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Performance-Based Seismic Design Criteria For Bridges

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Notations

The following symbols are used in this chapter. The section number in parentheses after definition of a symbol refers to the section where the symbol first appears or is defined.

A = cross-sectional area (Figure 16.9)

A_b = cross-sectional area of batten plate (Section 17.A.1)
A_{close} = area enclosed within mean dimension for a box (Section 17.A.1)
A_d = cross-sectional area of all diagonal lacings in one panel (Section 17.A.1)
A_e = effective net area (Figure 16.9)
A_{equiv} = cross-sectional area of a thin-walled plate equivalent to lacing bars considering shear transferring capacity (Section 17.A.1)
A_f = flange area (Section 17.A.1)
A_g = gross section area (Section 16.7.3)
A_{gt} = gross area subject to tension (Figure 16.9)
A_{gv} = gross area subject to shear (Figure 16.9)
A_i = cross-sectional area of individual component i (Section 17.A.1)
A_{nt} = net area subject to tension (Figure 16.9)
A_{nv} = net area subject to shear (Figure 16.9)
A_p = cross-sectional area of pipe (Section 16.7.3)
A_r = nominal area of rivet (Section 16.7.3)
A_s = cross-sectional area of steel members (Figure 16.8)
A_w = cross-sectional area of web (Figure 16.12)
A_i^* = cross-sectional area above or below plastic neutral axis (Section 17.A.1)
A_{equiv}^* = cross-sectional area of a thin-walled plate equivalent to lacing bars or battens assuming full section integrity (Section 17.A.1)
$\overline{B}$ = ratio of width to depth of steel box section with respect to bending axis (Section 17.A.4)
C = distance from elastic neutral axis to extreme fiber (Section 17.A.1)
C_b = bending coefficient dependent on moment gradient (Figure 16.10)
C_w = warping constant, in.^{6} (Table 16.2)
D_Δ = damage index defined as ratio of elastic displacement demand to ultimate displacement (Section 17.A.3)
DC_{accept} = Acceptable force demand/capacity ratio (Section 16.8.1)
E = modulus of elasticity of steel (Figure 16.8)
E_c = modulus of elasticity of concrete (Section 16.5.2)
E_s = modulus of elasticity of reinforcement (Section 16.5.2)
E_t = tangent modulus (Section 17.A.4)
$(EI)_{eff}$ = effective flexural stiffness (Section 17.A.4)
F_L = smaller of $(F_{yf} - F_r)$ or F_{yw}, ksi (Figure 16.10)
F_r = compressive residual stress in flange; 10 ksi for rolled shapes, 16.5 ksi for welded shapes (Figure 16.10)
F_u = specified minimum tensile strength of steel, ksi (Section 16.5.2)
F_{umax} = specified maximum tensile strength of steel, ksi (Section 16.5.2)
F_y = specified minimum yield stress of steel, ksi (Section 16.5.2)
F_{yf} = specified minimum yield stress of the flange, ksi (Figure 16.10)
F_{ymax} = specified maximum yield stress of steel, ksi (Section 16.5.2)
F_{yw} = specified minimum yield stress of the web, ksi (Figure 16.10)
G = shear modulus of elasticity of steel (Table 16.2)
I_b = moment of inertia of a batten plate (Section 17.A.1)
I_f = moment of inertia of one solid flange about weak axis (Section 17.A.1)
I_i = moment of inertia of individual component i (Section 17.A.1)
I_s = moment of inertia of the stiffener about its own centroid (Section 16.7.3)
I_{x-x} = moment of inertia of a section about x-x axis (Section 17.A.1)
I_{y-y} = moment of inertia of a section about y-y axis considering shear transferring capacity (Section 17.A.1)

I_y = moment of inertia about minor axis, in.^{4} (Table 16.2)
J = torsional constant, in.^{4} (Figure 16.10)
K_a = effective length factor of individual components between connectors (Figure 16.8)
K = effective length factor of a compression member (Section 16.7.2)
L = unsupported length of a member (Figure 16.8)
L_g = free edge length of gusset plate (Section 16.7.3)
M = bending moment (Figure 16.26)
M_1 = larger moment at end of unbraced length of beam (Table 16.2)
M_2 = smaller moment at end of unbraced length of beam (Table 16.2)
M_n = nominal flexural strength (Figure 16.10)
M_n^{FLB} = nominal flexural strength considering flange local buckling (Figure 16.10)
M_n^{LTB} = nominal flexural strength considering lateral torsional buckling (Figure 16.10)
M_n^{WLB} = nominal flexural strength considering web local buckling (Figure 16.10)
M_p = plastic bending moment (Figure 16.10)
M_r = elastic limiting buckling moment (Figure 16.10)
M_u = factored bending moment demand (Section 16.7.3)
M_y = yield moment (Figure 16.10)
$M_{p-\text{batten}}$ = plastic moment of a batten plate about strong axis (Figure 16.12)
$M_{\varepsilon c}$ = moment at which compressive strain of concrete at extreme fiber equal to 0.003 (Section 16.7.4)
N_s = number of shear planes per rivet (Section 16.7.3)
P = axial force (Section 17.A.4)
P_{cr} = elastic buckling load of a built-up member considering buckling mode interaction (Section 17.A.2)
P_L = elastic buckling load of an individual component (Section 17.A.2)
P_G = elastic buckling load of a global member (Section 17.A.2)
P_n = nominal axial strength (Figure 16.8)
P_u = factored axial load demands (Figure 16.13)
P_y = yield axial strength (Section 16.7.3)
P_n^* = nominal compressive strength of column (Figure 16.8)
P_n^{LG} = nominal compressive strength considering buckling mode interaction (Figure 16.8)
P_n^b = nominal tensile strength considering block shear rupture (Figure 16.9)
P_n^f = nominal tensile strength considering fracture in net section (Figure 16.9)
P_n^s = nominal compressive strength of a solid web member (Figure 16.8)
P_n^y = nominal tensile strength considering yielding in gross section (Figure 16.9)
P_n^{comp} = nominal compressive strength of lacing bar (Figure 16.12)
P_n^{ten} = nominal tensile strength of lacing bar (Figure 16.12)
Q = full reduction factor for slender compression elements (Figure 16.8)
Q_i = force effect (Section 16.4.1)
R_e = hybrid girder factor (Figure 16.10)
R_n = nominal shear strength (Section 16.7.3)
S = elastic section modulus (Figure 16.10)
S_{eff} = effective section modulus (Figure 16.10)
S_x = elastic section modulus about major axis, in.^{3} (Figure 16.10)
T_n = nominal tensile strength of a rivet (Section 16.7.3)
V_c = nominal shear strength of concrete (Section 16.7.4)
V_n = nominal shear strength (Figure 16.12)
V_p = plastic shear strength (Section 16.7.3)
V_s = nominal shear strength of transverse reinforcement (Section 16.7.4)
V_t = shear strength carried bt truss mechanism (Section 16.7.4)

V_u	=	factored shear demand (Section 16.7.3)
X_1	=	beam buckling factor defined by AISC-LRFD [4] (Figure 16.11)
X_2	=	beam buckling factor defined by AISC-LRFD [4] (Figure 16.11)
Z	=	plastic section modulus (Figure 16.10)
a	=	distance between two connectors along member axis (Figure 16.8)
b	=	width of compression element (Figure 16.8)
b_i	=	length of particular segment of (Section 17.A.1)
d	=	effective depth of (Section 16.7.4)
f'_c	=	specified compressive strength of concrete (Section 16.7.5)
f_{cmin}	=	specified minimum compressive strength of concrete (Section 16.5.2)
f_r	=	modulus of rupture of concrete (Section 16.5.2)
f_{yt}	=	probable yield strength of transverse steel (Section 16.7.4)
h	=	depth of web (Figure 16.8) or depth of member in lacing plane (Section 17.A.1)
k	=	buckling coefficient (Table 16.3)
k_v	=	web plate buckling coefficient (Figure 16.12)
l	=	length from the last rivet (or bolt) line on a member to first rivet (or bolt) line on a member measured along the centerline of member (Section 16.7.3)
m	=	number of panels between point of maximum moment to point of zero moment to either side [as an approximation, half of member length ($L/2$) may be used] (Section 17.A.1)
m_{batten}	=	number of batten planes (Figure 16.12)
m_{lacing}	=	number of lacing planes (Figure 16.12)
n	=	number of equally spaced longitudinal compression flange stiffeners (Table 16.3)
n_r	=	number of rivets connecting lacing bar and main component at one joint (Figure 16.12)
r	=	radius of gyration, in. (Figure 16.8)
r_i	=	radius of gyration of local member, in. (Figure 16.8)
r_y	=	radius of gyration about minor axis, in. (Figure 16.10)
t	=	thickness of unstiffened element (Figure 16.8)
t_i	=	average thickness of segment b_i (Section 17.A.1)
t_{equiv}	=	thickness of equivalent thin-walled plate (Section 17.A.1)
t_w	=	thickness of the web (Figure 16.10)
v_c	=	permissible shear stress carried by concrete (Section 16.7.4)
x	=	subscript relating symbol to strong axis or x-x axis (Figure 16.13)
x_i	=	distance between y-y axis and center of individual component i (Section 17.A.1)
x_i^*	=	distance between center of gravity of a section A_i^* and plastic neutral y-y axis (Section 17.A.1)
y	=	subscript relating symbol to strong axis or y-y axis (Figure 16.13)
y_i^*	=	distance between center of gravity of a section A_i^* and plastic neutral x-x axis (Section 17.A.1)
Δ_{ed}	=	elastic displacement demand (Section 17.A.3)
Δ_u	=	ultimate displacement (Section 17.A.3)
α	=	separation ratio (Section 17.A.2)
α_x	=	parameter related to biaxial loading behavior for x-x axis (Section 17.A.4)
α_y	=	parameter related to biaxial loading behavior for y-y axis (Section 17.A.4)
β	=	0.8, reduction factor for connection (Section 16.7.3)
β_m	=	reduction factor for moment of inertia specified by Equation 16.28 (Section 17.A.1)
β_t	=	reduction factor for torsion constant may be determined Equation 16.38 (Section 17.A.1)
β_x	=	parameter related to uniaxial loading behavior for x-x axis (Section 17.A.4)
β_y	=	parameter related to uniaxial loading behavior for y-y axis (Section 17.A.4)

δ_o = imperfection (out-of-straightness) of individual component (Section 17.A.2)
γ_{LG} = buckling mode interaction factor to account for buckling model interaction (Figure 16.8)
λ = width-thickness ratio of compression element (Figure 16.8)
λ_b = $\frac{L}{r_y}$ (slenderness parameter of flexural moment dominant members) (Figure 16.10)
λ_{bp} = limiting beam slenderness parameter for plastic moment for seismic design (Figure 16.10)
λ_{br} = limiting beam slenderness parameter for elastic lateral torsional buckling (Figure 16.10)
λ_{bpr} = limiting beam slenderness parameter determined by Equation 16.25 (Table 16.2)
λ_c = $\left(\frac{KL}{r\pi}\right)\sqrt{\frac{F_y}{E}}$ (slenderness parameter of axial load dominant members) (Figure 16.8)
λ_{cp} = 0.5 (limiting column slenderness parameter for 90% of the axial yield load based on AISC-LRFD [4] column curve) (Table 16.2)
λ_{cpr} = limiting column slenderness parameter determined by Equation 16.24 (Table 16.2)
λ_{cr} = limiting column slenderness parameter for elastic buckling (Table 16.2)
λ_p = limiting width-thickness ratio for plasticity development specified in Table 16.3 (Figure 16.10)
λ_{pr} = limiting width-thickness ratio determined by Equation 16.23 (Table 16.2)
λ_r = limiting width-thickness ratio (Figure 16.8)
$\lambda_{p-\text{Seismic}}$ = limiting width-thickness ratio for seismic design (Table 16.2)
μ_Δ = displacement ductility, ratio of ultimate displacement to yield displacement (Section 16.7.4)
μ_ϕ = curvature ductility, ratio of ultimate curvature to yield curvature (Section 17.A.3)
ρ'' = ratio of transverse reinforcement volume to volume of confined core (Section 16.7.4)
ϕ = resistance factor (Section 16.7.1)
ϕ = angle between diagonal lacing bar and the axis perpendicular to the member axis (Figure 16.12)
ϕ_b = resistance factor for flexure (Figure 16.13)
ϕ_{bs} = resistance factor for block shear (Section 16.7.1)
ϕ_c = resistance factor for compression (Figure 16.13)
ϕ_t = resistance factor for tension (Figure 16.9)
ϕ_{tf} = resistance factor for tension fracture in net (section 16.7.1)
ϕ_{ty} = resistance factor for tension yield (Figure 16.9)
σ_c^{comp} = maximum concrete stress under uniaxial compression (Section 16.7.5)
σ_c^{ten} = maximum concrete stress under uniaxial tension (Section 16.7.5)
σ_s = maximum steel stress under uniaxial tension (Section 16.7.5)
τ_u = shear strength of a rivet (Section 16.7.3)
ε_s = maximum steel strain under uniaxial tension (Section 16.7.5)
ε_{sh} = strain hardening strain of steel (Section 16.5.2)
$\varepsilon_c^{\text{comp}}$ = maximum concrete strain under uniaxial compression (Section 16.7.5)
γ_i = load factor corresponding to Q_i (Section 16.4.1)
η = a factor relating to ductility, redundancy, and operational importance (Section 16.4.1)

16.2 Introduction

16.2.1 Damage to Bridges in Recent Earthquakes

Since the beginning of civilization, earthquake disasters have caused both death and destruction — the structural collapse of homes, buildings, and bridges. About 20 years ago, the 1976 Tangshan earthquake in China resulted in the tragic death of 242,000 people, while 164,000 people were severely injured, not to mention the entire collapse of the industrial city of Tangshan [39]. More recently, the 1989 Loma Prieta and the 1994 Northridge earthquakes in California [27, 28] and the 1995 Kobe earthquake in Japan [29] have exacted their tolls in the terms of deaths, injuries, and the collapse of the infrastructure systems which can in turn have detrimental effects on the economies. The damage and collapse of bridge structures tend to have a more lasting image on the public.

Figure 16.1 shows the collapsed elevated steel conveyor at Lujiatuo Mine following the 1976 Tangshan earthquake in China. Figures 16.2 and 16.3 show damage from the 1989 Loma Prieta earthquake: the San Francisco-Oakland Bay Bridge east span drop off and the collapsed double deck portion of the Cypress freeway, respectively. Figure 16.4 shows a portion of the R-14/I-5 interchange following the 1994 Northridge earthquake, which also collapsed following the 1971 San Fernando earthquake in California while it was under construction. Figure 16.5 shows a collapsed 500-m section of the elevated Hanshin Expressway during the 1995 Kobe earthquake in Japan. These examples of bridge damage, though tragic, have served as full-scale laboratory tests and have forced bridge engineers to reconsider their design principles and philosophies. Since the 1971 San Fernando earthquake, it has been a continuing challenge for bridge engineers to develop a safe seismic design procedure so that the structures are able to withstand the sometimes unpredictable devastating earthquakes.

FIGURE 16.1: Collapsed elevated steel conveyor at Lujiatuo Mine following the 1976 Tangshan earthquake in China. (From California Institute of Technology, The Greater Tangshan Earthquake, California, 1996. With permission.)

FIGURE 16.2: Aerial view of collapsed upper and lower decks of the San Francisco-Oakland Bay Bridge (I-80) following the 1989 Loma Prieta earthquake in California. (Photo by California Department of Transportation. With permission.)

16.2.2 No-Collapse-Based Design Criteria

For seismic design and retrofit of ordinary bridges, the primary philosophy is to prevent collapse during severe earthquakes [13, 24, 25]. The structural survival without collapse has been a basis of seismic design and retrofit for many years [13]. To prevent the collapse of bridges, two alternative design approaches are commonly in use. First is the conventional force-based approach where the adjustment factor Z for ductility and risk assessment [12], or the response modification factor R [1], is applied to elastic member force levels obtained by acceleration spectra analysis. The second approach is the newer displacement-based design approach [13] where displacements are a major consideration in design. For more detailed information, reference is made to a comprehensive and state-of-the-art book by Prietley et al. [35]. Much of the information in this book is backed by California Department of Transportation (Caltrans)-supported research, directed at the seismic performance of bridge structures.

16.2.3 Performance-Based Design Criteria

Following the 1989 Loma Prieta earthquake, bridge engineers recognized the need for site-specific and project-specific design criteria for important bridges. A bridge is defined as “important” when one of the following criteria is met:

- The bridge is required to provide secondary life safety.
- Time for restoration of functionality after closure creates a major economic impact.
- The bridge is formally designated as critical by a local emergency plan.

FIGURE 16.3: Collapsed Cypress Viaduct (I-880) following the 1989 Loma Prieta earthquake in California.

FIGURE 16.4: Collapsed SR-14/I-5 south connector overhead following the 1994 Northridge earthquake in California. (Photo by James MacIntyre. With permission.)

Caltrans, in cooperation with various emergency agencies, has designated and defined the various important routes throughout the state of California. For important bridges, such as I-880 replacement [23] and R-14/I-5 interchange replacement projects, the design criteria [10, 11] including site-specific Acceleration Response Spectrum (ARS) curves and specific design procedures to reflect the desired performance of these structures were developed.

