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SCHOOL OF ENGINEERING AND APPLIED SCIENCE
DEPARTMENT OF CIVIL ENGINEERING

Proposed Criteria for

LOAD AND RESISTANCE FACTOR DESIGN

OF

STEEL BUILDING STRUCTURES

May 1976

American Iron and Steel Institute Project 163

"Load Factor Design of Steel Buildings"

T. V. Galambos, Project Director, Washington University, St. Louis, Mo.

 h Report No. 45 Civil Engineering Department

PRELIMINARY AND TENTATIVE

DO NOT PUBLISH AND/OR USE AS REFERENCE

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LOAD AND RESISTANCE FACTOR DESIGN

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PART 1: CRITERIA

Section 1: General Provisions

1.1 Scope

These Load and Resistance Factor Design (LRFD) criteria are intended as an alternate to the currently approved "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings" of the American Institute of Steel Construction* (approved February 12, 1969, with Supplements 1, 2 and 3, dated respectively, November 1, 1970, December 8, 1971 and June 12, 1974). Specifically, the LRFD criteria are intended for the design of steel building structures fabricated from hot-rolled steel elements, using the types and grades of material enumerated in Sec. 1.4 and 2.2 of the AISC Specification.

The LRFD criteria contain new provisions for loads and load-combinations, and new rules for the proportioning of structural members and connections. However, these LRFD criteria do not represent a complete set of structural steel specifications and they must be used in conjunction with the AISC Specifications with regard to types of steel, construction and shop practices, and structural details. The applicable portions of the AISC Specification are referenced in these criteria in the appropriate sections. The following general sections of the AISC Specification are

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*This Specification will be abbreviated herein as "AISC Specification".

applicable also in LRFD without change: Sec. 1.1, Plans and Drawings; Sec. 1.2, Types of Construction; Sec. 1.4, Material; Sec. 1.12, Simple and Continuous Spans; Sec. 1.14, Gross and Net Sections; Sec. 1.15, Connections; Sec. 1.16, Rivets and Bolts; Sec. 1.17, Welds; Sec. 1.19, Built-Up Members; Sec. 1.19, Camber; Sec. 1.20, Expansion; Sec. 1.21, Column Bases; Sec. 1.22, Anchor Bolts; Sec. 1.23, Fabrication; Sec. 1.24, Shop Painting; Sec. 1.25, Erection; Sec. 1.26, Quality Control; Sec. 2.2, Structural Steel; Sec. 2.10, Fabrication.

1.2 Definition of LRFD

Load and Resistance Factor Design (LRFD) is a method of proportioning structural elements (i.e. members and connections) such that any applicable limit state is not exceeded when the structure is subjected to any appropriate load combination.

Two types of limit states are to be considered: 1) the limit state of the capacity required to resist the extreme loads during the intended life of the structure, and 2) the limit state of the ability of the structure to perform its intended function during its life. These limit states will be called in these criteria, respectively, the Limit State of Strength and the Limit State of Serviceability.

1.2.1 Limit State: Strength

The design is satisfactory when the computed internal forces, as determined from the assigned mean loads which are multiplied by appropriate load factors, are smaller than or equal to the factored nominal strength of each structural element, i.e.:

$$(\phi R_n)_k \leq \gamma_o \left\{ \sum_{i=1}^{n} c_i \gamma_i Q_i \right\}_j \qquad (1.2\text{-}1)$$

≥

where $(\phi R_n)_k$ = factored nominal strength for limit state k

ϕ = resistance factor for the appropriate limit state

R_n = nominal strength for the appropriate limit state

$\left\{ \sum_{i=1}^{n} c_i \gamma_i Q_i \right\}_j$ = factored internal force for load combination j

c_i = influence factor by which the factored load intensity $\gamma_i Q_i$ is transformed into an internal force (i.e., bending moment, shear force, axial force, torque) by structural analysis

γ_i = load factor for load type i

Q_i = load or load intensity i

γ_o = analysis factor

Appropriate ϕ-factors are given throughout Sec. 2, provisions for the determination of the loads and the list of the load factors are given in Sec. 1.3.

1.2.2 Limit State: Serviceability

Serviceability is satisfactory if a factored nominal structural response (e.g. deflection, drift, stress, frequency, amplitude or acceleration) due to the applicable loads, excitations or temperatures is less than or more than, as appropriate, the corresponding acceptable or allowable value of this response*.

1.3 Loads and Load Combinations

The basis of these LRFD criteria, especially the determination of the resistance factor ϕ and the load factors γ_i, is the use of mean maximum expected loads for the time duration of the loading, and, therefore, the following load types are all mean values**.

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* Further discussion and guide-lines for serviceability criteria are given in Sec. C1.2.2 in the Commentary.

** See Sec. C.1.3 in the Commentary for guidelines to determine or estimate the mean loads.

1.3.1 Load Types*

D = Mean dead load due to the self weight of the structural elements and the permanent features on the structure

L = Mean maximum lifetime live-load due to occupancy

L_I = Mean instantaneous (or sustained) live-load due to occupancy which is expected to be on the structure at any time

W = Mean maximum lifetime wind load

W_A = Mean maximum annual wind load

W_D = Mean maximum daily wind load

S = Mean maximum lifetime snow load

S_A = Mean maximum annual snow load

P = Mean maximum lifetime ponding load

T = Mean extreme lifetime temperature effects

B = Mean maximum lifetime equipment loads, including impact factors where moving equipment is involved**

Construction loading?

1.3.2 Load Combinations

The structure must be designed for the appropriate most critical load combination. Several load combinations may need to be checked to assure that the critical combination is detected. While the determination of the proper combination is often a matter of judgment, the following combinations are frequently encountered***

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* Earthquake loading is omitted from this listing because a separate investigation has not yet provided the necessary input to include them here. It is expected that earthquake research will give the means for dealing with this loading case also within the framework of the LRFD format of Eq. 1.2-1.

** See Sec. 1.3.3 in the AISC Specification for appropriate impact factors.

*** Other load combinations and the general concept underlying the choice of combinations is discussed in Sec. C1.3.2 of the Commentary.

1) Mean dead plus mean maximum lifetime live-loads,

$$1.1\ (1.1\ D + 1.4\ L) \tag{1.2-2}$$

2) Mean dead plus mean instantaneous live plus mean maximum lifetime wind loads,

$$1.1\ (1.1\ D + 2.0\ L_I + 1.6\ W) \tag{1.2-3}$$

3) Mean dead plus mean instantaneous live plus mean maximum lifetime snow loads,

$$1.1\ (1.1\ D + 2.0\ L_I + 1.7S) \tag{1.2-4}$$

4) Mean maximum lifetime wind minus mean dead loads (overturning)

$$1.1\ (1.6\ W - 0.9\ D) \tag{1.2-5}$$

1.3.3 Load Factors for Strength Design

The following load factors are recommended*:

1) Analysis factor, $\gamma_o = 1.1$.

2) Load Factor for dead load, $\gamma_D = 1.1$, except that $\gamma_D = 0.9$ when overturning due to wind is the design consideration.

3) Live-load factors, $\gamma_L = 1.4$ for the mean maximum lifetime and $\gamma_{LI} = 2.0$ for the mean instantaneous live loads.

4) Wind load factors, $\gamma_W = 1.6$ for the mean maximum lifetime, $\gamma_{W_A} = 1.6$ for the mean maximum annual and $\gamma_{W_D} = 2.3$ for the mean maximum daily wind.

5) Snow load factors, $\gamma_S = 1.7$ for the mean maximum lifetime and $\gamma_{S_A} = 2.3$ for the mean maximum annual snow.

6) Load factor for ponding loads, $\gamma_p = 1.2$.

7) Load factor for equipment loads, $\gamma_B = 1.3$.

8) Load factor for construction loads, $\gamma_c = 1.4$.

9) Load factor for temperature effects, $\gamma_T = 1.6$.

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* The statistical bases for the determination of these load factors are given in Sec. C1.3 of the Commentary, where also methods are given for estimating load factors when other statistical premises apply.

Section 2: Design Criteria for the Limit State of Strength

2.1 Types of Structures

2.1.1 Material

The design criteria herein apply to the proportioning of steel building structures fabricated from hot-rolled steel elements using the types and grades of material defined in Sec. 1.4 and 2.2 of the AISC Specification.

2.1.2 Framing

With regard to framing the structure may be either "rigid", "simple" or "semi-rigid" in accordance with the definitions given in Sec. 1.2 "Types of Construction" of the AISC Specification. With regard to the ability of the structure to withstand frame instability the distribution between laterally braced (side-sway buckling prevented by bracing, shear walls, etc.) and unbraced frames (side-sway buckling not prevented) must be considered in design, where the two types of resistance are defined as in Sec. 1.8.2 and 1.8.3 of the AISC Specification, respectively.

2.2 Structural Analysis

The forces in the structural members and connections are determined for the factored loads for the appropriate load combinations. Indeterminate structures may be analyzed by elastic or by plastic analysis, except that plastic analysis may be used only if the appropriate slenderness parameter λ_b as defined in later portions of this section is equal to or less than the limiting value of λ_{bp}, and the structural steel used is limited to the types and grades of material listed in Sec. 2.2 of the AISC Specification.

Forces in multi-story frames shall be determined with due regard to secondary bending (P-delta effect), axial shortening and bending stiffness reduction due to axial shortening where appropriate*.

2.3 The Design of Members

Structural members are to be proportioned such that the maximum force (e.g. bending moment, shear force, axial force, torque), calculated for the appropriately factored loads (as defined in Sec. 1 of these criteria), is less than or equal to the corresponding factored resistance ϕR_n. The resistance factors ϕ and the nominal resistances R_n are presented in the following sections for the member types distinguished according to the kinds of forces acting on them.

2.3.1 Tension Members

2.3.1.1 Factored Maximum Strength

For members subjected to axial tension caused by static loads through the centroidal axis the factored maximum strength $\phi_t R_{nt}$ to be used in design is the lower value obtained according to the limit states of 1) yielding in the net section and 2) fracture in the net section.

Limit State: Yielding in the net section

$$\phi_{ty} = 0.88 \ ; \quad R_{nty} = U A_n F_y \qquad (2.3.1\text{-}1)$$

where A_n = net area of section

F_y = specified yield stress of the grade of steel

U = a coefficient equal to unity except that

$U = 0.75$ for the net section of pin-holes in eyebars, pin-connected plates or built-up members.

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* Further discussion of these effects is given in Sec. C.2.2 of the Commentary.

Limit State: fracture in the net section

$$\phi_{tu} = 0.74 \; ; \quad R_{ntu} = A_n F_u \qquad (2.3.1\text{-}2)$$

where F_u is the specified tensile strength of the grade of steel.

The determination of the net area is to be made in accordance with the provisions of Sec. 1.14.1 through 1.14.6 of the AISC Specification, except that for gusset plates in trusses a net area adjustment may be required*. For pin-connected members the various geometric requirements of Sec. 1.14.6 in the AISC Specification must also be considered.

2.3.1.2 Limiting Slenderness Ratios

The slenderness ratio L/r of tension members, other than rods, tubes or straps should preferably not exceed the limiting values of 240 for main members and 300 for secondary members (Sec. 1.8.4, AISC Specification).

2.3.2 Compression Members

2.3.2.1 Factored Maximum Strength

For members subjected to axial compression through the centroidal axis the factored maximum strength ϕR_n to be used in design is determined by the limit state of instability. The following formulas apply directly to prismatic doubly symmetric columns buckling in the direction of one of their principal axes:

$$\phi_c = 0.86 \quad \text{for} \quad \lambda \le 0.16$$
$$\phi_c = 0.90 - 0.25\lambda \quad \text{for } 0.16 \le \lambda \le 1.0 \qquad (2.3.2\text{-}1)$$
$$\phi_c = 0.65 \quad \text{for} \quad \lambda \ge 1.0$$

$$R_{nc} = P_u = A_g\, Q(F_{cr})_c \qquad (2.3.2\text{-}2)$$

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* See Ch. 6 in Ref. 16.

where A_g = gross area of cross section

Q = 1 if the width-thickness ratios are less than or equal to the limiting values given in Sec. 1.9 of the AISC Specification, and $Q < 1$ is determined by Appendix C of the AISC Specification if these ratios are exceeded.

$$(F_{cr})_c = F_y (1 - 0.25 \lambda^2) \quad \text{for } \lambda \leq \sqrt{2} \qquad (2.3.2\text{-}3)$$

$$(F_{cr})_c = \frac{F_y}{\lambda^2} \quad \text{for } \lambda \geq \sqrt{2} \qquad (2.3.2\text{-}4)$$

$$\lambda = \frac{1}{\pi} \left(\frac{KL}{r} \right) \sqrt{\frac{Q F_y}{E}} \qquad (2.3.2\text{-}5)$$

F_y = specified yield stress of the grade of steel

E = modulus of elasticity

$\frac{KL}{r}$ = effective slenderness ratio

2.3.2.2 Effective Length Factor

The effective length factor K shall be determined by stability analysis as outlined in Sec. C1.8 of the Commentary to the AISC Specification.

2.3.2.3 Effective Slenderness Ratio

The effective slenderness ratio KL/r shall not exceed 200.

2.3.2.4 Flexural-Torsional Buckling

Singly symmetric and unsymmetric columns, such as angle or Tee-shaped columns, and certain doubly symmetric columns such as cruciforms or built-up columns with very thin walls, may require the consideration of the limit state of flexural-torsional buckling. The resistance factor ϕ_c and the nominal resistance R_{nc} are determined by the formulae of Sec. 2.3.2.1 for an equivalent slenderness parameter

$$\lambda_{eq} = \sqrt{\frac{F_y}{(F_{cr})_e}} \qquad (2.3.2\text{-}6)$$

where $(F_{cr})_e$ is the elastic critical flexural-torsional buckling stress*.

2.3.2.5 Tapered Members

The resistance factor $\emptyset_c$ and the nominal resistance R_{nc} for tapered or stepped members shall be determined by the formulae of Sec. 2.3.2.1 for the equivalent slenderness parameter of Eq. 2.3.2-6, except that for members with a single web-taper the special charts given in Appendix D of the commentary to the AISC Specification may be used**.

2.3.2.6 Details of Built-Up Compression Members

Built-up member details shall comply with the provisions of Sec. 1.18.2 of the AISC Specification.

2.3.3 Flexural Members

2.3.3.1 Scope

This section concerns the design of singly or doubly symmetric beam and girder type members which are loaded in the plane of symmetry, and of channel section beams loaded in a plane passing through the shear center parallel to the web***.

Flexural members are subjected to shear force and bending moment. Design for the limit state of shear capacity is treated in Sec. 2.3.3.2, while design for the limit state of bending moment capacity is considered in Sec. 2.3.3.3. In plate girders it is necessary to consider interaction between shear force and bending moment for certain combinations of the two

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* See Commentary Sec. C.2.3.2 for methods of computing $(F_{cr})_e$ for flexural-torsional buckling and for tapered or stepped members.

** See Commentary Sec. C.2.3.2

*** Unsymmetric section beams, and beams subjected to biaxial bending and/or torsion are treated in Sec. 2.3.5. Members under combined bending and axial force are considered in Sec. 2.3.4.

effects, and the requirements are given in Sec. 2.3.3.2.4.

2.3.3.2 Factored Maximum Strength of Webs in Shear

The maximum strength of singly or doubly symmetric members subjected to a shear force in the plane of symmetry is provided by the ultimate shear capacity of the web (or webs in case of multiple web members). The factored maximum strength of webs in shear is $\phi_v R_{nv}$, where the shear resistance factor ϕ_v and the nominal maximum shear strength

$$R_{nv} = V_u \qquad (2.3.3.2\text{-}1)$$

are given in Sec. 2.3.3.2.1 for beams (no transverse stiffeners) and in Sec. 2.3.3.2.2 for plate girders (transverse stiffeners required).

Webs of composite beams must be able to support the total vertical factored design shear on the section.

2.3.3.2.1 Factored Maximum Strength of Beam Webs in Shear

No transverse stiffeners are required, and no interaction check for combined flexure and shear is necessary if $\frac{h}{t} \leq \frac{425}{\sqrt{F_{yw}}}$, where h is the web height and t is its thickness. Otherwise transverse stiffeners are required.

$$\phi_v = 0.86 \quad \text{and} \quad V_u = \left(\frac{1}{\sqrt{3}}\right) A_w F_{yw} \qquad (2.3.3.2\text{-}2)$$

where F_{yw} = specified yield stress of the steel in the web

A_w = web area

2.3.3.2.2 Factored Maximum Strength of Plate-Girder Webs in Shear

When $\frac{h}{t} > \frac{425}{\sqrt{F_{yw}}}$ transverse stiffeners may be needed and an interaction check for combined flexure and shear is required.

When $h/t \leq \frac{425\,\theta}{\sqrt{F_{yw}}}$

$$\phi_v = 0.78 \quad \text{and} \quad V_u = \left(\frac{1}{\sqrt{3}}\right) A_w F_{yw} \qquad (2.3.3.2\text{-}3)$$

and when $h/t \geq \frac{425\,\theta}{\sqrt{F_{yw}}}$

$$\phi_v = 0.78 \text{ and } V_u = \left(\frac{1}{\sqrt{3}}\right) A_w F_{yw} \left[C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right] \qquad (2.3.3.2\text{-}4)$$

except that for end-panels in non-hybrid plate-girders and for all panels in hybrid and web-tapered plate-girders

$$\phi_v = 0.78 \quad \text{and} \quad V_u = \left(\frac{1}{\sqrt{3}}\right) A_w F_{yw} C_v \qquad (2.3.3.2.5)$$

where $$\theta = \frac{\sqrt{(a/h)^2 + 1}}{a/h} \qquad (2.3.3.2\text{-}6)$$

When $\frac{425\,\theta}{\sqrt{F_{yw}}} \leq \frac{h}{t} \leq \frac{532\,\theta}{\sqrt{F_{yw}}}$

$$C_v = \frac{425\,\theta}{h/t\sqrt{F_{yw}}} \qquad (2.3.3.2\text{-}7)$$

and when $\frac{h}{t} > \frac{532\,\theta}{\sqrt{F_{yw}}}$

$$C_v = \frac{226{,}000\,\theta^2}{(h/t)^2 F_{yw}} \qquad (2.3.3.2\text{-}8)$$

and a is the panel length.

The aspect ratio a/h may not exceed 3 nor $\frac{260}{(h/t)}$

2.3.3.2.3 Stiffener Requirements

Transverse stiffeners are required in plate-girders when $h/t > 425/\sqrt{F_{yw}}$, except that stiffeners may be omitted in those portions of the girders where the factored design shear V_D, as determined by structural analysis for the factored design loads, is less than or equal to $\phi_v (1/\sqrt{3}) A_w F_{yw} C_v$, where C_v is determined for $\theta = 1$.

The moment of inertia I_{st} of a transverse stiffener about an axis in the web center shall not be less than $a t^3 j$,

where

$$j = \frac{2.5}{(a/h)^2} - 2 \text{ but not less than } 0.5 \qquad (2.3.3.2\text{-}9)$$

and the stiffener area A_{st} shall not be less than

$$\frac{F_{yw}}{F_{yst}} \left\{ 0.15 \, Dht \, (1 - C_v)\left(\frac{V_D}{V_u}\right) - 18 t^2 \right\} \qquad (2.3.3.2\text{-}10)$$

where F_{yst} = specified yield stress of the stiffener material

$D = 1$ for stiffeners in pairs

$D = 1.8$ for single angle stiffeners

$D = 2.4$ for single plate stiffeners.

and C_v and V_u are defined in Sec. 2.3.3.2.2 and V_D is the factored design shear at the location of the stiffener.

Bearing stiffeners shall be placed in pairs at unframed ends and at points of concentrated loads in the interior of the beam or girder span. They shall be designed as axially compressed members (columns) according to Sec. 2.3.2.1 with an effective length equal to 3/4 of the web depth h and for a cross section comprised of the two stiffeners and a strip of the web having a width of 25 t at interior stiffeners and 12 t at the ends of the member*.

2.3.3.2.4 Web Crippling

No bearing stiffeners are required at interior concentrated loads if

$$R_D \le \phi_{bs}\ t(N + 2\ k)F_{yw} \qquad (2.2.2.2\text{-}11)$$

and at end reactions if

$$R_D \le \phi_{bs}\ t(N + k)F_{yw} \qquad (2.3.3.2\text{-}12)$$

where R_D = factored design concentrated load or reaction

ϕ_{bs} = 0.92

t = web thickness

N = length of bearing, but not less than k at end reactions.

k = distance from outer face of flange to web toe of fillet.

The compressive stresses in the web directly under the flange due to the factored concentrated or distributed design loads at unstiffened portions of plate-girders must be less than $\phi\ F_{cr}$, where ϕ = 0.86 and

$$F_{cr} = \left[5.5 + \frac{4}{(a/h)^2} \right] \frac{26{,}200}{(h/t)^2} \qquad (2.3.3.2\text{-}13)$$

when the flange is restrained against rotation, nor

$$F_{cr} = \left[2 + \frac{4}{(a/h)^2} \right] \frac{26{,}200}{(h/t)^2} \qquad (2.3.3.2\text{-}14)$$

when the flange is not so restrained.

These stresses shall be computed as follows: Concentrated loads and loads distributed over a partial length of a panel shall be divided by the product of the web thickness and the girder depth or the length of the panel in which the load is placed, whichever is the lesser dimension. Continuous distributed loading shall be divided by the web thickness.

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* Note the additional provisions for end bearing details in Sec. 1.10.5.1 in the AISC Specification.

2.3.3.2.5 Interaction Between Bending Moment and Shear Force

When stiffeners are required and the ratio of the factored design shear V_D and the factored design moment M_D is within the limits $V_u/0.75\ M_u \geq V_D/M_D \geq 0.6\ V_u/M_u$ an interaction check must be made such that

$$\frac{M_D}{M_u} + 1.04\ \frac{V_D}{V_u} \leq 1.40 \qquad (2.3.3.2\text{-}15)$$

where M_u is the bending strength of plate-girders (Sec. 2.3.3.3.2) and V_u is the shear strength (Sec. 2.3.3.2.2), except that M_D may not exceed $\phi_b\ M_u$ (ϕ_b = 0.86, Sec. 2.3.3.3) and V_D may not exceed $\phi_v\ V_u$ (ϕ_v = 0.78, Sec. 2.3.3.2.2).

2.3.3.3 Factored Maximum Moment Capacity

The factored maximum moment capacity of singly and doubly symmetric beams and plate girders is $\phi_b\ R_{nb}$, where the resistance factor ϕ_b and the nominal resistance

$$R_{nb} = M_u \qquad (2.3.3.3\text{-}1)$$

is given for beams, plate-girders and composite beams in the following section.

2.3.3.3.1 Maximum Moment Capacity for Beams

This section applies to

1a) Doubly or singly symmetric wide-flange beams loaded in the plane of symmetry;

1b) Doubly or singly symmetric box-beams loaded in the plane of symmetry;

1c) Doubly or singly symmetric hybrid wide-flange beams loaded in the plane of symmetry

1d) Channels loaded through the shear center plane and bent about the major axis;

provided that for these sections the web slenderness $h/t \leq 970/\sqrt{F_{yw}}$, and to

2) Symmetric wide-flange beams and channels bent about their minor axis

3) Doubly symmetric solid sections (solid round, square or rectangular bars, etc.).

The resistance factor $\phi_b = 0.86$ for these sections.

The maximum moment capacity for these sections is determined by the following formulas:

$$M_u = M_p \quad \text{for } \lambda_b \leq \lambda_{bp} \qquad (2.3.3.3\text{-}2)$$

$$M_u = M_p - (M_p - M_r)\left(\frac{\lambda_b - \lambda_{bp}}{\lambda_{br} - \lambda_{bp}}\right) \qquad (2.3.3.3\text{-}3)$$

$$\text{for } \lambda_{bp} \leq \lambda_b \; \lambda_{br}$$

$$M_u = S(F_{cr})_b \quad \text{for } \lambda_b \geq \lambda_{br} \qquad (2.3.3.3\text{-}4)$$

where M_p = plastic moment

M_r = moment at elastic limit, including the effect of residual stress

S = elastic section modulus

F_{cr} = elastic buckling stress

λ_b = slenderness parameter defined as

1) L_b/r_y , the minor axis slenderness-ratio of the laterally unsupported length L_b for the limit state of lateral-torsional buckling (LTB)

2) the flange-plate width-thickness ratio when the limit state is flange local buckling (FLB)

3) the web-plate depth-thickness ratio when the limit state is web local buckling (WLB)

λ_{bp} = slenderness parameter up to which the maximum moment capacity is equal to M_p

λ_{br} = slenderness parameter below which elastic buckling no longer will take place.

