

A. Splices

Tests [5] [9] [10] have shown that flexural members can be proportioned on the basis of

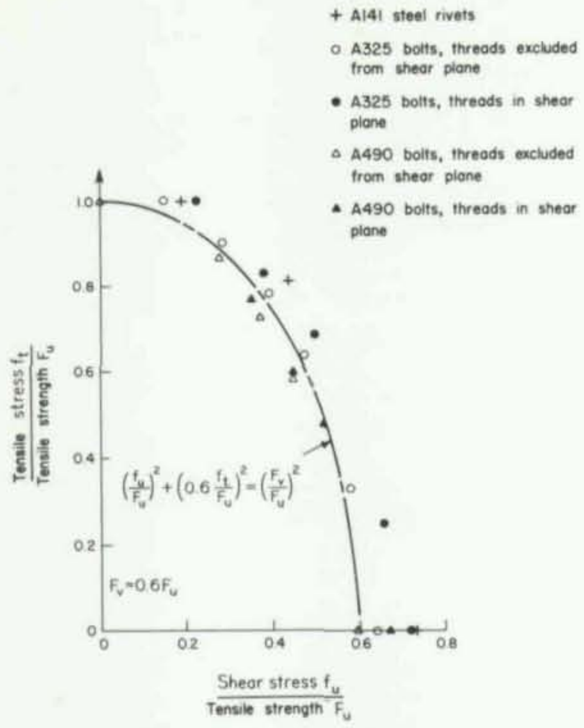

Fig. A-16–Interaction Curves

the gross cross section except when the holes comprise a large percentage of the section. Figure A-17 compares the behavior of bolted beam splices and shows clearly that the full plastic moment M_p was developed by both the friction type (curve 1) and the bearing-type (curve 2) connection. The bolt holes had no appreciable effect in either test even though 25% of the plastic strength was removed as indicated by the plastic moment M_{pn} at net section. This is the result of the recognized effect of strain hardening and the added splice material.

For tension members a similar behavior is experienced. An examination of the typical tension test in Fig. A-18 shows that yielding of the net section has no significant effect on the behavior; failure occurred only after yielding of the gross section. It is necessary that some reserve be available at the net section after yielding occurs on the gross section. Thus

$$\frac{F_u A_n}{F_y A_g} > 1.0$$

and

$$\frac{A_n}{A_g} > \frac{F_y}{F_u}$$

If the maximum permitted load on the net section were 85% of the tensile strength, then

$$A_n/A_g \geqslant \frac{F_y}{0.85 F_u}$$

To insure yielding of the gross section of A36 and A440 steel, the ratio of A_n/A_g must be greater than the value given by $F_y/0.85\ F_u$. For these steels, the resulting ratio of A_n/A_g will be between 0.7 and 0.84. The limiting value of 0.85 was selected. Hence, the net section is fully effective as long as less than 15% of the gross area is missing.

B. *Bolts Subjected to Prying Action*

Current specifications note that bolts required to support applied load by means of direct tension should resist the sum of the external load and any tension resulting from prying action produced by deformation of the connected parts. Recent studies have led to the development of formulas for estimating the magnitude of the force due to prying. The following semi-empirical expression was developed in Ref. 5:

$$Q = \left[\frac{\frac{1}{2} - \frac{gt^4}{30ab^2 A_b}}{\frac{a}{b}\left(\frac{a}{3b} + 1\right) + \frac{gt^4}{6ab^2 A_b}}\right] T$$

Because of the complexity of the above equation, the effects of the variables a, b, g, t and A_b were evaluated for the practical range of values. This study showed that the prying action can be approximated with reasonable accuracy using the empirical expression given by Equation 2.5.2-1. Comparisons of the simplified expression with the expression developed in Ref. 5 are given in Figs. A-19 and A-20 in which the dashed lines represent Equation 2.5.2-1 and the full lines represent the equation from Ref. 5.

Figure A-19 compares the force due to prying for various geometrical configurations and bolt sizes. The parameter b/a was varied from 1/2 to 4/3 for a constant length of T-stub, g, that is tributary to the bolt. The empirical approximation is seen to provide a conservative