FIGURE 16.5: Collapsed Hanshin Expressway following the 1995 Kobe earthquake in Japan. (Photo by Mark Yashinsky. With permission.)

In 1995, Caltrans, in cooperation with engineering consulting firms, began the task of seismic retrofit design for the seven major toll bridges including the San Francisco-Oakland Bay Bridge (SFOBB) in California. Since the traditional seismic design procedures could not be directly applied to these toll bridges, various analysis and design concepts and strategies have been developed [7]. These differences can be attributed to the different post-earthquake performance requirements. As shown in Figure 16.6, the performance requirements for a specific project or bridge must be the first item to be established. Loads, materials, analysis methods and approaches, and detailed acceptance criteria are then developed to achieve the expected performance. The no-collapse-based design criteria shall be used unless performance-based design criteria is required.

16.2.4 Background of Criteria Development

It is the purpose of this chapter to present performance-based criteria that may be used as a guideline for seismic design and retrofit of important bridges. More importantly, this chapter provides concepts for the general development of performance-based criteria. The appendices, as an integral part of the criteria, are provided for background and information of criteria development. However, it must be recognized that the desired performance of the structure during various earthquakes ultimately defines the design procedures.

Much of this chapter was primarily based on the Seismic Retrofit Design Criteria (*Criteria*) which was developed for the SFOBB West Spans [17]. The SFOBB *Criteria* was developed and based on past successful experience, various codes, specifications, and state-of-the-art knowledge.

The SFOBB, one of the national engineering wonders, provides the only direct highway link between San Francisco and the East Bay Communities. SFOBB (Figure 16.7) carries Interstate Highway 80 approximately 8-1/4 miles across San Francisco Bay since it first opened to traffic in 1936. The west spans of SFOBB, consisting of twin, end-to-end double-deck suspension bridges and a three-span double-deck continuous truss, crosses the San Francisco Bay from the city of San Francisco to Yerba Buena Island. The seismic retrofit design of SFOBB West Spans, as the top priority project of the California Department of Transportation, is a challenge to bridge engineers. A performance-based design *Criteria* [17] was, therefore, developed for SFOBB West Spans.

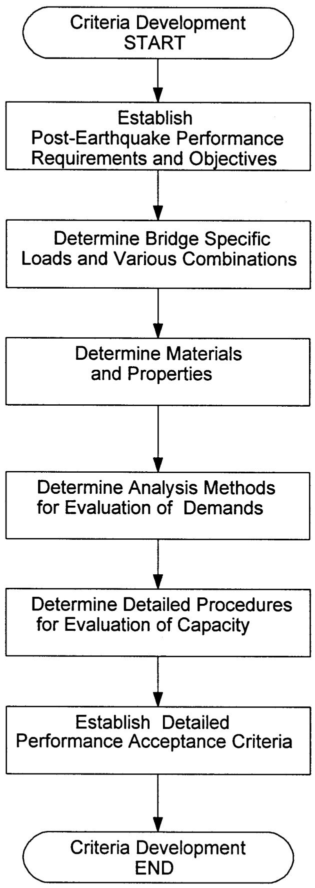

FIGURE 16.6: Development procedure of performance-based seismic design criteria for important bridges.

16.3 Performance Requirements

16.3.1 General

The seismic design and retrofit of important bridges shall be performed by considering both the higher level Safety Evaluation Earthquake (SEE), which has a mean return period in the range of 1000 to 2000 years, and the lower level Functionality Evaluation Earthquake (FEE), which has a mean return period of 300 years with a 40% probability of exceedance during the expected life of the bridge. It is important to note that the return periods of both the SEE and FEE are dictated by the engineers and seismologists.

16.3.2 Safety Evaluation Earthquake

The bridge shall remain serviceable after a SEE. Serviceable is defined as sustaining repairable damage with minimum impact to functionality of the bridge structure. In addition, the bridge will be open to emergency vehicles immediately following the event, provided bridge management personnel can provide access.

(a) West crossing spans.

(b) East crossing spans.

FIGURE 16.7: San Francisco-Oakland Bay Bridge. (Photo by California Department of Transportation. With permission.)

16.3.3 Functionality Evaluation Earthquake

The bridge shall remain fully operational after a FEE. Fully operational is defined as full accessibility to the bridge by current normal daily traffic. The structure may suffer repairable damage, but repair operations may not impede traffic in excess of what is currently required for normal daily maintenance.

16.3.4 Objectives of Seismic Design

The objectives of seismic design are as follows:

1. To keep the *Critical* structural components in the essentially elastic range during the SEE.
2. To achieve safety, reliability, serviceability, constructibility, and maintainability when the Seismic Response Modification Devices (SRMDs), i.e., energy dissipation and isolation devices, are installed in bridges.
3. To devise expansion joint assemblies between bridge frames that either retain traffic support or, with the installation of deck plates, are able to carry the designated traffic after being subjected to SEE displacements.
4. To provide ductile load paths and detailing to ensure bridge safety in the event that future demands might exceed those demands resulting from current SEE ground motions.

16.4 Loads and Load Combinations

16.4.1 Load Factors and Combinations

New and retrofitted bridge components shall be designed for the applicable load combinations in accordance with the requirements of AASHTO-LRFD [1].

The load effect shall be obtained by

$$\text{Load effect } = \eta \sum \gamma_i Q_i \tag{16.1}$$

where

Q_i = force effect

η = a factor relating to ductility, redundancy, and operational importance

γ_i = load factor corresponding to Q_i

.

The AASHTO-LRFD load factors or load factors $\eta = 1.0$ and $\gamma_i = 1.0$ may be used for seismic design.

The live load on the bridge shall be determined by ADTT (Average Daily Truck Traffic) value for the project. The bridge shall be analyzed for the worst case with or without live load. The mass of the live loads shall not be included in the dynamic calculations. The intent of the live load combination is to include the weight effect of the vehicles only.

16.4.2 Earthquake Load

The earthquake load – ground motions and response spectra shall be considered at two levels: SEE and FEE. The ground motions and response spectra may be generated in accordance with Caltrans Guidelines [14, 15].

16.4.3 Wind Load

1. Wind load on structures — Wind loads shall be applied as a static equivalent load in accordance with AASHTO-LRFD [1] .
2. Wind load on live load — Wind pressure on vehicles shall be represented by a uniform load of 0.100 kips/ft (1.46 kN/m) applied at right angles to the longitudinal axis of the structure and 6.0 ft (1.85 m) above the deck to represent the wind load on vehicles.
3. Wind load dynamics — The expansion joints, SRMDs, and wind locks (tongues) shall be evaluated for the dynamic effects of wind loads.

16.4.4 Buoyancy and Hydrodynamic Mass

The buoyancy shall be considered to be an uplift force acting on all components below design water level. Hydrodynamic mass effects [26] shall be considered for bridges over water.

16.5 Structural Materials

16.5.1 Existing Materials

For seismic retrofit design, aged concrete with specified strength of 3250 psi (22.4 MPa) can be considered to have a compressive strength of 5000 psi (34.5 MPa). If possible, cores of existing concrete should be taken. Behavior of structural steel and reinforcement shall be based on mill certificate or tensile test results. If they are not available in bridge archives, a nominal strength of 1.1 times specified yield strength may be used [13].

16.5.2 New Materials

Structural Steel

New structural steel used shall be AASHTO designation M270 (ASTM designation A709) Grade 36 and Grade 50.

Welds shall be as specified in the Bridge Welding Code ANSI/AASHTO/AWS D1.5-95 [8].

Partial penetration welds shall not be used in regions of structural components subjected to possible inelastic deformation.

High strength bolts conforming to ASTM designation A325 shall be used for all new connections and for upgrading strengths of existing riveted connections. New bolted connections shall be designed as bearing-type for seismic loads and shall be slip-critical for all other load cases.

All bolts with a required length under the head greater than 8 in. shall be designated as ASTM A449 threaded rods (requiring nuts at each end) unless a verified source of longer bolts can be identified.

New anchor bolts shall be designated as ASTM A449 threaded rods.

Structural Concrete

All concrete shall be normal weight concrete with the following properties:

Specified compressive strength:	$f_{\mathrm{cmin}} = 4,000$	psi (27.6MPa)
Modulus of elasticity:	$E_c = 57{,}000\sqrt{f_c'}$	psi
Modulus of rupture:	$f_r = 5\sqrt{f_c'}$	psi

Reinforcement

All reinforcement shall use ASTM A706 (Grade 60) with the following specified properties:

Specified minimum yield stress:	$F_y = 60$ ksi	(414 MPa)
Specified minimum tensile strength:	$F_u = 90$ ksi	(621 MPa)
Specified maximum yield stress:	$F_{y\max} = 78$ ksi	(538 MPa)
Specified maximum tensile strength:	$F_{u\max} = 107$ ksi	(738 MPa)
Modulus of elasticity:	$E_s = 29{,}000$ ksi	(200,000 MPa)

$$\text{Strain hardening strain:} \quad \varepsilon_{sh} = \begin{cases} 0.0150 & \text{for \#8 and smallers bars} \\ 0.0125 & \text{for \#9} \\ 0.0100 & \text{for \#10 and \#11} \\ 0.0075 & \text{for \#14} \\ 0.0050 & \text{for \#18} \end{cases}$$

16.6 Determination of Demands

16.6.1 Analysis Methods

Static Linear Analysis

Static linear analysis shall be used to determine member forces due to self weight, wind, water currents, temperature, and live load.

Dynamic Response Spectrum Analysis

1. Dynamic response spectrum analysis shall be used for the local and regional stand alone models and the simplified global model described in Section 16.6.2 to determine mode shapes, structure periods, and initial estimates of seismic force and displacement demands.
2. Dynamic response spectrum analysis may be used on global models prior to time history analysis to verify model behavior and eliminate modeling errors.
3. Dynamic response spectrum analysis may be used to identify initial regions or members of likely inelastic behavior which need further refined analysis using inelastic nonlinear elements.
4. Site specific ARS curves shall be used, with 5% damping.
5. Modal responses shall be combined using the Complete Quadratic Combination (CQC) method and the resulting orthogonal responses shall be combined using either the Square Root of the Sum of the Squares (SRSS) method or the “30%” rule, e.g., $R_H = \text{Max}(R_x + 0.3R_y, R_y + 0.3R_x)$ [13].
6. Due to the expected levels of inelastic structural response in some members and regions, dynamic response spectrum analysis shall not be used to determine final design demand values or to assess the performance of the retrofitted structures.

Dynamic Time History Analysis

Site specific multi-support dynamic time histories shall be used in a dynamic time history analysis. All analyses incorporating significant nonlinear behavior shall be conducted using nonlinear inelastic dynamic time history procedures.

1. Linear elastic dynamic time history analysis — Linear elastic dynamic time history analysis is defined as dynamic time history analysis with considerations of geometrical linearity

(small displacement), linear boundary conditions, and elastic members. It shall only be used to check regional and global models.

2. Nonlinear elastic dynamic time history analysis — Nonlinear elastic time history analysis is defined as dynamic time history analysis with considerations of geometrical nonlinearity, linear boundary conditions, and elastic members. It shall be used to determine areas of inelastic behavior prior to incorporating inelasticity into the regional and global models.
3. Nonlinear inelastic dynamic time history analysis – Level I — Nonlinear inelastic dynamic time history analysis – Level I is defined as dynamic time history analysis with considerations of geometrical nonlinearity, nonlinear boundary conditions, other inelastic elements (for example, dampers) and elastic members. It shall be used for the final determination of force and displacement demands for existing structures in combination with static gravity, wind, thermal, water current, and live load as specified in Section 16.4.
4. Nonlinear inelastic dynamic time history analysis – Level II — Nonlinear inelastic dynamic time history analysis – Level II is defined as dynamic time history analysis with considerations of geometrical nonlinearity, nonlinear boundary conditions, other inelastic elements (for example, dampers) and inelastic members. It shall be used for the final evaluation of response of the structures. Reduced material and section properties, and the yield surface equation suggested in the Appendix may be used for inelastic considerations.

16.6.2 Modeling Considerations

Global, Regional, and Local Models

The global models focus on the overall behavior and may include simplifications of complex structural elements. Regional models concentrate on regional behavior. Local models emphasize the localized behavior, especially complex inelastic and nonlinear behavior. In regional and global models where more than one foundation location is included in the model, multi-support time history analysis shall be used.

Boundary Conditions

Appropriate boundary conditions shall be included in the regional models to represent the interaction between the regional model and the adjacent portion of the structure not explicitly included. The adjacent portion not specifically included may be modeled using simplified structural combinations of springs, dashpots, and lumped masses.

Appropriate nonlinear elements such as gap elements, nonlinear springs, SRMDs, or specialized nonlinear finite elements shall be included where the behavior and response of the structure is determined to be sensitive to such elements.

Soil-Foundation-Structure-Interaction

Soil-Foundation-Structure-Interaction may be considered using nonlinear or hysteretic springs in the global and regional models. Foundation springs at the base of the structure which reflect the dynamic properties of the supporting soil shall be included in both regional and global models.

Section Properties of Latticed Members

For latticed members, the procedure proposed in the

Appendix may be used for member characterization.

Damping

When nonlinear member properties are incorporated in the model, Rayleigh damping shall be reduced, for example by 20%, compared with analysis with elastic member properties.

Seismic Response Modification Devices

The SRMDs, i.e., energy dissipation and isolation devices, shall be modeled explicitly using their hysteretic characteristics as determined by tests.

16.7 Determination of Capacities

16.7.1 Limit States and Resistance Factors

Limit States

The limit states are defined as those conditions of a structure at which it ceases to satisfy the provisions for which it was designed. Two kinds of limit states corresponding to SEE and FEE specified in Section 16.3 apply for seismic design and retrofit.

Resistance Factors

To account for unavoidable inaccuracies in the theory, variation in the material properties, workmanship, and dimensions, nominal strength of structural components should be modified by a resistance factor ϕ to obtain the design capacity or strength (resistance). The following resistance factors shall be used for seismic design:

- For tension fracture in net section $\phi_{tf} = 0.8$
- For block shear $\phi_{bs} = 0.8$
- For bolts and welds $\phi = 0.8$
- For all other cases $\phi = 1.0$

16.7.2 Effective Length of Compression Members

The effective length factor K for compression members shall be determined in accordance with Chapter 17 of this Handbook.

16.7.3 Nominal Strength of Steel Structures

Members

1. General — Steel members include rolled members and built-up members, such as latticed, battened, and perforated members. The design strength of those members shall be according to applicable provisions of AISC-LRFD [4]. Section properties of latticed members shall be determined in accordance with the Appendix.
2. Compression members — For compression members, the nominal strength shall be determined in accordance with Section E2 and Appendix B of AISC-LRFD [4]. For built-up members, effects of interaction of buckling modes shall be considered in accordance with the Appendix. A detailed procedure in a flowchart format is shown in Figure 16.8.
3. Tension Members — For tension members, the design strength shall be determined in accordance with Sections D1 and J4 of AISC-LRFD [4]. It is the smallest value obtained according to (i) yielding in gross section, (ii) fracture in net section, and (iii) block shear rupture. A detailed procedure is shown in Figure 16.9.

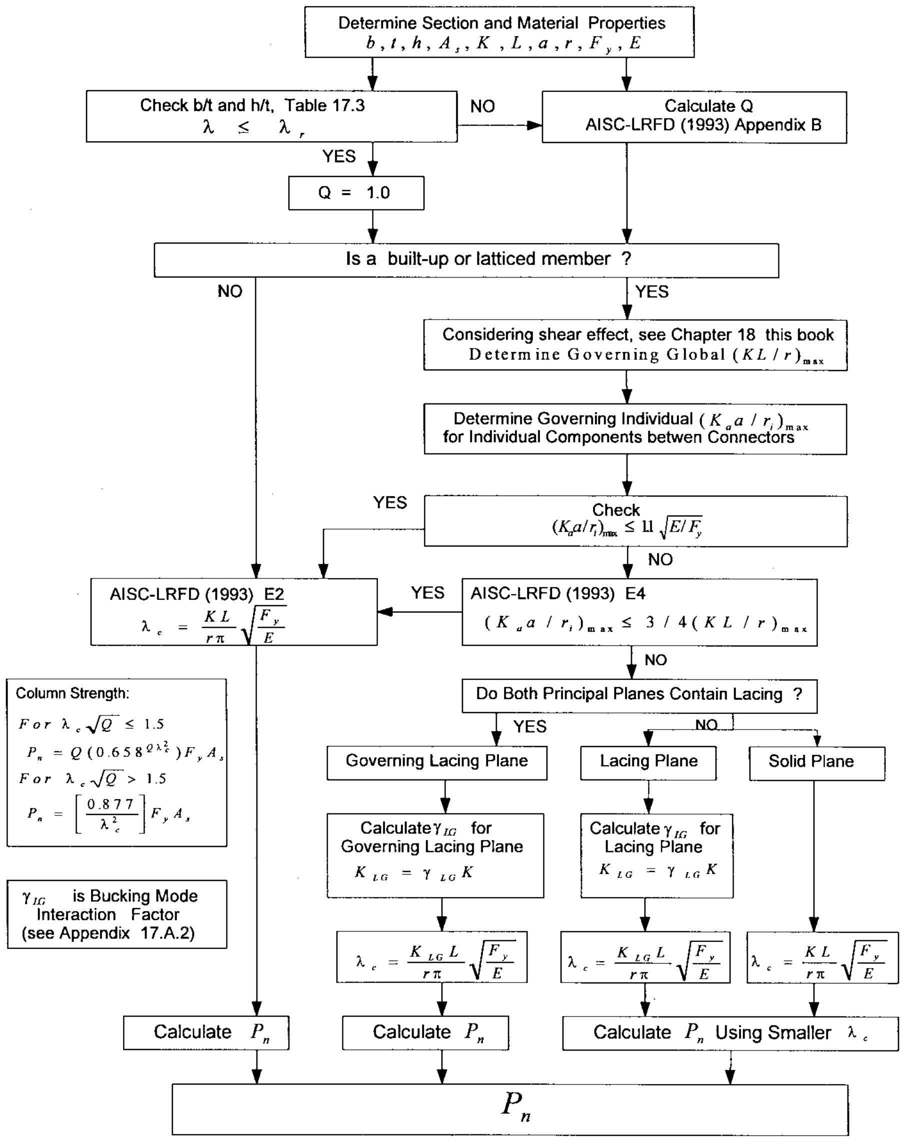

FIGURE 16.8: Evaluation procedure for nominal compressive strength of steel members.