M_u must be determined for all appropriate limit states (LTB, FLB, WLB), and the smallest M_u controls. Table 2.3.3.3 gives the relevant formulas for the appropriate cross-sections.

2.3.3.3.2 Maximum Moment Capacity for Plate Girders

This section applies to doubly or singly symmetric single-web plate-girders loaded in the plane of symmetry and for which $(h/t)_r \geq h/t \geq (h/t)_{max}$, where

$$\left(\frac{h}{t}\right)_r = \frac{970}{\sqrt{F_{yw}}} \qquad (2.3.3.3\text{-}5)$$

$$\left.\begin{aligned} \left(\frac{h}{t}\right)_{max} &= \frac{2000}{\sqrt{F_{yw}}} \quad \text{for } \frac{a}{h} \leq 1.5 \\ \left(\frac{h}{t}\right)_{max} &= \frac{14{,}000}{\sqrt{F_{yw}\,(F_{yw} + 16.5)}} \quad \text{for } \frac{a}{h} > 1.5 \end{aligned}\right\} \qquad (2.3.3.3\text{-}6)$$

The resistance factor ϕ_b = 0.86 for plate girders.

The maximum moment capacity is

$$M_u = S_x R_{PG} (F_{cr})_b \qquad (2.3.3.3\text{-}7)$$

where $$R_{PG} = 1 - 0.0005 \left(\frac{A_w}{A_f}\right)\left(\frac{h}{t} - \frac{970}{\sqrt{(F_{cr})_b}}\right) \qquad (2.3.3.3\text{-}8)$$

$$(F_{cr})_b = F_{yf} \quad \text{for} \quad \lambda_b \leq \lambda_{bp} \qquad (2.3.3.3\text{-}9)$$

$$(F_{cr})_b = F_{yf}\left\{1 - \frac{1}{2}\left[\frac{\lambda_b - \lambda_{bp}}{\lambda_{br} - \lambda_{bp}}\right]\right\} \text{for } \lambda_{bp} \leq \lambda_b \leq \lambda_{br} \qquad (2.3.3.3\text{-}10)$$

$$(F_{cr})_b = \frac{C_{PG}}{\lambda_b^2} \quad \text{for} \quad \lambda_b \geq \lambda_{br} \qquad (2.3.3.3\text{-}11)$$

For the limit state: tension flange yield, $(F_{cr})_b = F_{yf}$.

For the limit state: lateral buckling of the compression flange

$$\lambda_b = L_b/r_T \qquad (2.3.3.3\text{-}12)$$

$$\lambda_{bp} = \frac{146}{\sqrt{F_{yf}}} \qquad (2.3.3.3\text{-}13)$$

$$\lambda_{br} = \frac{757\sqrt{C_b}}{\sqrt{F_{yf}}} \qquad (2.3.3.3\text{-}14)$$

$$C_{PG} = 286{,}000\ C_b \qquad (2.3.3.3\text{-}15)$$

where r_T = radius of gyration of the compression flange plus one-sixth of the web

F_{yf} = specified yield stress of flange

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \qquad (2.3.3.3\text{-}16)$$

M_1 is the smaller and M_2 the larger end moment on the unbraced segment; M_1/M_2 is positive when the moments cause reverse curvature. When the bending moment at any point within an unbraced length is larger than at both ends of this length, $C_b = 1.0$.

For the limit state: local buckling of the compression flange,

$$\lambda_b = b_f/2\,t_f \qquad (2.3.3.3\text{-}17)$$

$$\lambda_{bp} = \frac{52.2}{\sqrt{F_{yf}}} \qquad (2.3.3.3\text{-}18)$$

$$\lambda_{br} = \frac{149}{\sqrt{F_{yf}}} \qquad (2.3.3.3\text{-}19)$$

$$C_{PG} = 11{,}140 \qquad (2.3.3.3\text{-}20)$$

For hybrid plate girders the smaller M_u from either Eq. 2.3.3.3-7 or from Table 2.3.3.3 is the controlling value.

2.3.3.3.3 Maximum Moment Capacity for Composite Beams

2.3.3.3.3.1 Definition

These criteria apply to the Load and Resistance Factor Design of composite beams and girders. Such composite members are defined in Sec. 1.11.1 of the AISC Specification, and they include concrete encased beams as well as steel-beam and concrete-slab assemblies connected by shear connectors. Section 1.11.1 in the AISC Specification defines the effective slab width, and this definition, as well as all other provisions therein, apply also to these LRFD criteria.

2.3.3.3.3.2 Factored Maximum Moment Capacities

The provisions of this section specifically pertain to strength limit states for load effects (factored design moments) determined from the factored ultimate loads. Serviceability criteria, such as yielding under permanent loads plus short-term live loads and environmental loads, and deflection under short-term live loads may also need to be considered*.

A. Unshored Beams Under Construction Loads**

The factored moment capacity $\phi_b M_u$ is determined for the steel section only, with $\phi_b = 0.86$ and M_u as the moment capacity of the steel beam. The maximum elastic stress may not exceed $\phi_y F_y$, where $\phi_y = 0.89$ in order to avoid permanent deformation.

B. Simple or Continuous Beams, Shored or Unshored Construction***

a) Positive Moment, Compact Web

The web is considered compact if its height-to-thickness ratio is less than $640/\sqrt{F_{yw}}$.

The factored maximum moment $\phi_b M_u$ is determined for $\phi_b = 0.84$ and M_u is the moment of the forces acting on the fully plastic steel beam and the force C in the concrete slab, as shown in Fig. 2.3.3.3-1.

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* See Commentary Sec. C2.3.3.3.3 for guidelines in the consideration of these serviceability criteria.

** According to Sec. C1.3.1-VIII, Construction loads are the weight of the wet concrete plus 20 psf.

*** The dead load for unshored beams should include the added concrete weight due to the thickening of the slab as a result of beam deflections.

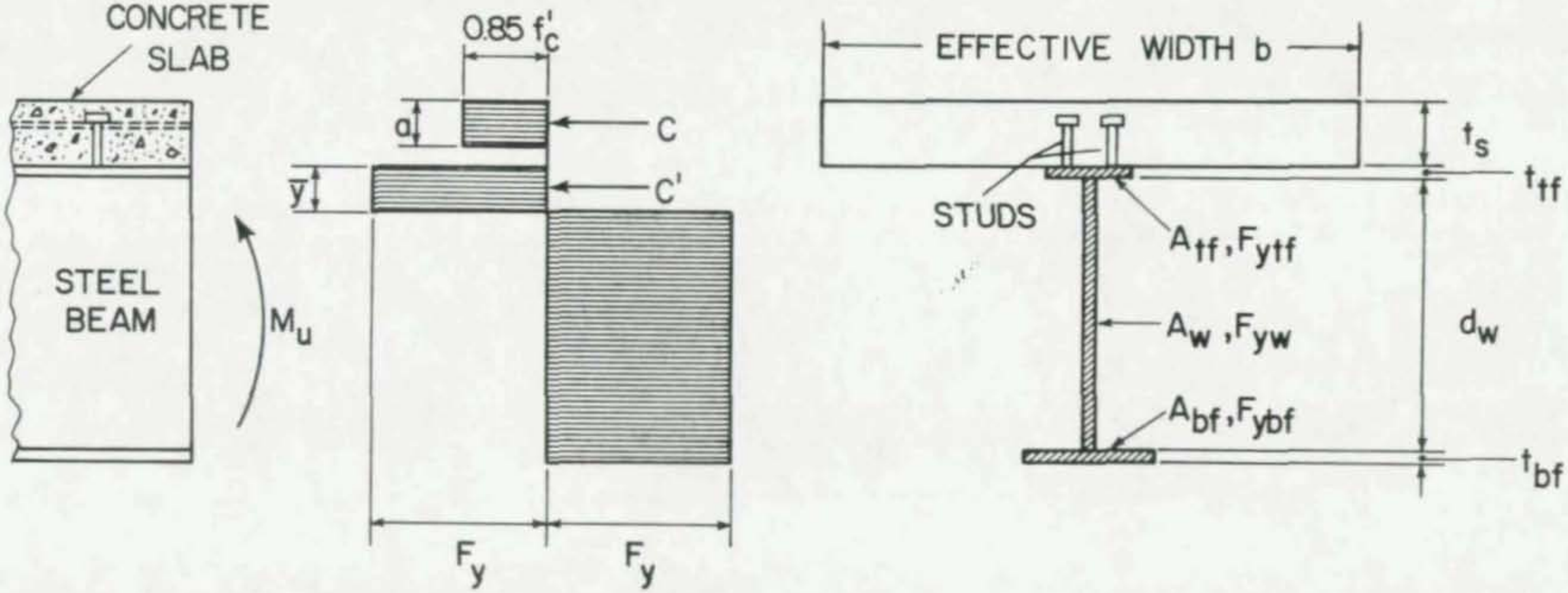

Fig. 2.3.3.3-1 Geometry and Force Distribution for Fully Plastic Composite Beams

The force C is the smallest of the following three values:

$$C = 0.85\ f'_c\ bt_s \qquad (2.3.3.3\text{-}21)$$

$$C = \sum (A_s\ F_y) \qquad (2.3.3.3\text{-}22)$$

$$C = \sum Q_u \qquad (2.3.3.3\text{-}23)$$

where f'_c = specified compression strength of concrete

b = effective slab width

t_s = slab thickness

$\sum (A_s\ F_y)$ = sum of the products of the steel element stress and their respective specified yield stresses

$\sum Q_u$ = the sum of the maximum capacities of the shear connectors between the point of maximum positive moment under consideration and the points of zero moment to either side.

When $C = \sum (A_s\ F_y)$, i.e., 2.3.3.3-22 governs, the plastic neutral axis is in the concrete slab, and the force C acts at a distance a/2 below the top of the slab, where

$$a = \frac{\sum (A_s\ F_y)}{0.85\ f'_c\ b} \quad \text{but not larger than } t_s \qquad (2.3.3.3\text{-}24)$$

When $C < \sum (A_s\ F_y)$ the plastic neutral axis is in the steel section, and the compressive force C' in the steel beam (Fig. 2.3.3.3-1) is equal to

$$C' = \frac{1}{2}\left[\sum (A_s\ F_y) - C\right] \qquad (2.3.3.3\text{-}25)$$

The plastic neutral axis is located by setting $\bar{y}$ in Fig. 2.3.3.3-1 equal to

$$\bar{y} = \frac{C' t_{tf}}{A_{tf}\ F_{ytf}} \quad \text{if} \quad C' \leq A_{tf}\ F_{ytf} \qquad (2.3.3.3\text{-}26)$$

and

$$\bar{y} = t_{tf} + \frac{(C' - A_{tf} F_{ytf}) d_w}{A_w F_{yw}} \quad \text{if} \quad C' \geq A_{tf} F_{ytf} \qquad (2.3.3.3\text{-}27)$$

where $\bar{y}$ is measured from the top of the steel section.

The maximum moment capacity of beams encased in concrete is to be determined by ultimate strength methods, neglecting any area of concrete in tension.

b) Section Under Positive Moment, Non-Compact Web

When the web height-to-thickness ratio exceeds $640/\sqrt{F_{yw}}$ the tensile stress in the bottom fiber of the steel beam must not exceed $\phi_y F_y$ where $\phi_y = 0.89$. For construction without temporary shores, stresses caused by factored loads applied before the concrete has reached 75 percent of its required strength shall be computed using the elastic section modulus of the steel beam. The stress from the factored loads acting on the composite beam is to be determined by using the elastic transformed area method, neglecting the contribution of the concrete in zones where it is in tension and transforming the concrete area in the compression zone into an equivalent steel area by dividing it by the modular ratio $n = E/E_c$.

When only partial shear connection is provided, i.e., $\sum Q_u < C_F$, where C_F is the slab force required for full shear connection and C_F is the smaller of $\sum (A_s F_y)$ and $0.85 f'_c bt_s$, an effective section modulus S_{eff} is to be used in determining the maximum elastic stresses in the steel and the concrete, where

$$S_{eff} = S_s + \sqrt[3]{\frac{\sum Q_u}{C_F}} (S_{tr} - S_s) \qquad (2.3.3.3\text{-}28)$$

In this equation S_s and S_{tr} are, respectively, the section moduli of the steel beam and the elastic transformed section for the composite beam.

c) Section Under Negative Moment

The factored maximum moment capacity $\phi_b M_u$ for composite beams under negative moment is determined for $\phi_b = 0.86$ and for M_u according to the capacity of the steel beam alone (Sec. 2.3.3.3.1 and 2.3.3.3.2), except that when sufficient shear connectors are present in the negative moment region (Sec. 2.3.3.3.3.4), suitably developed concrete slab reinforcement parallel to the steel section and within the design effective width of the concrete slab may be included in computing the maximum moment capacity of the composite section.

2.3.3.3.3.3 Concrete Slabs on Formed Steel Deck

Composite construction using concrete slabs on formed steel deck connected to steel beams and girders shall be designed according to the applicable provisions of Sec. 2.3.3.3.3.2 and 2.3.3.3.3.4 with the following modifications:

A. General

1) Deck ribs shall not be more than nominally 3 in. high.
2) Shear connectors shall not be less than h_r + 1.5 in. long, where h_r is the nominal rib height.
3) Concrete shall be connected to the steel beam with stud shear connectors 3/4 inches or less in diameter, welded directly through the deck or through pre-punched holes.
4) The total slab thickness including the ribs shall be used in determining the effective width of the concrete slab.
5) The minimum width of rib w_r shall be 2 inches.

B. Deck Ribs Running Perpendicular to the Steel Beam

1) Concrete below the top of the steel deck shall be neglected when determining C in Eq. 2.3.3.3-21.
2) No more than two studs shall be placed in any one transverse rib.

C. Deck Ribs Running Parallel to the Steel Beam

1) Concrete below the top of the steel deck may be included in the calculation of C according to Eq. 2.3.3.3-21.

2) For nominal rib heights of 1.5 in. or more, the average rib or haunch width w_r over the supporting member, divided by the number of connectors in one transverse row shall not be less than 2.25 inches. Preferably the steel deck should be split over the supporting member to form a haunch.

3) The shear capacity according to Eq. 2.3.3.3-29 may be used when $w_r/h_r \geq 1.5$.

4) When $w_r/h_r < 1.5$, the reduction factor from Eq. 2.3.3.3-31 shall apply.

5) The average width w_r of haunch or rib over the supporting member shall be 2 inches for the first stud plus 4 stud diameters for each additional stud in a transverse or staggered row.

2.3.3.3.3.4 Shear Connectors

This section applies to the stud diameters, minimum stud lengths, concrete strengths and unit weights cited in Sec. 1.11.4 of the AISC Specification.

The maximum capacity of stud shear connectors for solid concrete slabs is

$$(Q_u)_{sol.} = 0.5\, A_{sc}\sqrt{f'_c\, E_c} \qquad (2.3.3.3\text{-}29)$$

for each shear connector, where

A_{sc} = cross-sectional area of stud shear connector (in.^{2})

f'_c = specified compressive strength of concrete (Ksi)

E_c = modulus of elasticity of concrete (Sec. 8.3, ACI 318-71) (Ksi). $E_c = 1.044\, w^{1.5}\, (f'_c)^{0.5}$, where w is the unit weight of concrete in lbs/cu.ft.

The maximum capacity of stud shear connectors for slabs with formed steel deck subject to the conditions given in Sec. 2.3.3.3.3.3 is

$$(Q_u)_{red.} = C_r (Q_u)_{sol.} \qquad (2.3.3.3\text{-}30)$$

where $$C_r = 0.6 \left(\frac{w_r}{h_r}\right)\left(\frac{H}{h_r} - 1\right) \qquad (2.3.3.3\text{-}31)$$

but not greater than unity, except that $C_r = 1$ shall be used for steel decks with ribs parallel to the steel beam when $w_r/h_r \geq 1.5$.

In Eq. 2.3.3.3-30

w_r = average rib width for open rib deck or width of the top of the rib for trapezoidal ribs

h_r = rib height, not exceeding 3 in.

H = length of stud connector

The maximum capacity of channel shear connectors, for use in solid concrete slabs only, is

$$(Q_u)_c = 0.44\ (t_f + 0.5\ t_w)\ L_c \sqrt{f'_c} \qquad (2.3.3.3\text{-}32)$$

where t_f = average flange thickness of channel shear connector, (in.).

t_w = web thickness of channel shear connector, (in.).

L_c = length of channel shear connector, (in.).

For full composite action the number of shear connectors to be located on each side of the point of maximum bending moment, positive or negative as applicable, and distributed between that point and the adjacent point of zero moment shall be not less than

$$N = \frac{C}{Q_u} \qquad (2.3.3.3\text{-}33)$$

where C is the lesser of $\sum (A_s\ F_y)$ and $0.85\ f'_c\ b_s\ t_s$ for positive moment and $C = A_{sr}\ F_{yr}$ for negative moment, where A_{sr} is the area of the reinforcement in the negative moment region within the effective slab width and F_{yr}

is the yield stress of the reinforcement.

Shear connectors may be spaced uniformly except in regions of positive bending moment the number of shear connectors required between any concentrated load applied in that region and the nearest point of zero moment shall not be less than N from Eq. 2.3.3.3-33 times the factor $(M - M_s)/(M_u - M_s)$; M is the factored design moment, less than the maximum moment, at the point of the concentrated load, M_u is the maximum moment capacity for the composite beam according to Sec. 2.3.3.3.3.2 and M_s is the maximum moment capacity of the bare steel beam.

The use of partial shear connection is permitted, provided this is accounted for in determining M_u and if $\sum Q_u$ is more than one fourth the smaller value of C determined from Eqs. 2.3.3.3-21 and 2.3.3.3-22.

Except for connectors installed in the ribs of formed steel decks, shear connectors shall have at least 1 inch of lateral concrete cover.

Unless located directly over the web, the diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded. The minimum center-to-center spacing of stud connectors shall be 6 diameters along the longitudinal axis of the supporting composite beam and 4 diameters transverse to the longitudinal axis of the supporting composite beam. The maximum center-to-center spacing of stud connectors shall not exceed 8 times the total slab thickness if composite action is accounted for in design.

2.3.3.3.3.5 Vertical Shear Capacity

The total factored vertical design shear shall be supported by the resistance of the steel web in accordance with Sec. 2.3.3.2.

2.3.3.3.3.6 Special Cases

When composite construction does not conform to the requirements of Sects. 2.3.3.3.3.1 through 2.3.3.3.3.5, allowable load per shear connector and details of construction must be established by a suitable test program.

2.3.4 Members Under Combined Flexure and Axial Force

The provisions of this section apply for members of doubly symmetric shape subjected to axial force and bending moment about one or both axes of symmetry. Singly symmetric and unsymmetric shapes under combined loading are treated in Sec. 2.3.5.

2.3.4.1 Members in Flexure and Tension

For wide-flange shapes for which the slenderness parameter $\lambda_b \leq \lambda_{bp}$, where λ_b and λ_{bp} are as defined in Sec. 2.3.3.3.1, the following interaction equations apply:

Flexure about the major axis:

$$\frac{P_D}{\phi_b\ P_y} + \frac{M_D}{1.18\ \phi_b\ M_{px}} \leq 1.0 \qquad (2.3.4.1\text{-}1)$$

except that M_D may not exceed $\phi_b\ M_{px}$

Flexure about the minor axis:

$$\left(\frac{P_D}{\phi_b\ P_y}\right)^2 + \frac{M_D}{1.19\ \phi_b\ M_{py}} \leq 1.0 \qquad (2.3.4.1\text{-}2)$$

except that M_D may not exceed $\phi_b\ M_{py}$

If $\lambda_b > \lambda_{bp}$ and/or if flexure is about both principal axes*

$$\frac{P}{\phi_b\ P_y} + \frac{M_{Dx}}{\phi_b\ M_{ux}} + \frac{M_{Dy}}{\phi_b\ M_{uy}} \leq 1.0 \qquad (2.3.4.1\text{-}3)$$

2.3.4.2 Members in Flexure and Compression

Members in combined flexure and compression must be checked by the appropriate interaction equation from Sec. 2.3.4.1 (e.g. one of Eqs.

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* In some special cases a more liberal approach may be used, as discussed in the Commentary, Sec. C.2.3.4.

2.3.4.1-1 through 3) <u>and</u> by the equation*

$$\frac{P_D}{\phi_b \, P_u} + \frac{C_{mx} \, M_{Dx}}{\phi_b \, M_{ux}\left(1 - \dfrac{P_D}{\phi_b \, P_{Ex}}\right)} + \frac{C_{my} \, M_{Dy}}{\phi_b \, M_{uy}\left(1 - \dfrac{P_D}{\phi_b \, P_{Ey}}\right)} \le 1.0 \qquad (2.3.4.2\text{-}1)$$

except that P_D may not exceed $\phi_c \, P_u$

Definition of Terms in Sec. 2.3.4.1 and 2.3.4.2

P_D = Factored design axial force, tension or compression

M_D = Maximum factored design end moment; subscripts x and y define flexure about x and y-axis, respectively.

The factored design forces may be determined by elastic or plastic analysis, as defined in Sec. 2.2, except that plastic analysis may only be used if $\lambda_b \le \lambda_{bp}$ (Sec. 2.3.3.3.1) and if flexure is about only one of the principal axes.

ϕ_b = 0.86

ϕ_c = resistance factor for compression members as given by Eqs. 2.3.2-1.

$$P_y = A_g \, F_y \qquad (2.3.4.2\text{-}2)$$

A_g = gross area

F_y = specified yield stress of grade of steel

$M_p = Z \, F_y$

Z = plastic section modulus; subscripts x and y refer to flexure about the x and y axis, respectively

P_u = axial load capacity in the absence of flexure, as defined by Eq. 2.3.2-2

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* In some special cases a more liberal approach may be used, as discussed in the Commentary, Sec. C.2.3.4.

$$P_{Ex} = P_y/\lambda_x^2 \qquad (2.3.4.2\text{-}3)$$

$$P_{Ey} = P_y/\lambda_y^2 \qquad (2.3.4.2\text{-}4)$$

$$\lambda_x = \frac{K_x h}{r_x} \left(\frac{1}{\pi}\right)\sqrt{\frac{F_y}{E}} \qquad (2.3.4.2\text{-}5)$$

$$\lambda_y = \frac{K_y h}{r_y} \left(\frac{1}{\pi}\right)\sqrt{\frac{F_y}{E}} \qquad (2.3.4.2\text{-}6)$$

$\frac{K_x h}{r_x}$ and $\frac{K_y h}{r_y}$ are the effective slenderness ratios about the x and y-axes, respectively, as defined in Sec. 2.3.2.2. The effective length factors K_x and K_y, as appropriate, may be taken as unity for frames braced against side-sway buckling, and for unbraced planar frames under combined gravity and wind loads if the factored design forces are determined by considering secondary bending (P-delta effect included). Otherwise the effective length factors are larger than unity and they must be determined by stability analysis (Sec. 2.3.2.2).

M_u = Maximum moment capacity in the absence of axial force, as determined from Sec. 2.3.3.3-1 and Table 2.3.3.3; subscripts x and y refer to flexure about the x and y-axis, respectively.

M_u shall be determined with $C_b = 1.0$ (see Table 2.3.3.3 for a definition of C_b), except that the actual value of $C_b > 1$, if applicable, shall be used when $C_m = 0.85$.

$$C_m = 0.6 - 0.4\, M_1/M_2 < 0.4 \qquad (2.3.4.3\text{-}6)$$

where M_1/M_2 is the ratio of the numerically smaller to the larger factored design end moment, M_1/M_2 being positive when the end moments cause reverse curvature, except that $C_m = 0.85$ shall be used for unbraced frames when the factored design forces are determined without including the effect of secondary bending.

If transverse forces are present between the ends of the member, M_D is the maximum moment and C_m must be determined by separate analysis*.

