CHAPTER 3

Seismic Design of Steel Structures

Nomenclature

A_b	cross-sectional area of a horizontal boundary element	in^2
A_c	cross-sectional area of a vertical boundary element	in^2
A_g	gross area	in^2
A_{lw}	web area of link (excluding flanges)	in^2
A_{sc}	cross-sectional area of the yielding segment of steel core	in^2
A_{st}	horizontal cross-sectional area of the link stiffener	in^2
C_a	ratio of required strength to available axial yield strength	–
C_d	coefficient relating relative brace stiffness and curvature	–
D	dead load	kips
D	outside diameter of round HSS	in
E	seismic load effect	kips
E	modulus of elasticity of steel = 29,000	ksi
E_{mh}	horizontal seismic load effect, including the overstrength factor	kips, kip-in
F_e	elastic critical buckling stress	ksi
F_{cr}	critical stress	ksi
F_{cre}	critical stress calculated using expected yield stress	ksi
F_y	specified minimum yield stress	ksi
F_{yb}	specified minimum yield stress of beam	ksi
F_{yc}	specified minimum yield stress of column	ksi
F_u	specified minimum tensile strength	ksi
I	moment of inertia	in^4
I_b	moment of inertia of a horizontal boundary element	in^4
I_c	moment of inertia of a vertical boundary element	in^4
K	effective length factor	–
L	live load due to occupancy and moveable equipment	kips
L	length of column	in
L	length of brace	in

L	distance between vertical boundary element centerlines	in
L_b	unbraced length	in
L_{cf}	clear distance between column flanges	in
L_h	distance between beam plastic hinge locations	in
M_a	required flexural strength, using ASD load combinations	kip-in
M_f	maximum probable moment at the column face	kip-in
M_p	plastic bending moment	kip-in
M_p	plastic bending moment of a link	kip-in
$M_{p,exp}$	expected flexural strength	kip-in
M_{pr}	maximum probable moment at the location of the plastic hinge	kip-in
M_r	required flexural strength	kip-in
M_u	required flexural strength, using LRFD load combinations	kip-in
M_y	yield moment corresponding to yielding of the member in flexure	kip-in
P_a	required axial strength using ASD load combinations	kips
P_c	available axial strength	kips
P_n	nominal axial compressive strength	kips
P_r	required axial compressive strength	kips
P_u	required axial strength using LRFD load combinations	kips
P_y	axial yield strength	kips
P_{ysc}	axial yield strength of steel core	kips
Q_E	effect of horizontal seismic forces	kips, kip-in
R	seismic response modification coefficient	–
R_a	required tensile strength using ASD load combinations	kips
R_n	nominal strength	kips
R_t	ratio of the expected tensile strength to the specified minimum tensile strength	–
R_y	ratio of the expected yield stress to the specified minimum yield stress	–
S	snow load	kips
S_{DS}	design spectral response acceleration at short periods	ft/sec^2
S_h	hinge location distance from face of column	in
V_a	required shear strength using ASD load combinations	kips
V_n	nominal shear strength of link	kips
V_p	plastic shear strength of a link	kips
V_r	required shear strength using LRFD or ASD load combinations	kips
V_u	required shear strength using LRFD load combinations	kips
V_y	shear yield strength	kips

Z	plastic section modulus about the axis of bending	in^3
Z_c	plastic section modulus of column about the axis of bending	in^3
b_{bf}	width of beam flange	in
b_f	width of flange	in
d	overall depth of beam	in
d	overall depth of link	in
d^*	distance between centroids of beam flanges	in
e	length of link	in
h	clear distance between flanges less the fillet for rolled shapes	in
h	distance between horizontal boundary element centerlines	in
h_o	distance between flange centroids	in
r	governing radius of gyration	in
s_h	hinge location distance from center of column	in
t	thickness of column web or individual doubler plate	in
t_{bf}	thickness of beam flange	in
t_f	thickness of flange	in
t_w	thickness of web	in
t_w	web-plate thickness	in
w_z	width of panel zone between column flanges	in