4. Flexural members — For flexural members, the nominal flexural strength shall be determined in accordance with Section F1 and Appendices B, F, and G of AISC-LRFD [4].

   - For *critical* members, the nominal flexural strength is the smallest value according to (i) initial yielding, (ii) lateral-torsional buckling, (iii) flange local buckling, and (iv) web local buckling.
   - For *other* members, the nominal flexural strength is the smallest value according to (i) plastic moment, (ii) lateral-torsional buckling, (iii) flange local buckling, and (iv) web local buckling.

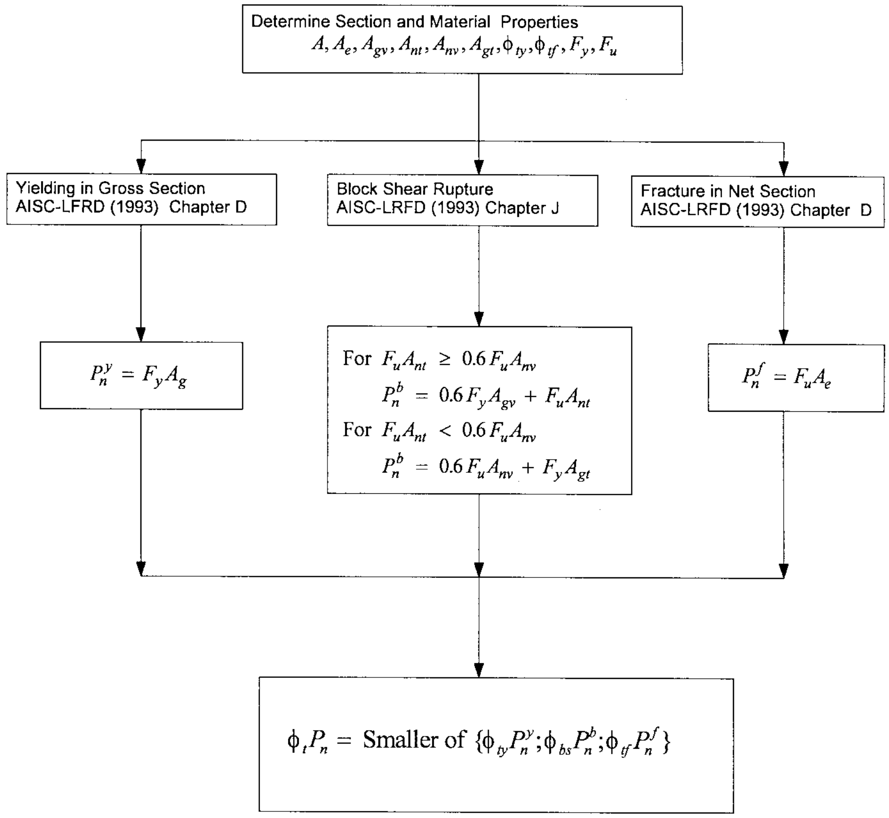

FIGURE 16.9: Evaluation procedure for tensile strength of steel members.

Detailed procedures for flexural strength of box- and I-shaped members are shown in Figures 16.10 and 16.11, respectively.

5. Nominal shear strength — For solid-web steel members, the nominal shear strength shall be determined in accordance with Appendix F2 of AISC-LRFD [4]. For latticed members, the shear strength shall be based on shear-flow transfer-capacity of lacing bar, battens, and connectors as discussed in the Appendix. A detailed procedure for shear strength is shown in Figure 16.12.
6. Members subjected to bending and axial force — For members subjected to bending and axial force, the evaluation shall be according to Section H1 of AISC-LRFD [4], i.e., the bi-linear interaction equation shall be used. The recent study on "Cyclic Testing of Latticed Members for San Francisco-Oakland Bay Bridge" at UCSD [37] recommends that the AISC-LRFD interaction equation can be used directly for seismic evaluation of latticed members. A detailed procedure for steel beam-columns is shown in Figure 16.13.

Gusset Plate Connections

1. General description — Gusset plates shall be evaluated for shear, bending, and axial forces according to Article 6.14.2.8 of AASHTO-LRFD [1]. The internal stresses in the gusset plate shall be determined according to Whitmore's method in which the *effective* area is defined as the width bound by two 30° lines drawn from the first row of the bolt or rivet

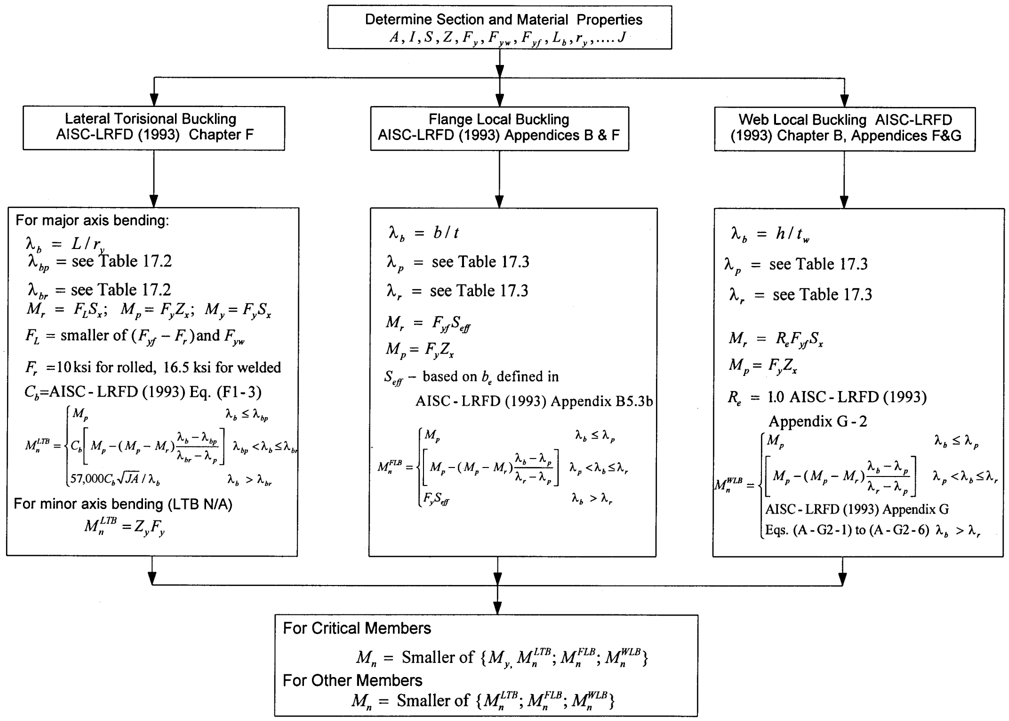

FIGURE 16.10: Evaluation procedure for nominal flexural strength of box-shaped steel members.

group to the last bolt or rivet line. The stresses in the gusset plate may be determined by more rational methods or refined computer models.

2. Tension strength — The tension capacity of the gusset plates shall be calculated according to Article 6.13.5.2 of AASHTO-LRFD [1].
3. Compressive strength — The compression capacity of the gusset plates shall be calculated according to Article 6.9.4.1 of AASHTO-LRFD [1]. In using the AASHTO-LRFD Equations (6.9.4.1-1) and (6.9.4.1-2), symbol l is the length from the last rivet (or bolt) line on a member to first rivet (or bolt) line on a chord measured along the centerline of the member; K is effective length factor $= 0.65$; A_s is average *effective* cross-section area defined by Whitmore's method.
4. Limit of free edge to thickness ratio of gusset plate — When the free edge length to thickness ratio of a gusset plate $L_g/t > 1.6\sqrt{E/F_y}$, the compression stress of a gusset plate shall be less than 0.8 F_y; otherwise the plate shall be stiffened. The free edge length to thickness ratio of a gusset plate shall satisfy the following limit specified in Article 6.14.2.8 of AASHTO-LRFD [1].

$$\frac{L_g}{t} \leq 2.06\sqrt{\frac{E}{F_y}} \tag{16.2}$$

When the free edge is stiffened, the following requirements shall be satisfied:

- The stiffener plus a width of $10t$ of gusset plate shall have an l/r ratio less than or equal to 40.

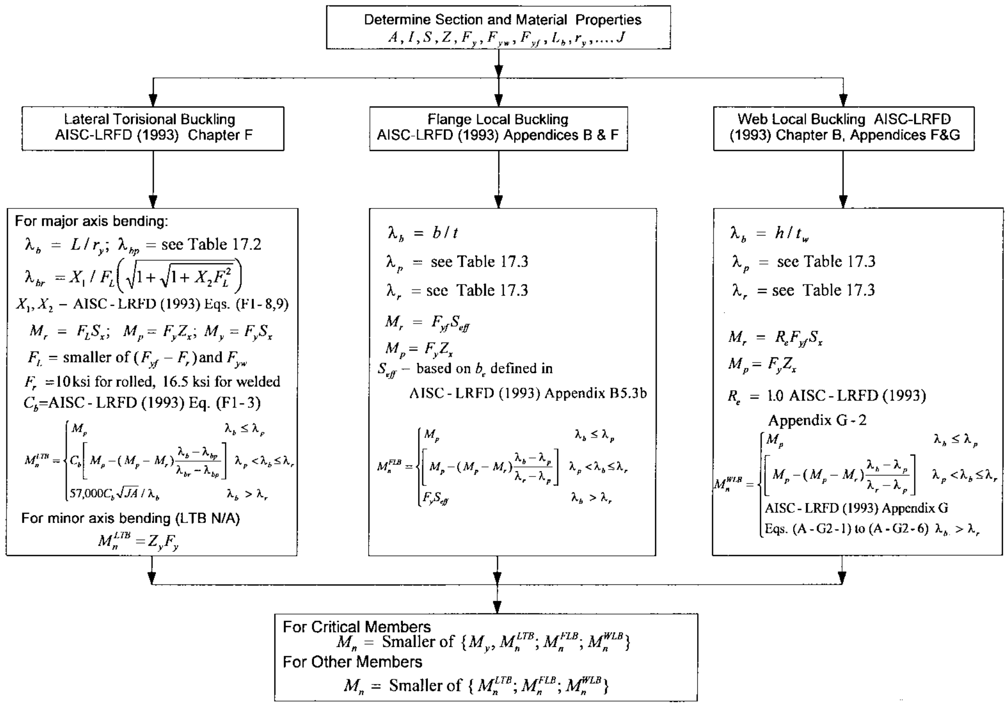

FIGURE 16.11: Evaluation procedure for nominal flexural strength of I-shaped steel members.

- The stiffener shall have an l/r ratio less than or equal to 40 between fasteners.
- The stiffener moment of inertia shall satisfy [38]:

$$I_s \geq \begin{cases} 1.83t^4\sqrt{(b/t)^2 - 144} \\ 9.2t^4 \end{cases} \tag{16.3}$$

where

I_s = the moment of inertia of the stiffener about its own centroid

b = the width of the gusset plate perpendicular to the edge

t = the thickness of the gusset plate

5. In-plane moment strength of gusset plate (strong axis) — The nominal moment strength of a gusset plate shall be calculated by the following equation in Article 6.14.2.8 of AASHTO-LRFD [1]:

$$M_n = SF_y \tag{16.4}$$

where
S = elastic section modulus about the strong axis

6. In-plane shear strength for a gusset plate — The nominal shear strength of a gusset plate shall be calculated by the following equations:

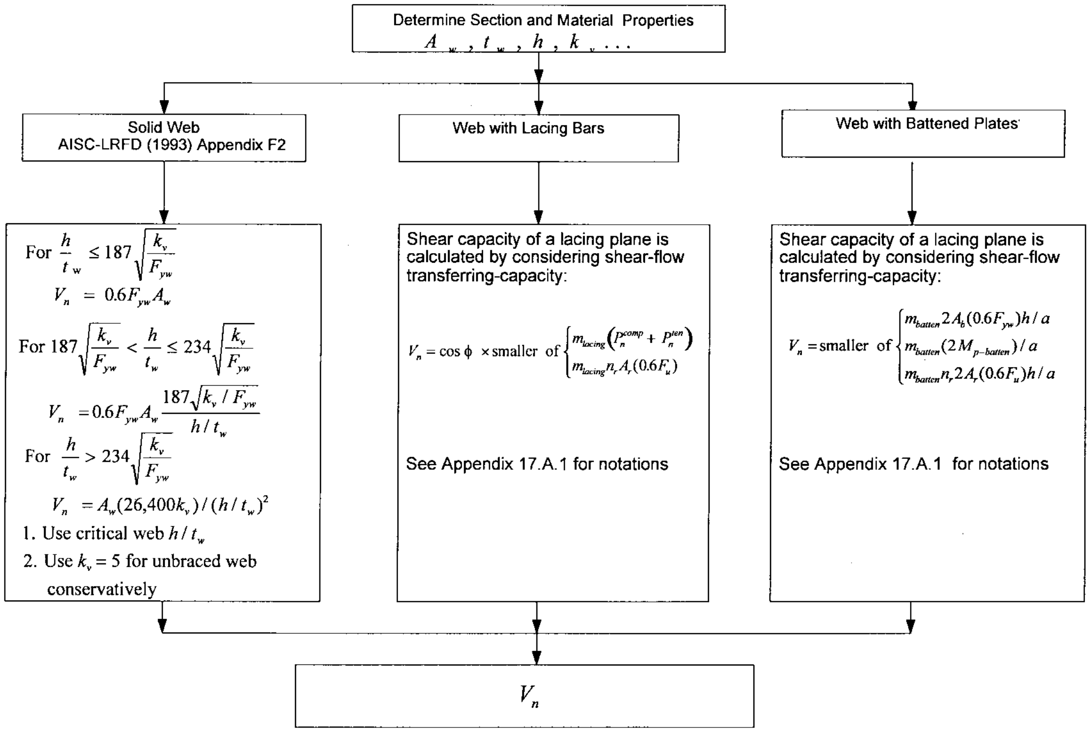

FIGURE 16.12: Evaluation procedure for nominal shear strength of steel members.

Based on gross section:

$$V_n = \text{smaller} \begin{cases} 0.4F_y A_{gv} & \text{for flexural shear} \\ 0.6F_y A_{gv} & \text{for uniform shear} \end{cases} \tag{16.5}$$

Based on net section:

$$V_n = \text{smaller} \begin{cases} 0.4F_u A_{nv} & \text{for flexural shear} \\ 0.6F_u A_{nv} & \text{for uniform shear} \end{cases} \tag{16.6}$$

where

A_{gv} = gross area subject to shear
A_{nv} = net area subject to shear
F_u = minimum tensile strength of the gusset plate

7. Initial yielding of gusset plate in combined in-plane moment, shear, and axial load — The initial yielding strength of a gusset plate subjected to a combined in-plane moment, shear, and axial load shall be determined by the following equations:

$$\frac{M_u}{M_n} + \frac{P_u}{P_y} \leq 1 \tag{16.7}$$

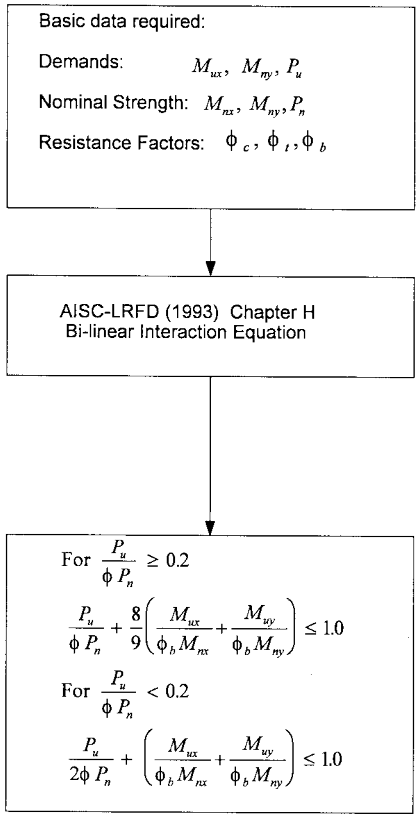

FIGURE 16.13: Evaluation procedure for steel beam-columns.

or

$$\left(\frac{V_u}{V_n}\right)^2 + \left(\frac{P_u}{P_y}\right)^2 \leq 1 \qquad (16.8)$$

where

V_u = factored shear
M_u = factored moment
P_u = factored axial load
M_n = nominal moment strength determined by Equation 16.4
V_n = nominal shear strength determined by Equation 16.5
P_y = yield axial strength $(A_g F_y)$
A_g = gross section area of gusset plate

8. Full yielding of gusset plate in combined in-plane moment, shear, and axial load — Full yielding strength for a gusset plate subjected to combined in-plane moment, shear, and axial load has the form [6]:

$$\frac{M_u}{M_p} + \left(\frac{P_u}{P_y}\right)^2 + \frac{\left(\frac{V_u}{V_p}\right)^4}{\left[1 - \left(\frac{P_u}{P_y}\right)^2\right]} = 1 \tag{16.9}$$

where
$M_p =$ plastic moment of pure bending (ZF_y)
$V_p =$ shear capacity of gusset plate $(0.6A_g F_y)$
$Z =$ plastic section modulus

9. Block shear capacity — The block shear capacity shall be calculated according to Article 6.13.4 of AASHTO-LRFD [1].
10. Out-of-plane moment and shear consideration — Moment will be resolved into a couple acting on the near and far side gusset plates. This will result in tension or compression on the respective plates. This force will produce weak axis bending of the gusset plate.