2.3.4.3 Tapered Beam-Columns

For tapered members with a single web taper under bending about the major axis, P_u and P_{Ex} are determined for the properties of the smaller end, using the effective length factors from Appendix D of the Commentary to the AISC Specification, and M_{ux}, M_D and M_{px} are determined for the larger end;

$$M_{ux} = \left(\frac{5}{3}\right) S_x F_{b\gamma} \qquad (2.3.4.3\text{-}1)$$

where S_x is the elastic section modulus of the larger end, and $F_{b\gamma}$ is the allowable flexural stress of tapered members as defined in Appendix D of the AISC Specification. Formulas for C_m may be also found in Appendix D.

2.3.5 Members Under Combined Stress

This section covers cases of loading (e.g. torsion alone or in combination with flexure and/or axial force), cross-sections (e.g. unsymmetric shapes), or cases of stability not considered in Sec. 2.3.1 through 2.3.4. For such cases the maximum normal stress f_{nD}, and the maximum shear stress f_{vD} shall be determined by elastic analysis for the factored design loads.

For the limit state, yielding under normal stress:

$$f_{nD} \leq \phi F_y \qquad (2.3.5\text{-}1)$$

where $\phi = 0.86$

For the limit state, yielding under shear stress:

$$f_{vD} < \phi F_y/\sqrt{3} \qquad (2.3.5\text{-}2)$$

where $\phi = 0.86$

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* See Sec. C1.6.1 of the Commentary of the AISC Specification or Chap. 8 of Ref. 17. Conservatively $C_m = 1.0$ may be used.

For the limit state of buckling:

$$f_{nD} \text{ or } f_{vD}\text{, as applicable } \leq \phi_c\,(F_{cr})_c \qquad (2.3.5\text{-}3)$$

where ϕ_c is determined from Eqs. 2.3.2-1, and $(F_{cr})_c$ is computed from either Eqs. 2.3.2-3 or 2.3.2-4 for an equivalent slenderness parameter

$$\lambda_{equ} = \sqrt{\frac{F_y}{(F_{cr})_e}} \qquad (2.3.5\text{-}4)$$

$(F_{cr})_e$ being the elastic buckling stress for the particular stability problem under investigation.

2.4 The Design of Connections

2.4.1 Definition

Connections consist of connecting elements (e.g., stiffeners, plates, angles, brackets) and connectors (welds, bolts, rivets). Forces acting on the parts of the connections are the forces determined by structural analysis for the factored loads acting on the structure, or the forces necessary to develop part or all of the strength of the members, whichever is appropriate.

2.4.2 Design of Connecting Elements

The factored nominal strength ϕR_n of connecting elements, such as shapes and plates (e.g., brackets, clip-angles, stiffeners, web plates, doubler plates, base plates) is to be determined for the appropriate limit state (e.g., yielding, plastification, buckling, rupture), using $\phi = 0.77$, to ascertain that ϕR_n is larger than or equal to the forces to be resisted.

The provisions concerning details of the connections contained in Sec. 1.15 of the AISC Specification apply also for the connections designed according to these LRFD criteria.

2.4.3 Connectors

2.4.3.1 Welds

The factored maximum stress ϕF_w of welds is determined as follows:

Complete penetration groove welds

a) tension or compression normal to the effective area or parallel to the axis of the weld

$$\phi = 0.88, \quad F_w = F_y \qquad (2.4.3\text{-}1)$$

b) shear on the effective area

$$\left.\begin{array}{ll} & \phi = 0.80, \quad F_w = 0.6\, F_{EXX} \\ \text{and} & \phi = 0.86, \quad F_{BM} = F_y/\sqrt{3} \end{array}\right\} \qquad (2.4.3\text{-}2)$$

Partial penetration groove welds

a) compression normal to effective area, tension or compression parallel to axis of the weld

$$\phi = 0.88, \quad F_w = F_y \qquad (2.4.3\text{-}3)$$

b) shear parallel to axis of weld

$$\left.\begin{aligned} &\phi = 0.80, \quad F_w = 0.6\ F_{EXX} \\ \text{and} \quad &\phi = 0.86, \quad F_{BM} = F_y/\sqrt{3} \end{aligned}\right\} \qquad (2.4.3\text{-}4)$$

c) tension normal to effective area

$$\left.\begin{aligned} &\phi = 0.80, \quad F_w = 0.6\ F_{EXX} \\ \text{and} \quad &\phi = 0.88, \quad F_{BM} = F_y \end{aligned}\right\} \qquad (2.4.3\text{-}5)$$

Fillet welds

a) stress on effective area

$$\left.\begin{aligned} &\phi = 0.80, \quad F_w = 0.6\ F_{EXX} \\ \text{and} \quad &\phi = 0.86, \quad F_{BM} = F_y/\sqrt{3} \end{aligned}\right\} \qquad (2.4.3\text{-}6)$$

b) tension or compression parallel to axis of weld

$$\phi = 0.88, \quad F_w = F_y \qquad (2.4.3\text{-}7)$$

Plug and slot welds

Shear parallel to faying surfaces (on effective area)

$$\left.\begin{aligned} &\phi = 0.80, \quad F_w = 0.6\ F_{EXX} \\ \text{and} \quad &\phi = 0.86, \quad F_{BM} = F_y/\sqrt{3} \end{aligned}\right\} \qquad (2.4.3\text{-}8)$$

In these equations F_w is the nominal maximum stress capacity of the weld electrode material, F_{EXX} is the specified tensile strength of the electrode material, F_y is the specified yield stress of the base metal, and F_{BM} is the nominal maximum stress capacity of the base metal.

The requirements regarding electrodes and matching base-metals given in Tables 1.5.3 and 1.17.3, as well as the provisions regarding welds given in Sec. 1.14.7, 1.15,6, 1.15.9, 1.15.10, 1.15.12, 1.17, 1.18.2.3 and 1.18.3 of the AISC Specification also apply to these LRFD criteria.

2.4.3.2 Bolts, Rivets and High-Strength Bolts

The factored maximum strength of bolts (ASTM-A307), rivets (ASTM-A502) and high-strength bolts (ASTM-A325 and A490) is ϕR_n, where ϕ and R_n are defined as follows:

2.4.3.2.1 Tension

$$R_n = A_{SA} F_u \qquad (2.4.3\text{-}9)$$

$\phi = 0.89$ for A502 rivets

$\phi = 0.84$ for A325 high-strength bolts

$\phi = 0.83$ for A490 high-strength bolts and A307 bolts

$\phi = 0.77$ for threaded rods made from material meeting the requirements of Sec. 1.4.1 of the AISC Specification

where F_u is the specified tensile strength* of the fastener material and A_{SA} is the stress area (e.g., thread area for bolts and gross area for rivets).

2.4.3.2.2 Shear

$$R_n = m A_{SA} (0.6 F_u) \qquad (2.4.3\text{-}10)$$

$\phi = 0.89$ for A502 rivets

$\phi = 0.86$ for A325 high-strength bolts

$\phi = 0.82$ for A490 high-strength bolts

* The specified tensile strength F_u of the fasteners is: A502 grade 1 rivets: 60 Ksi; A502 grade 2 rivets: 80 Ksi; A307 bolts: 60 Ksi; A325 high-strength bolts: 120 Ksi for 1/2 through 1 inch diameters, 105 Ksi for 1-1/8 through 1-1/2 inch diameters; A490 bolts: 150 Ksi for 1/2 through 1-1/2 inch diameters. (These values are quoted from Ref. 16).

$\phi = 0.75$ for threaded bolts made from material meeting the requirements of Sec. 1.4.1 of the AISC Specification.

where m is the number of shear planes per bolt and A_{SA} is the stress area, equal to the thread area if the shear plane passes through the threads, and the shank area if the shear plane passes through the shank.

2.4.3.2.3 Combined Tension and Shear

When a fastener is subject to forces producing both tension and shear, the following interaction equation must be satisfied:

$$S_D^2 + (m \times 0.6\ T_D)^2 \leq (\phi\ m\ A_{SA} \times 0.6\ F_u)^2 \qquad (2.4.3\text{-}11)$$

$\phi = 0.89$ for A502 rivets

$\phi = 0.80$ for A325 high-strength bolts

$\phi = 0.76$ for A490 high-strength bolts and A307 bolts

$\phi = 0.75$ for threaded bolts made from material meeting the requirements of Sec. 1.4.1 of the AISC Specification.

S_D and T_D are the factored design shear force and tension force, respectively, acting on the fastener.

2.4.3.2.5 Bearing Capacity of Bolt and Rivet Holes

The factored maximum strength of a bolt or rivet hole in bearing is $\phi\ R_n$, where $\phi = 0.65$ and

$$R_n = Lt\ F_u \qquad (2.4.3\text{-}12)$$

but not greater than 3 dt F_u

where L = distance from plate edge to center of hole or to the edge of the next hole, measured parallel to the direction of the load

d = hole diameter

t = plate thickness

F_u = specified tensile strength of plate material.

The ratio L/d may not be less than 1.5.

2.4.3.2.6 Bolt and Rivet Hole Details

The provisions concerning bolt and rivet hole details in Sec. 1.16.1 through 1.16.5 and 1.16.7 in the AISC Specification also apply to these LRFD criteria.

2.4.3.2.7 High-Strength Bolt Friction-Grip Joints*

The factored nominal strength of friction-grip joints is ϕR_n, where $\phi = 1.0$ and

$$R_n = mn K_s A_{SA} \times 0.7 F_u \qquad (2.4.3\text{-}13)$$

where m = number of slip planes

n = number of bolts per joint

K_s = friction coefficient**

A_{SA} = thread area

F_u = specified tensile strength of bolt material

The value of R_n from Eq. 2.4.3-14 must be multiplied by the following reduction factor when a factored tensile force T_D is present:

$$1 - \frac{T_D}{A_{SA} F_u} \qquad (2.4.3\text{-}14)$$

The factored design forces for friction-grip joints are to be determined for the service loading. An additional check for maximum capacity must also be made for these joints for the factored maximum lifetime levels using the resistances determined from Sec. 2.4.3.2.1, 2.4.3.2.2 and 2.4.3.2.3.

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* Since slip is a serviceability limit state, serviceability load combinations are to be used in design (see Sec. C.1.2.2 in the Commentary).

** For clean mill-scale contact surfaces $K_s = 0.33$. Values of K_s for other types of surfaces are given in Chap. 12 of Ref. 16.

2.4.4 Bearing Stresses on Contact Area

The factored nominal stress capacity of surfaces in bearing is ϕR_n, which is defined below for various types of bearing:

2.4.4.1 Milled Surfaces

For milled surfaces, including bearing stiffeners and pins in reamed, drilled or bored holes

$$\phi = 0.77 , \quad R_n = 1.5 F_y \qquad (2.4.3\text{-}15)$$

2.4.4.2 Expansion Rollers and Rockers

$$\phi = 0.77 , \quad R_n = \left(\frac{F_y - 13}{20} \right) \times 1.1 d \qquad (2.4.3\text{-}16)$$

where R_n is in kips per linear inch, and d is the diameter of the rocker in inches. When parts in contact have different yield stress values, the smaller value of F_y is to be used in Eqs. 2.4.3-15 and 2.4.3-16.

2.4.4.3 Masonry Bearing

ϕ = 0.70 and

R_n = 0.8 Ksi on sandstone or limestone

R_n = 0.5 Ksi on brick in cement mortar

R_n = 0.85 f'_c on the full area of a concrete support

where f'_c = specified compressive strength of concrete

When the supporting concrete area is wider on all sides than the loaded area, the value of $R_n = 0.85 f'_c$ may be increased by the factor $\sqrt{A_2/A_1}$ but not more than 2, where A_1 is the bearing area and A_2 is the concrete area.

2.5 Fatigue

The provisions of Sec. 1.7 in the AISC Specification shall apply for fatigue.

Table 2.3.3.3 Formulas for the Maximum Moment Capacity of Beams

This table gives the formulas for determining the maximum moment capacities for the beam sections in Sec. 2.3.3.3-1.* The factored maximum moment capacity is ϕM_u, where

$$\phi = 0.86$$

$$M_u = M_p \quad \text{for} \quad \lambda_b \le \lambda_{bp} \qquad (2.3.3.3\text{-}2)$$

$$M_u = M_p - (M_p - M_r)\left(\frac{\lambda_b - \lambda_{bp}}{\lambda_{br} - \lambda_{bp}}\right) \text{ for } \lambda_{bp} \le \lambda_b \le \lambda_{br} \qquad (2.3.3.3\text{-}3)$$

$$M_u = S(F_{cr})_b \quad \text{for} \quad \lambda_b \ge \lambda_{br} \qquad (2.3.3.3\text{-}4)$$

1. Doubly Symmetric Wide-Flange Beams Loaded in the Plane of Symmetry

$$M_p = Z_x F_y \qquad (A\text{-}2.3.3.3\text{-}1)$$

Limit State Lateral-Torsional Buckling (LTB)

$$\lambda_b = \frac{L_b}{r_y} \qquad (A\text{-}2.3.3.3\text{-}2)$$

$$M_r = S_x(F_y - 10) \qquad (A\text{-}2.3.3.3\text{-}3)$$

$$\left.\begin{aligned} \lambda_{bp} &= \frac{240}{\sqrt{F_y}} \quad \text{for} \quad -0.5 > \frac{M}{M_p} \ge -1 \\ \lambda_{bp} &= \frac{390}{\sqrt{F_y}} \quad \text{for} \quad +1 \ge \frac{M}{M_p} \ge -0.5 \end{aligned}\right\} \qquad (A\text{-}2.3.3.3\text{-}4)$$

$$S(F_{cr})_b = S_x(F_{cr})_b \qquad (A\text{-}2.3.3.3\text{-}5)$$

$$(F_{cr})_b = \frac{C_b X_1}{\lambda_b}\sqrt{1 + \frac{X_2}{\lambda_b^2}} \qquad (A\text{-}2.3.3.3\text{-}6)$$

λ_{br} is determined from Eq. A-2.3.3.3-6 by setting $F_{cr} = F_y - 10$ and solving for $\lambda_b = \lambda_{br}$.

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* The notation and definition of terms for the formulas in this table is given at the end of the table on p. 2-40.

Limit State: Flange Local Buckling (FLB)

$$\lambda_b = \frac{b_f}{2\ t_f} \qquad \text{(A-2.3.3.3-7)}$$

$$M_r = S_x(F_y - 10) \qquad \text{(A-2.3.3.3-8)}$$

In indeterminate beams if the moments are determined by plastic analysis

$$\lambda_{bp} = \frac{52.2}{\sqrt{F_y}} \qquad \text{(A-2.3.3.3-9)}$$

In indeterminate beams if the moments are determined by elastic analysis and in determinate beams

$$\lambda_{bp} = \frac{65}{\sqrt{F_y}} \qquad \text{(A-2.3.3.3-10)}$$

$$S(F_{cr})_b = S_x(F_{cr})_b \qquad \text{(A-2.3.3.3-11)}$$

$$(F_{cr})_b = \frac{21{,}500}{\lambda_b^2} \qquad \text{(A-2.3.3.3-12)}$$

$$\lambda_{br} = \frac{147}{\sqrt{F_y - 10}} \qquad \text{(A-2.3.3.3-13)}$$

Limit State: Web Local Buckling (WLB)

$$\lambda_b = \frac{d}{t} \qquad \text{(A-2.3.3.3-14)}$$

$$M_r = S_x \left[\frac{d}{d - 2\ t_f} \right] F_y \qquad \text{(A-2.3.3.3-15)}$$

In indeterminate beams if the moments are determined by plastic analysis

$$\left. \begin{aligned} \lambda_{bp} &= \frac{520}{\sqrt{F_y}} \left(1 - 1.54 \frac{P_D}{\phi_b\ P_y} \right) \text{for } \frac{P_D}{\phi_b\ P_y} \leq 0.125 \\ \lambda_{bp} &= \frac{152}{\sqrt{F_y}} \left(2.89 - \frac{P_D}{\phi_b\ P_y} \right) \text{for } \frac{P_D}{\phi_b\ P_y} > 0.125 \end{aligned} \right\} \qquad \text{(A-2.3.3.3-16)}$$

In indeterminate beams if the moments are determined by elastic analysis and in determinate beams;

$$\left.\begin{aligned} \lambda_{bp} &= \frac{640}{\sqrt{F_y}}\left(1 - 2.75\,\frac{P_D}{\phi_b\,P_y}\right) \text{for } \frac{P_D}{\phi_b\,P_y} \leq 0.125 \\ \lambda_{bp} &= \frac{152}{\sqrt{F_y}}\left(2.89 - \frac{P_D}{\phi_b\,P_y}\right) \text{ for } \frac{P_D}{\phi_b\,P_y} > 0.125 \end{aligned}\right\} \qquad \text{(A-2.3.3.3-17)}$$

$$\lambda_{br} = \frac{970}{\sqrt{F_y}} \qquad \text{(A-2.3.3.3-18)}$$

When $\lambda_b > \lambda_{br}$, the plate-girder formulas in Sec. 2.3.3.3.2 must be used.

For tapered members with a single web taper determine M_u as 5/3 of the allowable moment obtained from Appendix D of the AISC Specification.

2. <u>Channels Loaded Through the Shear Center Plane and Bent About the Major Axis.</u>

All the same formulas apply as for the doubly symmetric wide-flange shape except that $\lambda_b = \frac{b_f}{t_f}$ for the limit-state LTB.

3. <u>Doubly Symmetric Wide-Flange Beams and Channels Bent About the Minor Axis.</u>

$$M_p = Z_y\,F_y \qquad \text{(A-2.3.3.3-19)}$$

$$M_r = S_y\,F_y \qquad \text{(A-2.3.3.3-20)}$$

Limit-states LTB and WLB do not apply, i.e., $M_u = M_p$. For the limit-state FLB, $S(F_{cr})_b = S_y(F_{cr})_b$, and Eqs. A-2.3.3.3-9, 10, 12 and 13 apply in calculating M_u from Eqs. 2.3.3.3-2 through 4.

4. <u>Singly Symmetric Wide-Flange Shapes Loaded in the Plane of Symmetry.</u>

All equations given for the doubly symmetric wide-flange shapes apply, except that $r_y = b_f/\sqrt{12}$ is to be used for the radius of gyration of the compression flange in computing λ_{bp} for the limit-state LTB (Eq. A-2.3.3.3-2) and F_{cr} for the limit-state LTB (Eq. A-2.3.3.3-6) is to be determined by analysis*.

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* Formulas are provided in Chap. 6 in the Column Research Council Guide (Ref. 17) or in Refs. 27 through 29.

5. Tee-Shaped and Double-Angle Beams Loaded Through the Plane of Symmetry.

$$M_u = M_r = S_x F_y \qquad \text{for} \quad 0 \le \lambda_b \le \lambda_{br} \qquad \text{(A-2.3.3.3-21)}$$

$$M_u = S_x (F_{cr})_b \qquad \text{for} \quad \lambda_b \ge \lambda_{br} \qquad \text{(A-2.3.3.3-22)}$$

Limit State: Lateral-Torsional Buckling (LTB)

$$\lambda_b = \frac{L_b}{r_y} \qquad \text{(A-2.3.3.3-23)}$$

$$(F_{cr})_b = \frac{B_1 C_b}{\lambda_b^2} \left[1 \pm \sqrt{1 + B_2 \lambda_b^2} \right] \qquad \text{(A-2.3.3.3-24)}$$

where + applies when the flange is in compression and - applies when the flange is in tension. λ_{br} is determined by setting $F_{cr} = F_y$ and solving for λ_{br} from Eq. A-2.3.3.3-24).

Limit States: FLB and WLB

$$(F_{cr})_b = Q F_y \qquad \text{(A-2.3.3.3-25)}$$

where Q is determined by Appendix C of the AISC Specification if b_f/t_f of the flange, when it is in compression, or d/t of the web, when the flange is in tension, exceeds the limiting ratios of Sec. 1.9 of the AISC Specification. Otherwise Q = 1.0.

6. Solid Symmetric Shapes

$$M_u = M_p = Z F_y \qquad \text{(A-2.3.3.3-26)}$$

Limit states FLB and WLB do no apply, nor does LTB except for rectangular bars bent about their major axis; for these sections

$$M_p = Z_x F_y \qquad \text{(A-2.3.3.3-27)}$$

$$M_r = S_x F_y \qquad \text{(A-2.3.3.3-28)}$$

$$\lambda_{br} = \frac{253}{\sqrt{F_y - 10}} \qquad \text{(A-2.3.3.3-41)}$$

$$M_r = S_x(F_y - 10) \qquad \text{(A-2.3.3.3-42)}$$

$$F_{cr} = \frac{(S_x)_{eff}}{S_x} (F_y - 10) \qquad \text{(A-2.3.3.3-43)}$$

where $(S_x)_{eff}$ is an effective section modulus determined for a section with a reduced compression flange width b_{eff} if $b_{eff} < b_f$, where

$$b_{eff} = \frac{324\ t_f}{\sqrt{F_y - 10}} \left\{1 - \frac{54.5}{\lambda_b\sqrt{F_y - 10}}\right\} \qquad \text{(A-2.3.3.3-44)}$$

Limit State: Web Local Buckling (WLB)

Use the same formulas as those given for the web of the symmetric wide-flange shape.

8. Doubly and Singly Symmetric Hybrid Beams.

$$M_p = Z_x R_{HP} F_{yf} \qquad \text{(A-2.3.3.3-45)}$$

where $$R_{HP} = \frac{4 + ma_r}{4 + a_r} \qquad \text{(A-2.3.3.3-46)}$$

Limit State: Lateral-Torsional Buckling (LTB)

$$\lambda_b = \frac{L_b}{r_T} \qquad \text{(A-2.3.3.3-47)}$$

$$\left.\begin{array}{ll} \lambda_p = \dfrac{240}{\sqrt{F_{yf}}} \left(\dfrac{r_y}{r_T}\right) & \text{for } -0.5 > \dfrac{M}{M_p} \geq -1 \\ \lambda_p = \dfrac{390}{\sqrt{F_{yf}}} \left(\dfrac{r_y}{r_T}\right) & \text{for } +1 \geq \dfrac{M}{M_p} \geq -0.5 \end{array}\right\} \qquad \text{(A-2.3.3.3-48)}$$

$$M_r = S_x R_{HE} (F_{yf} - 10) \qquad (A\text{-}2.3.3.3\text{-}49)$$

where $$R_{HE} = 1 - \frac{\Psi a_r (1 - m)^2 (3 - \Psi + \Psi m)}{6 + \Psi a_r (3 - \Psi)} \qquad (A\text{-}2.3.3.3\text{-}50)$$

$$\lambda_r = \frac{536\sqrt{C_b}}{\sqrt{F_{yf} - 10}} \qquad (A\text{-}2.3.3.3\text{-}51)$$

$$(F_{cr})_b = \frac{286{,}000\, C_b\, R_{HE}}{\lambda_b^2} \qquad (A\text{-}2.3.3.3\text{-}52)$$

Limit State: Flange Local Buckling (FLB)

$$\lambda_b = \frac{b_f}{2\, t_f} \quad \text{of compression flange} \qquad (A\text{-}2.3.3.3\text{-}53)$$

M_r is same as Eq. A-2.3.3.3-49,

$$\lambda_{bp} = \frac{52.2}{\sqrt{F_{yf}}} \qquad (A\text{-}2.3.3.3\text{-}54)$$

$$\lambda_{br} = \frac{106}{\sqrt{F_y - 10}} \qquad (A\text{-}2.3.3.3\text{-}55)$$

$$F_{cr} = \frac{11{,}200\, R_{HE}}{\lambda_b^2} \qquad (A\text{-}2.3.3.3\text{-}56)$$

Limit State: Web Local Buckling (WLB)

$$\lambda_b = \frac{d}{t}$$

λ_{bp} is determined by Eqs. A-2.3.3.3-16

$$\lambda_{br} = \frac{970}{\sqrt{F_{yw}}} \qquad (A\text{-}2.3.3.3\text{-}58)$$

$$M_r = \left(\frac{I_x}{\bar{y} - t_{tf}} \right) F_{yw} \qquad (A\text{-}2.3.3.3\text{-}59)$$

When $\lambda_b > \lambda_{br}$, Sec. 2.3.3.3.2 must be used for determining the maximum moment capacity.

The formulas presented herein for R_{HP} and R_{HE} apply for the usual case when $F_{yf} > F_{yw}$ and $(A_{ft} \leq A_{fc} \leq 1.25\ A_{ft}$.