Symbols

α_s	force level adjustment factor = 1.0 for LRFD and 1.5 for ASD	–
β	compressive strength adjustment factor	–
γ_{total}	total link rotation angle	rad
Δ	design story drift	in
Δ_b	total brace axial deformation for the brace test specimen	in
θ	story drift angle	rad
λ_{hd}	slenderness parameter for highly ductile compression elements	–
λ_{md}	slenderness parameter for moderately ductile compression elements	–
ϕ	resistance factor	–
ϕ_c	resistance factor for compression	–
ϕ_v	resistance factor for shear	–
Ω	strain hardening adjustment factor	–
Ω_c	safety factor	–
Ω_c	safety factor for compression	–
Ω_0	system overstrength factor	–

3.1 General design requirements

In accordance with IBC[1] Section 2202.2, steel building structures assigned to seismic design category D, E, or F must be designed and detailed as specified by AISC 341.[2] In accordance with IBC Section 2202.2.1.1, steel building structures assigned to seismic design category B or C may also be designed and detailed as specified by AISC 341. In this case, the seismic loads are computed using the response modification coefficient, R, given in ASCE 7 Table 12.2-1.[3] However, in accordance with IBC Section 2202.2.1.1, steel building structures assigned to seismic design category B or C, with the exception of cantilever column systems, may be designed and detailed as specified by AISC 360.[4] In this case, the seismic loads are computed using a response modification coefficient of $R = 3$, and this alternative may often result in a more economical structure. For seismic design category A, special detailing is not required and steel building structures may be designed and detailed as specified by AISC 360.

3.2 Material strength and ductility

Structural steels used in seismic applications must exhibit the following characteristics:

- a pronounced stress-strain plateau at the yield stress
- a large inelastic strain capability
- good weldability

Elements of the structural system that undergo extremely large plastic rotations in excess of 0.04 radians under the design earthquake are designated as highly ductile members. These members have severe restrictions placed on their width-to-thickness ratios to prevent local buckling as plastic hinges develop. An example of this is the link in an eccentrically braced frame. As shown in Figure 3-1, inelastic action occurs primarily in the link, and the remaining members in the system remain essentially elastic.

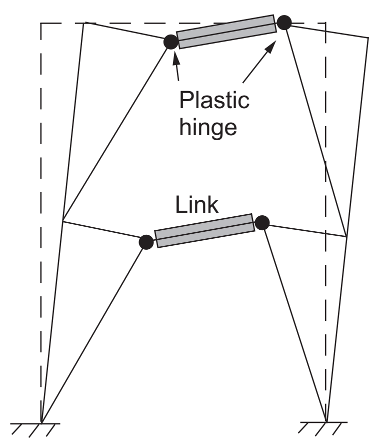

Figure 3-1 Eccentrically braced frame

Elements of the structural system that undergo moderate plastic rotations not exceeding 0.02 radians under the design earthquake are designated as moderately ductile members. These members have less restrictive limits placed on their width-to-thickness ratios. An example of this is the diagonal brace in an eccentrically braced frame. As shown in Figure 3-1, the link serves as a fuse to limit the load transferred to the diagonal braces, which are designed to remain essentially elastic without the possibility of buckling and are designed, as specified in AISC 341 Section F3.5a, as moderately ductile members. As specified in AISC 341 Section F3.5b(1), the link is designed as a highly ductile member.

The diagonal braces in a special concentrically braced frame with chevron configuration, as shown in Figure 3-2, act as the fuses in the system. Inelastic action occurs primarily in the braces and, as specified in AISC 341 Section F2.5a, these are designed as highly ductile members. Beams remain essentially elastic and are designed as moderately ductile members.