Connections Splices

The splice section shall be evaluated for axial tension, flexure, and combined axial and flexural loading cases according to AISC-LRFD [4]. The member splice capacity shall be equal to or greater than the capacity of the smaller of the two members being spliced.

Eyebars

The tensile capacity of the eyebars shall be calculated according to Article D3 of AISC-LRFD [4].

Anchor Bolts (Rods) and Anchorage Assemblies

1. Anchorage assemblies for nonrocking mechanisms shall be anchored with sufficient capacity to develop the lesser of the seismic force demand and plastic strength of the columns. Anchorage assemblies may be designed for rocking mechanisms where yield is permitted — at which point rocking commences. Shear keys shall be provided to prevent excess lateral movement. The nominal shear strength of pipe guided shear keys shall be calculated by:

$$R_n = 0.6F_y A_p \tag{16.10}$$

where
$A_p =$ cross-section area of pipe

2. Evaluation of anchorage assemblies shall be based on reinforced concrete structure behavior with bonded or unbonded anchor rods under combined axial load and bending moment. All anchor rods outside of the compressive region may be taken to full minimum tensile strength.
3. The nominal strength of anchor bolts (rods) for shear, tension, and combined shear and tension shall be calculated according to Article 6.13.2 of AASHTO-LRFD [1].
4. Embedment length of anchor rods shall be such that a ductile failure occurs. Concrete failure surfaces shall be based on a shear stress of $2\sqrt{f_c'}$ and account for edge distances and overlapping shear zones. In no case should edge distances or embedments be less than those shown in Table 8-26 of the AISC-LRFD Manual [3]. New anchor rods shall be threaded to assure development.

Rivets and Holes

1. The bearing capacity on rivet holes shall be calculated according to Article 6.13.2.9 of AASHTO-LRFD [1].
2. Nominal shear strength of a rivet shall be calculated by the following formula:

$$R_n = 0.75\beta F_u A_r N_s \tag{16.11}$$

where

β = 0.8, reduction factor for connections with more than two rivets and to account for deformation of connected material which causes nonuniform rivet shear force (see Article C6.13.2.7 of AASHTO-LRFD [1])

F_u = minimum tensile strength of the rivet

A_r = the nominal area of the rivet (before driving)

N_s = number of shear planes per rivet

It should be pointed out that the 0.75 factor is the ratio of the shear strength τ_u to the tensile strength F_u of a rivet. The research work by Kulak et al. [31] found that this ratio is independent of the rivet grade, installation procedure, diameter, and grip length and is about 0.75.

3. Tension capacity of a rivet shall be calculated by the following formula:

$$T_n = A_r F_u \tag{16.12}$$

4. Tensile capacity of a rivet subjected to combined tension and shear shall be calculated by the following formula:

$$T_n = A_r F_u \sqrt{1 - \frac{V_u}{R_n}} \tag{16.13}$$

where

V_u = factored shear force

R_n = nominal shear strength of a rivet determined by Equation 16.11

Bolts and Holes

1. The bearing capacity on bolt holes shall be calculated according to Article 6.13.2.9 of AASHTO-LRFD [1].
2. The nominal strength of a bolt for shear, tension, and combined shear and tension shall be calculated according to Article 6.13.2 of AASHTO-LRFD [1].

Prying Action

Additional tension forces resulting from prying action must be accounted for in determining applied loads on rivets or bolts. The connected elements (primarily angles) must also be checked for adequate flexural strength. Prying action forces shall be determined from the equations presented in AISC-LRFD Manual Volume 2, Part 11 [3].

16.7.4 Nominal Strength of Concrete Structures

Nominal Moment Strength

The nominal moment strength M_n shall be calculated by considering combined biaxial bending and axial loads. It is defined as:

$$M_n = \text{smaller} \begin{cases} M_y \\ M_{\varepsilon c} \end{cases} \tag{16.14}$$

where
M_y = moment corresponding to first steel yield
$M_{\varepsilon c}$ = moment at which compressive strain of concrete at extreme fiber equal to 0.003

Nominal Shear Strength

The nominal shear strength V_n shall be calculated by the following equations [12, 13].

$$V_n = V_c + V_s \tag{16.15a}$$

$$\text{or} \quad V_n = V_c + V_t \tag{16.15b}$$

$$V_c = 0.8 \nu_c A_g \tag{16.16}$$

$$\nu_c = \text{larger} \begin{cases} 2\left(1 + \frac{P_u}{2{,}000 A_g}\right)\sqrt{f_c'} \le 3\sqrt{f_c'} \\ \text{Factor 1} \times \left(1 + \frac{P_u}{2{,}000 A_g}\right)\sqrt{f_c'} \le 4\sqrt{f_c'} \end{cases} \tag{16.17}$$

$$\text{Factor 1} = \frac{\rho'' f_{yt}}{150} + 3.67 - \mu_\Delta \le 3.0$$

$$\rho'' = \frac{\text{volume of transverse reinforcement}}{\text{volume of confined core}} \tag{16.18}$$

$$V_s = \begin{cases} A_v f_{yt} d/s & \text{for rectangular sections} \\ \frac{A_s f_{yt} D'}{2s} & \text{for circular sections} \end{cases} \tag{16.19}$$

where
A_g = gross section area of concrete member
A_s = cross-sectional area of transverse reinforcement within space s
V_t = shear strength carried by truss mechanism
D' = hoop or spiral diameter
P_u = factored axial load associated with design shear V_u and P_u/A_g is in psi
d = effective depth of section
s = space of transverse reinforcement
f_{yt} = probable yield strength of transverse steel (psi)
μ_Δ = ductility demand ratio (1.0 will be used)

16.7.5 Structural Deformation Capacity

Steel Structures

Displacement capacity shall be evaluated by considering both material and geometrical non-linearity. Proper boundary conditions for various structures shall be carefully adjusted. The ultimate available displacement capacity is defined as the displacement corresponding to a load that drops a maximum of 20% from the peak load.

Reinforced Concrete Structures

Displacement capacity shall be evaluated using stand-alone push-over analysis models. Both the geometrical and material nonlinearities, as well as the foundation (nonlinear soil springs) shall be taken into account. The ultimate available displacement capacity is defined as the displacement corresponding to a maximum of 20% load reduction from the peak load, or to a specified stress-strain failure limit (surface), whichever occurs first.

The following parameters shall be used to define stress-strain failure limit (surface):

$$\begin{aligned} \sigma_c^{\text{comp}} &= 0.85 f_c' \\ \sigma_c^{\text{ten}} &= f_r = 5\sqrt{f_c'} \\ \varepsilon_c^{\text{comp}} &= 0.003 \\ \sigma_s &= F_u \\ \varepsilon_s &= 0.12 \end{aligned}$$

where

σ_c^{comp} = maximum concrete stress under uniaxial compression
σ_c^{ten} = maximum concrete stress under uniaxial tension
f_c' = specified compressive concrete strength
σ_s = maximum steel stress under uniaxial tension
ε_s = maximum steel strain under uniaxial tension
$\varepsilon_c^{\text{comp}}$ = maximum concrete strain under uniaxial compression

16.7.6 Seismic Response Modification Devices

General

The SRMDs include the energy dissipation and seismic isolation devices. The basic purpose of energy dissipation devices is to increase the effective damping of the structure by adding dampers to the structure thereby reducing forces, deflections, and impact effects. The basic purpose of isolation devices is to change the fundamental mode of vibration so that the structure is subjected to lower earthquake forces. However, the reduction in force may be accompanied by an increase in displacement demand that shall be accommodated within the isolation system and any adjacent structures.

Determination of SRMDs Properties

The properties of SRMDs shall be determined by the specified testing program. References are made to AASHTO-Guide [2], Caltrans [18], and JMC [30]. The following items shall be addressed rigorously in the testing specification:

- Scales of specimens; at least two full-scale tests are required
- Loading (including lateral and vertical) history and rate
- Durability — design life
- Expected levels of strength and stiffness deterioration

16.8 Performance Acceptance Criteria

16.8.1 General

To achieve the performance objectives stated in Section 16.3, the various structural components shall satisfy the acceptable demand/capacity ratios, DC_{accept}, specified in this section. The general design format is given by the formula:

$$\frac{\text{Demand}}{\text{Capacity}} \leq DC_{\text{accept}} \tag{16.20}$$

where demand, in terms of various factored forces (moment, shear, axial force, etc.), and deformations (displacement, rotation, etc.) shall be obtained by the nonlinear inelastic dynamic time history analysis – Level I defined in Section 16.6; and capacity, in terms of factored strength and deformations, shall be obtained according to the provisions set forth in Section 16.7. For members subjected to combined loadings, the definition of force D/C ratio:] D/C ratios is given in the Appendix.

16.8.2 Structural Component Classifications

Structural components are classified into two categories: critical and other. It is the aim that other components may be permitted to function as "fuses" so that the critical components of the bridge system can be protected during FEE and SEE. As an example, Table 16.1 shows structural component classifications and their definition for SFOBB West Span components.

TABLE 16.1 Structural Component Classification

Component classification	Definition	Example (SFOBB West Spans)
Critical	Components on a critical path that carry bridge gravity load directly. The loss of capacity of these components would have serious consequences on the structural integrity of the bridge	Suspension cables Continuous trusses Floor beams and stringers Tower legs Central anchorage A-Frame Piers W-1 and W2 Bents A and B Caisson foundations Anchorage housings Cable bents
Other	All components other than *critical*	All other components

Note: Structural components include members and connections.

16.8.3 Steel Structures

General Design Procedure

Seismic design of steel members shall be in accordance with the procedure shown in Figure 16.14. Seismic retrofit design of steel members shall be in accordance with the procedure shown in Figure 16.15.

Connections

Connections shall be evaluated over the length of the seismic event. For connecting members with force D/C ratios larger than one, 25% greater than the nominal capacities of the connecting members shall be used for connection design.

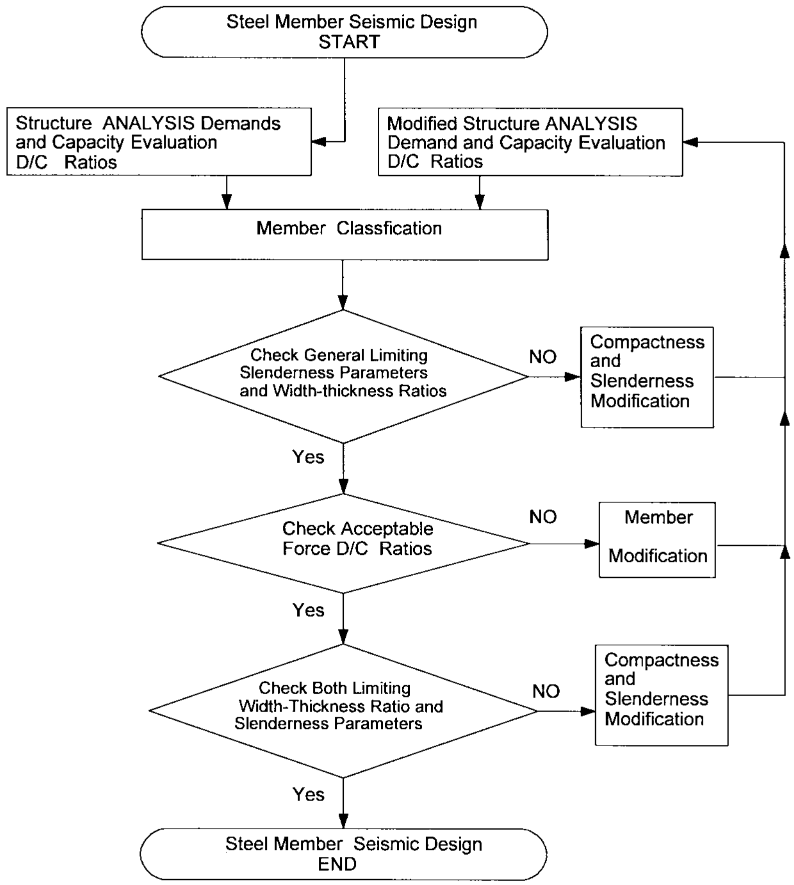

FIGURE 16.14: Steel member seismic design procedure.

General Limiting Slenderness Parameters and Width-Thickness Ratios

For all steel members regardless of their force D/C ratios, slenderness parameters λ_c for axial load dominant members, and λ_b for flexural dominant members shall not exceed the limiting values ($0.9\lambda_{cr}$ or $0.9\lambda_{br}$ for *critical*, λ_{cr} or λ_{br} for *other*) shown in Table 16.2.

Acceptable Force D/C Ratios and Limiting Values

Acceptable force D/C ratios, DC_{accept} and associated limiting slenderness parameters and width-thickness ratios for various members are specified in Table 16.2.

For all members with D/C ratios larger than one, slenderness parameters and width- thickness ratios shall not exceed the limiting values specified in Table 16.2. For existing steel members with D/C ratios less than one, width-thickness ratios may exceed λ_r specified in Table 16.3 and AISC-LRFD [4].

The following symbols are used in Table 16.2:

M_n = nominal moment strength of a member determined by Section 16.7
P_n = nominal axial strength of a member determined by Section 16.7
λ = width-thickness (b/t or h/t_w) ratio of compressive elements
λ_c = $(KL/r\pi)\sqrt{F_y/E}$, slenderness parameter of axial load dominant members
λ_b = L/r_y, slenderness parameter of flexural moment dominant members

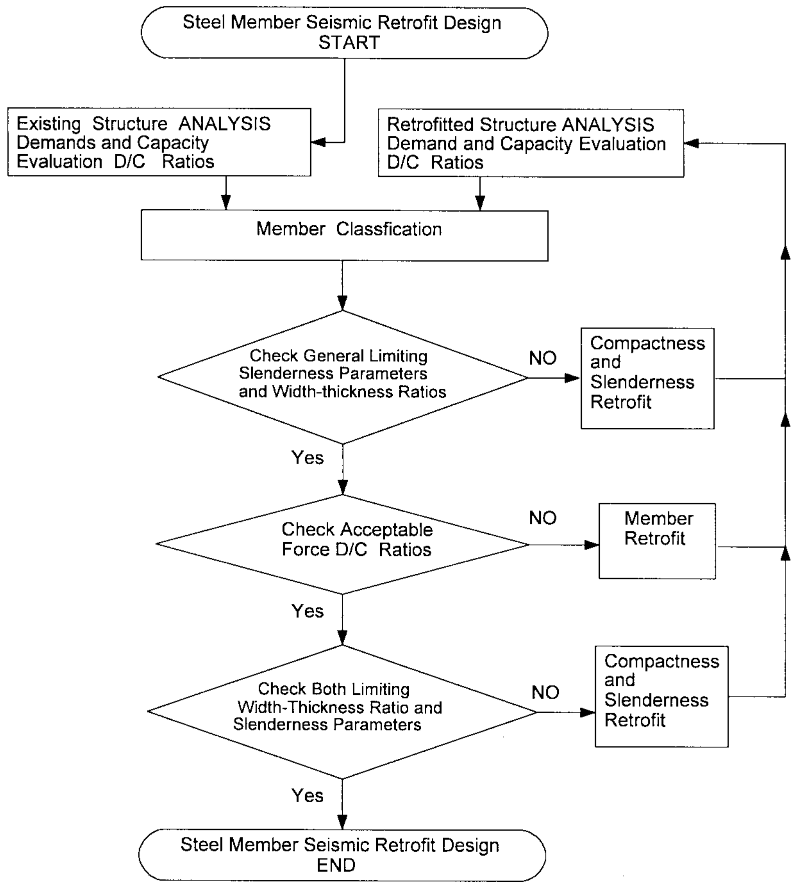

FIGURE 16.15: Steel member seismic retrofit design procedure.