Notation for Use with Table 2.3.3.3

A = Cross-sectional area; subscripts f, and w refer to flange, and web, respectively.

a_r = ratio of web area to compression flange area, A_w/A_{fc}

B_1, B_2 = oefficients in Eq. A-2.3.3.3-24

$$B_1 = \frac{\pi^2 E}{2}\left(\frac{A\ B_x}{S_x}\right) \quad \text{and} \quad B_2 = \frac{4G}{\pi^2 E}\left(\frac{J}{B_x^2 A}\right)$$

where

$$B_x = \frac{1}{I_x}\left\{\left(\frac{d + t_f}{2} - y\right)^3 A_w - \left(y - \frac{t_f}{2}\right)^3 A_f\right\} + 2y - t_f$$

$$\text{and} \quad J = \frac{1}{3}\left[A_f\, t_f^2 + A_w\, t_w^2\right]$$

b = width of rectangular section

b_f = flange width, subscripts fc and ft refer to compression and tension flange, respectively.

b_{eff} = effective flange width of box section (Eq. A-2.3.3.3-44)

C_b = equivalent moment factor

$$C_b = 1.75 + 1.05 \left(\frac{M_1}{M_2} \right) + 0.3 \left(\frac{M_1}{M_2} \right)^2 \ngtr 2.3$$

where M_1 is the smaller and M_2 the larger end-moment in the unbraced segment of the beam; M_1/M_2 is positive when the moments cause reverse curvature

C_w = warping constant

d = depth of a section

E = modulus of elasticity (E = 29,000 Ksi)

$(F_{cr})_b$ = critical elastic buckling stress of beam

F_r = compressive residual stress in flange (F_r = 10 Ksi)

F_y = specified yield stress, subscripts yf and yw refer to flange and web, respectively

G = shear modulus (G/E = 0.385)

I_x = second moment of area about x-axis

J = torsion constant;

$$J = \frac{db^3}{3} \left(1 - 0.630 \frac{b}{d}\right) \qquad \text{for solid rectangle}$$

$$J = \frac{2(b_f - t)^2 (d - t_f)^2}{\dfrac{b_f - t}{t_f} + \dfrac{d - t_f}{t}} \qquad \text{for box shape}$$

$$J = \frac{1}{3} \left(A_{fc}\, t_{fc}^2 + A_w\, t_w^2 + A_{ft}\, t_{ft}^2\right) \qquad \text{for singly symmetric W-shape}$$

L_b = unbraced length

M/M_p = ratio of moment M at the end of the unbraced section of a beam to the plastic moment at the other end; M/M_p is positive when the moments cause reverse curvature.

M_p = plastic moment

M_r = moment when yielding commences

M_u = maximum moment capacity

m = ratio of web to flange yield stress in hybrid beams

P_D = factored design axial load

P_y = $A_g F_y$, where A_g is the gross area

Q = plate buckling reduction factor from Appendix C of the AISC Specification

R_{HE}, R_{HP} = coefficients defined by Eqs. A-2.3.3.3-50 and 46, respectively

r_T = radius of gyration of compression flange plus one-sixth of the web

r_y = minor axis radius of gyration

S = elastic section modulus, subscripts x and y refer to major and minor axis, respectively

$(S_x)_{eff}$ = effective section modulus for box shapes

t = web thickness; twice the angle thickness for double angles

t_f = flange thickness; fc and ft refer to compression and tension flange, respectively

X_1, X_2 = coefficients in Eq. A-2.3.3.3-6; X_1 and X_2 are tabulated for all rolled shapes in the AISC Manual in Table C-2.3.3.3-1.

$$X_1 = \frac{\pi \sqrt{GE}\sqrt{JA}}{S_x} \text{ and } X_2 = \frac{\pi^2 E}{4\,G}\left[\frac{A(d - t_f)^2}{J}\right]$$

y = distance from the outside of the flange to the centroid for Tee and double-angle shapes

$\bar{y}$ = distance from bottom of tension flange to centroid for hybrid W-shapes

Z = plastic section modulus, subscripts x and y refer to major and minor axis, respectively.

Ψ = the ratio $\bar{y}$ to d for hybrid shapes.

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NOMENCLATURE

Section 1: General Provisions

A_I : influence area

B : equipment or machinery load

c : influence coefficients in structural analysis

C_s : shape factor for snow loading on roof

C_p, C_s : ponding parameters defined in Fig. C1.3-1

D : mean dead load intensity

E : modulus of elasticity

g : specific gravity of water

h : height of ponded water above support

I : second moment of area

j : ponding parameter defined in Fig. C1.3-1

K : coefficient defined in Fig. C1.3-1

L : mean maximum lifetime liveload intensity, psf

L_I : mean maximum instantaneous liveload intensity, psf

L_p : span of primary roof member

L_s : span of secondary roof member

$\overline{M}_p$, $\overline{M}_s$: ponding magnification factors defined in Fig. C1.3-1

P : mean ponding load effect

Q : load effect in general, subscripts m and n refer to mean and nominal values, respectively

q_{ANSI} : effective wind pressure according to ANSI-A58.1-1972 for 50 year mean recurrence interval, psf

q_{Am} : mean maximum annual ground snow intensity, psf

q_{Lm} : mean maximum lifetime ground snow intensity, psf

R : resistance in general, subscripts m and n refer to mean and nominal values, respectively

S : mean maximum lifetime roof snow load intensity, psf

S_A : meam maximum annual roof snow load intensity, psf

T : temperature effect

V : coefficient of variation, subscripts refer to the different variables

W : mean maximum lifetime wind load intensity, psf

W_A : mean maximum annual wind load intensity, psf

W_D : mean maximum daily wind load intensity, psf

α_s : ponding coefficient defined in Fig. 1.3-1

β : safety index

γ : load factor, subscripts refer to the different load types

γ_o : load factor accounting for the uncertainty of structural analysis

δ_a, δ_c : allowable and computed floor deflection, respectively

ϕ : resistance factor

ρ_a, ρ_c : allowable and computed story deflection, respectively

σ : standard deviation

Section 2: Design Criteria for the Limit State of Strength

A : cross-sectional area

A_1 : bearing area

A_2 : area of concrete

A_f : area of one flange; A_{ft} and A_{fb} refer to top and bottom flange, respectively

A_g : gross area

A_n : net area

A_s	:	area of steel section in composite beam
A_{SA}	:	stress area of bolt
A_{sc}	:	area of shear connector
A_{sr}	:	area of reinforcement in effective slab width
A_{st}	:	area of stiffener
A_w	:	web area
a	:	length of panel between transverse stiffeners in plate girder
a	:	depth of compression zone in concrete slab
b	:	effective slab width in composite beam
b_{eff}	:	effective flange width in box beam
b_f	:	flange width
C, C', C_F	:	forces in slab of composite beam
C_b, C_m	:	equivalent moment factors
C_w	:	warping constant
d	:	depth of section, hole diameter
E	:	modulus of elasticity of steel (E = 29,000 Ksi)
E_c	:	critical stress
F_{BM}	:	maximum stress in base metal
$F_{b\gamma}$	:	allowable bending stress for tapered beam
F_{cr}	:	critical stress
F_{cre}	:	elastic critical stress
F_{exx}	:	specified tensile strength of electrode
F_{yst}	:	specified yield stress of stiffener material
F_u	:	specified tensile strength
F_y	:	specified yield stress, subscripts f and w refer to flange and web, respectively

F_{yr} : specified yield stress of reinforcing bars

F_w : maximum stress in weld

f'_c : specified ultimate stress of concrete

f_nD, f_{vD} : factored design normal and shear stress, respectively

G : shear modulus ($G/E = 0.385$)

H : length of stud connector

h : web depth

h_r : rib height of formed steel deck

I, I_x, I_y : second moment of area

I_{eff} : effective I

I_{st} : I of stiffener

I_s : I of steel section in composite beam

I_{tr} : I of transformed area in composite beam

J : torsion constant

K, K_x, K_y : effective length factor

K_s : friction coefficient

k : distance between face of flange and toe of fillet

L : distance from edge of plate to center of bolt hole

L_b : unbraced length

L_c : length of channel shear connector

M, M_1, M_2 : moment

M_D, M_{Dx}, M_{Dy} : factored design moment

M_p, M_{px}, M_{py} : plastic moment

M_r : moment at elastic limit

M_s : yield moment of steel beam in composite section

M_u, M_{ux}, M_{uy} : maximum moment capacity

m : number of slip planes in a joint

m : ratio of web-to-flange yield stress in hybrid beams

N	:	bearing length at support
N	:	number of shear connectors
n	:	number of bolts per joint
P_D	:	factored design axial force
P_E, P_{Ex}, P_{Ey}	:	elastic column buckling load
P_u	:	axial capacity of column
P_y	:	yield load
Q	:	local buckling reduction factor
Q_u	:	maximum shear connector capacity
R_D	:	factored design reaction
R_n	:	nominal resistance
r, r_x, r_y, r_T	:	radius of gyration
S, S_x, S_y	:	elastic section modulus
S_D	:	factored design shear force in bolt
S_{eff}	:	effective section modulus
T_D	:	factored design tensile force in bolt
t, t_w	:	web thickness, plate thickness
t_f	:	flange thickness
t_s	:	slab thickness
V_D	:	factored design shear force
V_u	:	maximum shear capacity
w_r	:	average rib width
x, y	:	principal centroidal coordinates of cross-section
x_o, y_o	:	coordinates of shear center
y, $\bar{y}$	:	centroidal distance
Z, Z_x, Z_y	:	plastic section modulus
λ	:	slenderness parameter
ϕ	:	resistance factor

TENTATIVE

LOAD AND RESISTANCE FACTOR DESIGN

OF

STEEL BUILDING STRUCTURES

PART 2: COMMENTARY

Section C.1: General Provisions

C1.1 Scope

These Load and Resistance Factor Design (LRFD) criteria are intended to be an alternative to the currently approved AISC Specification by providing a method of design which is based on the use of load factors and resistance factors. The designation LRFD reflects the fact that both the resistance and the loading are factored. This factoring is in contrast to the criteria in Part 1 of the AISC Specification where only the resistance (i.e., the limiting stress) is multiplied by a factor, or Part 2, where only the load is so modified. The LRFD criteria have been developed to permit the designer of structural steel buildings a greater flexibility, rationality and possible economy. A number of structural specifications in the USA, in Canada and in other countries abroad have adopted an LRFD type specification, or they have provided such criteria as alternates. Others are seriously planning the implementation of a change-over.

The format using load factors ϕ and resistance factors γ (Eq. 1.2-1) is identical to the strength design criteria of the ACI-Code (ACI-318), to the alternate load-factor design procedure for steel highway bridges in the AASHTO Specification, and to the Canadian limit-states design specification which was adopted in 1975. Thus LRFD is not new to the designer,

nor is it radically different from the Allowable Stress Design or the Plastic Design in Parts 1 and 2 of the AISC Specification. These other methods, in fact, can be thought of as special cases of LRFD where only one instead of several factors are utilized. Nor should the new LRFD method give radically different designs from previous designs, since the new method was tuned, or "calibrated", to typical representative designs of the earlier methods. The advantage of LRFD with its multiple factors over the AISC Specification is that proper weight is given to the degree of accuracy with which the various loads and resistances can be determined resulting in a more rational design procedure and in a greater uniformity of reliability.

The LRFD criteria herein are not a full and entirely independent set of design rules and their use is definitely dependent on many of the provisions in Part 1 of the AISC Specification. They are a supplement much the same way as Part 2, Plastic Design, is an extension of Part 1, Allowable Stress Design. It is hoped that if the general specification trend tends everywhere toward an LRFD format that a unified single design criterion will evolve in the future.

C1.2 Definition of LRFD

The general format of the LRFD criteria is given by Eq. 1.2-1 in Sec. 1.2.1. The right side of this design criterion represents the forces which are computed by structural analysis from the factored loads; the left side represents a limiting structural capacity ("limit state"), which is multiplied by a resistance factor ϕ. The load factors γ and the resistance factor ϕ reflect the fact that loads, load effects (i.e., the computed forces in the structural element) and the resistance can only be determined to an imperfect degree of accuracy. Thus $\phi < 1$ indicates that the capacity

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which is computed by the formulas given in Sec. 2 of the LRFD criteria has a chance of being less, while the load factors $\gamma > 1$ reflect the fact that the computed forces may be more than the nominally determined values. These factors, then, in a way account for the unavoidable inaccuracies in theory, the variations in the material properties and the uncertainties in the loads. They do not, however, account for gross error and negligence.

The LRFD criteria are based (1) on a "first order" probabilistic model* which permits the incorporation of the statistical properties of the different variables of the design equation in a rational and simple manner; (2) on a calibration of the new criteria to the AISC Specification for selected common design cases (e.g. the simple compact braced beam under uniformly distributed dead and live loading; the simple column in a braced frame; the fillet welded joint; etc.) to ascertain that for these benchmark situations substantially the same designs emerge from both methods; and (3) on the evaluation of the resulting criteria by judgment and past experience, and from the results of a comparative design office study of representative structures (Ref. 3).

Following is a brief description of the basis for LRFD**: The resistance R and the load effect Q are random variables characterized by the frequency distributions shown in Fig. C.1.2-1, provided that it can be assumed that Q and R are independent. This is approximately so for most of the usual types of loading on steel structures. In Fig. C1.2-1 it can be seen that the probability of exceeding a limit state is equal to the

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* This model used only the mean and the standard deviation of the statistical properties of the variables involved, thus the name "first-order" or "second-moment" format. This method was developed and made practically usable by C. A. Cornell, N. C. Lind, A.H.-S. Ang and others (see description of the method in Refs. 1 and 2 where further bibliographic information and statistical and probabilistic fundamentals are also given).

** For a more detailed discussion see Ref. 1.

probability of $R < Q$. The representation in Fig. C.1.2-2 is an equivalent statement of this in a different way: the probability of exceeding the limit state is equal to the probability of the ratio $\ell n\ (R/Q) < 0$, and it is the shaded area to the left of the origin. As shown, the distance of the mean of $\ell n\ (R/Q)$ with respect to the origin can conveniently be measured as a number β times the standard deviation σ of $\ell n\ (R/Q)$. Generally, for a given distribution shape the magnitude of β defines the area to the left of the origin. For example, an increase of β implies either a movement to the right for a given standard deviation, or a reduction of the spread of the curve for a given mean; either change would result in a smaller probability of exceeding the limit state. A decrease in the value of β would have the reverse effect. If the actual distribution shape of $\ell n\ R/Q$ were known, and if a value of the probability of reaching the limit state could be agreed upon, one could establish a completely probability-based set of design criteria. Unfortunately so much information is not known. The distribution shape of each of the many variables (material, loads, etc.) has an influence on the shape of the distribution of $\ell n\ R/Q$. At best only the means and the standard deviations of the many variables involved in the make-up of the resistance and the load effect can be estimated. This information is enough to build a first-order approximate design criterion which is independent of the knowledge of the distribution, by stipulating the following design condition:

$$\beta\ \sigma_{\ell n(R/Q)} \cong \beta \sqrt{V_R^2 + V_Q^2} \leq (\ell n\ R/Q)_m \cong \ell n\left(\frac{R_m}{Q_m}\right) \qquad (C1.2\text{-}1)$$

In this formula the standard deviation has been replaced by the approximation $\sqrt{V_R^2 + V_Q^2}$, where $V_R = \frac{\sigma_R}{R_m}$ and $V_Q = \frac{\sigma_Q}{Q_m}$ (σ_R and σ_Q are the standard deviation, R_m and Q_m are the mean values, V_R and V_Q are the coefficients of variation, respectively, of the resistance R and the load effect Q). Since the distribution of ℓn R/Q is not known, nor is the probability of exceeding the limit state given, formula C1.2-1 is only a rough approximation. However, for structural elements and the usual loadings R_m, Q_m, V_R and V_Q can be estimated, and so a calculation of

$$\beta = \frac{\ell n \ (R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \tag{C1.2-2}$$

will give a comparative value of the measure of reliability of the design. The factor β is called, therefore, the "safety index". The determination of β for common structural situations for elements designed according to an existing specification and then choosing a single value of β is called "calibration". For example Fig. C1.2-3 shows the variation of β with tributary area, and dead and live load intensity according to present code requirements, for simply supported braced compact beams under dead and office occupancy live loading when these beams are designed according to part 2 of the AISC Specification*. It is evident that the safety index β, and thus the comparative reliability, varies considerably in present design. The value of β tends to increase as the code live load L_c and as the dead load D_c increase. A similar picture emerges for simple columns in braced frames (Fig. C1.2-4) where β is seen to vary also with the slenderness parameter λ.

One of the major features of the first-order probability based design method is that the large variations of β (a variation of one unit in β corresponds approximately to one order of magnitude variation in the probability of failure[(1)]) can be ironed out by specifying one value of β.

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*Figs. C1.2-3 and C1.2-4 are taken from Ref. 1, where the data basis for the development of the curves is fully explained and rationalized.

A value of β = 3.0 has been selected for members in LRFD. This single value of β, when used for all types of members and all kinds of loadings, will insure that all designs will have approximately the same reliability, and that this reliability will be characteristic of the set of all designs against which calibration was performed.

The basic selected value of β = 3.0 will thus level out variations in reliability, giving a more uniform design criterion, permitting smaller sections in many cases (e.g., when the dead load is large compared to the live load) and requiring somewhat larger sections in other situations, (e.g., when the live load contribution is large) than the existing AISC design.

The basic value of β = 3.0 applies to members (beams, columns, beam-columns); a study of connections[(4)] has shown that a larger value of β = 4.5 is representative of present practice. This is desirable, because it indicates that the probability of reaching a limit state is higher for members than for connections, reflecting current design philosophy. The value of β can also be increased or decreased, depending on the importance of the structure. In Ref. 1, for example, it is suggested that β = 3.0 is to be used for members in permanent structures, β = 2.5 or 2.0 for temporary structures and β = 4.5 for vital structures. Thus the resistance factor ϕ and the load factors γ, which are given herein for the basic value of β = 3.0, can be adjusted by a method to be described in Sec. C1.3 to account for the importance of the structure by varying β as required.

It is shown in Ref. 1 that by making suitable approximations involving separation of variables and error minimization procedures, the resistance factor ϕ and the load factors γ can be derived from Eq. C.1.2-2 and they can be expressed by the following formulas:

$$\text{Resistance factor}^{*} \quad \phi = (R_m/R_n) \exp(-0.55\,\beta\,V_R) \qquad \text{(C1.2-3)}$$

$$\text{Analysis factor } \gamma_o = \exp(0.55\,\beta\,V_o) \qquad \text{(C1.2-4)}$$

$$\text{Load factors} \quad \gamma_{Q_i} = 1 + 0.55\,\beta\,V_{Q_i} \qquad \text{(C1.2-5)}$$

where R_m = mean resistance

R_n = nominal resistance according to the formulas in Sec. 2 of these LRFD criteria

V_R = coefficient of variation of the resistance

V_o = coefficient of variation of the analysis

V_{Q_i} = coefficient of variation of the load effect Q_i

These approximations are used as the basis for the LRFD criteria when the limit state is the strength of the structure.

C1.2.1 Limit State: Strength

A limit state is a condition which represents a boundary of structural usefulness. Limit states may be arbitrary, such as maximum levels of stress beyond which the actual stresses should not rise; they may be dictated by functional requirements, such as maximum deflections or drift; they may be conceptual, such as a plastic hinge or mechanism formation; or they may represent the actual collapse of the whole or part of the structure, such as fracture or instability. Design criteria insure that a limit state is violated only with an acceptably small probability by selecting load and resistance factors and nominal load and resistance values which are shown by the design calculations never to be exceeded.

Two kinds of limit states apply for structures: limit states of strength which are required against the extreme loads during the intended life of the structure, and limit states of serviceability which define the functional requirements. These LRFD criteria, like all other structural

*Note that exp x is identical to the more familiar e^x.

specifications, focus on the limit states of strength because of the overriding considerations of public safety for the life, limb and property of human beings. This does not mean that limit states of serviceability are not important to the designer, who must equally insure functional performance and economy of design. However, these latter considerations permit more exercise of judgment on the part of the designers and they represent his competitive stock-in-trade. Minimum considerations of public safety, on the other hand, are not matters of individual judgment and, therefore, specifications dwell more on the limit states of strength than on the limit states of serviceability.

Limit states of strength vary from member to member, and several limit states may apply in every case. These are identified in Sect. 2 of these LRFD criteria. The following limit states of strength are the most common: onset of yielding, formation of a plastic hinge, formation of a plastic mechanism, overall frame or member instability, lateral-torsional buckling, local buckling, tensile fracture, development of fatigue cracks, deflection instability, alternating plasticity, and excessive deformation.

C1.2.2 Limit State: Serviceability

Serviceability criteria are formulated to ensure that malfunctions during the everyday use of the structure are rare. These malfunctions do not result in structural failure, but they can reduce or even eliminate any economic gain. There are three types of unserviceability:

1) Permanent deformations due to yielding at load levels which occur fairly frequently can result in unsightly sags and cracks in the finished structure.

2) Unacceptable elastic deflections may result in unsightly sags or cracks or which impair the functioning of the mechanical equipment or

the ancillary elements in the structure.

3) Fluctuations in the live load or in the dynamic deflections induced by live loads or wind can result in vibrations which are unacceptable.

In allowable stress design the problem of permanent set is taken care of by the factor of safety built into the allowable stress, live load deflections are controlled by deflection and drift limits, and vibrations are controlled by specifying limiting deflections and maximum length-to-depth ratios. By-and-large these rules work well, with perhaps the exception of large open floor areas without partitions, and satisfactory structural performance results.

Many serviceability criteria are common-sense or practice-tested rules relating to limiting dimensions such as slenderness-ratio limits and length-to-depth ratio restrictions. These are retained in the LRFD criteria and they are the same as those required in the AISC Specification. The following guide-lines will refer only to two limit states of serviceability: limits of yielding and limits of deflection. In the case of vibrations the present design state-of-the-art has not yet advanced to a clear definition of the acceptable limiting set of dynamic properties, nor has it yet crystallized as to what specific excitation should be used as the basis for computing dynamic response in building structures. Thus the subject of vibration will not be covered further here, and the reader is referred to the specialized literature in this field (see Refs. 5, 6 and 7 for a review of this subject).

In case that the strength limit state is a limit state which is either the attainment of the plastic moment at a section or the formation of a plastic mechanism, it may be necessary to insure that yielding does not occur during service conditions. This is especially so for composite beams which exhibit a larger shape factor than the non-composite wide-flange sections. The design criterion for this situation is of the same format as that given by Eq. 1.2-1, i.e.,

$$(\phi R_n)_{yield} \geq \gamma_o \left\{ \sum_{i=1}^{n} c_i \gamma_i Q_i \right\}_j \tag{C1.2-6}$$

However, the resistance factor ϕ and the load factors γ are based on a value of $\beta = 1.5$ rather than on $\beta = 3.0$, reflecting the lower degree of reliability demanded for a serviceability limit state than for a strength limit state. The nominal resistance R_n is a limiting elastic force or stress (e.g., the yield moment M_y, or the yield stress F_y which may be modified to include a residual stress, i.e., $F_y - F_r$), and the load effects $c_i Q_i$ are determined by linear elastic stress analysis. The following resistance factor and load factors correspond to the serviceability limit state of yielding:

Resistance factor	$\phi = 0.94$
Analysis factor	$\gamma_o = 1.05$
Load factors for dead load	$\gamma_D = 1.05$
Instantaneous live load	$\gamma_{L_I} = 1.50$
Maximum annual wind	$\gamma_{W_A} = 1.30$
Maximum annual snow	$\gamma_{S_A} = 1.65$
Equipment	$\gamma_B = 1.15$

Since the limit state represents a serviceability condition, the load combinations to be considered should involve only the dead and equipment loads, the instantaneous (or "sustained") live loads and the maximum annual wind and snow loads, as appropriate, instead of the expected maximum lifetime loads.

A case similar to the limit state of yielding is the initiation of slipping in a high-strength bolted friction-type connection under static loading. Since the onset of slipping is not an indication of the maximum capacity of the joint, its occurrence is a serviceability limit state. Thus the same type load combinations apply as above for the limit state of yielding. However, $\phi = 1.0$ should be used (see Sec. 2.4.3.2.8) in conjunction with the strength limit state load factors. This latter provision is stipulated in order to avoid the necessity of recalculating load factors, and an adjustment has been provided in the determination of ϕ (see Ref. 4).