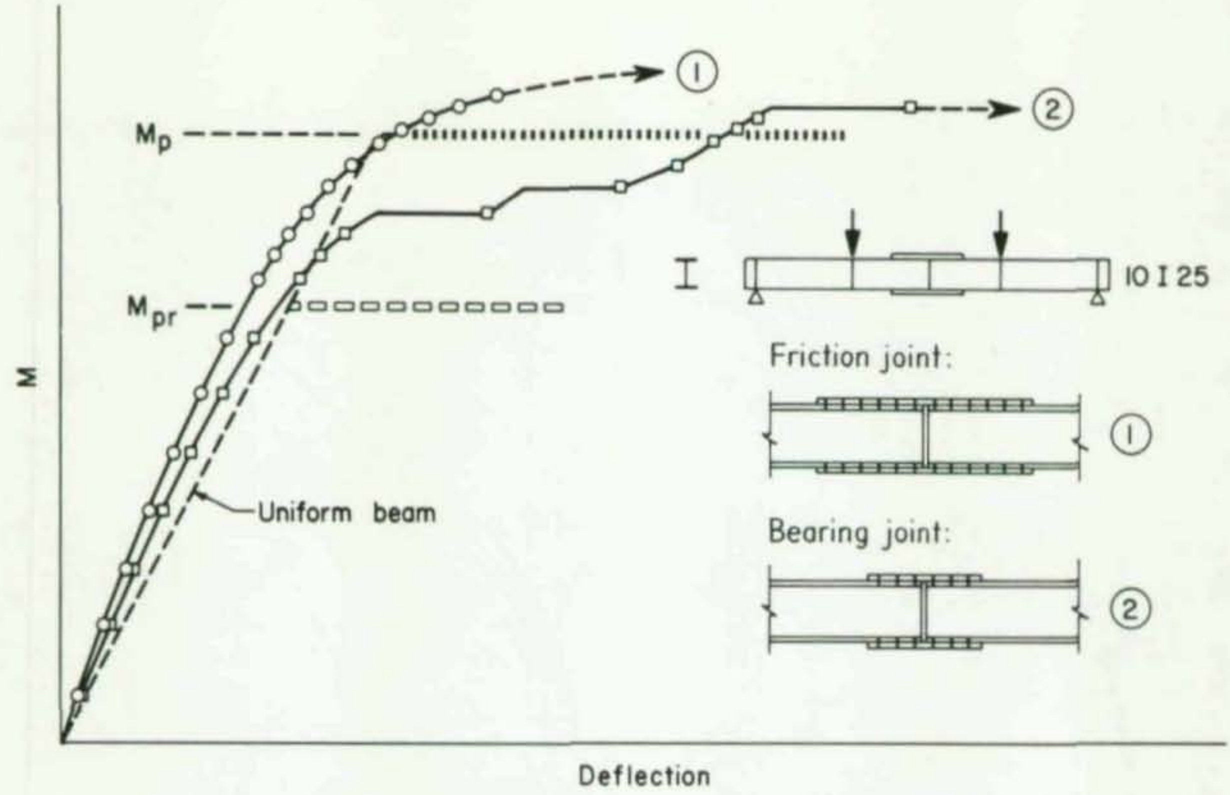

Fig. A-17–Moment Deflection Curves of Spliced Beams

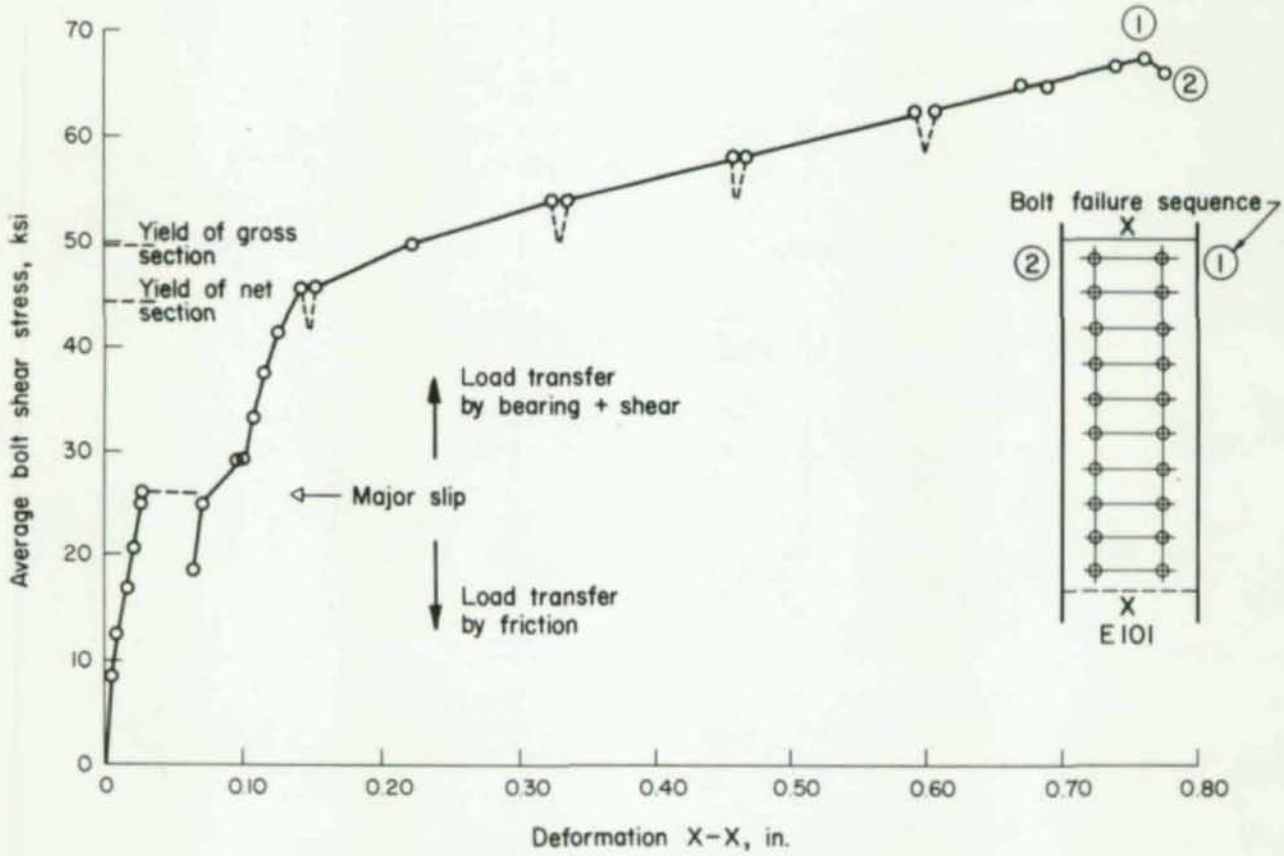

Fig. A-18 – Typical Load-Deformation Curve (A440 Steel Joint Connected by A325 Bolts)