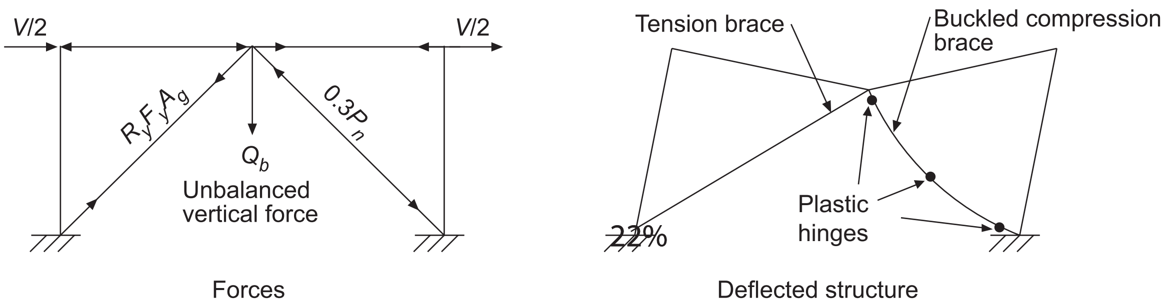

Figure 3-2 Buckled compression brace

In order to prevent local buckling in elements that undergo large plastic deformations, stringent width-to-thickness ratio limits are specified for highly ductile elements. Values of limiting width-to-thickness ratios for moderately ductile compression members, λ_{md}, and highly ductile compression members, λ_{hd}, are tabulated in AISC 341 Table D1.1 and are given in Table 3-1 for the more commonly used sections.

TABLE 3-1 Limiting width-to-thickness ratios

Element	Width-to-thickness ratio	Limiting width-to-thickness ratio	
		Moderately ductile, λ_{md}	Highly ductile, λ_{hd}
Round HSS used as diagonal braces[a]	D/t	$0.062E/R_yF_y$	$0.053E/R_yF_y$
Rectangular HSS used as diagonal braces	b/t	$0.76(E/R_yF_y)^{0.5}$	$0.65(E/R_yF_y)^{0.5}$
Rectangular HSS used in beams or columns	b/t	$1.00(E/R_yF_y)^{0.5}$	$0.55(E/R_yF_y)^{0.5}$
Angles	b/t	$0.38(E/R_yF_y)^{0.5}$	$0.30(E/R_yF_y)^{0.5}$
Flanges of I-shaped members and channels	b/t	$0.38(E/R_yF_y)^{0.5}$	$0.30(E/R_yF_y)^{0.5}$

(continued)

TABLE 3-1 Limiting width-to-thickness ratios—*continued*

Element	Width-to-thickness ratio	Limiting width-to-thickness ratio: Moderately ductile, λ_{md}	Limiting width-to-thickness ratio: Highly ductile, λ_{hd}
Webs of I-shaped sections used as beams or columns[b]	h/t_w	$3.76(1 - 3.05C_a)(E/R_yF_y)^{0.5}$ . . . for $C_a \le 0.113$	$2.45(1 - 1.04C_a)(E/R_yF_y)^{0.5}$ . . . for $C_a \le 0.113$
		$2.61(1.00 - 0.49C_a)(E/R_yF_y)^{0.5} \ge 1.56(E/R_yF_y)^{0.5}$ . . . for $C_a > 0.113$	$2.26(1.00 - 0.38C_a)(E/R_yF_y)^{0.5} \ge 1.56(E/R_yF_y)^{0.5}$ . . . for $C_a > 0.113$
Webs of I-shaped sections used as diagonal braces	h/t_w	$1.49(E/R_yF_y)^{0.5}$	$1.49(E/R_yF_y)^{0.5}$

$C_a = (\alpha_s P_r)/(R_y F_y A_g)$
where:
A_g = gross area, in^2 (mm^2)
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress, ksi (MPa)
P_r = required axial strength using LRFD or ASD load combinations, kips (N)
R_y = ratio of the expected yield stress to the specified minimum yield stress
α_s = LRFD-ASD force level adjustment factor
= 1,0 for LRFD and 1.5 for ASD

3.3 Capacity design and expected material strength

For the design of some elements, a capacity design, or capacity-limited, approach is adopted. One element of the system is designated as the yielding element, or structural fuse. The remaining elements in the system are designed to remain elastic for the anticipated force developed in the yielding element. An example of this is the link in an eccentrically braced frame. As shown in Figure 3-1, yielding occurs at the ends of the link and a mechanism is formed. Forces in the remaining elements of the system are obtained by removing the link and applying the gravity loads and link-induced loads to the remaining structure. The remaining beams and columns are designed to resist the force produced in the link so as to remain essentially elastic.

ASCE 7 Section C12.4.3.2 describes the basis of the capacity design method as the expected strength of one or more elements in a structure being used to generate the required strength for other elements, because the yielding of the former limits the forces delivered to the latter.

Steel sections invariably have a yield stress and a tensile strength greater than the specified minimum values. An accurate estimate of the link strength at yield is required and this requires an accurate estimate of the expected yield stress and tensile strength. Then,

expected yield stress $= R_y F_y$

expected tensile strength $= R_t F_u$