Two deflection limit states apply: limit states of beam deflection under live loads and limit states of building drift under wind loads. The AISC specification does not specify live load deflection or drift limits. These are left to the individual designer's judgmental choice. Common live load deflection limits are 1/360 of the span for floor beams, and 1/240 of the span for roof beams. Drift limits in common usage are of the order of 1/400 to 1/500 of the story height. No deflection or drift limit recommendations are intended here; it is only stipulated that the designer specify these.

The LRFD deflection and drift criteria are expressed by the general formula (Ref. 1):

$$\delta_a \geq \delta_c \exp (\beta_\delta \, V_\delta) \qquad \text{(C1.2-7)}$$

where δ_a is the limiting deflection or drift and δ_c is the calculated deflection or drift, computed by elastic theory for the mean instantaneous live load, the mean superimposed dead load, and the annual mean maximum snow and wind loads, as appropriate*. The value of the safety index β_δ is

* See discussion of these loads in Sec. C.1.3.1.

equal to 1.5 from calibration**, and the coefficient of variation V_δ is defined by the formula

$$V_\delta = \sqrt{V_P^2 + V_M^2 + V_F^2 + V_L^2} \qquad \text{(C1.2-8)}$$

In Eq. C1.2-8, V_P is the coefficient of variation reflecting the uncertainties of the deflection analysis procedure assumed to be $V_P = 0.05$ for beam deflection and $V_P = 0.10$ for wind drift), V_M is the coefficient of variation of the modulus of elasticity ($V_M = 0.06$), V_F is the coefficient of variation of the moment of inertia ($V_F = 0.05$) and V_L is the coefficient of variation of the loads.

For the <u>annual wind</u>, the value of $V_L = V_{W_A} = 0.37$ (Ref. 14) and thus for the drift calculation $\exp \beta_\delta V_\delta = 1.8$.

For the <u>instantaneous live load</u>, V_L depends on the tributary area (Ref. 9):

for $56 \text{ ft}^2 \leq A_T \leq 336 \text{ ft}^2$

$$V_L = V_{L_I} = 0.82 \left[1 - 0.00113 \left(A_T - 56\right)\right] \qquad \text{(C1.2-9)}$$

for $A_T \geq 336 \text{ ft}^2$

$$V_L = V_{L_I} = 0.56 \left[1 - 0.0001865 \left(A_T - 336\right)\right] \qquad \text{(C1.2-10)}$$

The factor $\exp \beta_\delta V_\delta$ can be determined for a given tributary area A_T with $V_P = 0.05$, $V_M = 0.06$, $V_F = 0.05$, V_L from either Eq. C1.2-9 or 10, as appropriate, and $\beta_\delta = 1.5$. Alternately, a representative tributary area of $A_T = 500 \text{ ft}^2$ may be used, for which $\exp \beta_\delta V_\delta = 2.3$.

For the <u>annual snow</u>, the coefficient of variation is equal to (Ref. 14)

$$V_L = V_{S_A} = \sqrt{\left[\exp\left(\sigma_{\ell n\, S_A}\right)^2 - 1\right] + V_{C_S}^2} \qquad \text{(C1.2-11)}$$

** A calculation of the safety index has shown that β in current design practice varies from 1 to 2 for deflections and drift under live, wind and snow loads (Refs. 1 and 14).

where $\sigma_{\ln S_A}$ is the standard deviation of the maximum annual snowfall in a given location, obtained from the map in Fig. C1.3-2b and V_{C_S} is the coefficient of variation of the roof shape factor. The value of $\sigma_{\ln S_A}$ varies from 1.0 to 0.3 on this map. The following table gives the magnitudes of exp $\beta_\delta V_\delta$ for snow load deflection calculations for $\beta_\delta = 1.5$, $V_P = 0.05$, $V_M = 0.06$, $V_F = 0.05$ and V_L from Eq. C1.2-10:

$\sigma_{\ln S_A}$	exp $\beta_\delta V_\delta$
1.0	7.3
0.9	5.5
0.8	4.2
0.7	3.4
0.6	2.8
0.5	2.3
0.4	2.0
0.3	1.7

C1.3 Loads and Load Combinations

Design criteria which are specifically intended for use with a given type of building material, steel in this instance, commonly do not, and they should not, concern themselves with the definitions of the loads and the load combinations. This first presentation of LRFD criteria for steel buildings, however, needs to contain such provisions because the whole basis of their development is the knowledge of the essential statistics of the loads: their means and their standard deviations. Current load codes, especially the local and regional building codes and to some extent the national model load code ANSI-A58.1-1972, are based on nominal loads from which these statistics are impossible or difficult to identify. The essential statistical properties of loads are not always available, and, therefore, a great deal of judgment has to be exercised at this stage of development. This is especially so for live loadings where extensive load survey data exists only for office occupancies. On the other hand, the statistics of the environmental loads are available. Research efforts are currently underway to determine essential data on live loads, and it is expected that future editions of the national model load code (ANSI-A58.1) will contain the statistics and methodologies necessary for utilization in material design criteria based on the first-order probabilistic concepts.

Following is a description of how loads are to be treated in these LRFD criteria for steel buildings. The loads will be related to the current ANSI-A58.1-1972 model load code as much as possible, and the text will indicate where the load determinations are founded on as yet unsupported estimates and assumptions. Obviously not all possible loadings can be covered, and the user of these criteria may need to examine the fundamentals more thoroughly (see Ref. 1 for an introduction and for further relevant literature) before estimating the loads to be used in design for the cases not covered herein.

The basic considerations in the design criterion (Eq. 1.2-1) are that the loads from which the load effects Q_i are determined are mean maximum loads over the period over which they are intended to act, and that the load factors γ_i are obtained from the formula

$$\gamma_i = 1 + 0.55\ \beta\ V_i \qquad \text{(C1.3-1)}$$

for the limit states of strength, where $\beta = 3.0$ and V_i is the coefficient of variation for the load type "i". The coefficients of variation underlying the load factors in Sec. 1.3.3 are given below for the purpose of 1) the determination of new values of γ_i when $\beta \neq 3.0$ is desired as the basis for design (e.g., for temporary or vital structures - see previous discussion in Sec. C1.2) and 2) the comparative examination of these values when the designer has at his disposal the actual statistical data for a specific load type for a specific structure and he wishes to adjust the load factor to present more correctly his given situation.

Load Type	V	Load Type	V
D	0.06	B	0.18
L and C	0.24	S	0.42
L_I	0.61	S_A	0.79
W and T	0.36	W_D	0.79
W_A	0.36	P	0.12

C1.3.1 Load Types

Following are comments, data and suggestions relating to the various common types of loads.

I. Dead loads are the self weight of the structural elements and the weight of the permanent fixtures on the structure. It is not always clear whether some types of loads are dead loads, live loads or equipment loads. It is suggested herein that in doubtful cases an estimate of the coefficient of variation could serve as the means for classification. For example, a fixture could be considered as equipment, permanent walls and partitions could be considered as dead loads, and moveable partitions could be considered as distributed live loads which are added to the uniformly distributed live loads due to occupancy. The mean dead loads can be computed from the usual published unit weights of the various materials. These values are assumed to be mean values. Should the variability of the particular dead load be higher than $V_D = 0.06$, a new value of γ_D should be computed from Eq. C1.3-1.

II. Live loads are the loads on the structure due to a specific type of occupancy. In common design practice it is customary to designate snow loads on roofs as live loads also. In these LRFD criteria such snow loads are treated separately, having their own load factors. Should the type of occupancy change, the structure needs to be reexamined in the light of the requirements of these changes.

Live loads may be classified according to the following categorizations:

A) Classification according to occupancy

1) Office
2) Residential (apartments, hotels, dormitories)
3) Parking
4) Industrial (manufacturing, utility)
5) Storage
6) Hospitals
7) Assembly
8) School

In addition there are possible unusual cases requiring special load investigations prior to design.

B) Classification according to location in building

1) Floor loads
2) Roof loads (usual and long-span)

c) Classification according to intended load combination

1) Maximum mean lifetime live load, L
2) Mean instantaneous live load, L_I

The former (L) is the maximum expected mean live load in the life of the structure and the latter (L_I) is the mean live load expected at any instant in time. The instantaneous live load is also referred to as the "sustained" live load.

At this time (1976) statistical data is available only for office type occupancy (Ref. 9), for which

$$L = 14.9 + \frac{763}{\sqrt{A_I}} \quad \text{in psf, but never more than 60 psf} \qquad (C1.3\text{-}2)$$

Alternately, for small areas for which $L > 60$ psf a moveable 2000 lb concentrated load at the critical location may be used.

$$L_I = 12 \text{ psf} \qquad (C1.3\text{-}3)$$

The term A_I is the "influence area" which is equal to

2 times the tributary area for floor beams

4 times the tributary area for columns

For the design of the slabs the distributed and concentrated loads given by ANSI-A58.1-1972 for office occupancy should be used as the appropriate mean loads. Load combinations in slabs involving L_I should use L_I = 12 psf.

For occupancies similar to the office type occupancy (i.e., residential, hospital, school) the relevant loads from Eqs. C1.3-2 and 3 are to be multiplied by the ratio of the appropriate live load intensity from ANSI-A58.1-1972 divided by 50 psf.

For the other occupancy types (i.e., not office or similar types as defined above) the appropriate live load intensities from ANSI-A58.1-1972 shall be used as the mean maximum lifetime live loads. The mean instantaneous live loads may be estimated by the designer as an appropriate fraction of the mean maximum lifetime live load.

It is evident that further research will permit a more rational treatment of these live loads. The recommendations above are, except for the case of office occupancy, based on judgment, resulting in load criteria which are as valid as the currently applicable ones. In case the necessary statistics (i.e., the mean and the coefficient of variation) are available to the designer a load factor can be determined from Eq. C1.3-1.

Roofs and members supporting roofs should be designed for load combinations involving the mean maximum lifetime live loads equal to the minimum roof loads specified in Sec. 3.3 and 3.8 of ANSI-A58.1-1972.

III. Equipment loads are due to moving or stationary equipment (trucks, cranes, hoists, monorails, machinery, computers, etc.) which cannot be considered to be part of the dead loading. In determining the load factor γ_B = 1.3 no statistical data was available, and thus it was assumed that the coefficient of variation of these loads was somewhere between that for dead loads and office live loads. In the case the designer should know the appropriate equipment loads with a greater degree of certainty, the load factor may be reduced but it should not become less than the load factor for dead loads, γ_D = 1.1. Where the equipment loads derive from moving machinery or cranes, the loads must be increased by the appropriate impact factors according to Sec. 3.4 of ANSI-A58.1-1972 or Sec. 1.3.3 and 1.3.4 of the AISC Specifications. Load combinations involving mean maximum lifetime equipment loads and occupancy and/or environmental loads should be based on the instantaneous and/or annual load values for the latter.

IV. Ponding loads are due to an accumulation of water on roofs. The load factor γ_p = 1.2 is based on an assumed coefficient of variation of 0.12 which accounts for the estimated uncertainties of water level, roof geometry and roof hydrology. The determination of the bending moment due to ponding in primary and secondary roof members must be performed according to the underlying basic theory of ponding loads (see Refs. 10 through 13).

Design aids given in Fig. C1.3-1 permit the rapid calculation of the maximum ponding moment if the roof members are simply supported, the secondary roof members framing into the primary roof member are equally spaced and of equal length on each side, and if the various assumptions stated in Sec. C.1.13.3 of the AISC Specification apply.

V. Wind loads required for the various load combinations are the mean maximum lifetime (W), annual (W_A) and daily (W_D) wind loads*. The wind load intensity determination for ordinary steel structures (as contrasted to unusual structures for which more careful studies, including wind tunnel studies, are recommended) involves the use of the effective wind velocity pressures given in Sec. 6 of ANSI-A58.1-1972 (Tables 5, 6 or 12 as required for determining external pressures on the whole or part of structures or internal pressures, respectively) for the type exposure (A, B or C), the height above ground for which the wind load is required, and for the 50 year mean recurrence interval basic wind speed obtained for the desired geographic location from Fig. 1 of ANSI-A58.1-1972. The 50 year wind speed map is the only one required to be used, regardless of the intended life of the structure.

The determination of the mean wind loads involves first the computation of the effective wind pressure, including all the modifications for shape, slope, type of structure, etc., contained in Sec. 6 of ANSI-A58.1-1972, q_{ANSI}. This pressure is then modified as follows to obtain the mean wind pressures required herein:

Mean Maximum Lifetime Wind Loads:

25 year life: $W_{25} = 1.00\ q_{ANSI}$

50 year life: $W_{50} = 1.17\ q_{ANSI}$

100 year life: $W_{100} = 1.36\ q_{ANSI}$

* The derivation of the wind load factors and the mean wind loads is given in Ref. 14.

Mean Maximum Annual Wind Loads:

$$W_A = 0.49\ q_{ANSI}$$

Mean Maximum Daily Wind Loads:

$$W_D = 0.07\ q_{ANSI}$$

The direct application of the wind load intensities and the wind load factors from these criteria results in substantially larger structural members than are obtained from the use of the AISC Specification where a one-third increase of allowable stress is permitted if wind acts alone or in combination with any other load. Since the statistical basis of the development of the LRFD wind criteria is formulated on well substantiated data and theory on the one hand, and the AISC criteria result in structures with satisfactory performance on the other hand, it is fair to question as to which approach is correct. While a clearly documented explanation for the difference is still lacking, it is evident that the theory predicting wind pressures on structures does not account for the following factors:

1) There is a substantial sharing of the wind load, which is applied to the structure as the computed wind pressure intensity, between the idealized structural elements being designed and the non-structural elements of the building as well as the portions of the structure which are ignored in the idealization. For example, a simple braced frame is usually designed such that all wind loads are resisted by the bracing. However, the "simple" connections have some moment resistance, and the cladding will also assist substantially in providing both stiffness and resistance against wind. This load sharing is especially active at the serviceability loads, where stiffness rather than strength are important.

2) The failure of steel buildings under catastrophic winds is a dynamic phenomenon, involving not only the static strength of the steel

structure, but also its dynamic properties and its ductility. While failure under wind forces is but incompletely understood, there are some parallels to the failure of steel structures under earthquake motions where strength, ductility and dynamic properties all play an important role.

While some current research is underway to study the behavior of the whole building system in wind, much more work needs to be done to fully quantify the behavior. Until such a time as more is known and explicit rules can be formulated, it is recommended that wind forces for all wind load types are multiplied by a factor 0.6. This factor brings the final designs essentially in line with structures designed for wind by all of the currently used codes (14). The factor should not, however, be used when considering overturning effects due to wind.

For buildings that are enclosed! See Report 39

VI. Snow loads used with the various load combinations are the mean maximum lifetime and annual snow loads. The roof snow loads are determined from the formulas given below:

Mean Maximum Lifetime Snow Load:

$$S = C_s \, q_{Lm} \qquad \text{(C1.3.2-4)}$$

Mean Maximum Annual Snow Load:

$$S_A = C_s \, q_{Am} \qquad \text{(C1.3.2-5)}$$

where C_s is a shape factor depending on the roof characteristics as per Sec. 7.2, ANSI-A58.1-1972 (usually $C_s = 0.8$) and q_{Lm} and q_{Am} are the ground snow load intensities obtained from the formulas

$$q_{Am} = \frac{62.4}{12} \left\{ \exp \left[(\ell n\, X)_m + \frac{1}{2} (\sigma_{\ell n\, X})^2 \right] \right\} \qquad \text{(C1.3.2-6)}$$

$$q_{Lm} = q_{Am} \left[1 + 3.70 \sqrt{\exp\,(\sigma_{\ell n\ X})^2 - 1} \right] \qquad (C1.3.2\text{-}7)$$

In these equations $(\ell n\ X)_m$ is the mean of the logarithm of the water equivalent of the ground snow, taken from the map in Fig. C1.3-2a, and $\sigma_{\ell n\ X}$ is the standard deviation of the logarithm of the water equivalent of ground snow, obtained from the map in Fig. C1.3-2b*.

The snow load factors given in Sec. 1.3.3 are average values (Ref. 14). The actual values can be determined as follows:

$$\gamma_S = 1 + 0.55\ \beta\ V_S \qquad (C1.2.3\text{-}8)$$

for the mean maximum lifetime snow load and

$$\gamma_S = 1 + 0.55\ \beta\ V_{S_A} \qquad (C1.2.3\text{-}9)$$

for the mean maximum annual snow load.

In these equations

$$V_S = \sqrt{\frac{1}{2}\left[\exp\left(\sigma_{\ell n\ S_A}\right)^2 - 1\right] + {V_{C_S}}^2} \qquad (C1.2.3\text{-}10)$$

and V_{S_A} is given by Eq. C1.2-10. The value of $\sigma_{\ell n\ S_A}$ is taken from the map in Fig. C1.3-2b. With $\beta = 3.0$ and $V_{C_S} = 0.15$, the following table of snow load factors can be calculated:

* These maps are reproduced here and in Ref. 14 from Ref. 15. The snow load values in Table C1.3.1-1 were determined from the data from the maps to permit a rapid estimation of the snow loads. It should be recognized that the snow data is for average conditions, and it does not reflect the situation in deep valley or mountainous regions.

$\sigma_{\ln S_A}$	γ_S	γ_{S_A}
1.0	2.5	3.1
0.9	2.3	2.9
0.8	2.1	2.6
0.7	2.0	2.3
0.6	1.8	2.1
0.5	1.7	1.9
0.4	1.6	1.7
0.3	1.4	1.6

VII. Temperature induced forces are usually determined for the extreme ranges of temperature to which various portions of structures are subjected. In case a careful analysis is required it is recommended that local records of the temperature data be examined and the appropriate mean temperatures and the corresponding load factors be determined (Eq. C1.2-5). Otherwise it is recommended in Sec. 1.3.3 that the load factor for temperature be $\gamma_T = 1.6$, the same as the load factor for the mean maximum lifetime wind loads.

VIII. Construction loads depend on the type of construction and there is very little data to back the development of load factors. The designer must use his judgment in the estimation of these loads. In the absence of statistical evidence it is recommended that the same load factor be used as for the mean maximum lifetime live loads, i.e., $\gamma_c = 1.4$. For composite beams it is recommended that construction loads equal the weight of the wet concrete plus 20 psf.

C1.3.2 Load-Combinations

The load-combinations listed in Sec. 1.3.2 encompass the usual possibilities. Other load-combinations may apply and should be considered if necessary. Of special importance may be partially loaded members or structures where many judgmental factors might need to be considered in determining the most critical loading. In individual cases often only a few of the load-combinations apply, and often it is possible to eliminate the combinations which surely will not control.

In considering load combinations it should be realized that dead load is always present and that other lifetime maximum loads should be combined with instantaneous or annual maximum loads. For example, the maximum lifetime wind loads are to be considered in combination with the instantaneous live loads (Eq. 1.2-3).

Because of the multiplicity of the combinations and load factors it is essential that great care be exercised in the bookkeeping. Forces from various load types should be identifiable as to origin (D, L, W etc.), and special care should be taken that unfactored load effects are determined at material interface locations in the structure where two structural specifications may demand different load factors (e.g., interface between steel and concrete or wood, or the interface at the foundation).

C1.3.3 Load Factors

The load factors in Sec. 1.3.3 were determined according to the available statistical information, and the previous sections in this Commentary have indicated the extent to which this information was available and where estimates had to be made. It should be realized that the bases of the loads and load factors are at least as valid as those of the current practice. The background, the data, theoretical bases and the derivations for loads and load factors is given in greater detail in Refs. 1 and 14.

City	State	q_{Am} (psf)	q_{Lm} (psf)
Paducah	Kentucky	3	15
New Orleans	Louisiana	1	7
Shreveport	Louisiana	1	5
Augusta	Maine	23	78
Baltimore	Maryland	5	24
Boston	Massachusetts	6	21
Marquette	Michigan	25	64
Detroit	Michigan	5	15
Minneapolis	Minnesota	11	48
Duluth	Minnesota	20	50
Jackson	Mississippi	1	10
St. Louis	Missouri	4	22
Great Falls	Montana	6	14
Billings	Montana	4	29
North Platte	Nebraska	6	11
Lincoln	Nebraska	9	41
Winnemucca	Nevada	3	6
Las Vegas	Nevada	2	10
Concord	New Hampshire	13	43
Trenton	New Jersey	7	32
Raton	New Mexico	3	12
Albuquerque	New Mexico	1	9
Las Cruces	New Mexico	2	9
Albany	New York	11	34
New York	New York	7	32
Raleigh	North Carolina	3	14

City	State	q_{Am} (psf)	q_{Lm} (psf)
Wilmington	North Carolina	2	13
Bismarck	North Dakota	9	24
Fargo	North Dakota	7	29
Cleveland	Ohio	6	16
Columbus	Ohio	4	11
Cincinnati	Ohio	3	9
Oklahoma City	Oklahoma	3	9
Tulsa	Oklahoma	2	10
Blue Mountains	Oregon	7	32
Eugena	Oregon	2	10
Portland	Oregon	5	19
Pittsburgh	Pennsylvania	5	14
Harrisburg	Pennsylvania	6	24
Philadelphia	Pennsylvania	5	24
Providence	Rhode Island	7	26
Columbia	South Carolina	2	13
Rapid City	South Dakota	6	18
Sioux Falls	South Dakota	9	39
Memphis	Tennessee	2	10
Knoxville	Tennessee	3	17
Amarillo	Texas	3	11
Forth Worth	Texas	2	7
Austin	Texas	1	5
Salt Lake City	Utah	4	8
Lake Powell Area	Utah	3	6
Montpelier	Vermont	17	58

City	State	q_{Am} (psf)	q_{Lm} (psf)
Richmond	Virginia	4	20
Seattle	Washington	3	10
Spokane	Washington	6	21
Charleston	West Virginia	3	12
Green Bay	Wisconsin	9	32
Madison	Wisconsin	7	32
Worland	Wyoming	6	21
Cheyenne	Wyoming	4	10