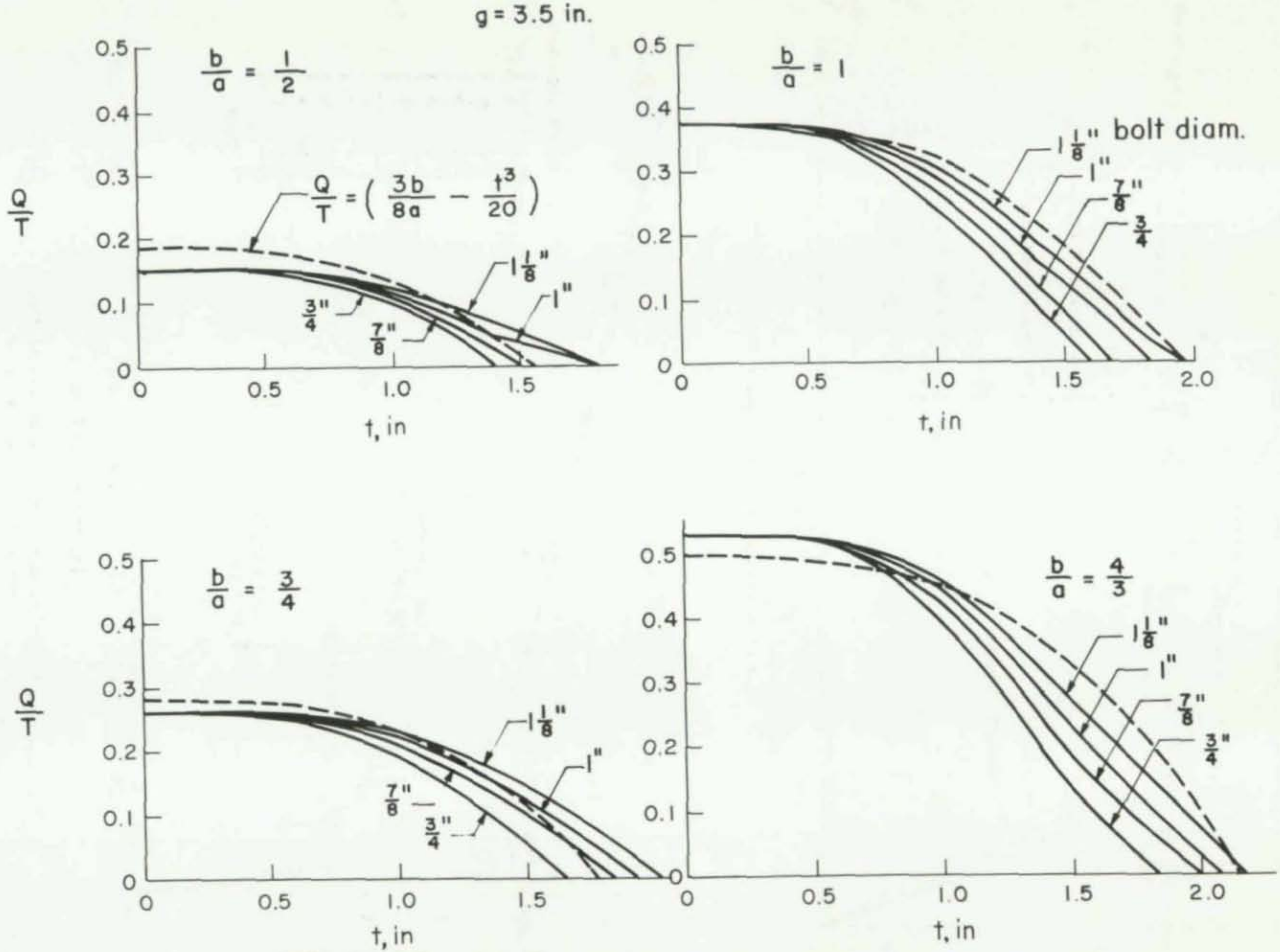

Fig. A-19–Comparison of design recommendations with semi-empirical formula

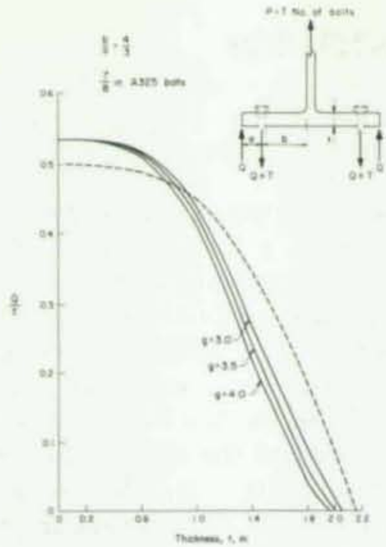

Fig. A-20 – Influence of variable g

estimate of the prying force for all bolt diameters.

Figure A-20 shows that a large variation in the value of *g* has little effect on the prying action. Hence, this variable need not be considered.

C. *Rigid Connections*

The provisions for rigid frame connections are well documented in Chapter 8 of Ref. 10. This type of connection is used in rigid frames. The provision for checking the beam or connection web insures adequate strength and stiffness of the steel frame connection.

In bridge structures diagonal stiffeners of minimum thickness will provide sufficient stiffness. Alternately, web thickness may be increased in the connection region.

The provisions for checking a member subjected to concentrated forces applied to its flange by the flanges of another member framing into it are intended to prevent crippling of the web and distortions of the flange. It is conservative to provide stiffeners of a thickness equal to that of the flanges of the other member.

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Section 3—Service Behavior

3.1 OVERLOAD

There is no question about the strength of steel flexural members. Their toughness has been well established by service conditions and demonstrated by the AASHO test road bridges [1]. There is, however, a definite need for a control on the possibility of permanent deformation under infrequent overloads which may impair the riding qualities of the bridge. The AASHO Road Test Report No. 4, Bridge Research, gives data bearing on permanent deformations.

In those tests noncomposite bridges showed permanent set under loads producing stresses below the yield strength by an amount about equal to the average residual stress in the flanges. The supporting data are shown in Table 39 on Page 68 of Report No. 4. It may be significant that beams 1-A, 9-A and 9-B showed permanent set under nominal stresses equivalent to 0.80, 0.75 and 0.77, respectively, of F_y. The permanent set usually was not significant but may be indicative of the fact that beams stressed to this level are at the border line approaching possible undesirable profiles under a number of excessive loads. Bridge 3-A, under a load stressing it to 0.90 F_y, suffered a permanent set of 3.41 in.

For a beam designed as a noncompact section with a moment equal to F_yS, the factor 1.25 in the load factor formula means that under dead load plus 5/3 live load (double live load in one lane only) the member will be stressed to $F_y/1.25 = 0.80\ F_y$, which has been taken as the reasonable upper limit for avoiding objectionable permanent set. However, if the member under consideration is a compact section with a Z value equal to 1.15 S, the moment capacity is 1.15 F_yS, and the member will be stressed to 1.15 x 0.80 F_y = 0.92 F_y. For such beams the design will be governed by the overload provision.

The effect of overload on composite beams is different. In the AASHO bridge tests only Bridge 2-B of the composite bridges showed permanent set at a stress below F_y. It had a set of 0.67 in. at a stress of 0.88 F_y.

The permanent sets at midspan measured at the AASHO Road Test are plotted in Figure A-21 against the ratio of maximum test stress to the yield stress. The test stresses include the dead load stress and the stress measured during the passages of the test vehicles. The permanent sets are the totals accumulated during the full period of test traffic on any one bridge, varying from a low of 392,400 (Bridge 3A) to a high of 558,400 (Bridge 2B) passages. The difference between the permanent set of composite and noncomposite bridges is evident. In recognition of this difference the Criteria permit a computed stress up to 95% of F_y for composite beams under an overload.

