Errata List, September 4, 2001

AISC Load and Resistance Factor Design Specification for Structural Steel Buildings, December 27, 1999

The following editorial corrections have been made in the First Revision, September 4, 2001. To facilitate the incorporation of these corrections, this booklet has been constructed using copies of the revised pages, with corrections noted. The user may find it convenient in some cases to hand-write a correction; in others, a cut-and-paste approach may be more efficient.

Load and Resistance Factor Design Specification for Structural Steel Buildings

December 27, 1999

Supersedes the *Load and Resistance Factor Design Specification for Structural Steel Buildings* dated December 1, 1993 and all previous versions.

Prepared by the American Institute of Steel Construction, Inc. Under the Direction of the AISC Committee on Specifications and approved by the AISC Board of Directors.

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Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance, ASTM A847 Errata 9/4/01

and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges, ASTM A709/A709M

Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi (485 MPa) Minimum Yield Strength to 4 in. (100 mm) Thick, ASTM A852/A852M

High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST), ASTM A913/A913M

Steel for Structural Shapes for Use in Building Framing, ASTM A992/A992M

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M, Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling or A568/A568M, Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for, as applicable, shall constitute sufficient evidence of conformity with one of the above ASTM standards. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

1b. Unidentified Steel

Unidentified steel, if surface conditions are acceptable according to criteria contained in ASTM A6/A6M, is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

1c. Heavy Shapes

For ASTM A6/A6M Group 4 and 5 rolled shapes to be used as members subject to primary tensile stresses due to tension or flexure, toughness need not be specified if splices are made by bolting. If such members are spliced using complete-joint-penetration groove welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch (CVN) impact testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall meet a minimum average value of 20 ft-lbs. (27 J) absorbed energy at +70°F (+21°C) and shall be conducted in accordance with ASTM A673/A673M, with the following exceptions:

(1) The center longitudinal axis of the specimens shall be located as near as practical to midway between the inner flange surface and the center of the flange thickness at the intersection with the web mid-thickness.

(2) Tests shall be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests.

For plates exceeding two-in. (50 mm) thick used for built-up cross-sections with bolted splices and subject to primary tensile stresses due to tension or flexure, material toughness need not be specified. If such cross-sections are spliced using complete-joint-penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall be conducted by the producer in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 20 ft-lbs. (27 J) absorbed energy at +70°F (+21°C).

The above supplementary requirements also apply when complete-joint-penetra-

TABLE B5.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios: λ_p (compact)	Limiting Width-Thickness Ratios: λ_r (noncompact)
Stiffened Elements	Flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t		
	for uniform compression		$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$
	for plastic analysis		$0.939\sqrt{E/F_y}$	-
	Unsupported width of cover plates perforated with a succession of access holes [b]	b/t	NA	$1.86\sqrt{E/F_y}$
	Webs in flexural compression [a]	h/t_w	$3.76\sqrt{E/F_y}$ [c], [g]	$5.70\sqrt{E/F_y}$ [h]
	Webs in combined flexural and axial compression	h/t_w	for $P_u/\phi_b P_y \le 0.125$ [c],[g] $3.76\sqrt{\dfrac{E}{F_y}}\left(1-\dfrac{2.75P_u}{\phi_b P_y}\right)$ for $P_u/\phi_b P_y > 0.125$ [c][g] $1.12\sqrt{\dfrac{E}{F_y}}\left(2.33-\dfrac{P_u}{\phi_b P_y}\right) \ge 1.49\sqrt{\dfrac{E}{F_y}}$	[h] $5.70\sqrt{\dfrac{E}{F_y}}\left(1-0.74\dfrac{P_u}{\boxed{\phi_b P_y}}\right)$ Errata 9/4/01
	All other uniformly compressed stiffened elements, i.e., supported along two edges	b/t h/t_w	NA	$1.49\sqrt{E/F_y}$
	Circular hollow sections In axial compression In flexure	D/t	[d] NA $0.07E/F_y$	 $0.11E/F_y$ $0.31E/F_y$

[a] For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.

[b] Assumes net area of plate at widest hole.

[c] ~~Assumes an inelastic rotation capacity of 3 radians. For structures in zones of high seismicity, a greater rotation capacity may be required.~~

[d] For plastic design use $0.045E/F_y$.