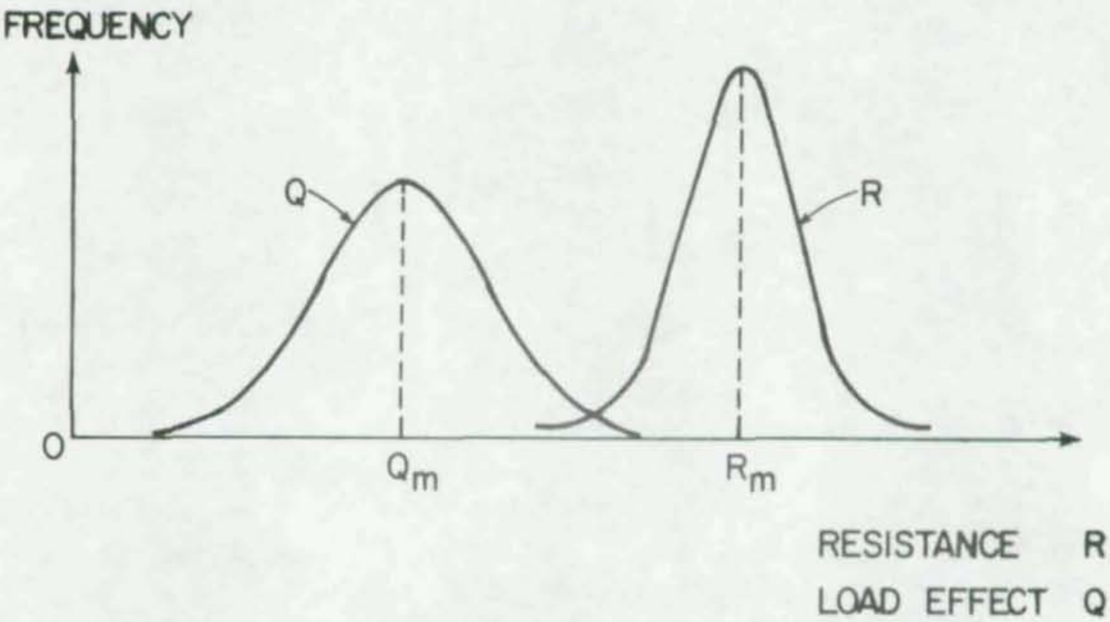


Fig. C1.2-1 Frequency Distribution of Load Effect Q and Resistance R

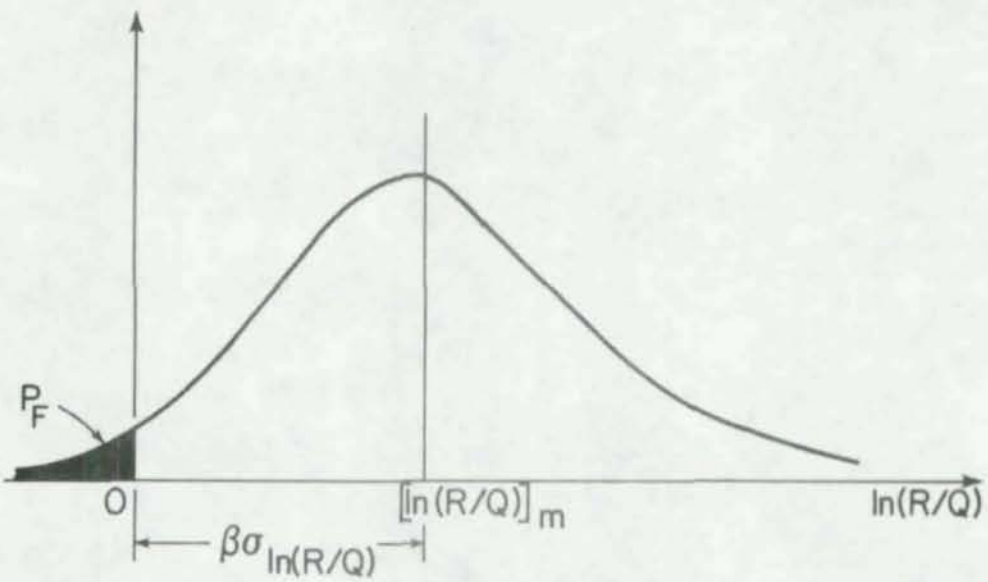


Fig. C1.2-2 Definition of Safety Index

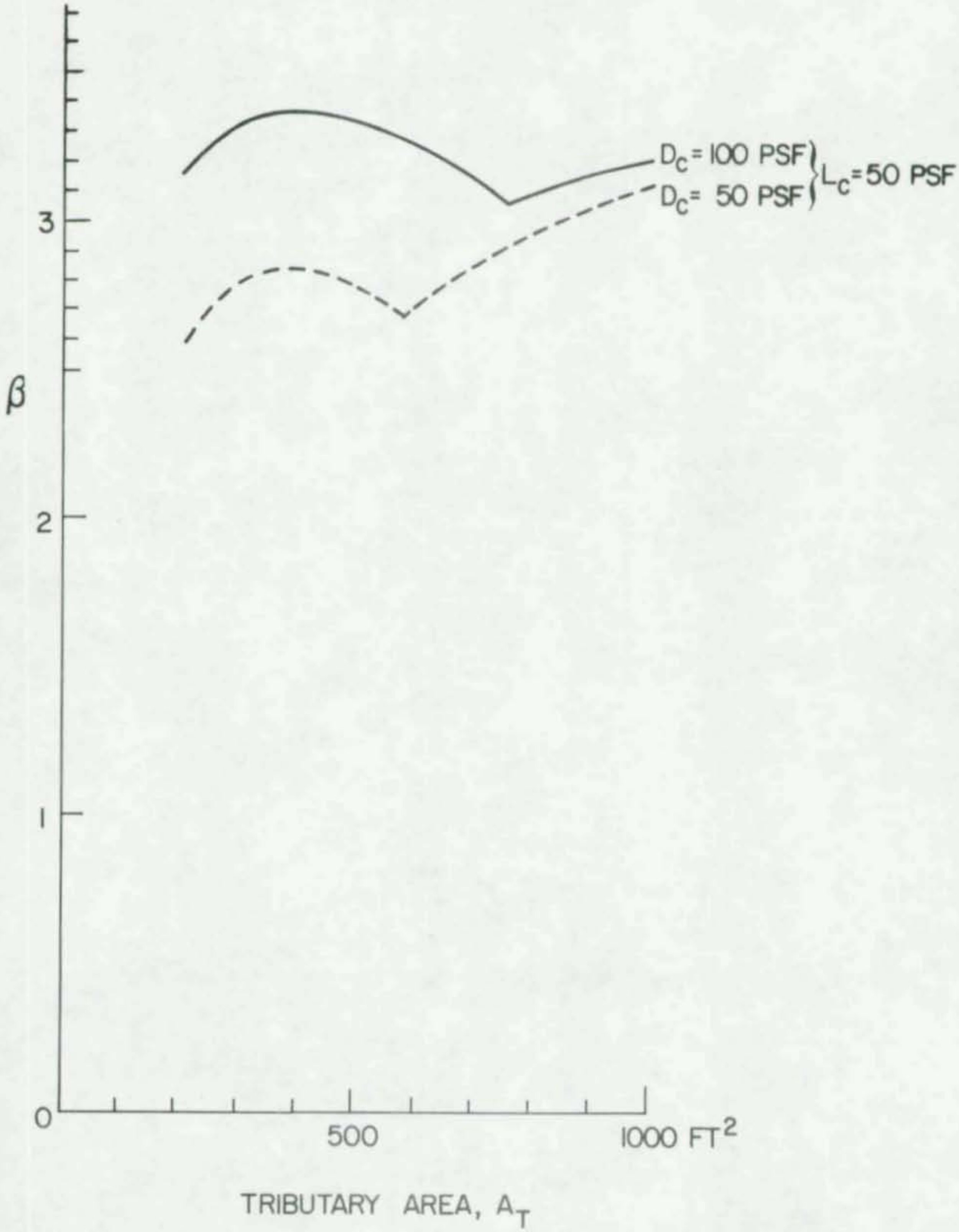
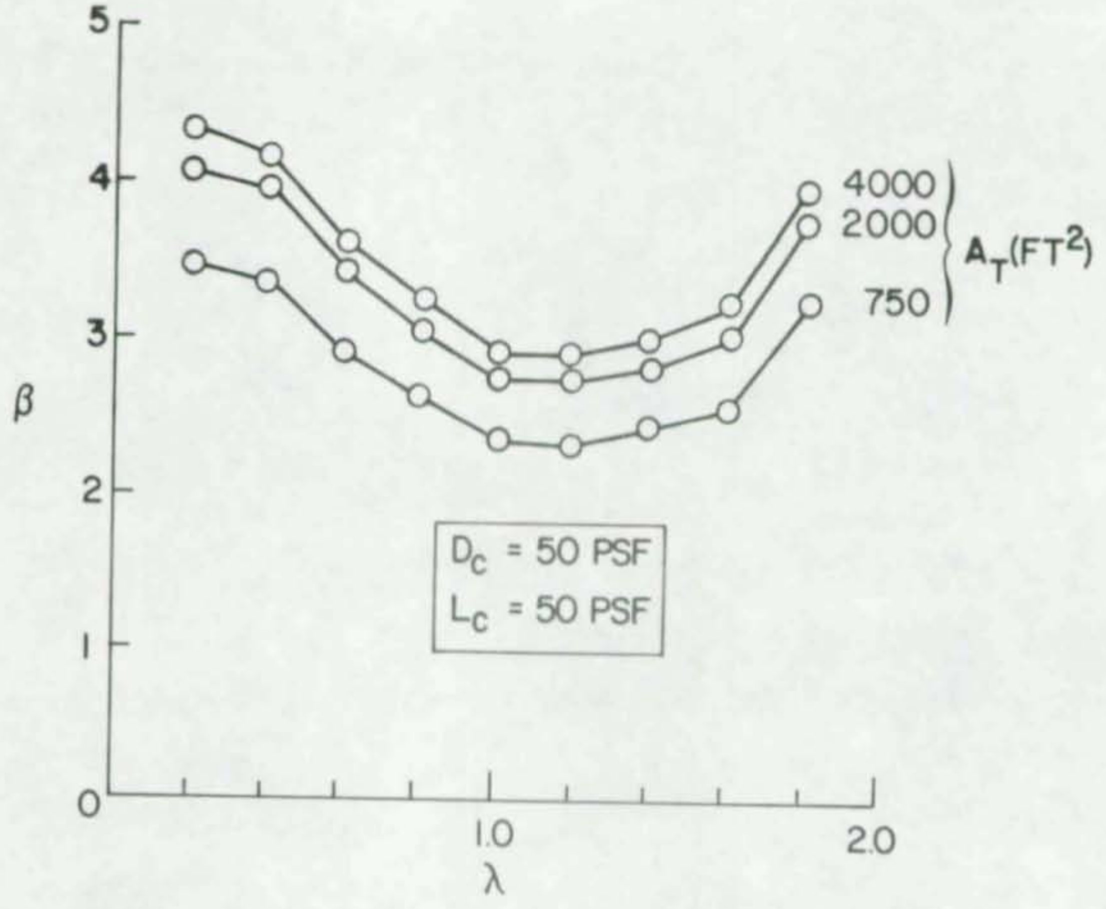


Fig. C1.2-3 Variation of β for Beams



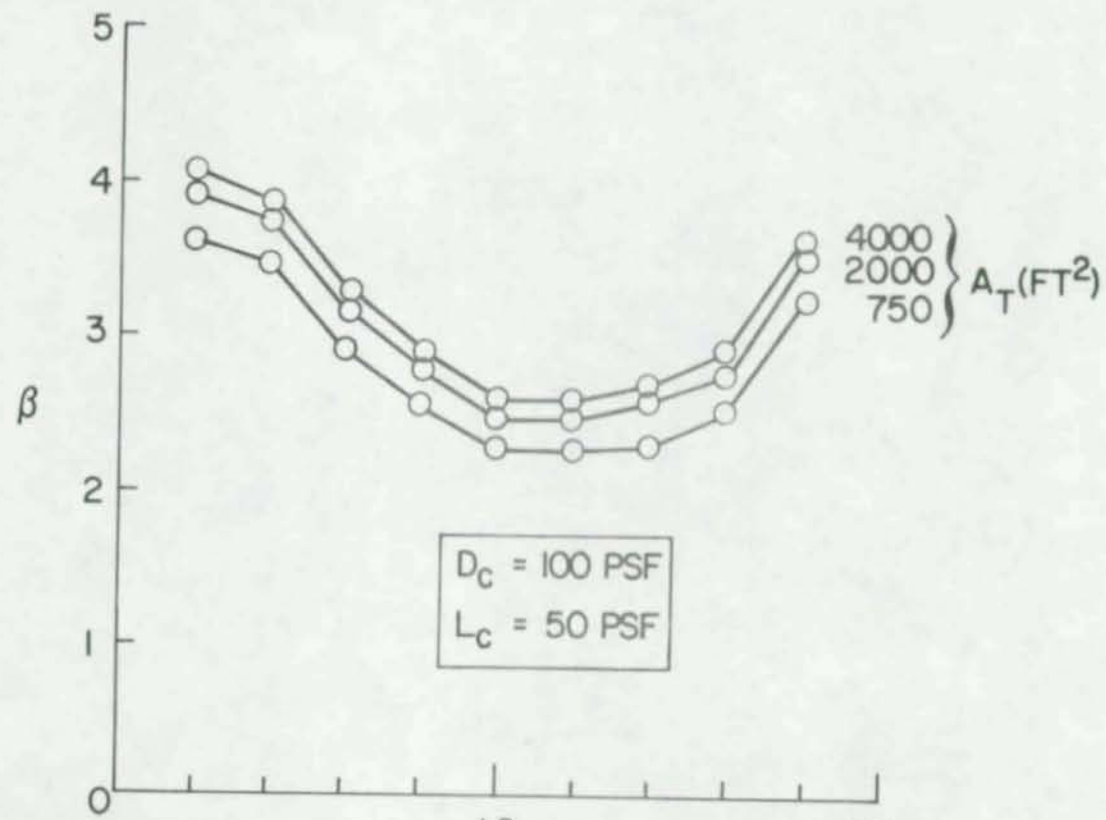


Fig. C1.2-4 Variation of β for Columns

Fig. C1.3-1 MAGNIFICATION FACTORS FOR MAXIMUM MID-SPAN BENDING MOMENTS FOR SIMPLY SUPPORTED PRIMARY AND SECONDARY ROOF MEMBERS UNDER PONDING LOADS

Assumptions:

1) Height of water over support of primary members: "h"
2) Ends of primary members rest on unmoving supports
3) All deflections are sinusoidal
4) Secondary members frame at right angles to the primary member, they are equally spaced, their length is equal on both sides of the primary member, their ends deflect the same amount on each end
5) No camber
6) Elastic behavior

These assumptions are the same as those in Sec. 1.13 of the AISC Specification.

Multiply the maximum moment due to the uniformly distributed water load g'h

$$(M_{max})_{\text{Primary beam}} = \frac{g'h\ L_S\ L_P^2}{8}$$

$$(M_{max})_{\text{Secondary beam}} = \frac{g'h\ S_S\ L_S^2}{8}$$

by the magnification factors $\bar{M}_P$ and $\bar{M}_S$, respectively

C1.3-1a

where $\overline{M}_P$ and $\overline{M}_S$ are given in the charts of Figs. C1.3-1b and C1.3-1c, or by the formulas

$$\overline{M}_P = j \left[1 + \frac{8K}{\pi^2} \left(\frac{j\, C_P}{1 - j\, C_P} \right) \right]$$

$$\overline{M}_S = \left[\left(1 - \frac{4}{\pi} \right) + \left(\frac{4}{\pi} \right)_j \right] \left[1 + K \left(\frac{j\, C_P}{1 - j\, C_P} \right) \right]$$

where: g' = specific gravity of water

S_S = spacing of secondary beams

L_S = length of secondary beams

L_P = length of primary beams

h = height of water above support of primary beams

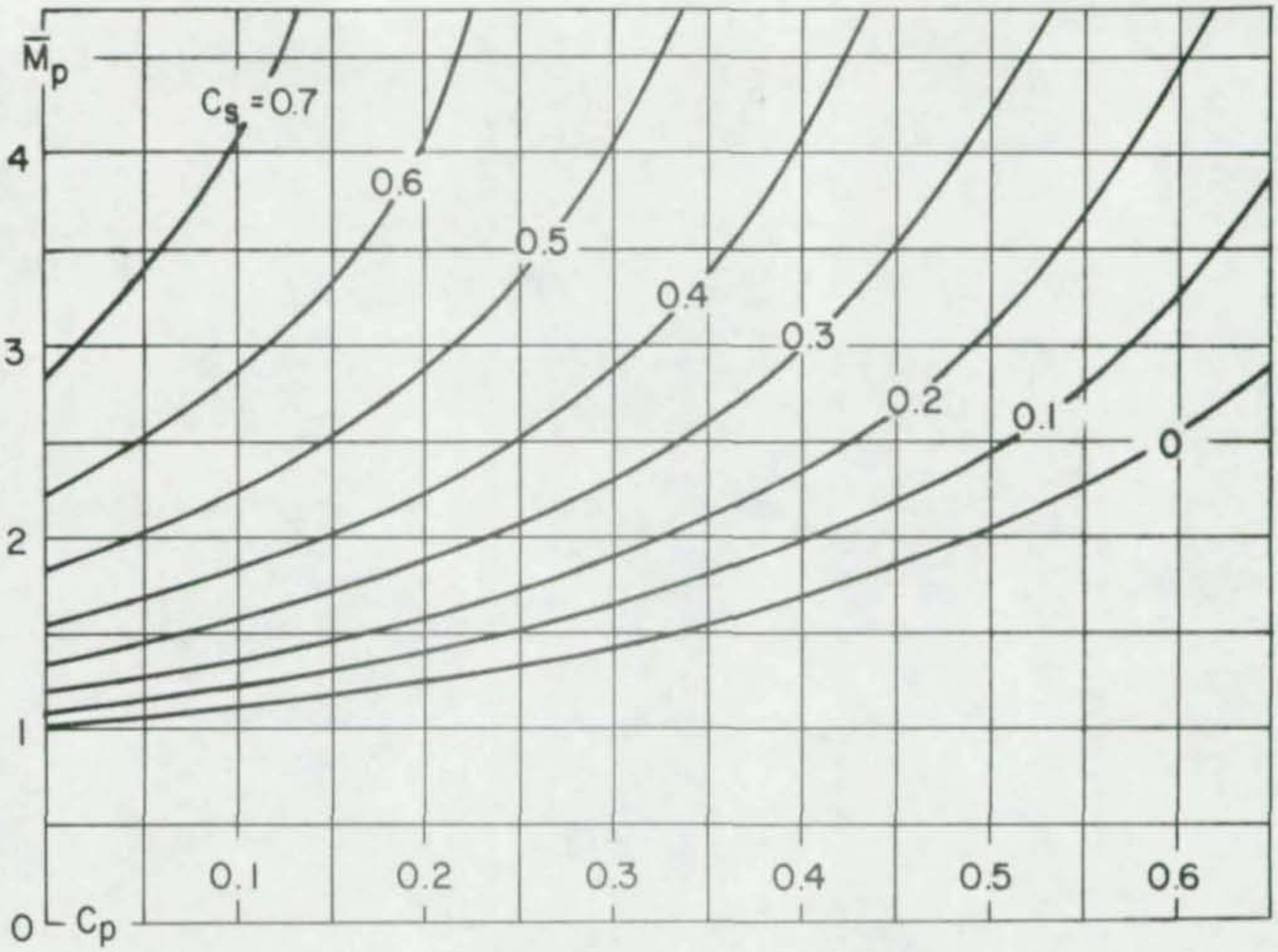
$$C_P = \frac{g' L_S\, L_P^4}{\pi^4\, E\, I_P} \; ; \quad C_S = \frac{g' S_S\, L_S^4}{\pi^4\, E\, I_S} \; ; \quad \alpha_S = \frac{C_S}{1 - C_S}$$

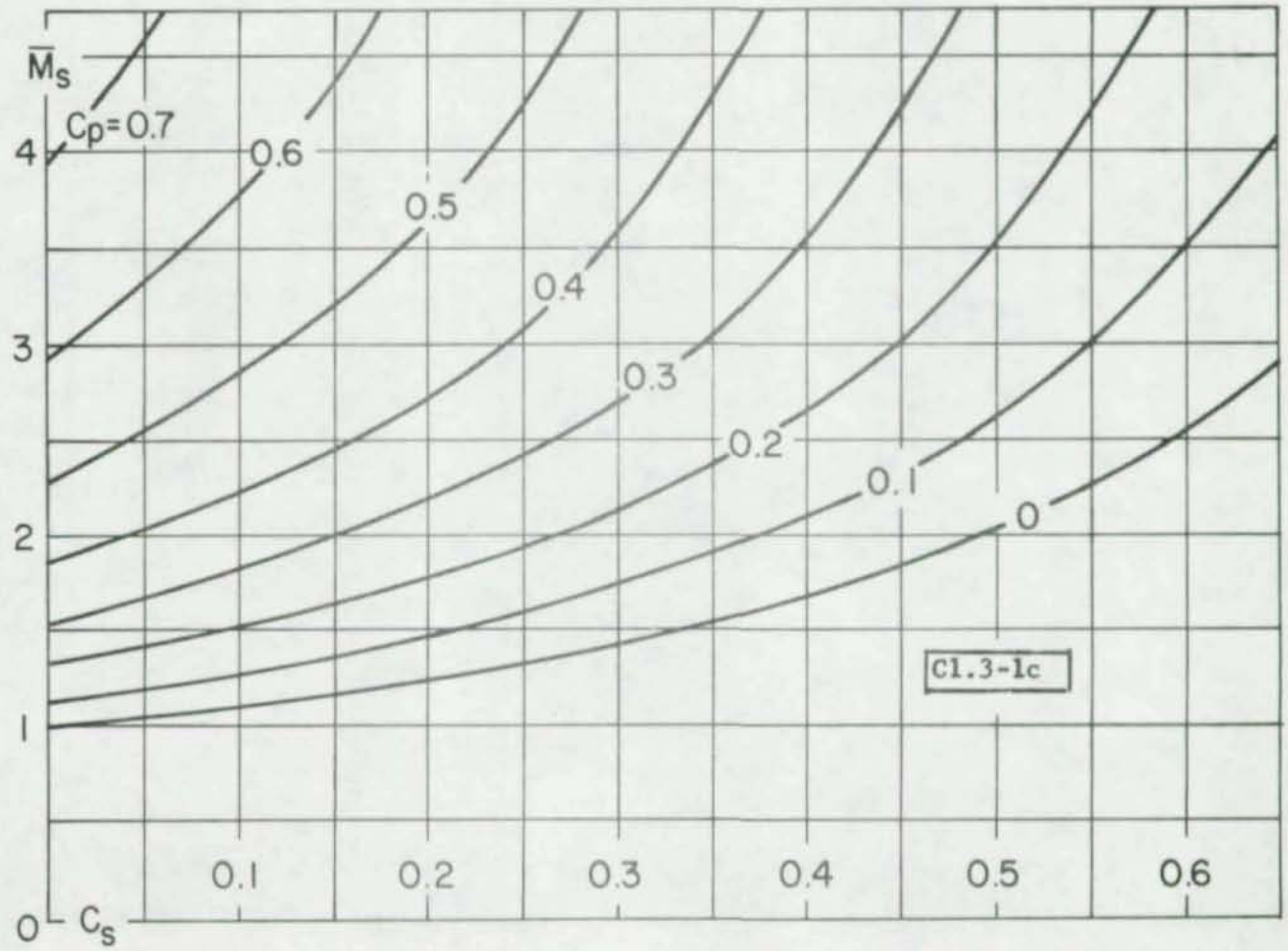
$$j = 1 + \frac{2\, K\, \alpha_S}{\pi} \; ; \quad K = \frac{5\, \pi^4}{384}$$

E = modulus of elasticity

I_S = second moment of area of secondary beam

I_P = second moment of area of primary beam


M̄p
Cs = 0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
4
3
2
1
0
0.1
0.2
0.3
0.4
0.5
0.6
Cp


M̄s
Cp = 0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
4
3
2
1
0
0.1
0.2
0.3
0.4
0.5
0.6
Cs

C1.3-1c

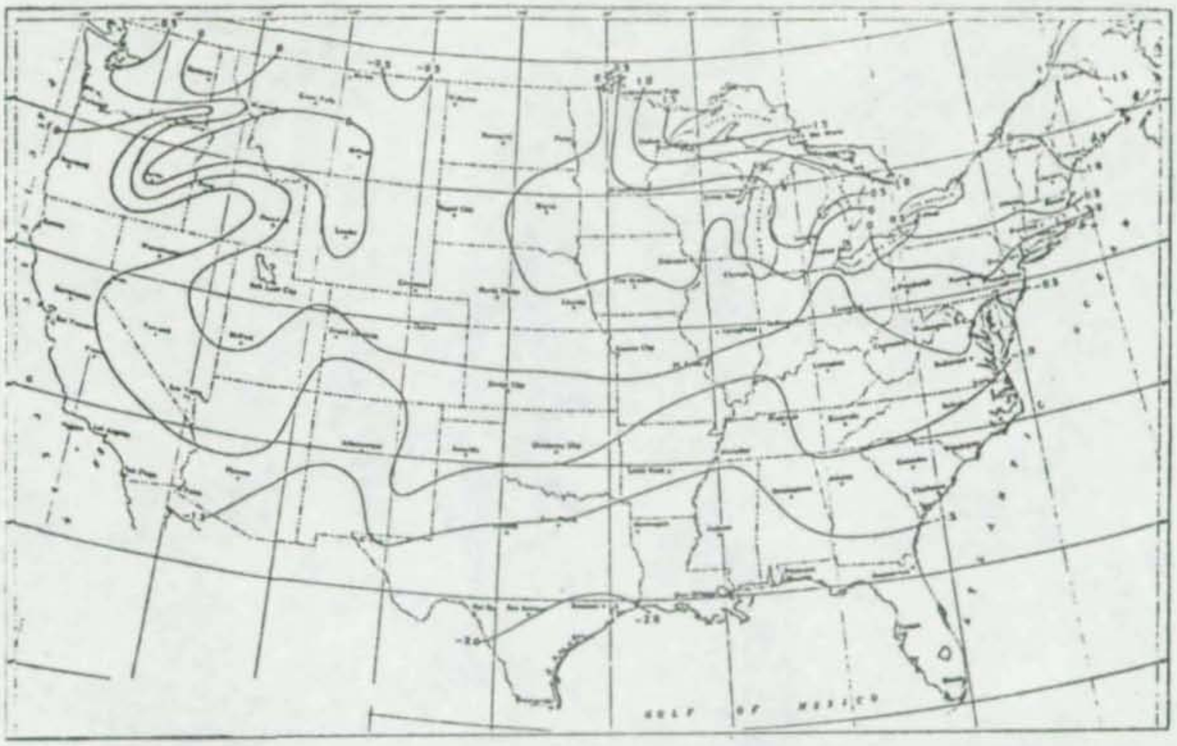


Fig. C1.3-2a Mean of the logarithms of the water equivalent of ground snow.