3.1.3 Friction Joints

The maximum shear values for friction-type joints have been selected so that under permissible overloads the joint is just at the limit of slip. The Research Council for Riveted and Bolted Structural Joints recognizes a slip factor of 0.35 as representative of values likely to be encountered in actual construction. Hence, under permissible overload the limiting shear stress is

$$F_v = \frac{0.35 \text{ x minimum bolt tension}}{A_b}$$

or

$$F_v = \frac{0.35 \text{ x } 0.7 \text{ x } F_u \text{ x } A_s}{A_b}$$

For A325 bolts, this yields

$$F_v = 0.35 \text{ x } 0.7 \text{ x } 115 \text{ x } 0.76 = 21 \text{ ksi}$$

for A490 bolts

$$F_v = 0.35 \text{ x } 0.7 \text{ x } 150 \text{ x } 0.76 = 28 \text{ ksi}$$

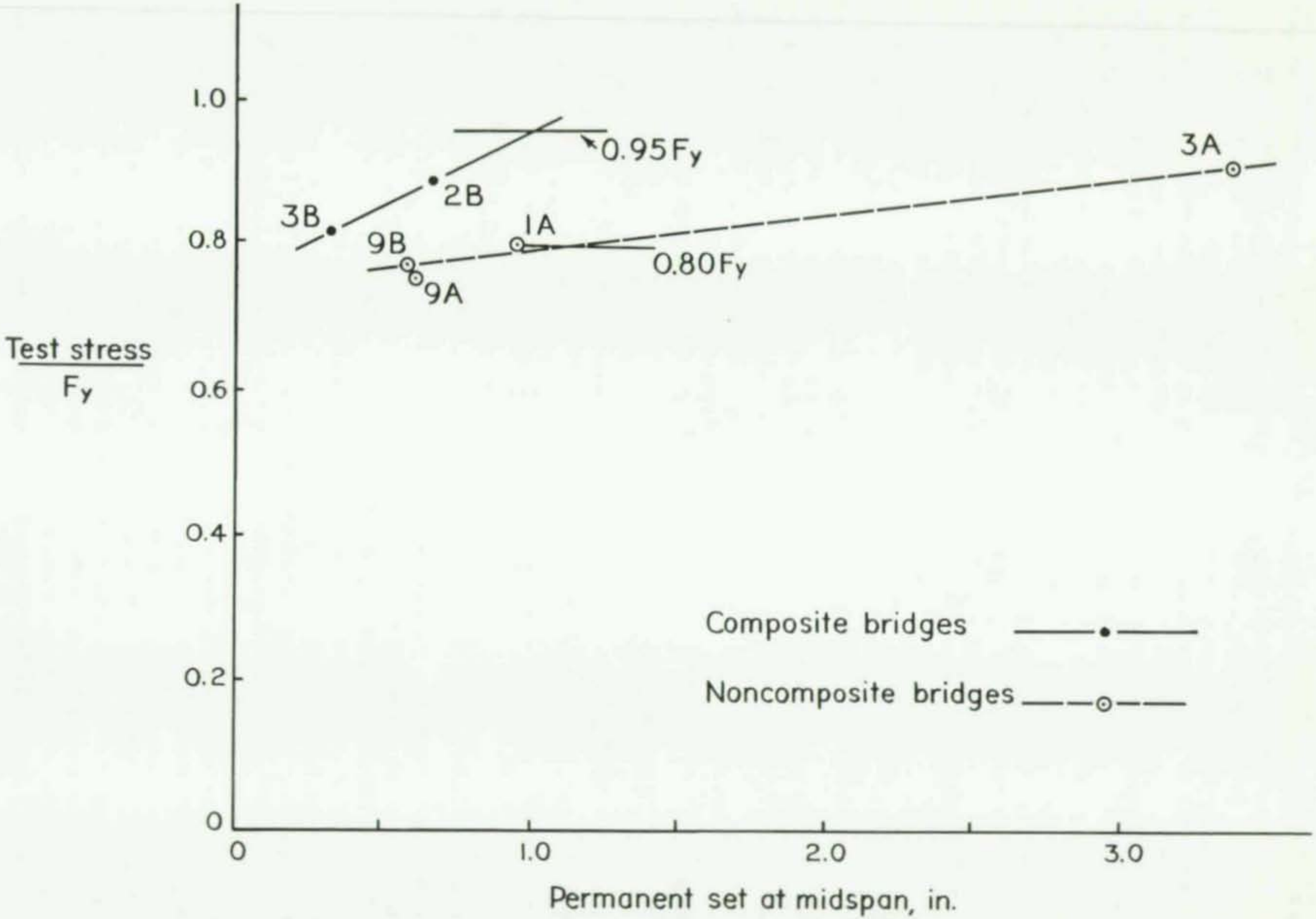

Fig. A-21–Permanent Set of AASHO Road Test Bridges

When a friction-type joint is loaded by a tensile component P, the clamping force is reduced to T_i-P and the frictional resistance is also reduced. Because frictional resistance is proportional to the bolt clamping force, the allowable shear is also proportional to the change. Hence

$$\frac{f_v}{F_v} = \frac{T_i - P}{T_i}$$

$$f_v = F_v\left(1 - \frac{P}{T_i}\right)$$

$$= F_v\,(1 - f_t A_b / 0.70 \times F_u \times A_s)$$

$$= F_v\,(1 - f_t / 0.70 \times F_u \times A_s / A_b)$$

$$= F_v\,(1 - f_t / 0.53 \times F_u)$$

where f_v = reduced shear stress,
T_i = initial bolt tension,
f_t = tensile stress due to applied load
A_b= nominal bolt area
A_s= stress area of a bolt

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3.3 DEFLECTION

Historically [1] it appears that the primary purpose of limiting depth-span ratios was to limit live load deflection. The deflection limitations were introduced into the specifications to reduce vibration in highway and railway bridges. They were not very restrictive for the materials and allowable stresses in use at that time. Observations through the years have cast doubt on the optimum level or even the effectiveness of deflection limitation. This and the increasing restrictiveness of such limitations when applied to design with high-strength materials have led to a reappraisal of the causes, effects and control of bridge vibration.

