CHAPTER 2

CODE PROVISIONS FOR SEISMIC DESIGN OF STEEL STRUCTURES

Abstract: Relevant design provisions used in the subsequent chapters for code-compliant seismic design (member sizing) of plane steel moment frame structures are provided in this chapter. These include 2000 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA-368 2002), AISC Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings (AISC 1994), and AISC Seismic Provisions for Structural Steel Buildings (AISC 1997) and supplemental documents (AISC 2000).

2.1 2000 NEHRP equivalent lateral force procedure

The equivalent lateral force procedure of the 2000 NEHRP provisions permits an elastic analysis for seismic design of SMRF structures that are represented by simple linear elastic models with centerline dimensions. The design response spectrum is based on 2/3 of the 5%-damped maximum considered elastic response spectrum at a hazard level corresponding to an exceedance probability of 2% in 50 years (return period of 2,475 years) and with proper site class effects adjustment. A structure designed in accordance with these provisions is equipped with a lateral force resistance capacity that is typically lower than what can keep the structure elastic when subject to design earthquakes. Seismic energy imparted to the structure is expected to dissipate through hysteretic behavior of structural components and, as a result, an economical structural design is obtained. Note that changes to 1997 NEHRP provisions (FEMA-302 1998) relevant to seismic design of steel SMRF structures include a new lower bound for the seismic base shear coefficient, a modified formula for the empirical fundamental period, and increased upper bounds on the rationally calculated fundamental period.

2.1.1 Seismic base shear

The seismic base shear V is determined by

$$V = C_s W \tag{2.1}$$

where

W = total dead load plus 25% of floor live load, with proper reduction for tributary area;

C_s = seismic design coefficient, as determined by

$$C_s = \max\left\{0.044\, I\, S_{D1},\ \min\left[\frac{S_{DS}}{R/I}, \frac{S_{D1}}{TR/I}\right]\right\} \tag{2.2}$$

with

R = response modification (or elastic strength reduction) factor and equal to 8 for the present steel SMRF;

I = occupancy importance factor and equal to 1.0 in this study for *Seismic Use Group I* structures;

S_{D1}, S_{DS} = design spectral response acceleration ordinates at one-second and short periods, which are equal to $(2/3)S_{M1}$ and $(2/3)S_{MS}$, respectively, with S_{M1} and S_{MS} the site class effects adjusted maximum considered earthquake spectral response acceleration ordinates at one-second and short periods, respectively, which are obtained from a 5%-damped acceleration response spectrum with a 2% probability of exceedance in 50 years;

T = the fundamental period.

The fundamental period can be calculated either from a rational dynamic analysis or by use of the following empirical formula

$$T_a = C_r h_n^{\lambda} \tag{2.3}$$

where

$h_n =$ from-the-base structural height in ft.;

$C_r = 0.028$ and $\lambda = 0.8$ for steel SMRF structures.

In order to maintain a conservative base shear level, the rationally calculated fundamental period T shall not exceed the product of an S_{D1}-dependent coefficient for upper limit on calculated period, C_u, and the approximate empirical fundamental period T_a.

2.1.2 Vertical distribution of seismic forces

The lateral seismic force at the *j*-th level is determined by

$$F_j = \left(\frac{w_j h_j^k}{\sum_{i=1}^{n} w_i h_i^k} \right) V \tag{2.4}$$

where

w_i = portion of the total gravity load of the building assigned to level i;

h_i = height from the base to level i; n = total number of levels;

$k =$ fundamental period T dependent exponent with k =1 for $T \leq 0.5s$, k =2 for $T \geq 2.5s$, and linear interpolation allowed for $0.5s < T < 2.5s$.

2.1.3 Determination of design story drifts

The nominal deflection at the *x*-th floor level is determined by

$$\delta_x = \frac{C_d \, \delta_{xe}}{I} \tag{2.5}$$

where

$C_d =$ deflection amplification factor, which is 5.5 in this study;

$\delta_{xe} =$ deflection determined by an elastic analysis. For computation of these (nominal) design drift demands, it is permissible to use the rationally calculated fundamental period without the upper bound limitation as used in determination of seismic design force level.