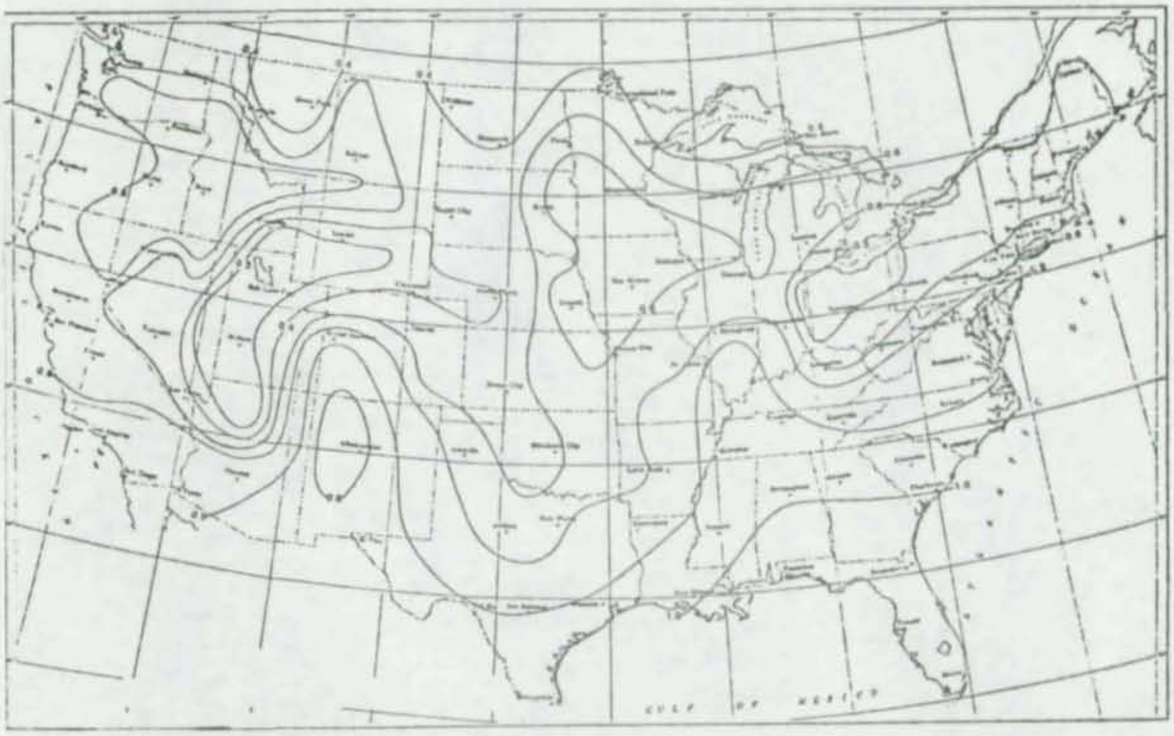


Fig. C1.3-2b Standard deviation of the logarithms of the water equivalent of ground snow.

Section C.2: Design Criteria for the Limit State of Strength

The design rules in Sec. 2 of these LRFD criteria pertain to the condition when the limit state is the strength or, using an equivalent definition, the ultimate capacity of a steel structure.

C2.1 Types of Structures

The design criteria given herein are meant to be applied to the design of the same kinds of structures for which currently the AISC Specifications are used: building structures fabricated from hot-rolled plates and/or shapes. They should not be used for other types of steel structures because the load and resistance statistics may be different, and the level of reliability against exceeding a limit state (β = 3.0 herein) may not be the same.

C2.1.1 Material

The steel types and grades recommended for use in these LRFD criteria are the same as in the AISC Specification. No further restrictions are placed on the material requirements than those contained in the present ASTM Specifications.

C2.1.2 Framing

The AISC Specification recognizes three types of framing: "simple", where the structure or an element of it, is idealized to be statically determinate; "rigid", where the joints of the structure are rigid so that for purposes of analysis it can be assumed that the original slopes between elements remain the same after the structure is loaded, and the structure is analyzed as statically indeterminate; and "semi-rigid" where the joints are intermediate in stiffness between the simple and the rigid condition, and the flexibility of these joints must be accounted for in the force analysis.

Two conditions are recognized in the LRFD criteria, as well as in the AISC Specification, with regard to the stability of the whole frame: 1) braced frames and 2) unbraced frames. The condition of bracing refers to a joint at story level at which location side-sway buckling is either prevented by diagonal or other positive bracing or by attachment to a shear-wall or to another structure, or it is not prevented. This distinction results in the choice of an effective column length as either being less than or equal to the column length for the side-sway prevented case, or larger than the column length for the case where side-sway is not prevented. Individual members in either the braced or the unbraced frames may or may not be laterally braced, and this must be considered in the design of these members.

C2.2 Structural Analysis

The forces in the members are determined from the factored loads given in Sec. 1 of the LRFD criteria. More than one analysis may need to be performed when multiple load combinations apply and when it is not evident which combination is critical.

In the large majority of structural steel design situations structural analysis is performed by formulating the equilibrium on the undeformed structure. This is known as "first-order" analysis, and many standard computer programs are available to the designer in performing such analyses for statically indeterminate structures according to elastic theory.

The forces in statically indeterminate structures may be determined by either plastic analysis or by elastic analysis. In plastic analysis the strength limit state is the formation of a plastic mechanism, while in elastic analysis the limit state is the attainment of a moment capacity determined by the full plastification of one section (i.e., the first plastic hinge formation is the limiting criterion) or by the attainment of a force causing instability of the member. In case plastic analysis is used it is necessary to insure that the hinge-rotations required for the development of a mechanism can take place by limiting the unbraced length and the flange and web width-thickness ratios as defined in Sec. 2.3.3.3.1.

In multi-story frames of more than two stories subjected to combined gravity and wind loads it is usually necessary to consider secondary bending effects due to the increase of story shears caused by the product of the story deflection and the gravity loads. Approximate and iterative methods of accounting for these P-delta forces are given in Chap. 15 of the Column Research Council Guide (Ref. 17) for the design case where the strength limit state is either the formation of the first plastic hinge or instability. If these P-delta forces have been determined explicitly, then the beam-columns in such frames may be designed with an effective length factor equal to unity and for the actual computed end moments (see Sec. 2.3.4.2). Alternately, if the P-delta forces have not been included explicitly (i.e., the forces in the frame have been determined by a first-order analysis), then the beam-column design must reflect this in using an effective length factor larger than unity and a modified moment amplification factor by which the first-order moments must be multiplied (see Sec. 2.3.4.2).

While the designer has a choice whether or not to determine the second-order forces explicitly or to account for them indirectly when the limit state is not a plastic mechanism, it is essential that P-delta forces be considered when plastic design is used for multi-story frames. Various analysis methods for braced (Ref. 18 and Chap. 10 of Ref. 19) and unbraced frames (Chap. 10, Ref. 19) are available for the use of the designer.

Multi-story frames are usually designed for stringent drift limits, and the design of slender frames is usually controlled by this serviceability criterion. A study on regular multi-story frames (Ref. 20) designed by Part 1 of the AISC Specification has suggested that the control of the drift under serviceability limit states loading serves also to control frame stability under the maximum life-time loads. This study resulted in considerable relaxations in the requirements for unbraced frames (see Sec. C.1.8 in the AISC Specification, Supplement No. 3). Unfortunately no equivalent studies for LRFD have yet been made, and even though intuitively it is reasonable to expect a similar outcome, it is necessary to await the results of a comparable analysis before similar relaxations can be included herein.

C2.3 The Design of Members

This section of the LRFD criteria contains the requirements for the strength limit states of structural members. Structural members are classified as the elements between the joints or supports in the structure. Structural analysis, elastic or plastic, first-order or second-order, as appropriate, of the assumed preliminary structure under the factored loads provides the designer with the factored design forces which must be shown to be less than the factored nominal resistance ϕR_n. This section furnishes values of the resistance factor ϕ and formulas for the nominal resistance R_n for members classified according to the predominant forces acting on them: tension members (Sec. 2.3.1), compression members (Sec. 2.3.2), flexural members (Sec. 2.3.3), members under combined flexure and axial force (Sec. 2.3.4), and members under combined stress (Sec. 2.3.5). This latter section contains provisions for such items as the design of unsymmetric shapes and the design for combined compression, flexure and torsion, as well as guide-lines for unusual situations not covered in the other sections. The first four sections, e.g. Sec. 2.3.1 through Sec. 2.3.4, contain the provisions for the commonly encountered elements in steel structures: columns, beams (including plate-girders, composite beams, and hybrid girders) and beam-columns.

The resistance factors ϕ have been estimated from analyses of experimental and analytical data available in the literature and by applying engineering judgment where such data were incomplete or entirely absent. The basis of determining ϕ has been described in Ref. 1, and the details of the data analysis are given further in Ref. 21 (Beams), 22 (Plate-girders), 23 (Beam-Columns), 24 (Composite Beams), and also in Ref. 1 (compact beams, columns). The estimation of ϕ was based on the formula (from Ref. 1)

$$\phi = \frac{R_m}{R_n} \exp(-0.55\,\beta\,V_R) \qquad (C2.3\text{-}1)$$

where R_m = mean resistance as determined by structural theory and/or experiment for the mean material properties appropriate to the case under consideration

R_n = the nominal resistance based on the appropriate formula which is used and for the specified material properties

β = safety index determined by calibration, as discussed in Sec. C.1.2. The basic safety index of $\beta = 3.0$ is used, except in a few cases in Sec. 2.3 where the Commentary will note the exception, and in Sec. 2.4 where $\beta = 4.5$ is used for connections. Since the value of β also influences the load factors (Eqs. C1.2-4 and C1.2-5), and these are given in Sec. 1 for $\beta = 3.0$, a change in $\beta \neq 3$ results in some adjustments which are absorbed in the value of ϕ (see Ref. 4 and Sec. C.2.4 of this Commentary) rather than in changing the load factors for the different types of members.

V_R = Coefficient of variation of the resistance.

The value of V_R is determined from the formula

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \quad \text{(C2.3-2)}$$

where the three coefficients of variation V_M, V_F and V_P account for the variability of the material properties, the fabrication tolerances and the ratio of the prediction of the mean resistance to the experimental results, respectively (Ref. 1). The material property statistics depend on the particular type of property used, e.g., F_y, F_u, E, the fabrication tolerance is assumed to have a value of 5% (e.g., $V_F = 0.05$) throughout and V_P depends on the particular type member. The appropriate statistical parameters used in determining ϕ will be noted in this Commentary.

C2.3.1 Tension Members

The limit states relevant to the design of tension members are 1) full plastification (i.e., onset of overall yielding) of the net section and 2) tensile rupture of the net section. Distinction is made in the criteria for the limit state of plastification in pin holes, where the net section strength $A_n F_y$ is multiplied by the factor 0.75 to account for localized plastic deformation caused by stress concentrations at the sides of the hole (see p. 325, Ref. 26 or Sec. C.1.5.1.1 in the Commentary to the AISC Specification). The resistance factor ϕ_{ty} is based on the statistics $F_{ym} = 1.05\ F_y$ (where $R_m = A_n\ F_{ym}$, F_{ym} being the mean yield stress - Ref. 1), $V_M = 0.1$, $V_F = 0.05$ and $V_P = 0$. The resistance factor ϕ_{tu} is based on

$F_{um} = 1.10\ F_u$, $V_M = 0.10$, $V_F = 0.05$, $V_P = 0$ and a $\beta = 4.5$. This latter value of β is the same as that used for connections, reflecting the implied increased reliability for this type of failure over yielding in the AISC Specification where F.S. = 2.0 is used for the former and F.S. = 5/3 is used for the latter. To account for the increased β, ϕ from Eq. (C2.3-1) is multiplied by 0.88 (see Ref. 4).

C2.3.2 Compression Members

C2.3.2.1 Factored Maximum Strength

The basis for the formulas for ϕ_c and R_{nc} given in this section is presented in detail in Ref. 1. In order to retain continuity with the AISC Specification the same basic column formula is used in the LRFD criteria. Table C2.3.2.1-1 contains values of the ratio $\phi_c\ F_{cr}/F_y$ for the range of the slenderness parameter λ from 0 to 2.10 in intervals of 0.01. The column strength statistics used in the development of ϕ_c are based on the column research performed at Lehigh University under the guidance of Task Group 1 of the Column Research Council, and this research is described and fully referenced in Chap. 3 of the Column Research Council Guide (Ref. 17). The data base is for solid uniform columns of symmetric rolled or welded built-up shapes made from hot-rolled elements, and the variability underlying V_R reflects the spread for the whole range of column types which were investigated. In case the designer has data available from the literature (e.g. Chap. 3 of the Column Research Council Guide, Ref. 17) which provide a formula for the mean strength of a particular column-type being used, a uniform value of $\phi_c = 0.86$ may be used in lieu of the variable ϕ_c from Eqs. 2.3.2-1, which varies with the slenderness parameter λ.

C2.3.2.2 Effective Length Factor

The comments and charts in Sec. C1.8 of the AISC Specification regarding frame stability and effective length factors apply also to these LRFD criteria. Further analysis methods, formulas, charts and references are provided in Chap. 15 of the Column Research Council Guide (Ref. 17) for the determination of the effective length.

C2.3.2.3 Flexural-Torsional Buckling

A possible mode of buckling of columns is torsional buckling for symmetric shapes and flexural-torsional buckling for unsymmetric shapes. These modes are usually not considered in design for the hot-rolled columns because they generally do not govern, or the critical load differs very little from the weak-axis planar buckling load. Such a buckling mode may, however, control the capacity of columns made from plate elements which are relatively thin, and for unsymmetric columns. Formulas for determining the flexural-torsional elastic buckling loads of such columns are derived in texts on structural stability (Refs. 27 through 29, for example). They are given below for the convenience of the designer: For symmetric shapes, the critical elastic torsional buckling stress is

$$F_{cr} = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right]\left[\frac{1}{I_x + I_y}\right] \tag{C2.3.4-1}$$

For singly symmetric shapes one of the critical loads is buckling in the plane of symmetry, and the flexural-torsional elastic buckling stress is

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4 F_{cry} F_{crz} H}{(F_{cry} + F_{crz})^2}}\right] \tag{C2.3.4-2}$$

For unsymmetric shapes the elastic flexural-torsional buckling stress is the lowest root of the cubic equation

$$(F_{cr} - F_{crx})(F_{cr} - F_{cry})(F_{cr} - F_{crz}) - F_{cr}^2 (F_{cr} - F_{cry})\left(\frac{x_o^2}{r_o^2}\right) - F_{cr}^2 (F_{cr} - F_{crx})\left(\frac{y_o^2}{r_o^2}\right) = 0 \tag{C2.3.4-3}$$

In these equations the terms are defined as follows:

K_zL = effective length of torsional buckling

E = modulus of elasticity

G = shear modulus

C_w = Warping constant (in.^{6})

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

I_x, I_y = Moment of inertia about x and y axis, respectively

x_o, y_o = Coordinates of shear center with respect to the centroid

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A} \qquad \text{(C2.3.4-3)}$$

$$H = 1 - \frac{y_o^2}{\bar{r}^2} \qquad \text{(C2.3.4-4)}$$

$$F_{crx} = \frac{\pi^2 E}{(K_x L/r_x)^2} \qquad \text{(C2.3.4-5)}$$

$$F_{cry} = \frac{\pi^2 E}{(K_y L/r_y)^2} \qquad \text{(C2.3.4-6)}$$

$$F_{crz} = \left\{ \frac{\pi^2 E C_w}{(K_z L)^2} + GJ \right\} \frac{1}{A \bar{r}_o^2} \qquad \text{(C2.3.4-7)}$$

K_x , K_y = effective length factors in x and y direction, respectively

r_x , r_r = radii of gyration about x and y direction, respectively

Since these equations for torsional flexural buckling apply only to elastic buckling, they must be modified for inelastic buckling when $F_{cr} > 0.5\ F_y$. This is accomplished through the use of the equivalent slenderness factor $\lambda_{eq} = F_y/F_{cr}$ (Eq. 2.3.2-6).

C2.3.2.4 Tapered Members

The factored resistance of wide-flange columns with a single web-taper and constant flanges follows the same procedure as for uniform columns according to Sec. 3.2.1, except that λ for major axis buckling is determined for a slenderness ratio $K_\gamma L/r_{ox}$ and for minor axis buckling for KL/r_{oy}, where K_γ is an effective length factor for tapered members (see AISC Specification Commentary Section D.2, Supplement No. 3 for charts to determine K_γ), K is the effective length factor for prismatic members and r_{ox} and r_{oy} are the radii of gyration about the x and the y axes, respectively, taken at the smaller end of the tapered members.

For stepped columns or columns with other than a single web-taper the elastic critical stress is determined by analysis or from data in reference texts or research reports (see Refs. 28, 29 and Chap. 11 and 13 in Ref. 17), and then the same procedure of using λ_{eq} is utilized in calculating the factored resistance.

This same approach is recommended for open-section built-up columns (columns with lacing, battens or perforated cover-plates) where the elastic critical buckling stress determination must include a reduction for the effect of shear. Methods for calculating the elastic buckling stress of such columns are given in Refs. 28 and 29, and in Chap. 12 of the Column Research Council Guide (Ref. 17)

C2.3.3 Flexural Members

This section covers the design of beams and girders, i.e., members which are subjected to forces which cause flexure and shear in a plane of symmetry. Included herein are the formulas for the nominal resistance of beam and plate-girder webs in shear, (Sec. 2.3.3.2), beams (Sec. 2.3.3.3.1) and plate girders (Sec. 2.3.3.3.2) in flexure, and composite beams (Sec. 2.3.3.3.3). The basis for the particular values of the resistance factor ϕ and the formulas for the nominal resistance is presented in Refs. 1 ("compact" beams), 21 (beams), 22 (plate-girders) and 24 (composite beams). The nominal resistance formulas are based on maximum capacities in flexure and shear, and so they appear to differ considerably from corresponding provisions in Part 1 of the AISC Specification where allowable stresses are used. However, the same fundamental research results have been used herein and so a closer inspection will reveal many similarities. This is especially so for plate and hybrid girders. The provisions for beams have been streamlined, and the tabular representation in Table 2.3.3.3 and the design aid tables in Table C2.3.3.1-1 permit a reasonably simple way to determine the maximum capacity of beams. The provisions for composite beams have been modified from the treatment in the AISC Specifications by basing the flexural capacity on either the fully plastic capacity of the composite cross section or on the capacity as determined by shear-connector strength. It should be noted that partial shear connection is permitted and that the material has been expanded to include composite beams having slabs on formed steel deck.

C2.3.3.2 Factored Maximum Strength of Webs in Shear

The limit state for compact webs in shear ($h/t \leq 425/\sqrt{F_{yw}}$) is the full plastification of the web (Eq. 2.3.3.2-2). For slender webs in interior panels of plate girders for which the web and the flanges are fabricated from the same grade of steel the limit state is the formation of a tension-field (Eq. 2.3.3.2-4), while the limit state in end panels and in all panels of hybrid plate-girders the limit state is plate buckling (Eq. 2.3.3.2-5). As in the AISC Specification, no provisions are given for longitudinally stiffened plate-girders.

The stiffener requirements for transverse stiffeners and end-stiffeners are essentially the same as in the AISC Specification. Stiffeners in interior panels need to be checked for an area requirement (Eq. 2.3.3.2-10) only if tension-field action is present. Otherwise only the moment of inertia requirement must be considered.

The difference in the resistance factor ϕ between compact and slender webs (i.e., $\phi = 0.86$ versus $\phi = 0.78$) reflects the larger scatter of test results for the shear strength of plate-girders. These differences appear implicitly in the interaction equation (Eq. 2.3.3.2-13) when both high shear force and high bending moment are present.

C2.3.3.3 Factored Maximum Moment Capacity

Studies of the test-performance of the maximum capacity of beams and plate girders (Refs. 1, 21 and 22) have indicated that a single resistance factor $\phi = 0.86$ is sufficient for all problems involving flexural failure.

Flexural members are subdivided into two categories: beams and plate girders, depending on the web slenderness ratio, $d/t \cong h/t = 970/\sqrt{F_{yw}}$ being the slenderness ratio separating the two types of members. The capacity of flexural members depends on the slenderness of the unbraced length, the compression flange and the web. If these slenderness ratios λ_b are less than the limiting values λ_{bp}, the member can be counted on to resist the plastic moment, M_p. When the slenderness ratios are larger than λ_{br}, elastic buckling takes place. Throughout these criteria a linear transition range is assumed between the points M_p, λ_{bp} and M_r, λ_{br} (Fig. C2.3.3.3a). In this region instability sets in after some portion of the member cross section has yielded.

Throughout this section three limit states criteria must be investigated: lateral-torsional buckling of the unbraced member, local buckling of the compression flange, and web failure. The smallest resulting moment is the governing maximum moment capacity, M_u.

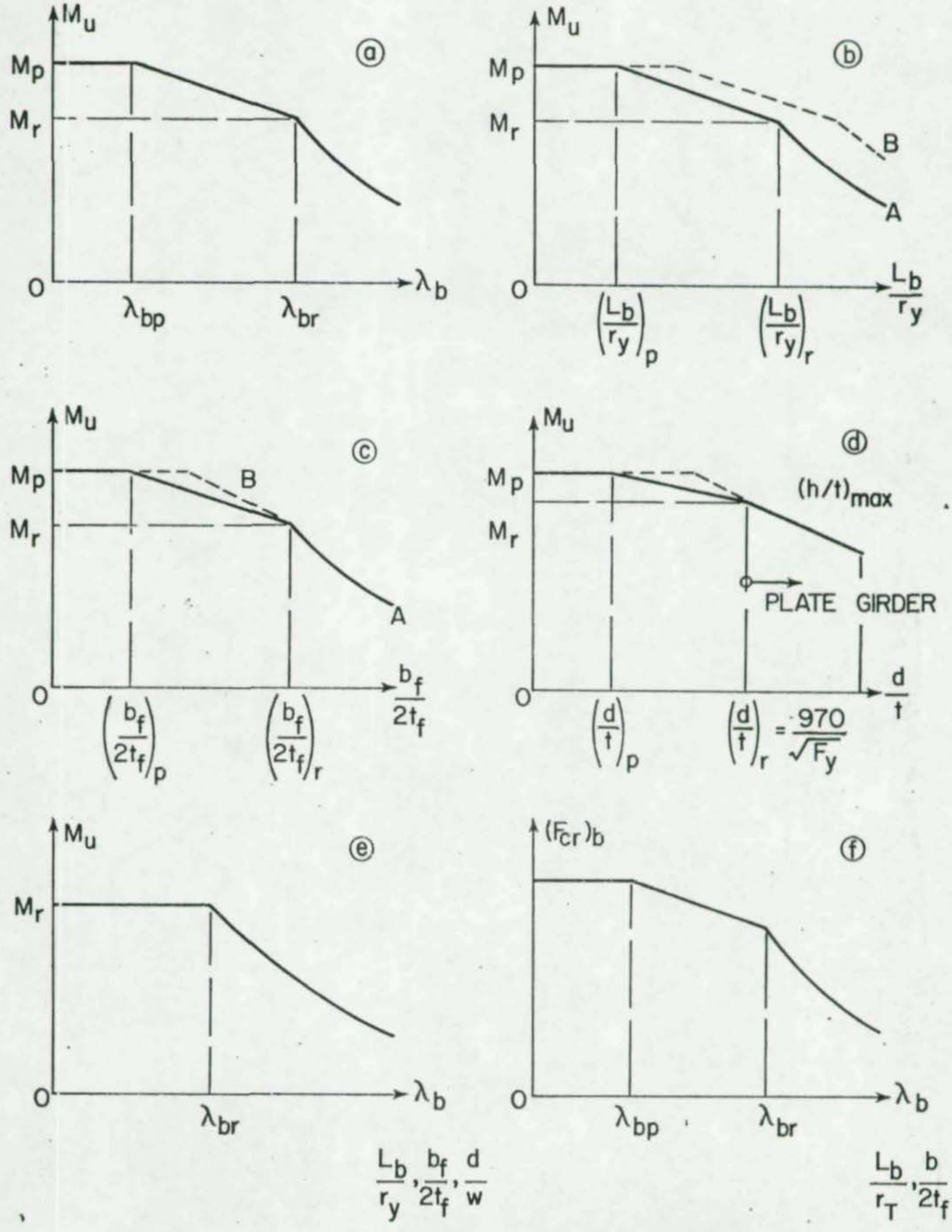
The limit state of lateral-torsional buckling is illustrated in Fig. C2.3.3.3b, where $\lambda_b = L_b/r_y$, the weak-axis slenderness ratio of the unbraced length, L_b. The limiting values at the end of the plastic range, $(L_b/r_y)_p$, and at the beginning of the elastic buckling range, $(L_b/r_y)_r$, depend on the moment gradient, uniform moment resulting in the lowest moment capacity (curve A in Fig. C2.3.3.3b). The formulas relating to the cases where this limit state governs are listed in Table 2.3.3.3 for a variety of common shapes, e.g. singly and doubly symmetric wide-flange shapes, channels, Tee and double angle shapes (for these latter cross sections the maximum capacity is the moment at yield, and the M_u versus λ_b curve is illustrated in Fig. C2.3.3.3e), solid rectangular shapes, box-beams and hybrid beams. For other singly or doubly symmetric shapes an analysis for lateral stability must be performed according to the available literature (Ref. 17). The formulas as given do not include the effect of restraint from adjacent elements. A more elaborate analysis is recommended in case that it is necessary to include the effect of restraint (see Ref. 17).