[e] F_L = smaller of $(F_{yf} - F_r)$ or F_{yw}, ksi (MPa)
F_r = compressive residual stress in flange
= 10 ksi (69 MPa) for rolled shapes
= 16.5 ksi (114 MPa) for welded shapes

[f] $k_c = \dfrac{4}{\sqrt{h/t_w}}$ and $0.35 \le k_c \le 0.763$

[g] For members with unequal flanges, use h_p instead of h when comparing to λ_p.

[h] For members with unequal flanges, see Appendix B5.1.

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Assumes an inelastic ductility ratio (ratio of strain at fracture to strain at yield) of 3. When the seismic response modification factor ***R*** is taken greater than 3, a greater rotation capacity may be required.

2. Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in symmetric shapes shall be limited by Equations H1-1a and H1-1b

where

P_u = required compressive strength, kips (N)
P_n = nominal compressive strength determined in accordance with Section E2, kips (N)
ϕ = ϕ_c = resistance factor for compression = 0.85 (see Section E2)
ϕ_b = resistance factor for flexure = 0.90

H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

The design strength, $\phi\boxed{F_n,}$ of the member shall equal or exceed the required strength expressed in terms of the normal stress f_{un} or the shear stress f_{uv}, determined by elastic analysis for the factored loads:

(a) For the limit state of yielding under normal stress:

$$f_{un} \leq \phi\boxed{F_n} \tag{H2-1}$$

$$\phi = 0.90$$

$$\boxed{F_n = F_y}$$

(b) For the limit state of yielding under shear stress:

$$f_{uv} \leq 0.6\phi\boxed{F_n} \tag{H2-2}$$

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$$\phi = 0.90$$

$$\boxed{F_n = F_y}$$

(c) For the limit state of buckling:

$$f_{un} \text{ or } f_{uv} \leq \phi_c\boxed{F_n,} \text{ as applicable} \tag{H2-3}$$

$$\phi_c = 0.85$$

$$\boxed{F_n = F_{cr}}$$

Some constrained local yielding is permitted adjacent to areas which remain elastic.

H3. ALTERNATIVE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

See Appendix H3.

TABLE J2.1
Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Welding Process	Welding Position	Included Angle at Root of Groove	Effective Throat Thickness
Shielded metal arc Submerged arc	All	J or U joint	Depth of chamfer
Gas metal arc		Bevel or V joint ≥ 60°	
Flux-cored arc		Bevel or V joint < 60° but ≥ 45°	Depth of chamfer Minus ⅛-in. (3 mm)

TABLE J2.2
Effective Throat Thickness of Flare Groove Welds

Type of Weld	Radius (*R*) of Bar or Bend	Effective Throat Thickness
Flare bevel groove	All	$\frac{5}{16}R$
Flare V-groove	All	$\boxed{\frac{1}{2}R\ [a]}$

[a] Use $\frac{3}{8}R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq 1$ in. (25 mm)

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TABLE J2.3
Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material Thickness of Thicker Part Joined, in. (mm)	Minimum Effective Throat Thickness [a], in. (mm)
To ¼ (6) inclusive	⅛ (3)
Over ¼ (6) to ½ (13)	3/16 (5)
Over ½ (13) to ¾ (19)	¼ (6)
Over ¾ (19) to 1½ (38)	5/16 (8)
Over 1½ (38) to 2¼ (57)	⅜ (10)
Over 2¼ (57) to 6 (150)	½ (13)
Over 6 (150)	⅝ (16)

[a] See Table J2.1

exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

2. Fillet Welds

2a. Effective Area

The effective area of fillet welds shall be as defined in AWS D1.1 Section 2.4.3 and 2.11. The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the diagrammatic weld, except that for fillet welds made by the submerged arc process, the effective throat thickness shall be

3a. Unstiffened Compression Elements

The design strength of unstiffened compression elements whose width-thickness ratio exceeds the applicable limit λ_r as stipulated in Section B5.1 shall be subject to a reduction factor Q_s. The value of Q_s shall be determined by Equations A-B5-3 through A-B5-10, as applicable. When such elements comprise the compression flange of a flexural member, the design flexural strength, in ksi, shall be computed using $\phi_b F_y Q_s$, where $\phi_b = 0.90$. The design strength of axially loaded compression members shall be modified by the appropriate reduction factor Q, as provided in Appendix B5.3d.