TABLE 16.2 Acceptable Force Demand/Capacity Ratios and Limiting Slenderness Parameters and Width/Thickness Ratios

Member classification		Limiting ratios: Slenderness parameter (λ_c and λ_b)	Limiting ratios: Width/thickness λ (b/t or h/t_w)	Acceptable force D/C ratio D/C_{accept}
Critical	Axial load	$0.9\ \lambda_{cr}$	λ_r	$DC_r = 1.0$
	dominant	λ_{cpr}	λ_{pr}	$1.0 \sim 1.2$
	$P_u/P_n \geq M_u/M_n$	λ_{cp}	λ_p	$DC_p = 1.2$
	Flexural moment	$0.9\ \lambda_{br}$	λ_r	$DC_r = 1.0$
	dominant	λ_{bpr}	λ_{pr}	$1.2 \sim 1.5$
	$M_u/M_n > P_u/P_n$	λ_{bp}	λ_p	$DC_p = 1.5$
Other	Axial load	λ_{cr}	λ_r	$DC_r = 1.0$
	dominant	λ_{cpr}	λ_{pr}	$1.0 \sim 2.0$
	$P_u/P_n \geq M_u/M_n$	λ_{cp}	$\lambda_{p-\text{Seismic}}$	$DC_p = 2$
	Flexural moment	λ_{br}	λ_r	$DC_r = 1.0$
	dominant	λ_{bpr}	λ_{pr}	$1.0 \sim 2.5$
	$M_u/M_n > P_u/P_n$	λ_{bp}	$\lambda_{p-\text{Seismic}}$	$DC_p = 2.5$

TABLE 16.3 Limiting Width-Thickness Ratio

No	Description of elements	Examples	Width-thickness ratios	λ_r	λ_p	$\lambda_{p\text{-}Seismic}$
	UNSTIFFENED ELEMENTS					
1	Flanges of I-shaped rolled beams and channels in flexure	Figures 16.16a, c	b/t	$\mathbf{\frac{141}{\sqrt{F_y - 10}}}$	$\mathbf{\frac{65}{\sqrt{F_y}}}$	$\mathbf{\frac{52}{\sqrt{F_y}}}$
2	Outstanding legs of pairs of angles in continuous contact; flanges of channels in axial compression; angles and plates projecting from beams or compression members	Figures 16.16d, e, f	b/t	$\mathbf{\frac{95}{\sqrt{F_y}}}$	$\frac{65}{\sqrt{F_y}}$	$\frac{52}{\sqrt{F_y}}$
	STIFFENED ELEMENTS					
3	Flanges of square and rectangular box and hollow structural section of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds.	Figure 16.16b	b/t	$\mathbf{\frac{238}{\sqrt{F_y}}}$	$\mathbf{\frac{190}{\sqrt{F_y}}}$	$\mathbf{110/\sqrt{F_y}}$ (tubes) $\mathbf{150/\sqrt{F_y}}$ (others)
4	Unsupported width of cover plates perforated with a succession of access holes	Figure 16.16d	b/t	$\mathbf{\frac{317}{\sqrt{F_y}}}$	$\mathbf{\frac{253}{\sqrt{F_y}}}$	$\mathbf{\frac{152}{\sqrt{F_y}}}$
5	All other uniformly compressed stiffened elements, i.e., supported along two edges.	Figures 16.16a, c, d, f	b/t h/t_w	$\mathbf{\frac{253}{\sqrt{F_y}}}$	$\frac{190}{\sqrt{F_y}}$	$110/\sqrt{F_y}$ (w/lacing) $150/\sqrt{F_y}$ (others)
6	Webs in flexural compression	Figures 16.16a, c, d, f	h/t_w	$\mathbf{\frac{970}{\sqrt{F_y}}}$	$\mathbf{\frac{640}{\sqrt{F_y}}}$	$\mathbf{\frac{520}{\sqrt{F_y}}}$
7	Webs in combined flexural and axial compression	Figures 16.16a, c, d, f	h/t_w	$\mathbf{\frac{970}{\sqrt{F_y}} \times \left(1 - \frac{0.74P}{\phi_b P_y}\right)}$	For $\mathbf{P_u \le 0.125\phi_b P_y}$ $\mathbf{\frac{640}{\sqrt{F_y}}\left(1 - \frac{2.75P}{\phi_b P_y}\right)}$ For $\mathbf{P_u > 0.125\phi_b P_y}$ $\mathbf{\frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P}{\phi_b P_y}\right) \ge \frac{253}{\sqrt{F_y}}}$	For $\mathbf{P_u \le 0.125\phi_b P_y}$ $\mathbf{\frac{520}{\sqrt{F_y}}\left(1 - \frac{1.54P}{\phi_b P_y}\right)}$ For $\mathbf{P_u > 0.125\phi_b P_y}$ $\mathbf{\frac{191}{\sqrt{F_y}}\left(2.33 - \frac{P}{\phi_b P_y}\right) \ge \frac{253}{\sqrt{F_y}}}$
8	Longitudinally stiffened plates in compression	Figure 16.16e	b/t	$\frac{113\sqrt{k}}{\sqrt{F_y}}$	$\frac{95\sqrt{k}}{\sqrt{F_y}}$	$\frac{75\sqrt{k}}{\sqrt{F_y}}$

Notes:

1. Width-thickness ratios shown in **bold** are from AISC-LRFD [4] and AISC-Seismic Provisions [5].
2. k = buckling coefficient specified by Article 6.11.2.1.3a of AASHTO-LRFD [1]

   for $n = 1$, $k = (8I_s / bt^3)^{1/3} \le 4.0$ for $n = 2,3,4$ and 5, $k = (14.3I_s / bt^3n^4)^{1/3} \le 4.0$

   n = number of equally spaced longitudinal compression flange stiffeners

   I_s = moment of inertia of a longitudinal stiffener about an axis parallel to the bottom flange and taken at the base of the stiffener

λ_{cp} = 0.5, limiting column slenderness parameter for 90% of the axial yield load based on AISC-LRFD [4] column curve
λ_{bp} = limiting beam slenderness parameter for plastic moment for seismic design
λ_{cr} = 1.5, limiting column slenderness parameter for elastic buckling based on AISC-LRFD [4] column curve
λ_{br} = limiting beam slenderness parameter for elastic lateral torsional buckling

$$\lambda_{br} = \begin{cases} \frac{57{,}000\sqrt{JA}}{M_r} & \text{for solid rectangular bars and box sections} \\ \frac{X_1}{F_L}\sqrt{1+\sqrt{1+X_2F_L^2}} & \text{for doubly symmetric I-shaped members and channels} \end{cases}$$

$$M_r = \begin{cases} F_L S_x & \text{for I-shaped member} \\ F_{yf} S_x & \text{for solid rectangular and box section} \end{cases}$$

$$X_1 = \frac{\pi}{S_x}\sqrt{\frac{EGJA}{2}}; \qquad X_2 = \frac{4C_w}{I_y}\left(\frac{S_x}{GJ}\right)^2; \qquad F_L = \text{ smaller } \begin{cases} F_{yw} \\ F_{yf} - F_r \end{cases}$$

where
A = cross-sectional area, in.^{2}
L = unsupported length of a member
J = torsional constant, in.^{4}
r = radius of gyration, in.
r_y = radius of gyration about minor axis, in.
F_{yw} = yield stress of web, ksi
F_{yf} = yield stress of flange, ksi
E = modulus of elasticity of steel (29,000 ksi)
G = shear modulus of elasticity of steel (11,200 ksi)
S_x = section modulus about major axis, in.^{3}
I_y = moment of inertia about minor axis, in.^{4}
C_w = warping constant, in.^{6}

For doubly symmetric and singly symmetric I-shaped members with compression flange equal to or larger than the tension flange, including hybrid members (strong axis bending):

$$\lambda_{bp} = \begin{cases} \frac{[3{,}600+2{,}200(M_1/M_2)]}{F_y} & \text{for } \textit{other} \text{ members} \\ \frac{300}{\sqrt{F_{yf}}} & \text{for } \textit{critical} \text{ members} \end{cases} \tag{16.21}$$

in which
M_1 = larger moment at end of unbraced length of beam
M_2 = smaller moment at end of unbraced length of beam
(M_1/M_2) = positive when moments cause reverse curvature and negative for single curvature

For solid rectangular bars and symmetric box beam (strong axis bending):

$$\lambda_{bp} = \begin{cases} \frac{[5{,}000+3{,}000(M_1/M_2)]}{F_y} \geq \frac{3{,}000}{F_y} & \text{for } \textit{other} \text{ members} \\ \frac{3{,}750}{M_p}\sqrt{JA} & \text{for } \textit{critical} \text{ members} \end{cases} \tag{16.22}$$

in which
M_p = plastic moment $(Z_x F_y)$
Z_x = plastic section modulus about major axis

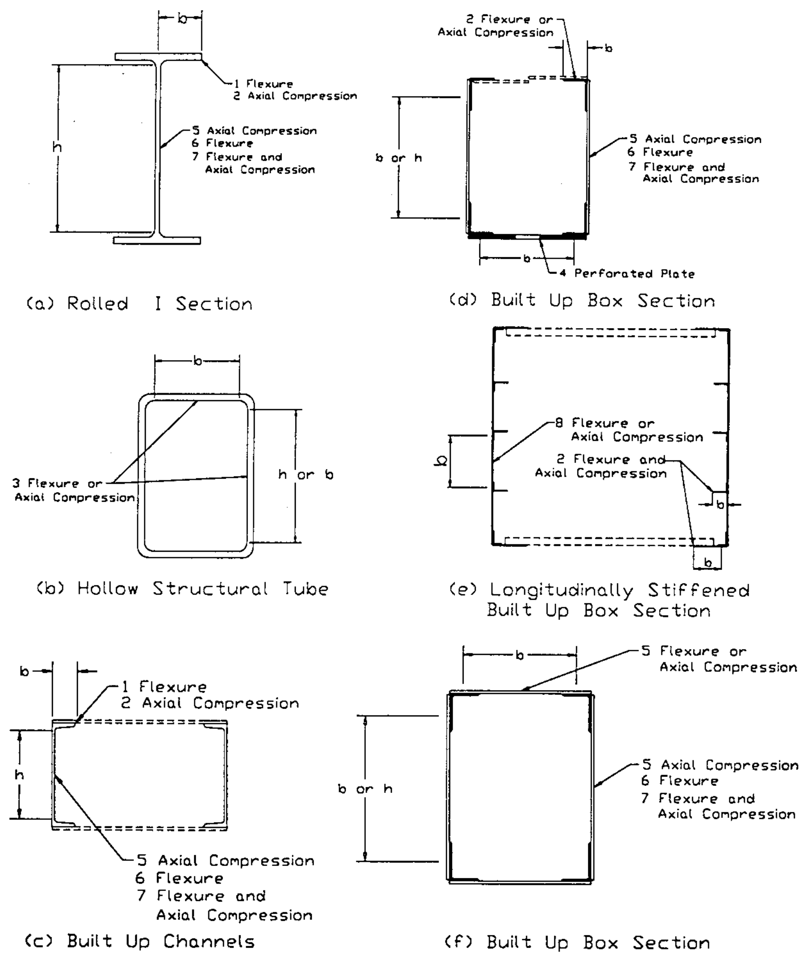

FIGURE 16.16: Typical cross-sections for steel members (SFOBB west spans).

$\lambda_r, \lambda_p, \lambda_{p-\text{Seismic}}$ are limiting width thickness ratios specified by Table 16.3

$$\lambda_{pr} = \begin{cases} \left[\lambda_p + \left(\lambda_r - \lambda_p\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right)\right] & \text{for } \textit{critical} \text{ members} \\ \left[\lambda_{p-\text{Seismic}} + \left(\lambda_r - \lambda_{p-\text{Seismic}}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right)\right] & \text{for } \textit{other} \text{ members} \end{cases} \tag{16.23}$$

For axial load dominant members ($P_u/P_n \geq M_u/M_n$)

$$\lambda_{cpr} = \begin{cases} \lambda_{cp} + \left(0.9\lambda_{cr} - \lambda_{cp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{critical} \text{ members} \\ \lambda_{cp} + \left(\lambda_{cr} - \lambda_{cp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{other} \text{ members} \end{cases} \tag{16.24}$$

For flexural moment dominant members ($M_u/M_n > P_u/P_n$)

$$\lambda_{bpr} = \begin{cases} \lambda_{bp} + \left(0.9\lambda_{br} - \lambda_{bp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{critical} \text{ members} \\ \lambda_{bp} + \left(\lambda_{br} - \lambda_{bp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{other} \text{ members} \end{cases} \tag{16.25}$$

16.8.4 Concrete Structures

General

For all concrete compression members regardless of their force D/C ratios, slenderness parameters KL/r shall not exceed 60.

For *critical* components, force $DC_{\text{accept}} = 1.2$ and deformation $DC_{\text{accept}} = 0.4$.

For *other* components, force $DC_{\text{accept}} = 2.0$ and deformation $DC_{\text{accept}} = 0.67$.

Beam-Column (Bent Cap) Joints

For concrete box girder bridges, the beam-column (bent cap) joints shall be evaluated and designed in accordance with the following guidelines [16, 40]:

1. Effective Superstructure Width — The effective width of a superstructure (box girder) on either side of a column to resist longitudinal seismic moment at bent (support) shall not be taken as larger than the superstructure depth.
   - The immediately adjacent girder on either side of a column within the effective superstructure width is considered effective.
   - Additional girders may be considered effective if refined bent-cap torsional analysis indicates that the additional girders can be mobilized.
2. Minimum Bent-Cap Width — Minimum cap width outside the column shall not be less than $D/4$ (D is column diameter or width in that direction) or 2 ft (0.61 m).
3. Acceptable Joint Shear Stress
   - For existing unconfined joints, acceptable principal tensile stress shall be taken as $3.5\sqrt{f_c'}$ psi ($0.29\sqrt{f_c'}$ MPa). If the principal tensile stress demand exceeds this value, the joint shear reinforcement specified in (4) shall be provided.
   - For new joints, acceptable principal tensile stress shall be taken as $12\sqrt{f_c'}$ psi ($1.0\sqrt{f_c'}$ MPa).
   - For existing and new joints, acceptable principal compressive stress shall be taken as $0.25\,f_c'$.
4. Joint Shear Reinforcement
   - Typical flexure and shear reinforcement (see Figures 16.17 and 16.18) in bent caps shall be supplemented in the vicinity of columns to resist joint shear. All joint shear reinforcement shall be well distributed and provided within D/2 from the face of column.
   - Vertical reinforcement including cap stirrups and added bars shall be 20% of the column reinforcement anchored into the joint. Added bars shall be hooked around main longitudinal cap bars. Transverse reinforcement in the joint region shall consist of hoops with a minimum reinforcement ratio of 0.4 (column steel area)/(embedment length of column bar into the bent cap)^2.
   - Horizontal reinforcement shall be stitched across the cap in two or more intermediate layers. The reinforcement shall be shaped as hairpins, spaced vertically at not more than 18 in. (457 mm). The hairpins shall be 10% of column reinforcement. Spacing shall be denser outside the column than that used within the column.
   - Horizontal side face reinforcement shall be 10% of the main cap reinforcement including top and bottom steel.

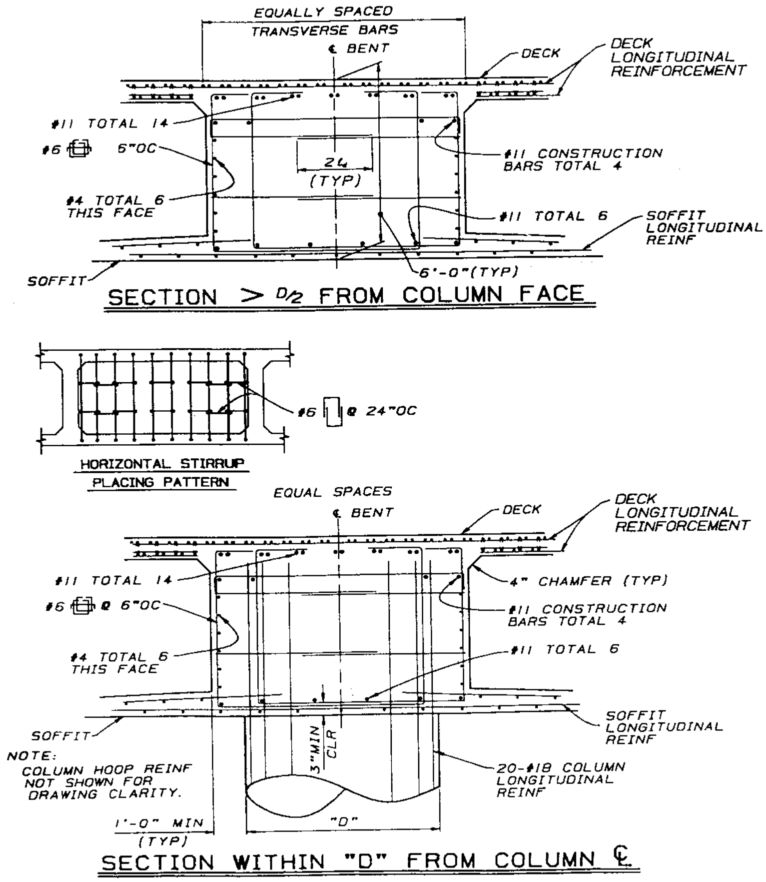

FIGURE 16.17: Example cap joint shear reinforcement — skews 0° to 20°.