The flange local buckling limit state results in a similar type of curve (Fig. C2.3.3.3c), however, two limit states for λ_{bp} are used for wide-flange shapes. The larger limit, $(b_f/2\ t_f)_p = 65/\sqrt{F_y}$ applies when the beam is statically determinate or if the forces are calculated by elastic analysis for indeterminate beams and the partial redistribution as outlined in Sec. 1.5.1.4.1 of the AISC Specification is used (Curve B in

Fig. 2.3.3.3c). This approach demands a smaller rotation capacity than the case where plastic analysis is used to determine the forces, and so in this latter case a smaller value of $(b_f/2\ t_f)_p = 52.2/\sqrt{F_y}$ (Curve A in Fig. 2.3.3.3c) is used. These limiting values are consistent with the AISC Specification, and the more liberal provisions have been adopted in Supplement No. 3 of the AISC Specification on the basis of tests presented in Ref. 30. While the tests indicate that the limit $52.2/\sqrt{F_y}$ is conservative, no evidence has yet been presented to indicate that $65/\sqrt{F_y}$ is completely adequate for plastic design. Probably a value intermediate between the two extremes, say $60/\sqrt{F_y}$, would be perfectly satisfactory in plastic design and it is anticipated that upon the completion of research currently in progress such a liberalization can be adopted in these criteria.

A similar dual situation exists for the limiting $(d/t)_p$ for the limit state of web failure (Fig. 2.3.3.3d). The formulas given in Table 2.3.3.3 do not, however, reflect exactly the AISC Specification. For the plastic design these new rules (Eqs. A-2.3.3.3-16) are more liberal, and they have been recommended in Ref. 31 on the basis of tests and theoretical derivations. The new rules for compact elastic design (Eqs. A-2.3.3.3-17) give the AISC Specification Supplement No. 3 value of $(d/t)_p = 640\sqrt{F_y}$ when the axial force is zero (this liberalized value was adopted upon the completion of the tests on continuous beams in Refs. 32 and 33), but they include a relaxation of the present requirement when axial force is present.

The determination of M_u for plate-girders (Sec. 2.3.3.3.2) is based on the same limit states, but here web slenderness is accounted for by considering the flange-to-web area ratio (Eq. 2.3.3.3-8 and Fig. C2.3.3.3d), and the limit states of lateral-torsional and flange local buckling are accomplished by a reduction of the critical stress (Fig. C2.3.3.3f).



M_u
(a)
M_p
M_r
O
λ_b
λ_{bp}
λ_{br}
M_u
(b)
M_p
M_r
B
A
O
$\frac{L_b}{r_y}$
$\left(\frac{L_b}{r_y}\right)_p$
$\left(\frac{L_b}{r_y}\right)_r$
M_u
(c)
M_p
B
M_r
A
O
$\frac{b_f}{2t_f}$
$\left(\frac{b_f}{2t_f}\right)_p$
$\left(\frac{b_f}{2t_f}\right)_r$
M_u
(d)
M_p
$(h/t)_{max}$
M_r
PLATE GIRDER
O
$\frac{d}{t}$
$\left(\frac{d}{t}\right)_p$
$\left(\frac{d}{t}\right)_r = \frac{970}{\sqrt{F_y}}$
M_u
(e)
M_r
O
λ_b
λ_{br}
$\frac{L_b}{r_y}, \frac{b_f}{2t_f}, \frac{d}{w}$
$(F_{cr})_b$
(f)
O
λ_b
λ_{bp}
λ_{br}
$\frac{L_b}{r_T}, \frac{b}{2t_f}$

Fig. C.2.3.3.3

C2.3.3.3.3 Maximum Moment Capacity for Composite Beams

The resistance factor $\phi = 0.84$ for composite beams with compact webs is based on the available tests (Ref. 24), and $\phi = 0.89$ for tension-flange yield and $\phi = 0.81$ for concrete crushing is based on the available material data. The provisions for partially composite beams with formed steel deck derive from the results presented in Refs. 34 and 35. Similar provisions are under discussion for inclusion in the AISC Specification.

The provisions of Sec. 2.3.3.3.3 concern the maximum factored moment capacity of composite beams for the limit state of strength. In addition to these criteria it may be necessary to insure that 1) the composite beam will not yield under the combination of permanent loads (dead and equipment loads) and instantaneous live and short-term environmental loads and 2) the live load deflection is kept at or below an allowable value. The load and resistance factors for such serviceability criteria are presented in Section C1.2.2 of the Commentary. Since the limit states of yielding and deflection are elastic phenomena, the stresses and deflections under the appropriate factored loads (Sec. C1.2.2) are determined by elastic theory for the transformed section, including provisions for creep where appropriate.

Cognizance should be taken in the determination of the stresses whether the construction is shored or unshored. When composite action is only partial, the effective section moduli given by Eq. 2.3.3.3-29 in the Criteria should be used in calculating stresses.

C2.3.4 Members Under Combined Flexure and Axial Force

The provisions in this section were derived in Ref. 23, and they are essentially the same as the corresponding provisions in Part 2 of the AISC Specification.

Under some conditions of geometry the interaction relationships given by Eqs. 2.3.4.1-3 and 2.3.4.2-1 are quite conservative when flexure is about both principal axes (biaxial bending). The following interaction equations have been recommended for biaxially-loaded H and wide-flange shapes in Ref. 17 and 36:

$$\left(\frac{M_{Dx}}{\phi_b\ M_{pcx}}\right)^{\zeta} + \left(\frac{M_{Dy}}{\phi_b\ M_{pcy}}\right)^{\zeta} \leq 1.0 \qquad \text{(C2.3.4-1)}$$

$$\left(\frac{C_{mx}\ M_{Dx}}{\phi_b\ M_{ucx}}\right)^{\eta} + \left(\frac{C_{my}\ M_{Dy}}{\phi_b\ M_{ucy}}\right)^{\eta} \leq 1.0 \qquad \text{(C2.3.4-2)}$$

In these equations M_{Dx}, M_{Dy}, ϕ_b, C_{mx}, and C_{my}, are defined as in Sec. 2.3.4, and

$$\zeta = 1.6 - \frac{\dfrac{P_D}{\phi_b\ P_y}}{2\ \ell n\left(\dfrac{P_D}{\phi_b\ P_y}\right)} \qquad \text{(C2.3.4-3)}$$

$$\eta = 0.4 + \frac{P_D}{\phi_b\ P_y} + \frac{b_f}{d} \geq 1.0 \qquad \text{(C2.3.4-4a)}$$

when $b_f/d \geq 0.3$

$$\eta = 1.0 \quad \text{when} \quad b_f/d < 0.3 \qquad \text{(C2.3.4-4b)}$$

where b_f is the flange width and d is the member depth.

$$M_{pcx} = 1.18\ M_{px}\left(1 - \frac{P_D}{\phi_b\ P_y}\right) \leq 1.0 \qquad \text{(C2.3.4-5)}$$

$$M_{pcy} = 1.19\ M_{py}\left[1 - \left(\frac{P_D}{\phi_b\ P_y}\right)^2\right] \leq 1.0 \qquad \text{(C2.3.4-6)}$$

$$M_{ucx} = M_{ux} \left(1 - \frac{P_D}{\phi_b P_u}\right)\left(1 - \frac{P_D}{\phi_b P_{Ex}}\right) \qquad (C2.3.4\text{-}7)$$

$$M_{ucy} = M_{uy} \left(1 - \frac{P_D}{\phi_b P_u}\right)\left(1 - \frac{P_D}{\phi_b P_{Ey}}\right) \qquad (C2.3.4\text{-}8)$$

The terms P_D, P_y, ϕ_b, P_u, P_{Ex}, P_{Ey}, M_{px}, M_{py}, M_{ux} and M_{uy} are defined in Sec. 2.3.4.

These equations represent a considerable liberalization over the provisions given in Sec. 2.3.4, and it is, therefore, necessary to check also yielding under service loads, using the appropriate load and resistance factors for the serviceability limit state (Sec. C.1.2.2) in Eq. 2.3.4.1-3 with $M_{ux} = S_x F_y$ and $M_{uy} = S_y F_y$.

While concrete-filled tubular columns are not treated in the AISC Specification, it is possible to use LRFD for such members. An interaction equation for concrete-filled tubular members has been recommended by Furlong in Ref. 37 and 38, and in Ref. 39 it was shown that the resistance factor $\phi = 0.75$ is an appropriate value to use multiplying both the axial and the flexural capacity of the member by ϕ in this interaction equation.

C2.3.5 Members Under Combined Stress

This section is essentially a catch-all provision, giving general rules for treating cases not specifically covered in the previous sections. It concerns especially unsymmetric members, and members under combined normal stress and torsion.

C2.4 The Design of Connections

This section deals with the design of connections, with special emphasis on giving the ϕ-factors and the nominal maximum capacities of connectors: welds, rivets, bolts and high-strength bolts. The basis for the determination of the ϕ-factors is given in Ref. 4, where the experimental data available on fasteners are analyzed. The ϕ-factors for connectors are based on $\beta = 4.5$, providing the traditionally higher reliability for connections as compared to members. In order to avoid having to adjust the load factors, the ϕ-factors obtained from Eq. C2.3-1 are multiplied by 0.88, as shown in Ref. 4.

The shear capacity R_n of long joints is to be reduced to 80% of the value given by Sec. 2.4.3.2.2 if the joint length exceeds 50 inches (Ref. 16).

TABLE C2.3.2.1-1 FACTORED COLUMN STRESS RATIOS

λ	$\frac{\phi_c F_{cr}}{F_y}$	λ	$\frac{\phi_c F_{cr}}{F_y}$	λ	$\frac{\phi_c F_{cr}}{F_y}$
0	0.860	0.24	0.828	0.48	0.735
0.01	0.860	0.25	0.824	0.49	0.731
0.02	0.860	0.26	0.821	0.50	0.727
0.03	0.860	0.27	0.817	0.51	0.722
0.04	0.860	0.28	0.814	0.52	0.718
0.05	0.859	0.29	0.810	0.53	0.714
0.06	0.859	0.30	0.806	0.54	0.709
0.07	0.859	0.31	0.803	0.55	0.705
0.08	0.859	0.32	0.799	0.56	0.700
0.09	0.858	0.33	0.795	0.57	0.696
0.10	0.858	0.34	0.791	0.58	0.692
0.11	0.857	0.35	0.788	0.59	0.687
0.12	0.857	0.36	0.784	0.60	0.683
0.13	0.856	0.37	0.780	0.61	0.678
0.14	0.856	0.38	0.776	0.62	0.673
0.15	0.855	0.39	0.772	0.63	0.669
0.16	0.854	0.40	0.768	0.64	0.664
0.17	0.851	0.41	0.764	0.65	0.660
0.18	0.848	0.42	0.760	0.66	0.655
0.19	0.845	0.43	0.756	0.67	0.650
0.20	0.842	0.44	0.752	0.68	0.646
0.21	0.838	0.45	0.748	0.69	0.641
0.22	0.835	0.46	0.743	0.70	0.636
0.23	0.831	0.47	0.739	0.71	0.631

TABLE C2.3.2.1-1 CONTINUED

λ	$\frac{\phi_c F_{cr}}{F_y}$	λ	$\frac{\phi_c F_{cr}}{F_y}$	λ	$\frac{\phi_c F_{cr}}{F_y}$
0.72	0.627	0.96	0.508	1.20	0.416
0.73	0.622	0.97	0.503	1.21	0.412
0.74	0.617	0.98	0.498	1.22	0.408
0.75	0.612	0.99	0.493	1.23	0.404
0.76	0.607	1.00	0.488	1.24	0.400
0.77	0.603	1.01	0.484	1.25	0.396
0.78	0.598	1.02	0.481	1.26	0.392
0.79	0.593	1.03	0.478	1.27	0.388
0.80	0.588	1.04	0.474	1.28	0.384
0.81	0.583	1.05	0.471	1.29	0.380
0.82	0.578	1.06	0.467	1.30	0.375
0.83	0.573	1.07	0.464	1.31	0.371
0.84	0.568	1.08	0.460	1.32	0.367
0.85	0.563	1.09	0.457	1.33	0.363
0.86	0.558	1.10	0.453	1.34	0.358
0.87	0.553	1.11	0.450	1.35	0.354
0.88	0.548	1.12	0.446	1.36	0.349
0.89	0.543	1.13	0.443	1.37	0.345
0.90	0.538	1.14	0.439	1.38	0.341
0.91	0.533	1.15	0.435	1.39	0.336
0.92	0.528	1.16	0.431	1.40	0.332
0.93	0.523	1.17	0.428	1.41	0.327
0.94	0.518	1.18	0.424	1.42	0.322
0.95	0.513	1.19	0.420	1.43	0.318

TABLE C2.3.2.1-1 CONTINUED

λ	$\frac{\phi_c F_{cr}}{F_y}$	λ	$\frac{\phi_c F_{cr}}{F_y}$	λ	$\frac{\phi_c F_{cr}}{F_y}$
1.44	0.313	1.68	0.230	1.91	0.176
1.45	0.309	1.69	0.228	1.92	0.175
1.46	0.305	1.70	0.225	1.93	0.173
1.47	0.301	1.71	0.222	1.94	0.171
1.48	0.297	1.72	0.220	1.95	0.169
1.49	0.293	1.73	0.217	1.96	0.167
1.50	0.289	1.74	0.215	1.97	0.166
1.51	0.285	1.75	0.212	1.98	0.164
1.52	0.281	1.76	0.210	1.99	0.163
1.53	0.278	1.77	0.207	2.00	0.161
1.54	0.274	1.78	0.205	2.01	0.159
1.55	0.271	1.79	0.203	2.02	0.158
1.56	0.267	1.80	0.201	2.03	0.156
1.57	0.264	1.81	0.198	2.04	0.155
1.58	0.260	1.82	0.196	2.05	0.153
1.59	0.257	1.83	0.194	2.06	0.152
1.60	0.254	1.84	0.192	2.07	0.150
1.61	0.251	1.85	0.190	2.08	0.149
1.62	0.248	1.86	0.188	2.09	0.147
1.63	0.245	1.87	0.186	2.10	0.146
1.64	0.242	1.88	0.184		
1.65	0.239	1.89	0.182		
1.66	0.236	1.90	0.180		
1.67	0.233	1.91	0.178		

TABLE C-2.3.3.3-1: Coefficients X_1 and X_2 for Eq. A-2.3.3.3-6

SHAPE	X_1	X_2
W36x300	3,834	10,820
280	3,613	12,250
260	3,350	14,270
245	3,163	16,040
230	2,972	18,020
W36x194	3,036	20,390
182	2,854	23,050
170	2,678	26,085
160	2,520	29,790
150	2,370	34,160
135	2,149	43,830
W33x240	3,534	12,740
220	3,257	15,060
200	2,970	18,148
W33x152	2,736	24,370
141	2,535	28,760
130	2,339	34,630
118	2,143	43,260
W30x210	3,647	11,760
190	3,318	14,170
172	3,009	17,180
W30x132	2,893	22,020
124	2,719	25,014
116	2,548	28,960
108	2,380	34,280
99	2,196	41,410
W27x177	3,707	11,360
160	3,358	13,700
145	3,060	16,400
W27x114	2,963	20,310
102	2,662	25,130
94	2,467	29,930
84	2,182	38,670
W24x160	3,806	10,180
145	3,459	12,360
130	3,103	15,440
W24x120	3,224	14,900
110	2,965	17,540
100	2,710	20,940

TABLE C-2.3.3.3-1 (CONTINUED)

SHAPE	X_1	X_2
W24x94	3,079	18,610
84	2,751	23,140
76	2,498	28,690
68	2,253	36,860
W24x61	2,378	37,180
55	2,168	42,770
W21x142	4,283	8,050
127	3,849	9,833
112	3,405	12,480
W21x96	3,875	11,370
82	3,327	15,270
W21x73	3,017	19,174
68	2,826	21,870
62	2,576	26,610
55	2,303	34,430
W21x49	2,400	34,850
44	2,189	44,300
W18x114	4,494	7,192
105	4,160	8,394
96	3,799	9,904
W18x85	4,222	8,828
77	3,869	10,500
70	3,519	12,550
64	3,233	14,840
W18x60	3,244	16,110
55	2,979	19,130
50	2,720	22,900
45	2,451	28,680
W18x40	2,552	28,260
35	2,240	38,580
W16x96	4,488	6,998
88	4,140	8,206
W16x78	4,645	7,310
71	4,256	8,664
64	3,837	10,620
58	3,484	12,810

TABLE C-2.3.3.3-1 (CONTINUED)

SHAPE	X_1	X_2
W16x50	3,296	15,230
45	2,996	18,580
40	2,672	22,990
36	2,405	29,650
W16x31	2,465	29,900
26	2,088	44,120
W14x730	24,660	292.0
665	23,030	329.8
605	21,380	369.8
550	19,960	418.9
500	18,500	476.8
455	17,180	543.9
W14x426	16,240	595.1
398	15,350	658.5
370	14,460	734.8
342	13,560	828.3
314	12,550	938.9
287	11,610	1,085
264	10,770	1,236
246	10,110	1,386
W14x237	9,775	1,474
228	9,433	1,567
219	9,078	1,679
211	8,795	1,788
202	8,436	1,919
193	8,088	2,071
184	7,732	2,243
176	7,420	2,429
167	7,084	2,656
158	6,728	2,915
150	6,392	3,201
142	6,067	3,534
W14x320	13,020	953.4
W14x136	6,081	3,557
127	5,694	3,996
119	5,367	4,484
111	5,023	5,104
103	4,655	5,824
95	4,305	6,745
87	3,976	7,901

TABLE C-2.3.3.3-1 (CONTINUED)

SHAPE	X_1	X_2
W14x84	4,504	6,447
78	4,194	7,422
W14x74	4,630	6,506
68	4,258	7,580
61	3,839	9,220
W14x53	3,987	9,138
48	3,628	10,960
43	3,280	13,300
W14x38	3,086	16,700
34	2,770	20,740
30	2,458	27,340
W14x26	2,658	25,130
22	2,273	35,830
W12x190	11,240	1,171
161	9,698	1,525
133	8,103	2,865
120	7,400	2,531
106	6,570	3,108
99	6,165	3,503
92	5,771	3,999
85	5,338	4,572
79	4,993	5,236
72	4,577	6,196
65	4,155	7,410
W12x58	4,337	6,960
53	3,982	
W12x50	4,432	8,293
45	4,054	7,020
40	3,658	8,452
W12x36	3,645	10,320
31	3,166	11,200
27	2,761	14,750
W12x22	3,071	19,400
19	2,670	20,060
16.5	2,372	27,610
14	2,036	38,350

TABLE C-2.3.3.3-1 (CONTINUED)

SHAPE	X_1	X_2
W10x112	9,966	1,443
100	8,994	1,745
89	8,074	2,118
77	7,071	2,708
72	6,636	3,061
66	6,109	3,527
60	5,593	4,170
54	5,062	5,000
49	4,615	5,962
W10x45	5,123	5,092
39	4,476	6,724
33	3,833	9,314
W10x29	4,081	8,931
25	3,534	11,780
21	3,000	17,290
W10x19	3,430	15,050
17	3,069	19,780
15	2,774	25,730
11.5	2,194	41,426
W8x67	9,335	1,627
58	8,191	2,082
48	6,879	2,817
40	5,789	3,995
35	5,112	5,000
31	4,553	6,267
W8x28	4,872	5,711
24	4,229	7,484
W8x20	3,995	9,283
17	3,441	12,920
W8x15	3,718	12,720
13	3,296	16,930
10	2,555	26,750
W6x25	[illegible],524	3,560
20	5,043	5,240
15.5	4,022	8,647
W6x16	5,672	4,657
12	4,401	8,251
8.5	3,203	15,480

TABLE C-2.3.3.3-1 (CONTINUED)

SHAPE	X_1	X_2
W5x18.5	7,198	2,606
16	6,295	3,378
W4x13	7,955	2,314
M14x17.2	1,997	55,450
M12x11.8	2,020	58,180
M10x29.1	4,764	8,419
22.9	3,639	11,330
M10x9	2,154	49,370
M8x34.3	5,336	4,928
32.6	5,054	5,188
M8x22.5	5,194	6,613
18.5	4,193	8,390
M8x6.5	2,337	39,510
M7x5.5	2,562	32,190
M6x22.5	6,587	3,482
20	5,732	4,043
M6x4.4	2,675	28,090
M5x18.9	8,111	2,178
M4x13.8	9,767	1,586
13	9,179	1,692
S24x120	4,805	9,124
105.9	4,308	10,050
S24x100	4,255	13,210
90	3,828	15,020
79.9	3,466	16,440
S20x95	5,394	7,697
85	4,788	8,801
S20x75	4,453	11,360
65.4	3,927	12,980
S18x70	5,074	9,531
54.7	3,906	13,040

TABLE C-2.3.3.3-1 (CONTINUED)

SHAPE	X_1	X_2
S15x50	4,870	9,187
42.9	4,178	10,840
S12x50	7,165	4,297
40.8	5,722	5,620
S12x35	4,936	8,022
31.8	4,508	8,728
S10x35	7,009	4,627
25.4	4,858	7,157
S8x23	6,739	4,518
x18.4	5,293	5,921
S7x20	7,608	3,649
15.3	5,607	5,225
S6x17.25	8,876	2,765
12.5	6,023	4,455
S5x14.75	10,990	1,881
10	6,652	3,611
S4x9.5	9,649	2,048
7.7	7,553	2,727
S3x7.5	13,000	1,168
5.7	9,121	1,826
HP14x117	5,427	4,954
102	4,788	6,346
89	4,202	8,136
73	3,458	11,710
HP12x74	4,870	6,235
53	3,548	11,386
HP10x57	5,517	4,901
42	4,131	8,479
HP8x36	5,422	5,035

TABLE C-2.3.3.3-1 (CONTINUED)

SHAPE	X_1	X_2
C15x50	6,570	7,293
40	5,046	10,666
33.9	4,269	13,010
C12x30	5,783	8,640
25	4,677	11,510
20.7	3,952	13,910
C10x30	8,958	4,238
25	6,995	6,244
20	5,277	9,316
15.3	4,076	12,474
C9x20	6,650	6,477
15	4,803	9,971
13.4	4,352	11,020
C8x18.75	7,965	4,690
13.75	5,441	8,018
11.5	4,621	9,576
C7x14.75	7,827	4,557
12.25	6,210	6,307
9.8	4,981	8,095
C6x13	9,364	3,259
10.5	7,108	4,838
8.2	5,476	6,563
C5x 9	8,518	3,399
6.7	6,202	5,028
C4x 7.25	10,320	2,284
5.4	7,387	3,495
C3x 6	14,683	1,149
5	11,460	1,629
4.1	9,289	2,136