(a) For single angles:

when $0.45\sqrt{E/F_y} < b/t < 0.91\sqrt{E/F_y}$:

$$Q_s = 1.340 - 0.76(b/t)\sqrt{F_y/E} \tag{A-B5-3}$$

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when $b/t \boxed{\geq} 0.91\sqrt{E/F_y}$:

$$Q_s = 0.53E/\left[F_y\left(b/t\right)^2\right] \tag{A-B5-4}$$

(b) For flanges, angles, and plates projecting from rolled beams or columns or other compression members:

when $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.415 - 0.74(b/t)\sqrt{F_y/E} \tag{A-B5-5}$$

when $b/t \geq 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69E/\left[F_y\left(b/t\right)^2\right] \tag{A-B5-6}$$

(c) For flanges, angles and plates projecting from built-up columns or other compression members:

when $0.64\sqrt{E/(F_y/k_c)} < b/t < 1.17\sqrt{E/(F_y/k_c)}$:

$$Q_s = 1.415 - 0.65(b/t)\sqrt{(F_y/k_c E)} \tag{A-B5-7}$$

when $b/t \geq 1.17\sqrt{E/(F_y/k_c)}$:

$$Q_s = 0.90Ek_c/\left[F_y\left(b/t\right)^2\right] \tag{A-B5-8}$$

The coefficient, k_c, shall be computed as follows:

(a) For I-shaped sections:

$$k_c = \frac{4}{\sqrt{h/t_w}},\ 0.35 \le k_c \le 0.763$$

where

h = depth of web, in. (mm)
t_w = thickness of web, in. (mm)

(b) For other sections:

$k_c = 0.763$

(d) For stems of tees:

when $0.75\sqrt{E/F_y} < \boxed{d/t} < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.908 - 1.22\boxed{(d/t)}\sqrt{F_y/E} \qquad \text{(A-B5-9)}$$

when $\boxed{d/t} \ge 1.03\sqrt{E/F_y}$: Errata 9/4/01

$$Q_s = 0.69 \quad / \left[F_y \boxed{(d/t)}^2\right] \qquad \text{(A-B5-10)}$$

where

$\boxed{d}$ = width of unstiffened compression element as defined in Section B5.1, in. (mm)
t = thickness of unstiffened element, in. (mm)

3b. Stiffened Compression Elements

When the width-thickness ratio of uniformly compressed stiffened elements (except perforated cover plates) exceeds the limit λ_r stipulated in Section B5.1, a reduced effective width b_e shall be used in computing the design properties of the section containing the element.

(a) For flanges of square and rectangular sections of uniform thickness:

when $\frac{b}{t} \ge 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \text{(A-B5-11)}$$

otherwise $b_e = b$.

(b) For other uniformly compressed elements:

when $\frac{b}{t} \ge 1.49\sqrt{\frac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \text{(A-B5-12)}$$

otherwise $b_e = b$.

where

TABLE A-F1.1
Nominal Strength Parameters

Shape	Plastic Moment M_p	Limit State of Buckling	Limiting Buckling Moment M_r
Channels and doubly and singly symmetric I-shaped beams (including hybrid beams) bent about major axis [a]	$F_y Z_x$ [b]	LTB doubly symmetric members and channels	$F_L S_x$
		LTB singly symmetric members	$F_L S_{xc} \le F_{yt} S_{xt}$
		FLB	$F_L S_x$
		WLB	$R_e F_{yf} S_x$
Channels and doubly and singly symmetric I-shaped members bent about minor axis [a]	$F_y Z_y$	FLB	$F_y S_y$

NOTE: LTB applies only for strong axis bending.
[a] Excluding double angles and tees.
[b] Computed from fully plastic stress distribution for hybrid sections.

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[c] $X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}}$ $\boxed{X_2 = 4\frac{C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2}$

[d] $\lambda_r = \frac{X_1}{F_L}\sqrt{1+\sqrt{1+X_2F_L^2}}$

[e] $F_{cr} = \frac{M_{cr}}{S_{xc}}$, where $M_{cr} = \frac{2EC_b}{L_b}\sqrt{I_yJ}\left[B_1+\sqrt{\left(1+B_2+B_1^2\right)}\right] \le M_p$