- For bent caps skewed greater than 20°, the vertical J-bars hooked around longitudinal deck and bent cap steel shall be 8% of the column steel (see Figure 16.18). The J-bars shall be alternatively 24 in. (600 mm) and 30 in. (750 mm) long and placed within a width of the column dimension on either side of the column centerline.
- All vertical column bars shall be extended as high as practically possible without interfering with the main cap bars.

16.8.5 Seismic Response Modification Devices

General

Analysis methods specified in Section 16.6 shall apply for determining seismic design forces and displacements on SRMDs. Properties or capacities of SRMDs shall be determined by specified tests.

Acceptance Criteria

SRMDs shall be able to perform their intended function and maintain their design parameters for the design life (for example, 40 years) and for an ambient temperature range (for example from 30°

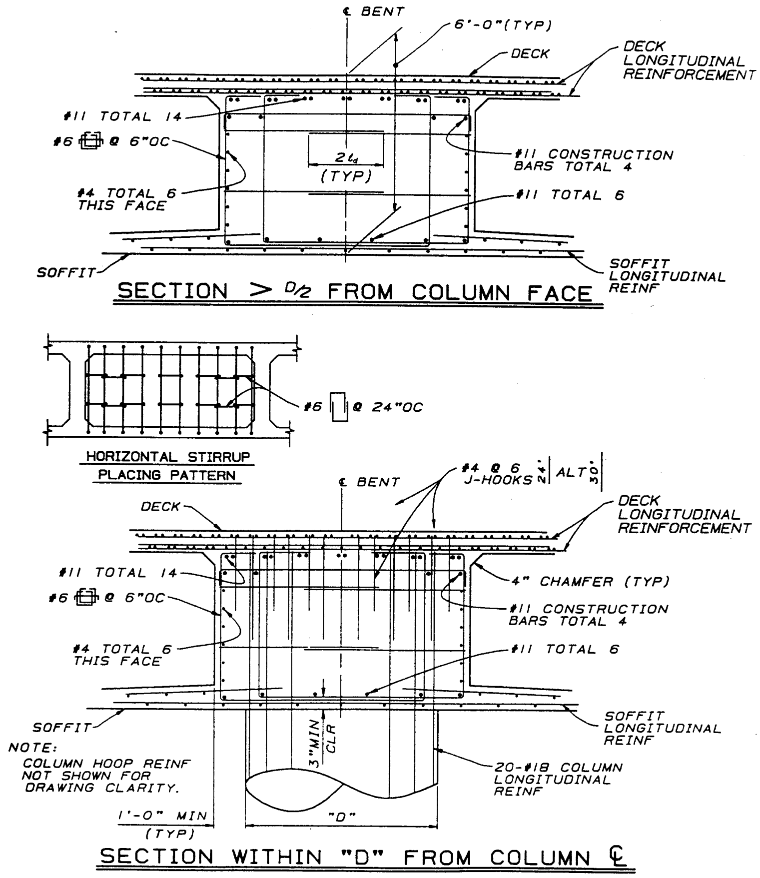

FIGURE 16.18: Example cap joint shear reinforcement — skews $> 20°$.

to 125°F). The devices shall have accessibility for periodic inspections, maintenance, and exchange. In general, the SRMDs shall satisfy at least the following requirements:

- To remain stable and provide increasing resistance with the increasing displacement. Stiffness degradation under repeated cyclic load is unacceptable.
- To dissipate energy within the design displacement limits. For example: provisions may be made to limit the maximum total displacement imposed on the device to prevent device displacement failure, or the device shall have a displacement capacity 50% greater than the design displacement.
- To withstand or dissipate the heat build-up during reasonable seismic displacement time history.
- To survive for the number of cycles of displacement expected under wind excitation during the life of the device and to function at maximum wind force and displacement levels for at least, for example, five hours.

Defining Terms

Bridge: A structure that crosses over a river, bay, or other obstruction, permitting the smooth and safe passage of vehicles, trains, and pedestrians.

Buckling model interaction: A behavior phenomenon of compression built-up member; that is, interaction between the individual (or local) buckling mode and the global buckling mode.

Built-up member: A member made of structural metal elements that are welded, bolted, and/or riveted together.

Capacity: factored strength and deformation capacity obtained according to specified provisions.

Critical components: Structural components on a critical path that carry bridge gravity load directly. The loss of capacity of these components would have serious consequences on the structural integrity of the bridge.

Damage index: A ratio of elastic displacement demand to ultimate displacement.

D/C ratio: A ratio of demand to capacity.

Demands: In terms of various forces (moment, shear, axial force, etc.) and deformation (displacement, rotation, etc.) obtained by structural analysis.

Ductility: A nondimensional factor, i.e., ratio of ultimate deformation to yield deformation.

Effective length factor K: A factor that when multiplied by actual length of the end-restrained column gives the length of an equivalent pin-ended column whose elastic buckling load is the same as that of the end-restrained column

Functionality evaluation earthquake (FEE): An earthquake that has a mean return period of 300 years with a 40% probability of exceedance during the expected life of the bridge.

Latticed member: A member made of metal elements that are connected by lacing bars and batten plates.

LRFD (Load and Resistance Factor Design): A method of proportioning structural components (members, connectors, connecting elements, and assemblages) such that no applicable limit state is exceeded when the structure is subjected to all appropriate load combinations.

Limit states: Those conditions of a structure at which it ceases to satisfy the provision for which it was designed.

No-collapse-based design: Design that is based on survival limit state. The overall design concern is to prevent the bridge from catastrophic collapse and to save lives.

Other components: All components other than *critical.*

Performance-based seismic design: Design that is based on bridge performance requirements. The design philosophy is to accept some repairable earthquake damage and to keep bridge functional performance after earthquakes.

Safety evaluation earthquake (SEE): An earthquake that has a mean return period in the range of 1000 to 2000 years.

Seismic design: Design and analysis considering earthquake loads.

Seismic response modification devices (SRMDs): Seismic isolation and energy dissipation devices including isolators, dampers, or isolation/dissipation (I/D) devices.

Ultimate deformation: Deformation refers to a loading state at which structural system or a structural member can undergo change without losing significant load-carrying capacity.

The ultimate deformation is usually defined as the deformation corresponding to a load that drops a maximum of 20% from the peak load.

Yield deformation: Deformation corresponds to the points beyond which the structure starts to respond inelastically.

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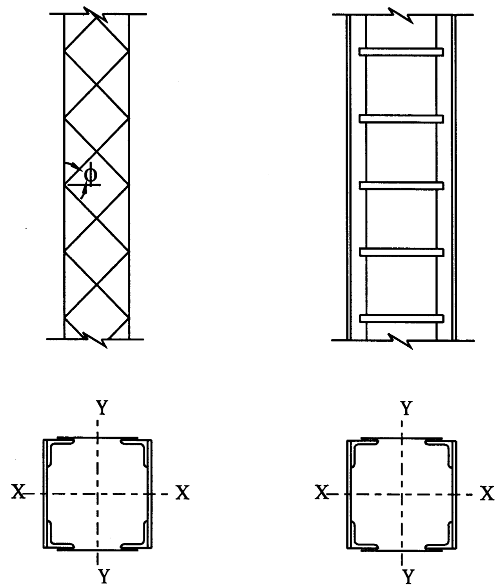
Appendix A

16.A.1 Section Properties for Latticed Members

This section presents practical formulas proposed by Duan and Reno [21] for calculating section properties for latticed members.

Concept

It is generally assumed that section properties can be computed based on cross-sections of main components if the lacing bars and battens can assure integral action of the solid main components [33, 36]. To consider actual section integrity, reduction factors β_m for moment of inertia, and β_t for torsional constant are proposed depending on shear-flow transferring-capacity of lacing bars and connections. For clarity and simplicity, typical latticed members as shown in Figure 16.19 are discussed.



(a) laced member (b) battened member

FIGURE 16.19: Typical latticed members.

Section Properties

1. Cross-sectional area — The contribution of lacing bars is assumed negligible. The cross-sectional area of latticed member is only based on main components.

$$A = \sum A_i \tag{16.26}$$

where A_i is cross-sectional area of individual component i.

2. Moment of inertia — For lacing bars or battens within web plane (bending about y-y axis in Figure 16.19)

$$I_{y-y} = \sum I_{(y-y)i} + \beta_m \sum A_i x_i^2 \tag{16.27}$$

where

I_{y-y} = moment of inertia of a section about y-y axis considering shear transferring capacity

I_i = moment of inertia of individual component i

x_i = distance between y-y axis and center of individual component i

β_m = reduction factor for moment of inertia and may be determined by the following formula:

For laced member (Figure 16.19a):

$$\beta_m = \frac{m \sin\phi \times \text{ smaller of } \begin{cases} m_{\text{lacing}}\left(P_n^{\text{comp}} + P_n^{\text{ten}}\right) \\ m_{\text{lacing}} n_r A_r \, (0.6 F_u) \end{cases}}{F_{yf} A_f} \leq 1.0 \tag{16.28a}$$

For battened member (Figure 16.19b)

$$\beta_m = \frac{m \times \text{ smaller of } \begin{cases} m_{\text{batten}} A_b \left(0.6 F_{yw}\right) \\ m_{\text{batten}} \left(2 M_{p-\text{batten}}\right) / h \\ m_{\text{batten}} n_r A_r \, (0.6 F_u) \end{cases}}{F_{yf} A_f} \leq 1.0 \tag{16.28b}$$

in which

ϕ = the angle between the diagonal lacing bar and the axis perpendicular to the member axis (see Figure 16.19)
A_b = cross-sectional area of batten plate
A_f = flange area
F_{yf} = yield strength of flange
F_{yw} = yield strength of web member (battens or lacing bars)
F_u = ultimate strength of rivets
m = number of panels between point of maximum moment to point of zero moment to either side (as an approximation, half of member length $L/2$ may be used)
m_{batten} = number of batten planes
m_{lacing} = number of lacing planes
n_r = number of rivets of connecting lacing bar and main component at one joint
h = depth of member in lacing plane
A_r = nominal area of rivet
$M_{p-\text{batten}}$ = plastic moment of a batten plate about strong axis
P_n^{comp} = nominal compressive strength of lacing bar and can be determined by AISC-LRFD [4] column curve
P_n^{ten} = nominal tensile strength of lacing bar and can be determined by AISC-LRFD [4]

Since the section integrity mainly depends on the shear transference between various components, it is rational to introduce the β_m factor in Equation 16.27. As seen in Equations 16.28a and 16.28b, β_m is defined as the ratio of the shear capacity transferred by lacing bars/battens and connections to the shear-flow ($F_{yf} A_f$) required by the plastic bending moment of a fully integral section. For laced members, the shear transferring capacity is controlled by either lacing bars or connecting rivets, the smaller of the two values should be used in Equation 16.28a. For battened members, the shear transferring capacity is controlled by either pure shear strength of battens (0.6 $F_{yw} A_b$), or flexural strength of battens or connecting rivets, the smaller of the three values should be used in Equation 16.28b. It is important to point out that the limiting value unity for β_m implies a fully integral section when shear can be transferred fully by lacings and connections.

For lacing bars or battens within flange plane (bending about x-x axis in Figure 16.19).

The contribution of lacing bars is assumed negligible and only the main components are considered.

$$I_{x-x} = \sum I_{(x-x)i} + \sum A_i y_i^2 \tag{16.29}$$

3. Elastic section modulus

$$S = \frac{I}{C} \tag{16.30}$$

where
S = elastic section modulus of a section
C = distance from elastic neutral axis to extreme fiber

4. Plastic section modulus
For lacing bars or battens within flange plane (bending about x-x axis in Figure 16.19)

$$Z_{x-x} = \sum y_i^* A_i^* \tag{16.31}$$

For lacing bars or battens within web plane (bending about y-y axis in Figure 16.19)

$$Z_{y-y} = \beta_m \sum x_i^* A_i^* \tag{16.32}$$

where
Z = plastic section modulus of a section about plastic neutral axis
x_i^* = distance between center of gravity of a section A_i^* and plastic neutral y-y axis
y_i^* = distance between center of gravity of a section A_i^* and plastic neutral x-x axis
A_i^* = cross-section area above or below plastic neutral axis

It should be pointed out that the plastic neutral axis is generally different from the elastic neutral axis. The plastic neutral axis is defined by equal plastic compression and tension forces for this section.

5. Torsional constant
For a box-shaped section

$$J = \frac{4\left(A_{\text{close}}\right)^2}{\sum \frac{b_i}{t_i}} \tag{16.33}$$

For an open thin-walled section

$$J = \sum \frac{b_i t_i^3}{3} \tag{16.34}$$

where
A_{close} = area enclosed within mean dimension for a box
b_i = length of a particular segment of the section
t_i = average thickness of segment b_i
For determination of torsional constant of a latticed member, it is proposed that the lacing bars or batten plates be replaced by reduced equivalent thin-walled plates defined as:

$$A_{\text{equiv}} = \beta_t A^*_{\text{equiv}} \tag{16.35}$$

For laced member (Figure 16.19a)

$$A^*_{\text{equiv}} = 3.12 A_d \sin\phi \cos^2\phi \tag{16.36a}$$

For battened member (Figure 16.19b)

$$A^*_{\text{equiv}} = \frac{74.88}{\frac{2ah}{I_b} + \frac{a^2}{I_f}} \tag{16.36b}$$

$$t_{\text{equiv}} = \frac{A_{\text{equiv}}}{h} \tag{16.37}$$

where

a = distance between two battens along member axis

A_{equiv} = cross-section area of a thin-walled plate equivalent to lacing bars considering shear transferring capacity

A^*_{equiv} = cross-section area of a thin-walled plate equivalent to lacing bars or battens assuming full section integrity

t_{equiv} = thickness of equivalent thin-walled plate

A_d = cross-sectional area of all diagonal lacings in one panel

I_b = moment of inertia of a batten plate

I_f = moment of inertia of a side solid flange about the weak axis

β_t = reduction factor for torsion constant may be determined by the following formula:

For laced member (Figure 16.19a)

$$\beta_t = \frac{\cos\phi \times \text{ smaller of } \begin{cases} P_n^{\text{comp}} + P_n^{\text{ten}} \\ n_r A_r \,(0.6 F_u) \end{cases}}{0.6 F_{yw} A^*_{\text{equiv}}} \leq 1.0 \tag{16.38a}$$

For battened member (Figure 16.19b)

$$\beta_t = \frac{\text{smaller of } \begin{cases} A_b \left(0.6 F_{yw}\right) h/a \\ 2M_{p-\text{batten}}/a \\ n_r A_r \,(0.6 F_u)\, h/a \end{cases}}{0.6 F_{yw} A^*_{\text{equiv}}} \leq 1.0 \tag{16.38b}$$

The torsional integrity is from lacings and battens. A reduction factor β_t, similar to that used for the moment of inertia, is introduced to consider section integrity when the lacing is weaker than the solid plate side of the section. β_t factor is defined as the ratio of the shear capacity transferred by lacing bars and connections to the shear-flow $(0.6F_{yw} A^*_{\text{equiv}})$ required by the equivalent thin-walled plate. It is seen that the limiting value of unity for β_t implies a fully integral section when shear in the equivalent thin-walled plate can be transferred fully by lacings and connections.

Based on the equal lateral stiffness principle, an equivalent thin-walled plate for a lacing plane, Equation 16.36a and 16.36b can be obtained by considering $E/G = 2.6$ for steel material and shape factor for shear $n = 1.2$ for a rectangular section.

6. Warping constant
   For a box-shaped section

$$C_w \approx 0 \tag{16.39}$$

For an I-shaped section

$$C_w = \frac{I_f h^2}{2} \tag{16.40}$$

where
I_f = moment of inertia of one solid flange about the weak axis (perpendicular to the flange) of the cross-section
h = distance between center of gravity of two flanges

16.A.2 Buckling Mode Interaction For Compression Built-up members

An important phenomenon of the behavior of these compressive built-up members is the interaction of buckling modes [9]; i.e., interaction between the individual (or local) component buckling (Figure 16.20a) and the global member buckling (Figure 16.20b). This section presents the practical approach proposed by Duan, Reno, and Uang [22] that may be used to determine the effects of interaction of buckling modes for capacity assessment of existing built-up members.

Buckling Mode Interaction Factor

To consider buckling mode interaction between the individual components and the global member, it is proposed that the usual effective length factor K of a built-up member be multiplied by a *buckling mode interaction factor*, γ_{LG}, that is,

$$K_{LG} = \gamma_{LG} K \tag{16.41}$$

where
K = usual effective length factor of a built-up member
K_{LG} = effective length factor considering buckling model interaction

Limiting Effective Slenderness Ratios

For practical design, the following two limiting effective slenderness ratios are suggested for consideration of buckling mode interaction:

$$K_a a/r_f = 1.1\sqrt{E/F} \tag{16.42}$$
$$K_a a/r_f = 0.75(KL/r) \tag{16.43}$$

where $(K_a a/r_f)$ is the largest effective slenderness ratio of individual components between connectors and (KL/r) is governing effective slenderness ratio of a built-up member.