Extensive and varied field and laboratory tests as well as theoretical studies have shown that the causes and effects of bridge vibration are very complex. Objectionable vibration cannot be consistently prevented by a simple deflection limitation alone. On the other hand, little if any damage to the structure can be attributed to vibration, except perhaps when the bridge has been previously damaged due to other causes such as a badly cracked or loosened concrete deck. The objection to vibration arises only from the response which it induces in persons on the bridge or in stationary vehicles on it.

A perceptible vibration is set up when a smoothly rolling load passes across an elastic beam. This may be considerably amplified if the relation of speed to span is such as to cause resonance or sub-harmonic excitation of a natural mode of oscillation of the bridge. Further excitation may be caused by roughness of the deck or approaches, resonant oscillation of the sprung and unsprung parts of the vehicle, and other factors. Investigators at the University of Illinois [2, 3] and at the Massachusetts Institute of Technology [4] have studied this and are able to predict these vibrations with engineering accuracy, both theoretically and with models.

Specifications have tried to minimize the vibrations by limiting the deflection. When the stiffness of the bridge is increased, there is less deflection per unit load. At the same time, the increased stiffness results in a greater "impact factor". At some limiting value of stiffness, the deflection per unit of live load becomes asymptotic to a horizontal line. D.T. Wright [5] showed that if the total stiffness of the bridge exceeded 200 kips per inch deflection (load placed and deflection measured at midspan), the median amplitude factor was about 0.00050 inches per kip of live load. Increased stiffness above this value reduced the mean amplitude factor but little.

Frequency and damping are parameters which affect the human response to the vibration as well as the amplitude. The general range of frequency at which a bridge vibrates is from 2 to 7 cycles per second. The damping is usually 1 to 2% of the critical damping (logarithmic decrement of 0.06 to 0.13). In this range of frequency, Janeway [6] has indicated that the human responds directly to the change in acceleration or jerk rather than to amplitude, velocity, or acceleration. Reiher and Meister [6] and Goldman [6] have shown results of subjective tests on humans which are in reasonable agreement and indicate the response to change of acceleration previously mentioned is correct. This would indicate that a bridge designed with a total stiffness of above 200 kips per inch would probably have perceptible vibrations when a vehicle weighing 20 kips passed over if all axles or wheels were responding together. Increasing the stiffness of the bridge would not decrease the amplitude of vibration sufficiently to remove it from the perceptible range.

The vibration would be sensed by a standing or sitting subject looking for the vibration. The question of the use and location of the bridge must then be evaluated. Moving pedestrians would probably not sense the vibration. Passengers in moving vehicles would not feel the vibration. People in parked vehicles would probably sense the vibration only if the frequency of the bridge was close to the natural frequency of the vehicle. Thus, it would seem that the use of the bridge should control specifications in regards to vibration. Vibrations will probably be sensed only on bridges with pedestrian traffic.

Unfortunately, coupled with the sensing of vibration is a psychological effect. The human tends to exaggerate any movement or vibration. Engineers who have investigated blasts, sonic booms, and bridge or building vibration feel that this magnification factor seems in the order of 100 to 1.

The response of humans to vibration can be reduced if sufficient damping is present [7]. If a vibration is damped to a small amplitude in less than 10 cycles, the human will respond at a reduced scale. With amplitudes of about .010 inches, the human will not sense them if they are damped to about .001 inches in 5 cycles or less. This requires damping of 7.5% of the critical or more, but there normally is only 1 to 2% of critical damping in a bridge. Successful vibration dampers have been devised but usually the cost has been considered too high for most installations. Recent work with viscoelastic material in buildings has indicated an acceptable method of introducing the necessary damping. Thus, vibration can be eliminated if it is economically feasible to do so.

From the previous discussion it has been suggested that the specifications for the deflection limitation and depth-to-span ratio of bridges might be altered to reclassify bridges in three categories with the following restrictions:

1. Bridges restricted to vehicular traffic should have stress restrictions only. The bridge need not be designed to minimize vibrations for the occasional emergency stop or for workmen.

2. Bridges in urban areas with moving pedestrian traffic and parking. A minimum stiffness of 200 kips per inch of deflection to practically minimize the vibrations.

3. Bridges with benches, fishing, or other loitering pedestrian traffic. A minimum stiffness of 200 kips per inch of deflection, plus damping of 7.5% critical damping of the bridge, to eliminate vibrations.

While suggestion 1 is considered to have merit it should receive further study.

Suggestions 2 and 3 would increase the stiffness and cost of some types of bridges for spans greater than 100 ft. This is not considered warranted by the degree of improvement that might result.

Pending further investigations no change in the AASHO Specification is recommended at this time.

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Comparative Designs

INTRODUCTION

A study was made to compare steel highway bridges designed in accordance with the Tentative Criteria for Load Factor Design of Steel Highway Bridges with bridges designed using the ninth edition of the AASHO Standard Specifications for Highway Bridges.

The comparative study involved 15 representative bridges of the following types:

Simple spans
- Rolled beam, noncomposite
- Rolled beam, composite, with cover plates
- Welded girder, composite

Two span continuous
- Rolled beam, composite, with cover plates
- Welded girder, noncomposite
- Welded girder, composite

Three-span continuous
- Welded girder, composite

Five-span continuous, hinges in center span
- Welded girder, composite

The comparative designs were made by Richardson, Gordon, and Associates, Consulting Engineers.

NOTES ON INDIVIDUAL DESIGNS

Depths, arrangement of cover plates or flange transitions, and grades of steel have been kept the same as in the conventional designs.

Design No. 1–40 ft. Simple Span Composite Rolled Beam with Cover Plate

The choice of rolled beam and cover plate in the load factor design was controlled by service behavior:

$$0.95\,F_yS \geqslant D + \frac{5}{3}(L + I)$$

The strength of the section was more than adequate since it could be considered compact. Since fatigue stresses are calculated in the same manner for elastic and load factor designs, fatigue becomes more critical in the load factor design due to the lighter section used. Cover plate cutoffs are 4 ft.–6 in. from the supports in the load factor design vs. 6 ft.–6 in. in the elastic design. The cover plate cutoff location also is compatible with ultimate strength of the bare beam section and first yielding at service overload.