where

$$B_1 = 2.25\left[2(I_{yc}/I_y)-1\right](h/L_b)\sqrt{(I_y/J)}$$

$$B_2 = 25(1-I_{yc}/I_y)(I_{yc}/J)(h/L_b)^2$$

$$C_b = 1.0 \text{ if } I_{yc}/I_y < 0.1 \text{ or } I_{yc}/I_y > 0.9$$

TABLE A-J3.1
Nominal Tension Stress (F_t), ksi (MPa)
Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in the Shear Plane	Threads Excluded from the Shear Plane
A307 bolts (Metric)	$\sqrt{45^2 - 6.25f_v^2}$ $\left(\sqrt{310^2 - 6.25f_v^2}\right)$	
A325 bolts (A325M bolts)	$\sqrt{90^2 - 6.25f_v^2}$ $\left(\sqrt{621^2 - 6.25f_v^2}\right)$	$\sqrt{90^2 - 4.00f_v^2}$ $\left(\sqrt{621^2 - 4.00f_v^2}\right)$
A490 bolts (A490M bolts)	$\sqrt{113^2 - 6.31f_v^2}$ $\boxed{\left(\sqrt{779^2 - 6.31f_v^2}\right)}$	$\sqrt{113^2 - 4.04f_v^2}$ $\boxed{\left(\sqrt{779^2 - 4.04f_v^2}\right)}$
Threaded parts A449 bolts over 1½ in. (38 mm)	$\sqrt{(0.75F_u)^2 - 6.25f_v^2}$	$\sqrt{(0.75F_u)^2 - 4.00f_v^2}$
A502 Gr. 1 rivets (Metric)	$\sqrt{45^2 - 5.76f_v^2}$ $\left(\sqrt{310^2 - 5.76f_v^2}\right)$	
A502 Gr. 2 rivets (Metric)	$\sqrt{60^2 - 5.86f_v^2}$ $\left(\sqrt{414^2 - 5.86f_v^2}\right)$	

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TABLE A-J3.2
Slip-Critical Resistance to Shear at Service Loads, F_v, ksi (MPa), of High-Strength Bolts[a]

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Type of Bolt	Resistance to Shear at Service Loads, ksi (MPa)			
	Standard Size Holes	Oversized and Short-slotted Holes	Long-slotted Holes: Perpendicular to Line of Force	Long-slotted Holes: Parallel to Line of Force
A325 (A325M)	17 (117)	15 (103)	12 (83)	10 (69)
A490 (A490M)	21 (145)	18 (124)	15 (103)	13 (90)

[a] For each shear plane.

proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in. (mm)

$= r_i \Delta_u / r_{crit}$

Δ_u $= 1.087(\theta + 6)^{-0.65} w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in. (mm)

w = leg size of the fillet weld, in. (mm)

r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u / r_i ratio, in. (mm)

J3. BOLTS AND THREADED PARTS

7. Combined Tension and Shear in Bearing-Type Connections

As an alternative to the use of the equations in Table J3.5, the use of the equations in Table A-J3.1 is permitted.

8. High-Strength Bolts in Slip-Critical Connections

8b. Slip-Critical Connections Designed at Service Loads

The design resistance to shear per bolt $\phi F_v A_b$ for use at service loads shall equal or exceed the shear per bolt due to service loads,

where

ϕ = 1.0 for standard, oversized, and short-slotted holes and long-slotted holes when the long slot is perpendicular or parallel to the line of force

F_v = ~~nominal~~ slip-critical shear resistance tabulated in Table A-J3.2, ksi (MPa). The values for F_v in Table A-J3.2 are based on Class A surfaces with slip coefficient $\mu = 0.33$. When specified by the designer, the ~~nominal~~ slip resistance for connections having special faying surface conditions is permitted to be adjusted to the applicable values in the RCSC Load and Resistance Factor Design Specification. Errata 9/4/01

When the loading combination includes wind loads in addition to dead and live loads, the total shear on the bolt due to combined load effects, at service load, may be multiplied by 0.75.

9. Combined Tension and Shear in Slip-Critical Connections

9b. Slip-Critical Connections Designed at Service Loads

When a slip-critical connection is subjected to an applied tension T that reduces the net clamping force, the slip resistance per bolt, $\phi F_v A_b$, according to Appendix J3.8b shall be multiplied by the following factor:

$$1 - \frac{T}{0.8 T_b N_b}$$

where

T_b = minimum fastener tension from Table J3.1, kips (N)

N_b = number of bolts carrying service-load tension T

APPENDIX K

CONCENTRATED FORCES, PONDING, AND FATIGUE

Appendix K2 provides an alternative determination of roof stiffness. Appendix K3 pertains to the design of members and connections subject to high cyclic loading (fatigue).