The first limit Equation 16.42 is based on the argument that if an individual component is very short, the failure mode of the component would be material yielding (say 95% of the yield load), not member buckling. This implies that no interaction of buckling modes occurs when an individual component is very short.

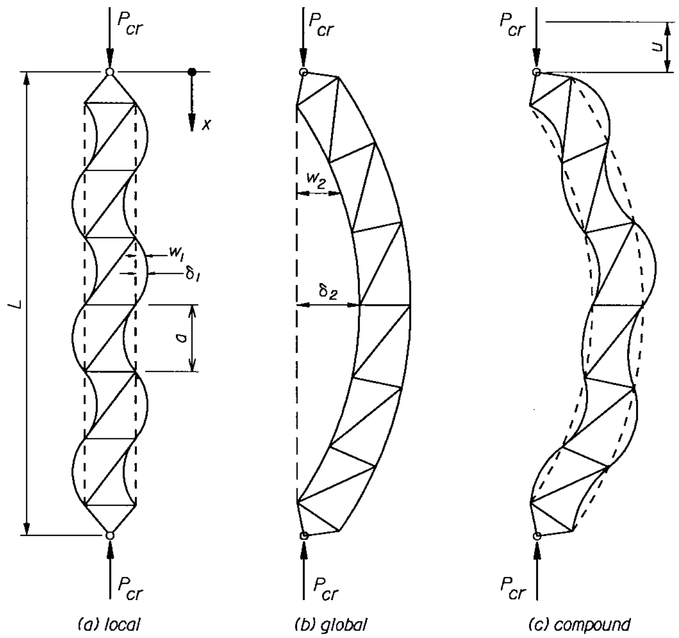


FIGURE 16.20: Buckling modes of built-up members.

The second limit Equation 16.43 is set forth by the current design specifications. Both the AISC-LRFD [4] and AASHTO-LRFD [1] imply that when the effective slenderness ratio $(K_a a/r_i)$ of each individual component between the connectors does not exceed 75% of the governing effective slenderness ratio of the built-up member, no strength reduction due to interaction of buckling modes needs to be considered. The study reported by Duan, Reno, and Uang [22] has justified that the rule of $(K_a a/r_i) < 0.75(KL/r)$ is consistent with the theory.

Analytical Equation

The buckling mode interaction factor γ_{LG} is defined as:

$$\gamma_{LG} = \sqrt{\frac{P_G}{P_{cr}}} = \sqrt{\frac{\pi^2 EI}{(KL)^2 P_{cr}}} \tag{16.44}$$

where

P_{cr} = elastic buckling load considering buckling mode interaction
P_G = elastic buckling load without considering buckling mode interaction
L = unsupported member length
I = moment of inertia of a built-up member

γ_{LG} can be computed by the following equation:

$$\gamma_{LG} = \sqrt{\frac{1+\alpha^2}{1+\frac{\alpha^2}{1+\frac{(\delta_o/a)^2(K_a a/r_f)^2}{2\left[1-\frac{(K_a a/r_f)^2}{(\gamma_{LG} KL/r)^2}\right]^3}}}} \tag{16.45}$$

where

(δ_o/a) = imperfection (out-of-straightness) parameter of individual component (see Figure 16.21)

α = separation as defined as:

$$\alpha = \frac{h}{2}\sqrt{\frac{A_f}{I_f}} = \frac{h}{2r_f} \tag{16.46}$$

in which

I_f = moment inertia of one side individual components (see Figure 16.21)

A_f = cross-section area of one side individual components (see Figure 16.21)

h = depth of latticed member, distance between center of gravity of two flanges in lacing plane (see Figure 16.21)

r = radius of gyration of a built-in member

r_f = radius of gyration of individual component

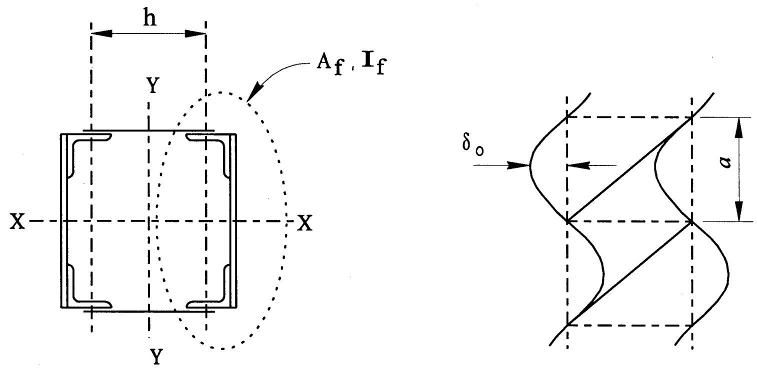


FIGURE 16.21: Typical cross-section and local components.

For widely separated built-up members with $\alpha \geq 2$, the buckling mode interaction factor γ_{LG} can be accurately estimated by the following equation on the conservative side:

$$\gamma_{LG} = \sqrt{1 + \frac{(\delta_o/a)^2 (K_a a/r_j)^2}{2\left[1 - \frac{(K_a a/r_j)^2}{(\gamma_{LG} KL/r)^2}\right]^3}} \tag{16.47}$$

Graphical Solution

Although γ_{LG} can be obtained by solving Equation 16.45, an iteration procedure must be used. For design purposes, solutions in chart forms are more desirable. Figures 16.22 to 16.24 provide engineers with alternative graphic solutions for widely separated built-up members with $\alpha \geq 2$. In these figures, the out-of-straightness ratios (δ_o/a) considered are 1/500, 1/1000, and 1/1500, and the effective slenderness ratios (KL/r) considered are 20, 40, 60, 100, and 140. In all these figures, the top line represents $KL/r = 20$ and the bottom line represents $KL/r = 140$.

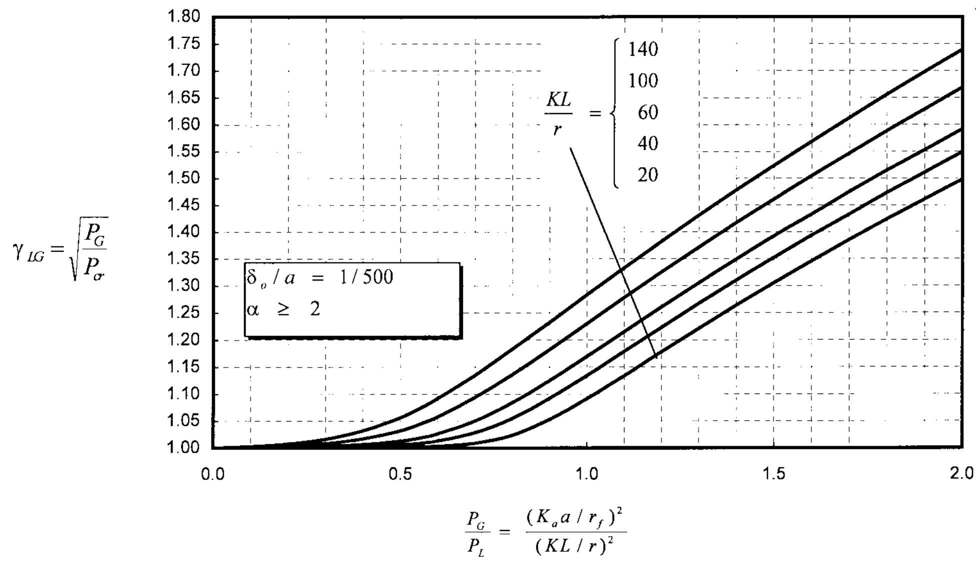


FIGURE 16.22: Buckling mode interaction factor γ_{LG} for $\delta_0/a = 1/500$.

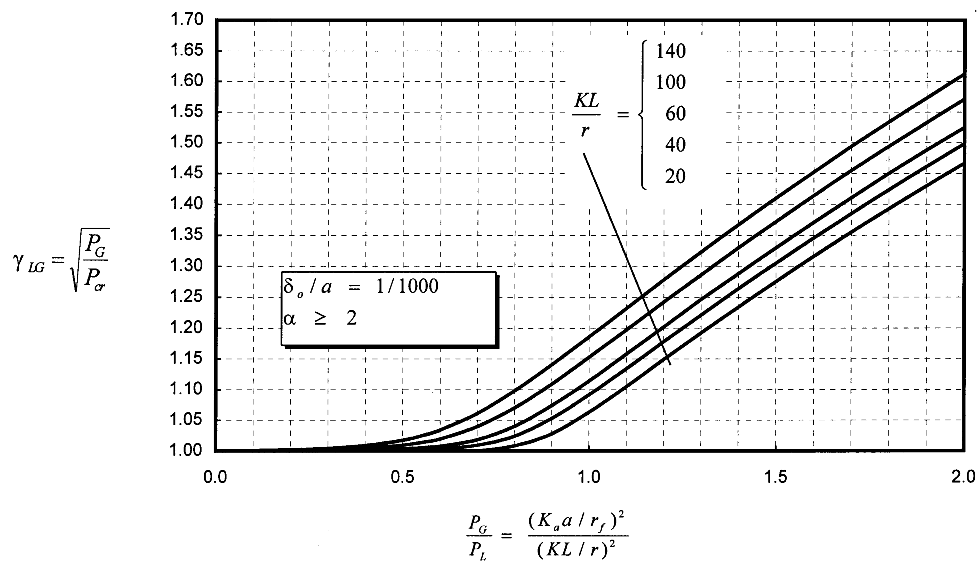


FIGURE 16.23: Buckling mode interaction factor γ_{LG} for $\delta_0/a = 1/1000$.

16.A.3 Acceptable Force D/C Ratios and Limiting Values

Since it is uneconomical and impossible to design bridges to withstand seismic forces elastically, the non-linear inelastic responses of the bridges are expected. The performance-based criteria accepts certain seismic damage in some *other* components so that the *critical* components and the bridges will be kept essentially elastic and functional after the SEE and FEE. This section presents the concept of acceptable force D/C ratios, limiting member slenderness parameters and limiting width-thickness ratios, as well as expected ductility.

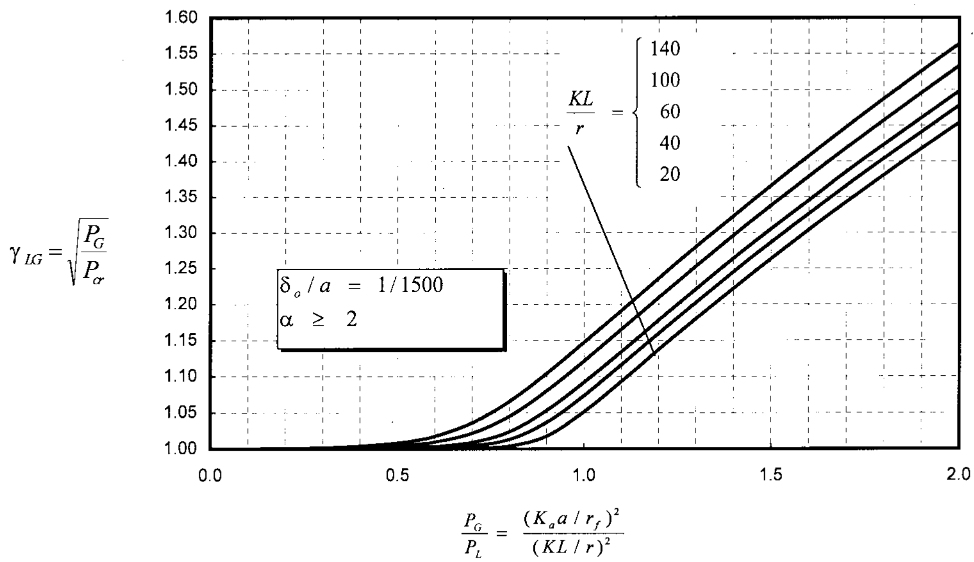


FIGURE 16.24: Buckling mode interaction factor γ_{LG} for $\delta_0/a = 1/1500$.

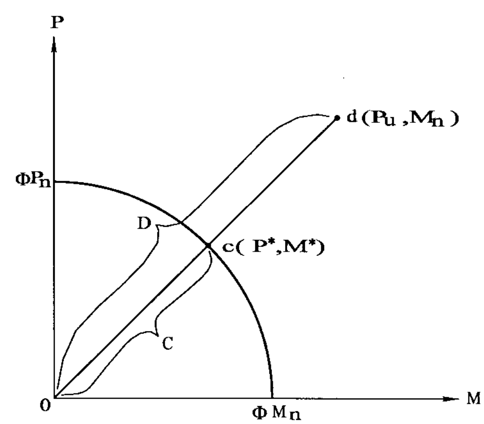


FIGURE 16.25: Definition of force D/C ratios for combined loadings.

Definition of Force Demand/Capacity (D/C) Ratios

For members subjected to an individual load, force demand is defined as a factored individual force, such as factored moment, shear, axial force, etc. which shall be obtained by the nonlinear dynamic time history analysis – Level I specified in Section 16.6, and capacity is defined according to the provisions in Section 16.7.

For members subjected to combined loads, force D/C ratio is based on the force interaction. For example, for a member subjected to combined axial load and bending moment (Figure 16.25), force demand D is defined as the distance from the origin point O(0, 0) to the factored force point $d(P_u, M_u)$, and capacity C is defined as the distance from the origin point O(0, 0) to the point $c(P^*, M^*)$ on the specified interaction surface or curve (failure surface or curve).

Ductility and Load-Deformation Curves

"Ductility" is usually defined as a nondimensional factor, i.e., ratio of ultimate deformation to yield deformation [20, 34]. It is normally expressed by two forms:

1. curvature ductility ($\mu_\phi = \phi_u/\phi_y$)
2. displacement ductility ($\mu_\Delta = \Delta_u/\Delta_y$)

Representing section flexural behavior, *curvature ductility* is dependent on the section shape and material properties. It is based on the moment-curvature curve. Indicating structural system or member behavior, *displacement ductility* is related to both the structural configuration and its section behavior. It is based on the load-displacement curve.

A typical load-deformation curve including both the ascending and descending branches is shown in Figure 16.26. The yield deformation (Δ_y or ϕ_y) corresponds to a loading state beyond which the structure starts to respond inelastically. The ultimate deformation (Δ_u or ϕ_u) refers to the a loading state at which a structural system or a structural member can undergo without losing significant load-carrying capacity. It is proposed that the ultimate deformation (curvature or displacement) is defined as the deformation corresponding to a load that drops a maximum of 20% from the peak load.

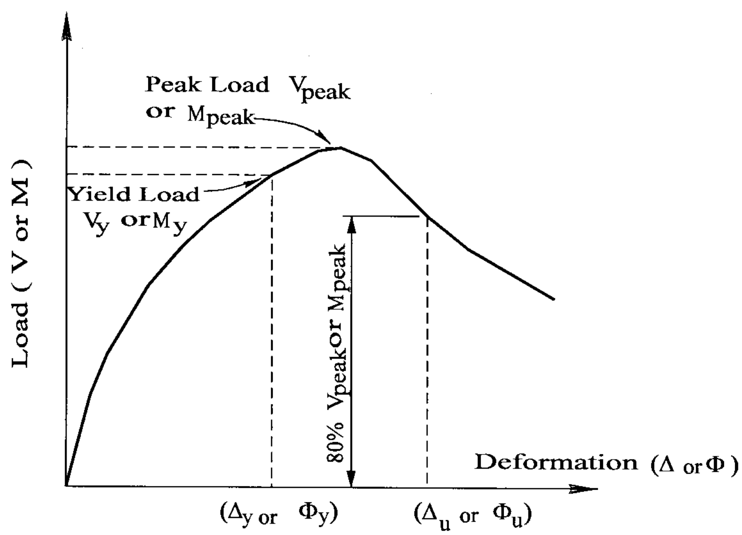


FIGURE 16.26: Load-deformation curves.