Design No. 2–51 ft. Simple Span Noncomposite Rolled Beam

The beam section for this bridge is compact and noncomposite, governed by service behavior:

$$0.80F_yS \geqslant D + 2.2\,(L + I)$$

Since the structure was designed for H15-44 live loading, provision for an infrequent heavy load, $2.2\ (L + I)$, was made. The overload governed the design; however, its effect was reduced for an interior stringer due to the AASHO rules for distribution of loads to stringers designed for one or multiple traffic lanes. For example:

Distribution to interior stringer, two or more lanes, conventional H15-44

$$= \frac{S}{5.5}\,;\ M = 1.25 \times \frac{5}{3} \times \frac{S}{5.5} \times M_{L+I}$$

$$= 0.379 \times S \times M_{L+I}$$

Distribution to interior stringer, one traffic lane, infrequent heavy load

$$= \frac{S}{7.0}\,;\ M = 1.25 \times 2.2 \times \frac{S}{7} \times M_{L+I}$$

$$= 0.393 \times S \times M_{L+I}$$

Design No. 3–65 ft. Simple Span Noncomposite Rolled Beam

The same comments as were made for Design No. 2 pertain to the interior stringers in this design. The infrequently heavy load distributed according to AASHO Specifications, however, increases the live load on an exterior stringer by the factor

$$\frac{2.2}{5/3} \times \frac{5.5}{5.83} = 1.25$$

For this reason the savings in material are smaller for an exterior stringer.

Design No.4–81 ft. Simple Span Composite Rolled Beam with Cover Plate

Since the working stress designs for this structure were somewhat understressed, they were remade to provide a more valid comparison with the load factor designs.

Interior Stringer:

Again, the compact section puts design on the basis of service behavior, $0.95\,F_yS \geqslant D+2.2\,(L+I)$. The high savings afforded by this design are primarily due to the high ratio of dead to live load (H15-44 Overload). The load factor design results in a very efficient section, a 33WF118 with a 10-1/2 in. x 1-1/8 in. cover plate. On the other hand the working stress design would yield a much less economical section requiring a heavier beam in the 36 in. series to keep the top flange stress within the allowable value. In order not to overemphasize the general savings to be realized by load factor design, a slight overstress in the top flange is permitted in the working stress design. It is felt that the safety of the bridge is not impaired thereby, since the cover plate stress is held within the allowable value, and any increase in top flange stress due to even a large overload would be very small.

Exterior Stringer:

The design of this stringer is similar to that of the interior stringer. Due to the increase in live load on the exterior stringer as explained under *Design No. 3*, the savings is less than that for the interior stringer.

Design No. 5–80 ft. Simple Span Composite Welded Girder

In this design a welded plate girder section is required to carry the heavier HS20-44 live loading, although the span length is slightly less than that of Design No. 4. This example illustrates the marked differences encountered between designs utilizing rolled sections and designs using welded plate girders. Rolled sections nearly always satisfy compactness requirements and are proportioned for service behavior,

$$0.95\,F_yS \text{ or } 0.80\,F_yS \geqslant D + \frac{5}{3}(L+I),$$

while welded plate girders are virtually never compact sections and are governed by strength:

$$F_yS \text{ or } F_yS \text{ multiplied by a reduction factor} \geqslant 1.25\left[D + \frac{5}{3}(L+I)\right]$$

It was judged initially that a stiffened web would be provided. The web thickness was set at 5/16 in. to satisfy the requirements on material thickness and $D/t \leqslant 190$. Since the entire span is under positive moment with the top flange embedded in the concrete slab, the reduction for unbraced length of compression flange did not have to be considered. Enough transverse web stiffeners are provided so that no reduction in moment capacity need be taken. Thus the criterion for design of the girder is strength as follows:

$$F_yS \geqslant 1.25\left[D + \frac{5}{3}(L+I)\right]$$

The 4 ft. x 9/16 in. transverse web stiffeners are minimum size and are at maximum spacing for the given diaphragm spacing. These stiffeners are the same size but fewer in number than those in the working stress design.

Design No. 6–60 ft. Simple Span Composite Rolled Beam with Cover Plate

The AASHO depth restriction of 1/30 the

span length permits a minimum depth of 24 in. A 24WF68 beam section with a 12 in. x 3/4 in. cover plate satisfies compactness requirements and is controlled by service behavior overload: $0.95F_yS \geqslant D + 2.2\ (L + I)$. As in Design No. 1, and all composite designs with cover plates, the cutoff locations are compatible with ultimate strength and with first yielding under service loading, in the weaker section.

The primary factor in the much larger savings (21.5%) for this design, as compared with the savings of Designs 2 and 3, is the combination of H15-44 loading and composite design with service behavior loading resisted by $0.95\ F_yS$. The higher the ratio of dead to live load, the greater will be the savings since the load factor applied to *LL* is higher than the factor applied to *DL*.

The conventional design for this example was obtained from the Bureau of Public Roads Standards, meeting requirements of the 1961 AASHO Specifications. The design is not in accordance with the fatigue considerations of the 1965 AASHO Specifications. For this reason the comparison with Load Factor Design may not be entirely legitimate.

Design No. 7–Two Span Continuous Composite Rolled Beam with Cover Plates (70-70 ft.)

Fatigue restrictions are at a maximum severity at cover plate cutoffs near the inflection points in continuous rolled beams. At these points stress reversal occurs, reducing allowable fatigue stresses to low levels.

For this reason the load factor design was made using the same basic 36 WF135 beam as the conventional design. With this section it was assured that cover plates could be cut off at the same points as in the conventional design. In the positive moment region, load factor design was governed by service behavior:

$$0.95\ F_yS \geqslant D + \frac{5}{3}\ (L + I)$$

On this basis it was possible to eliminate entirely the 10 in. x 3/8 in. bottom cover plate. The negative moment section was controlled by strength with a reduction for unbraced length of compression flange:

$$0.951\ F_yS \geqslant 1.25 \left[D + \frac{5}{3}(L + I) \right] .$$

Cover plates were reduced from 10 ft. x 1 in. in the conventional design to 10 ft. x 7/8 in.

Investigation shows that a 33 WF130 section with cover plates will also satisfy requirements at maximum positive and negative moment sections. However, it is questionable whether cover plate cutoffs may be made with this section, and it is clear that not much reduction in weight is achievable even if the section can be used.

Extra diaphragms to brace the compression flange and thereby eliminate the reduction factor 0.951 were investigated. It was concluded that maximum economy could not be attained in this manner.