K2. PONDING

The provisions of this Appendix are permitted to be used when a more exact determination of flat roof framing stiffness is needed than that given by the provision of Section K2 that $C_p + 0.9C_s \leq 0.25$.

For any combination of primary and secondary framing, the stress index is computed as

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$$U_p = \left[\frac{F_y - f_o}{f_o}\right]_p \quad \text{for the primary member} \qquad \text{(A-K2-1)}$$

$$U_s = \left[\frac{F_y - f_o}{f_o}\right]_s \quad \text{for the secondary member} \qquad \text{(A-K2-2)}$$

where

f_o = the stress due to $1.2D + 1.2R$ (D = nominal dead load, R = nominal load due to rain water or ice exclusive of the ponding contribution),* ksi (MPa)

Enter Figure A-K2.1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required. In the above,

$$C_p = \frac{32 L_s L_p^4}{10^7 I_p}$$

$$\left(\text{Metric: } C_p = \frac{504 L_s L_p^4}{I_p}\right)$$

$$C_s = \frac{32 S L_s^4}{10^7 I_s}$$

*Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves. A load factor of 1.2 shall be used for loads resulting from these phenomena.

ticular structure. Rather, the decision as to which welding process and which filler metal is to be utilized is usually left with the fabricator or erector. To ensure that the proper filler metals are used, codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode.

A4. LOADS AND LOAD COMBINATIONS

The load factors and load combinations are developed in Ellingwood, MacGregor, Galambos, and Cornell (1982) based on the recommended minimum loads given in ASCE 7 (ASCE, 1998).

The load factors and load combinations recognize that when several loads act in combination with the dead load (e.g., dead plus live plus wind), only one of these takes on its maximum lifetime value, while the other load is at its "arbitrary point-in-time value" (i.e., at a value which can be expected to be on the structure at any time). For example, under dead, live, and wind loads the following combinations are appropriate:

$$\gamma_D D + \gamma_L L \tag{C-A4-1}$$

$$\gamma_D D + \gamma_{L_a} L_a + \gamma_W W \tag{C-A4-2}$$

$$\gamma_D D + \gamma_L L + \gamma_{W_a} W_a \tag{C-A4-3}$$

where γ is the appropriate load factor as designated by the subscript symbol.Subscript a refers to an "arbitrary point-in-time" value.

The mean value of arbitrary point-in-time live load L_a is on the order of 0.24 to 0.4 times the mean maximum lifetime live load L for many occupancies, but its dispersion is far greater. The arbitrary point-in-time wind load W_a, acting in conjunction with the maximum lifetime live load, is the maximum daily wind. It turns out that $\gamma_{W_a} W_a$ is a negligible quantity so only two load combinations remain:

$$1.2D + 1.6L \tag{C-A4-4}$$

$$1.2D + 0.5L + \boxed{1.6}W \tag{C-A4-5}$$

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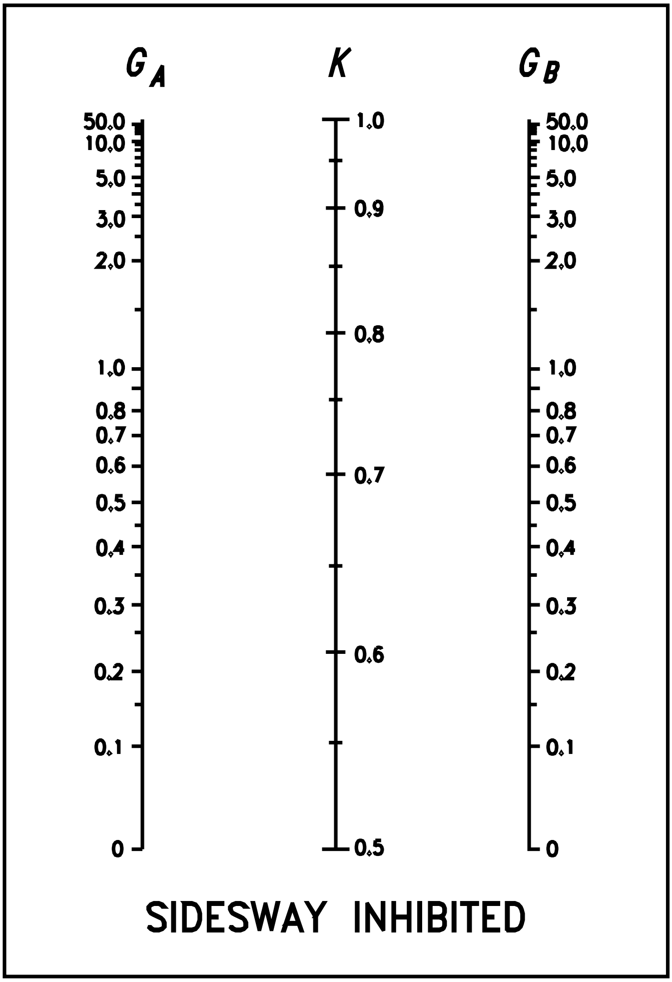
The load factor 0.5 assigned to L in the second formula reflects the statistical properties of L_a, but to avoid having to calculate yet another load, it is reduced so it can be combined with the maximum lifetime wind load.