Force D/C Ratios and Ductility

The following discussion will give engineers a direct measure of seismic damage of structural components during an earthquake. Figure 16.27 shows a typical load-response curve for a single degree of freedom system. Displacement ductility is:

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \tag{16.48}$$

A new term, *Damage Index*, is defined herein as the ratio of elastic displacement demand to ultimate

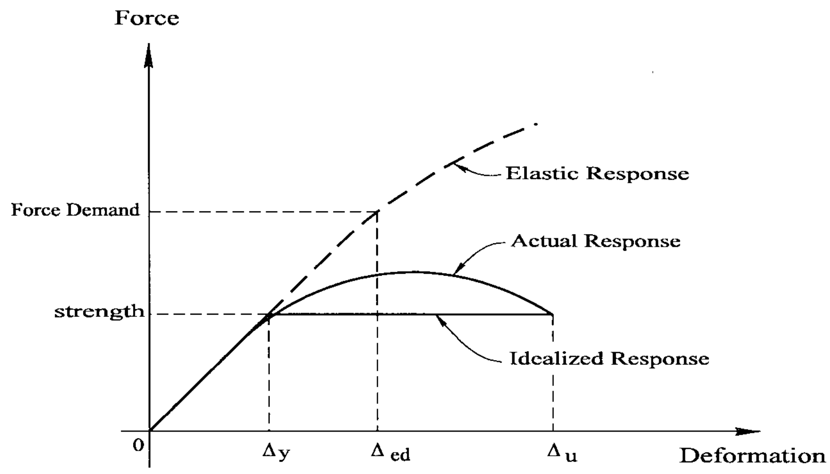


FIGURE 16.27: Response of a single degree of freedom of system.

displacement:

$$D_{\Delta} = \frac{\Delta_{ed}}{\Delta_u} \tag{16.49}$$

When damage index $D_{\Delta} < 1/\mu_{\Delta}$ ($\Delta_{ed} < \Delta_y$), it implies that no damage occurs and the structure responds elastically; $1/\mu_{\Delta} < D_{\Delta} < 1.0$ indicates certain damages occur and the structure responds inelastically; $D_{\Delta} > 1.0$, however, means that a structural system collapses.

Based on the equal displacement principle, the following relationship is obtained as:

$$\frac{\text{Force Demand}}{\text{Force Capacity}} = \frac{\Delta_{ed}}{\Delta_y} = \mu_{\Delta} D_{\Delta} \tag{16.50}$$

It is seen from Equation 16.50 that the force D/C ratio is related to both the structural characters in terms of ductility μ_{Δ} and the degree of damage in terms of damage index D_{Δ}. Table 16.4 shows detailed data for this relationship.

TABLE 16.4 Force D/C Ratio and Damage Index

Force D/C ratio	Damage index D_{Δ}	Expected system displacement ductility μ_{Δ}
1.0	No damage	No requirement
1.2	0.4	3.0
1.5	0.5	3.0
2.0	0.67	3.0
2.5	0.83	3.0

General Limiting Values

To ensure the important bridges have ductile load paths, general limiting slenderness parameters and width-thickness ratios are specified in Sections 16.8.3 and 16.8.4.

For steel members, λ_{cr} is the limiting member slenderness parameter for column elastic buckling and is taken as 1.5 from AISC-LRFD [4] and λ_{br} is the limiting member slenderness parameter for beam elastic torsional buckling and is calculated by AISC-LRFD [4]. For a *critical* member, a more strict requirement, 90% of those elastic buckling limits is proposed. Regardless of the force D/C ratios, all steel members must not exceed these limiting values. For existing steel members with D/C ratios less than one, this limit may be relaxed.

For concrete members, the general limiting parameter $KL/r = 60$ is proposed.

Acceptable Force D/C Ratios DC_{accept}

Acceptable force D/C ratios (DC_{accept}) depends on both the structural characteristics in terms of ductility μ_Δ and the degree of damage to the structure that can be accepted by practicing engineers in terms of damage index D_Δ.

To ensure a steel member has enough inelastic deformation capacity during an earthquake, it is necessary to limit both the member slenderness parameters and the section width-thickness ratios within the specified ranges so that the acceptable D/C ratios and the energy dissipation can be achieved.

Upper Bound Acceptable D/C Ratio DC_p

1. For *other* members, the large acceptable force D/C ratios ($DC_p = 2$ to 2.5) are proposed in Table 16.2. This implies that the damage index equals $0.67 \sim 0.83$ and more damage will occur at *other* members and large member ductility will be expected. To achieve this,
   - the limiting width-thickness ratio was taken as $\lambda_{p-\text{Seismic}}$ from AISC-Seismic Provisions [5], which can provide flexural ductility 8 to 10.
   - the limiting slenderness parameters were taken as λ_{bp} for flexural moment dominant members from AISC-LRFD [4], which can provide flexural ductility 8 to 10.
2. For *critical* members, small acceptable force D/C ratios ($DC_p = 1.2$ to 1.5) are proposed in Table 16.2, as the design purpose is to keep *critical* members essentially elastic and allow little damage (damage index equals to $0.4 \sim 0.5$). Thus, small member ductility is expected. To achieve this,
   - the limiting width-thickness ratio was taken as λ_p from AISC-LRFD [5], which can provide flexural ductility at least 4.
   - the limiting slenderness parameters were taken as λ_{bp} for flexural moment dominant members from AISC-LRFD [4], which can provide flexural ductility at least 4.
3. For axial load dominant members, the limiting slenderness parameters were taken as $\lambda_{cp} = 0.5$, corresponding to 90% of the axial yield load by the AISC-LRFD [4] column curve. This limit will provide the potential for axial load dominated members to develop expected inelastic deformation.

Lower Bound Acceptable D/C Ratio DC_r

The lower bound acceptable force D/C ratio $DC_{rc} = 1$ is proposed in Table 16.2. For $DC_{\text{accept}} = 1$, it is unnecessary to enforce more strict limiting values for members and sections. Therefore, the limiting slenderness parameters for elastic global buckling specified in Table 16.2 and the limiting width-thickness ratios specified in Table 16.3 for elastic local buckling are proposed.

Acceptable D/C Ratios Between Upper and Lower Bounds $DC_r < DC_{\text{accept}} < DC_p$

When acceptable force D/C ratios are between the upper and the lower bounds, $DC_r < DC_{\text{accept}} < DC_p$, a linear interpolation as shown in Figure 16.28 is proposed to determine the limiting slenderness parameters and width-thickness ratios. The following formulas can be used:

$$\lambda_{pr} = \begin{cases} \left[\lambda_p + \left(\lambda_r - \lambda_p\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right)\right] & \text{for } \textit{critical} \text{ members} \\ \left[\lambda_{p-\text{Seismic}} + \left(\lambda_r - \lambda_{p-\text{Seismic}}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right)\right] & \text{for } \textit{other} \text{ members} \end{cases} \tag{16.51}$$

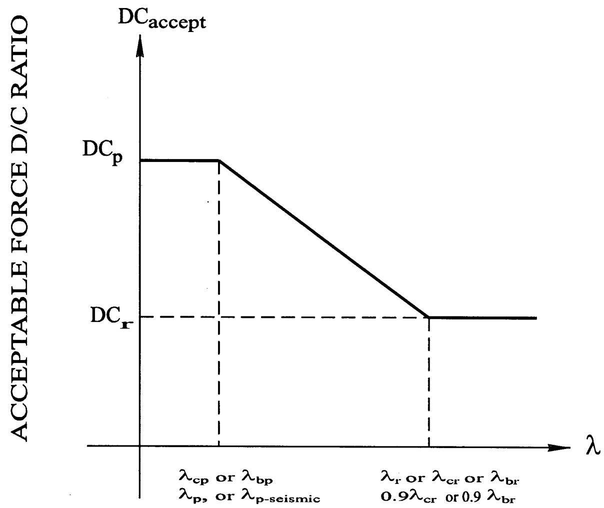


FIGURE 16.28: Acceptable D/C ratios and limiting slenderness parameters and width-thickness ratios.

For axial load dominant members ($P_u/P_n \geq M_u/M_n$)

$$\lambda_{cpr} = \begin{cases} \lambda_{cp} + \left(0.9\lambda_{cr} - \lambda_{cp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{critical} \text{ members} \\ \lambda_{cp} + \left(\lambda_{cr} - \lambda_{cp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{other} \text{ members} \end{cases} \tag{16.52}$$

For flexural moment dominant members ($M_u/M_n > P_u/P_n$)

$$\lambda_{bpr} = \begin{cases} \lambda_{bp} + \left(0.9\lambda_{br} - \lambda_{bp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{critical} \text{ members} \\ \lambda_{bp} + \left(\lambda_{br} - \lambda_{bp}\right)\left(\frac{DC_p - DC_{\text{accept}}}{DC_p - DC_r}\right) & \text{for } \textit{other} \text{ members} \end{cases} \tag{16.53}$$

where λ_r, λ_p, $\lambda_{p-\text{Seismic}}$ are limiting width-thickness ratios specified by Table 16.3.

Limiting Width-Thickness Ratios

The basic limiting width-thickness ratios λ_r, λ_p, $\lambda_{p-\text{Seismic}}$ specified in Table 16.3 are proposed for important bridges.

16.A.4 Inelastic Analysis Considerations

This section presents concepts and formulas of reduced material and section properties and yield surface for steel members for possible use in inelastic analysis.

Stiffness Reduction

Concepts of stiffness reduction — tangent modulus — have been used to calculate inelastic effective length factors by AISC-LRFD [4], and to account for both the effects of residual stresses and geometrical imperfection by Liew [32].

To consider inelasticity of a material, the tangent modulus of the material E_t may be used in analysis. For practical application, stiffness reduction factor (SRF) $= (E_t/E)$ can be taken as the ratio of the inelastic to elastic buckling load of the column:

$$SRF = \frac{E_t}{E} \approx \frac{(P_{cr})_{\text{inelastic}}}{(P_{cr})_{\text{elastic}}} \tag{16.54}$$

where $(P_{cr})_{\text{inelastic}}$ and $(P_{cr})_{\text{elastic}}$ can be calculated by AISC-LRFD [4] column equations:

$$(P_{cr})_{\text{inelastic}} = 0.658^{\lambda_c^2} A_s F_y \tag{16.55}$$

$$(P_{cr})_{\text{elastic}} = \left[\frac{0.877}{\lambda_c^2}\right] A_s F_y \tag{16.56}$$

in which A_s is the gross section area of the member and λ_c is the slenderness parameter. By utilizing the calculated axial compression load P, the tangent modulus E_t can be obtained as:

$$E_t = \begin{cases} E & \text{for } P/P_y \le 0.39 \\ -3(P/P_y)\ln(P/P_y) & \text{for } P/P_y > 0.39 \end{cases} \tag{16.57}$$

Reduced Section Properties

In an initial structural analysis, the section properties based on a fully integral section of a latticed member may be used. If section forces obtained from this initial analysis are lower than the section strength controlled by the shear-flow transferring capacity, assumed fully integral section properties used in the analysis are rational. Otherwise, section properties considering a partially integral section, as discussed in Section 17.A.1, I_{equiv} and J_{equiv}, may be used in the further analysis. This concept is similar to "cracked section" analysis for reinforced concrete structures [13].

1. Moment of inertia — latticed members

   (a) For lacing bars or battens within web plane (bending about y-y axis in Figure 16.19)

   The following assumptions are made:

   - Moment-curvature curve (Figure 16.29) behaves bi-linearly until the section reaches its ultimate moment capacity.
   - For moments less than $M_d = \beta_m M_u$, the moment at first stiffness degradation, the section can be considered fully integral.
   - For moments larger than M_d and less than M_u, the ultimate moment capacity of the section, the section is considered as a partially integral one; that is, bending stiffness should be based on a reduced moment of inertia defined by Equation 16.27.

   An equivalent moment of inertia, I_{equiv}, based on the secant stiffness can be obtained as:

$$I_{\text{equiv}} = \frac{I^*_{y-y} I_{y-y}}{\beta_m I_{y-y} + (1-\beta_m) I^*_{y-y}} \tag{16.58}$$

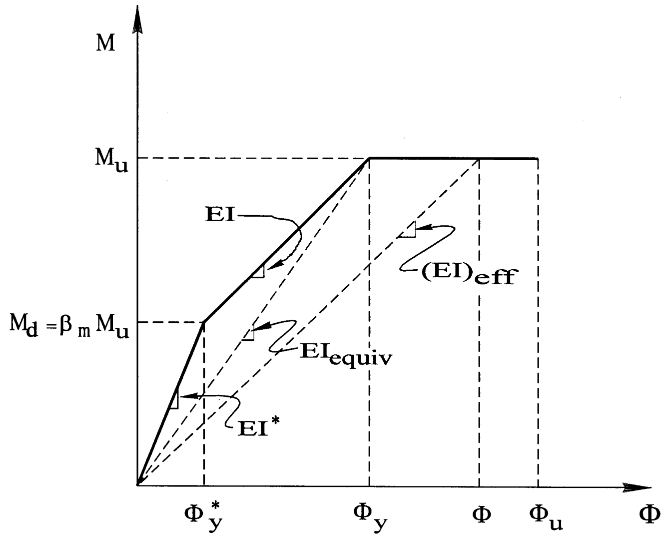


FIGURE 16.29: Idealized moment-curvature curve of a latticed member section.

where

I_{y-y} = moment of inertia of a section about the y-y axis considering shear transferring capacity

I^*_{y-y} = moment of inertia of a section about the y-y axis assuming full section integrity

β_m = reduction factor for the moment of inertia and may be determined by Equation 16.28

(b) For lacing bars or battens within flange plane (bending about x-x axis in Figure 16.19)

Equation 16.29 is still valid for structural analysis.

2. Effective flexural stiffness — For steel members, when $\Phi_y < \Phi < \Phi_u$, the further reduced section property, effective flexural stiffness $(EI)_{\text{eff}}$ may be used in the analysis.

$$\frac{M_u}{\Phi_u} \leq (EI)_{\text{eff}} = \frac{M_u}{\Phi} \leq EI_{\text{equiv}} \tag{16.59}$$

3. Torsional constant — latticed members
Based on similar assumptions and principles used for moment of inertia, an equivalent torsional constant, J_{equiv}, is derived as follows:

$$J_{\text{equiv}} = \frac{J^* J}{\beta_t J + (1 - \beta_t) J^*} \tag{16.60}$$

where

J = torsional constant of a section considering shear transferring capacity (See Section 17.A.1)

J^* = torsional constant of a section assuming full section integrity

β_t = reduction factor for torsional constant may be determined by Equation 16.38

Yield Surface Equation for Doubly Symmetrical Sections

The yield or failure surface concept has been conveniently used in inelastic analysis to describe the full plastification of steel sections under action of axial force combined with biaxial bending. A four parameter yield surface equation for doubly symmetrical steel sections (I, thin-walled circular tube, thin-walled box, solid rectangular and circular sections), developed by Duan and Chen [19], is presented in this section for possible use in a nonlinear analysis.

The general shape of the yield surface for a doubly symmetrical steel section as shown in Figure 16.30 can be described approximately by the following general equation:

$$\left(\frac{M_x}{M_{pcx}}\right)^{\alpha_x} + \left(\frac{M_y}{M_{pcy}}\right)^{\alpha_y} = 1.0 \tag{16.61}$$

where M_{pcx} and M_{pcy} are the moment capacities about the respective axes, reduced for the presence

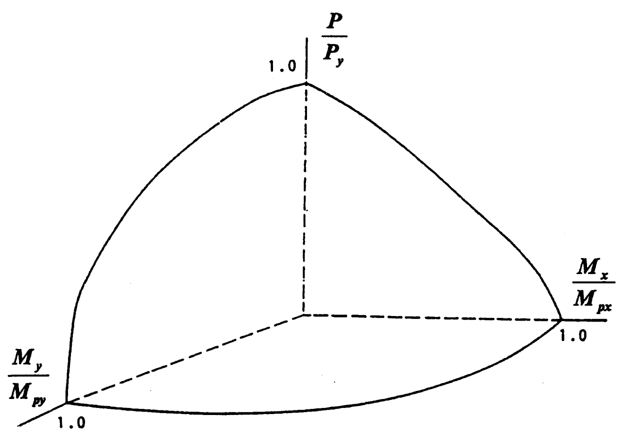


FIGURE 16.30: Typical yield surface for doubly symmetrical sections.

of axial load; they can be obtained by the following formulas:

$$M_{pcx} = M_{px}\left[1 - \left(\frac{P}{P_y}\right)^{\beta_x}\right] \tag{16.62}$$

$$M_{pcy} = M_{py}\left[1 - \left(\frac{P}{P_y}\right)^{\beta_y}\right] \tag{16.63}$$

where

P = axial force

M_x = bending moment about the x-x principal axis

M_y = bending moment about the y-y principal axis

M_{px} = plastic moment about x-x principal axis

M_{py} = plastic moment about y-y principal axis

The four parameters α_x, α_y, β_x, and β_y are dependent on sectional shapes and area distribution. It is seen that α_x and α_y represent biaxial loading behavior, while β_x and β_y describe uniaxial loading behavior. They are listed in Table 16.5:

Equation 16.61 represents a smooth and convex surface in the three-dimensional stress-resultant space. It meets all special conditions and is easy to implement in a computer-based structural analysis.

TABLE 16.5 Parameters for Doubly Symmetrical Sections

Section types	α_x	α_y	β_x	β_y
Solid rectangular	$1.7 + 1.3\ (P/P_y)$	$1.7 + 1.3\ (P/P_y)$	2.0	2.0
Solid circular	2.0	2.0	2.1	2.1
I-shape	2.0	$1.2 + 2\ (P/P_y)$	1.3	$2 + 1.2\ (A_w/A_f)$
Thin-walled box	$1.7 + 1.5\ (P/P_y)$	$1.7 + 1.5\ (P/P_y)$	$2 - 0.5\ \overline{B} \geq 1.3$	$2 - 0.5\ \overline{B} \geq 1.3$
Thin-walled circular	2.0	2.0	1.75	1.75

Note: $\overline{B}$ is the ratio of width-to-depth of the box section with respect to the bending axis.