Design No. 8–Two Span Continuous Composite Welded Girder (151.75-120.75 ft.)

The minimum web thicknesses were computed at 3/8 in. for the A36 steel used in positive moment regions, and 7/16 in. for the A441 steel used over the center support. These thicknesses of web furnished adequate shear capacity with a minimum of stiffeners, providing a value of V_u greater than $V/0.6$. Thus the design of positive moment sections was governed by strength, will full moment capacity,

$$F_yS \geqslant 1.25 \left[D + \frac{5}{3}(L + I) \right]$$

and the design of negative moment sections was governed by the service behavior relationship,

$$0.80\ F_yS \geqslant D + \frac{5}{3}(L + I) .$$

The skew of the structure, and the resultant stagger of diaphragms, shortened the unbraced length of compression flange. No strength reduction was required when the 20% allowable increase in strength due to moment gradient was taken advantage of.

The section transition to the left of the center pier could have been made as close as 13 ft. from the pier and still satisfied strength and fatigue requirements. The actual transition is made 30 ft. from the pier to provide a field splice location reasonably near the inflection point.

Transverse intermediate stiffener plates 4 in. x 5/16 in. are used, although these do not quite satisfy the area requirement immediately adjacent to the left end of the girder. This is justified since the extra required stiffener in the end space is ignored in computing the area. Stiffeners are omitted entirely in one diaphragm panel near the middle of the 151 ft.–9 in. span.

Design No. 9–Two-Span Continuous Noncomposite Welded Girder; Stringer/Floorbeam Construction (150–150 ft.)

For this design HS15-44 loading is used for the girders and H20-44 truck loading is used for the floor system. Again the lighter HS15-44 loading tends to produce greater savings than would be achieved with HS20-44 loading; however, at this span length, live load is a smaller fraction of the total load and the effect is not as great as with shorter spans.

The original working stress design was based on the 1957 AASHO Specifications which set the minimum thickness of the longitudinally stiffened A441 web at $D/220$. In order to obtain a more valid comparison with the load factor design the original design is revised to conform to the 1965 AASHO Specifications. Among other things, this revision permits the use of a 3/8 in. web instead of the original 1/2 in. web.

This design provides an example of a noncompact, noncomposite girder for which the strength and service behavior criteria are equivalent, disregarding reduction factors:

Strength: $$F_yS \geqslant 1.25 \left[D + \frac{5}{3}(L + I)\right]$$

Service Behavior: $$0.80\, F_yS \geqslant D + \frac{5}{3}(L + I)$$

In cases where strength and service behavior are equivalent, service behavior is cited as the governing factor. (This is done in anticipation of possible future changes in the criteria that might allow for higher strength.)

With the 100 in. web, a longitudinal stiffener as well as transverse stiffeners are utilized. A 5/16 in. thick web satisfies the requirement that

$$\frac{D}{t_w} \leqslant 330 \text{ for A441 steel.}$$

In order to increase shear capacity and avoid excessively tight transverse stiffener spacing a 3/8 in. web is used over all of the span except in the negative moment region over the pier where a 7/16 in. web is used. Over the pier the negative moment reduction factor for shear capacity is 0.987. At the adjacent flange transitions approximately 10% reductions in moment capacity are required for unbraced length of compression flange. No reduction is required for the maximum positive moment section.

For the longitudinal stiffener, a 7 in. x 1/2 in. plate furnished the necessary rigidity and radius of gyration. The 8 in. x 5/8 in. size of the transverse stiffeners is governed by the area requirement.

The design procedure for continuous welded girders is slightly tedious since one must always check the shear and see if it is greater than 0.6 times the web shear capacity. If it is, the moment capacity of the section must be reduced. It is not clearly obvious whether an increase in shear or moment capacity is more economical. In the end the judgment of the designer must be used to arrive at a reasonable design.

Computation of the shear capacity of the web is found to be time consuming and a small computer program is used to perform this task.

Design No. 10–Two-Span Continuous Composite Welded Girder (100–100 ft.)

The positive moment region is composite and non-compact and therefore governed by strength:

$$F_yS \geqslant 1.25 \left[D + \frac{5}{3}(L + I)\right]$$

The negative moment is governed by service behavior:

$$0.80\, F_yS \geqslant D + \frac{5}{3}(L + I)$$

No reduction factors are required. The minimum allowable thickness for a stiffened web, 5/16 in., is used from the end of span to the field splice. In the negative moment region the web thickness is increased to 3/8 in. to provide the required shear capacity without excessively close stiffener spacing; shear capacity is maintained greater than $V/0.6$ so that no reduction need be taken. When the reduction for unbraced length of compression flange incorporates the 20% allowable increase in strength for the condition where the moment at one end of the unbraced length is less than 0.7 of the moment at the other end of the unbraced length, no actual reduction is required.

Design No. 11–Three-Span Continuous Composite Welded Girder (156–200–156 ft.)

While the conventional design employs a 3/8 in. web the full length of the girder, the load factor design increases the web thickness to 7/16 in. in the negative moment region to provide adequate shear capacity without excessive numbers of stiffeners. The 3/8 in. thickness is sufficient in the positive moment section. Over the interior piers a slight reduction in moment capacity is accepted for the premium of fewer stiffeners. This is a matter of judgment that varies from one case to another; if the number of stiffener spaces in a given diaphragm panel can be decreased by one or two without sacrificing much moment capacity, then it may often be advantageous to take this approach. Since stiffeners increase fabrication costs, weight is not the only factor to be considered in deciding which alternative to follow in a particular design.

This design requires only the minimum size transverse stiffeners.

Design No. 12–Five-Span Continuous Composite Girder (280–360–360–360–280 ft.)

The depth of the web was held the same as in the elastic design with both designs requiring the use of transverse and longitudinal stiffeners. The load factor design produced lighter girder sections and resulted in a smaller total weight of stiffener material.

Due to the L/b' ratio of the girder flanges some moment reductions were necessary in the negative moment areas.

Computation of the shear capacity of the web was done by computer to save time. The shear capacity was kept large enough to hold any reductions in moment capacity to a minimum and keep the transverse stiffeners to a reasonable size. After examination of area requirements and some modifications of the stiffener spacing it was judged most economical to use two different sizes. While the transverse stiffeners in the load factor design are 7% to 25% heavier than those in the conventional design, they were 31% fewer in number. Area requirements governed the size of these stiffeners. It should be mentioned that this phase of the load factor design was somewhat tedious.