The nominal loads D, L, W, E, and S are the code loads or the loads given in ASCE 7. The latest edition of the ASCE 7 Standard on structural loads released in 1998 has adopted, in most aspects, the seismic design provisions from NEHRP (1997), as has the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997 and 1999). The reader is referred to the commentaries to these documents for an expanded discussion on seismic loads, load factors, and seismic design of steel buildings.



Notes for Fig. C-C2.2a and b: The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\Sigma(I_c / L_c)}{\Sigma(I_g / L_g)}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying on the plane in which buckling of the column is being considered. I_c is the moment of inertia and L_c the unsupported length of a column section, and I_g is the moment of inertia and L_g the unsupported length of a girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered.

For column ends supported by but not rigidly connected to a footing or foundation, G is theoretically infinity, but, unless actually designed as a true friction-free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis. Errata 9/4/01

Fig. C-C2.2a. Alignment chart for effective length of columns in continuous frames – Sidesway Inhibited.

There are practical cases in the design of structures where slip of the connection is desirable in order to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the directions normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to insure that the nut does not back off under service conditions. Thread deformation is commonly accomplished with a cold chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is discouraged.

2. Size and Use of Holes

To provide some latitude for adjustment in plumbing up a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.3 or J3.3M. The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections J3.3 and J3.4.

3. Minimum Spacing

The *maximum* factored strength R_n at a bolt or rivet hole in bearing requires that the distance between the centerline of the first fastener and the edge of a plate toward which the force is directed should not be less than $1\frac{1}{2}d$ where d is the fastener diameter (Kulak et al., 1987). By similar reasoning the distance measured in the line of force, from the centerline of any fastener to the nearest edge of an adjacent hole, should not be less than $3d$, to ensure maximum design strength in bearing. Plotting of numerous test results indicates that the critical bearing strength is directly proportional to the above defined distances up to a maximum value of $3d$, above which no additional bearing strength is achieved (Kulak et al., 1987). ~~Table J3.7 lists the increments that must be added to adjust the spacing upward to compensate for an increase in hole dimension parallel to the line of force.~~ Section J3.10 gives the bearing strength criteria as a function of spacing. Errata 9/4/01

4. Minimum Edge Distance

Critical bearing stress is a function of the material tensile strength, the spacing of

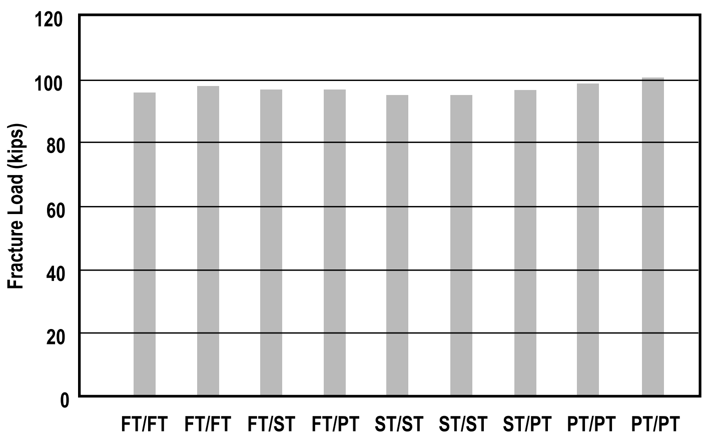


Fig. C-J3.2. Johnson (1996) tests, $3\frac{1}{4}$-in.-long, $\frac{3}{4}$-in.-diameter ASTM A325 bolts.