Two different sizes were used for the longitudinal stiffeners. Moment of inertia requirements controlled the design.

Design No. 13–73 ft.-4 in. Simple Span Composite Rolled Beam with Cover Plate

The rolled section of this design is a compact section, governed by service behavior:

$$0.95\,F_yS \geqslant D + \frac{5}{3}(L + I).$$

Here, load factor design permitted a reduction in beam size from 36WF150 to 36WF130, and a reduction in the cover plate size from 10-1/2 in. x 1-1/8 in. to 10 in. x 7/8 in. As usual, the cover plate is longer due to fatigue requirements.

Design No. 14–Two-Span Continuous Composite Welded Girder (118.25–118.25 ft.)

A 50 in. x 5/16 in. web in the positive moment region and a 50 in. x 7/16 in. web in the negative moment region satisfy the minimum allowable thickness criteria and provide sufficient shear capacity, without excessive numbers of stiffeners, to assure full moment capacity throughout the span. The girder is governed by strength,

$$F_yS \geqslant 1.25\left[D + \frac{5}{3}(L + I)\right]$$

in the positive moment region, and by service behavior,

$$0.80\,F_yS \geqslant D + \frac{5}{3}(L + I),$$

(equivalent to

$$F_yS \geqslant 1.25\left[D + \frac{5}{3}(L + I)\right])$$

in the negative moment region.

Due to the skew of the bridge, diaphragms on a given girder are staggered providing more than adequate bracing for the compression flange so that no moment capacity reduction need be used.

Design No. 15–Three-Span Continuous Composite Welded Girder (50–85.5–50 ft.)

This bridge was originally designed using a noncomposite rolled beam with cover plates. For comparison with load factor design a new conventional design was made using a 42 in. deep welded girder. Both the conventional and load factor designs took advantage of composite action only in the positive moment area of the center span.

To provide a reasonable load factor design it was judged advisable to increase the web thickness over that of the conventional design at the interior supports. The load factor design requires only minimum size transverse stiffeners which are identical to those used in the conventional design.

The design procedure is very similar to that of Design No. 11.

SUMMARY

The results of the comparative designs are summarized in Table A-6.

TABLE A-6
SUMMARY OF COMPARATIVE DESIGNS

Design No.	Span	Bridge Type	Location	Steel Type	Loading	Stringer Spacing	*Weight of Steel in lbs.		Per Cent Saving
							Conventional Design	Load Factor Design	
Simple Spans									
2	51'	Non-Comp. Rolled Beam	Georgia	A36	H15-44	7'–4"	7,020. 7,020.	6,760. 6,760.	3.7 3.7
3	65'	Non-Comp. Rolled Beam	Georgia	A36	H15-44	7'–4"	12,804. 11,220.	12,012. 11,220.	6.2 0.0
1	40'	Composite Rolled Beam W. Cov. Pl.	U.S.S. Report	A36	HS20-44	8'–4"	3,811.	3.249.	14.7
6	60'	Composite Rolled Beam W. Cov. Pl.	B.P.R. Stds.	A36	H15-44	7'–4"	7,059.	5,539.	21.5
13	73'–4"	Composite Rolled Beam W. Cov. Pl.	W. Va.	A36	HS20-44	7'–7½"	13,140.	11,320.	13.8
4	81'	Composite Rolled Beam W. Cov. Pl.	Georgia	A36	H15-44	8'–0"	15,302. 13,133.	11,913. 11,261.	22.1 14.2
5	80'	Composite Welded Girder	U.S.S. Report	A36	HS20-44	8'–4"	11,227.	10,438.	7.0
Two–Span Continuous									
7	70'–70'	Composite Rolled Beam W. Cov. Pls.	U.S.S. Report	A36	HS20-44	8'–4"	21,291.	20,227.	5.0
9	150'–150'	Non-Comp. Welded Girder□	Summersville, W. Va.	A441	H20-44 Truck HS15-44 Lane	23'–0"	⊛114,824.	⊛102,673.	10.6
10	100'–100'	Composite Welded Girder	U.S.S. Report	A36	HS20-44	8'–4"	30,861.	27,964.	9.4
14	118'–118'	Composite Welded Girder	Utah	A36	HS20-44	9'–3"	52,296.	45,514.	13.0
8	151'9"–120'9"	Composite Welded Girder	Georgia	A36 A441	HS20-44	7'–0"	⊛65,002.	⊛55,655.	14.4
Three–Span Continuous									
15	50'–85.5'–50'	Composite Welded Girder△	W. Va.	A36	HS20-44	7'–5"	20,364.	19,251.	5.5
11	156'–200'–156'	Composite Welded Girder△	U.S.S. Report	A36	HS20-44	8'–4"	155,822.	134,730.	13.5
Five–Span Continuous–Hinges in Center Span									
12	280'–360'–360'–360'–280'	Composite Welded Girder	San Mateo Creek, Calif.	A36 A441 A514	HS20-44	20'–0"	⊛1,494,849.	⊛1,319,755.	11.7

*Weight for one stringer including stiffeners. No diaphragms, bracing or other details included.

⊛Steel weights expressed in terms of equivalent weight of A36: Weight A36 x 1.0

Weight A441 x $\frac{31}{29}$

Weight A514 x $\frac{45}{29}$

□The conventional design for this bridge was redone in accordance with the 1965 AASHO Specifications.

△The conventional design for this bridge was originally a noncomposite rolled beam. It was redesigned as a welded girder to afford a comparison with a welded girder design using the load factor design criteria.

NOTES

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No. 1 Current Paving Practices on Orthotropic Bridge Decks
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No. 11 Response of Steel Frames to Earthquake Forces —Single Degree of Freedom Systems
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No. 12 Response of Multistory Steel Frames to Earthquake Forces
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No. 13 Behavior of Steel Building Connections Subjected to Inelastic Strain Reversals
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No. 14 Behavior of Steel Building Connections Subjected to Inelastic Strain Reversals —Experimental Data
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No. 15 Tentative Criteria for Load Factor Design of Steel Highway Bridges
George S. Vincent, March, 1969

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