To include the P-delta effects, the above story drift should be multiplied by $1/(1-\theta)$, where θ is the stability coefficient defined by

$$\theta = \frac{P_x \, \Delta}{V_x \, h_{sx} \, C_d} \tag{2.6}$$

where

P_x = total vertical design load at and above level x;

$V_x =$ seismic shears acting between levels x and x-1;

$\Delta =$ design story drift associated with V_x; h_{sx} = story height below level x.

The stability factor θ shall not exceed $\theta_{\max} = 0.5/(\beta C_d) \leq 0.25$, where β is the ratio of shear demand to shear capacity for the story between levels x and x-1. For structures having a stability factor greater than 0.25, 2000 NEHRP suggests that the structure may be potentially unstable and should be redesigned. The allowable story drift in this study is 2% of story height for *Seismic Use Group I* structures.

2.2 AISC-LRFD seismic steel design specifications

AISC-LRFD steel specifications (AISC 1994) and AISC seismic provisions (AISC 1997) as well as their supplemental documents (AISC 2000) define relevant load combinations, member strength requirements, proportioning of member flexural strength at moment connections, and cross-section slenderness ratio limits for steel SMRF designs. Specifically, in addition to load combination scenarios prescribed in AISC-LRFD, two more load combinations with amplified horizontal earthquake load effects are required in AISC seismic provisions to check axial strength of column members, which are sensitive to the effects of structural overstrength (Uang et al. 2001); the beam/column member strength is checked by interaction equations of axial forces and flexure; a strong-column-weak-beam mechanism needs to be ensured for SMRF designs that are classified in seismic design category D and higher; appropriate width-thickness limits are applied for web and flange elements of beams and column members, respectively.

2.2.1 Load combinations

Unlike AISC-Allowable Stress Design provisions (ASD 1989), AISC-LRFD provisions use separate factors for load and resistance in order to gain a uniform reliability level. The following load combination patterns are prescribed:

$$\begin{cases} 1.4D \\ 1.2D + 1.6L + 0.5\,L_r \\ 1.2D + 1.6L_r + (0.5L \text{ or } 0.8W) \\ 1.2D + 1.3W + 0.5L + 0.5L_r \\ 1.2D \pm 1.0E + 0.5L \\ 0.9D \pm (1.3W \text{ or } 1.0E) \end{cases} \tag{2.7}$$

where D, L, L_r, W, and E are the effects of dead load, live load, roof live load, wind load, and seismic load, respectively. Here the wind load is determined in accordance with ASCE-7 (1998).

2.2.2 Effects of seismic loads

Based on the uniform hazard spectra (e.g., USGS), the effect of earthquake-induced forces E is defined by

$$E = \rho Q_E \pm 0.2 S_{DS} D \tag{2.8}$$

where

S_{DS} = design spectral response acceleration at the short period, as defined in Section 2.1.1;

ρ = a reliability/redundancy factor;

Q_E = effects of horizontal seismic forces, which are the same as height-wise lateral seismic forces defined in Section 2.1.2.

Additional load combinations using amplified horizontal earthquake loads to check axial strength of column members, which are sensitive to structural overstrength, are defined by

$$\begin{cases} 1.2D + 0.5L + \Omega_0 Q_E \\ 0.9D - \Omega_0 Q_E \end{cases} \tag{2.9}$$

where Ω_0 = system overstrength factor, which is equal to 3 for the present seismic design of steel SMRF structures.

2.2.3 Member strength checking

Interaction of axial (tensile or compressive) forces and flexure in symmetric structural members of a planar frame is governed by the following equations:

$$
\begin{cases} \dfrac{P_u}{\phi P_n} + \dfrac{8}{9}\dfrac{M_u}{\phi_b M_n} \le 1.0 & \text{for } \dfrac{P_u}{\phi P_n} \ge 0.2 \\ \dfrac{P_u}{2\phi P_n} + \dfrac{M_u}{\phi_b M_n} \le 1.0 & \text{for } \dfrac{P_u}{\phi P_n} < 0.2 \end{cases} \tag{2.10}
$$

where

P_u = required axial strength;

ϕ = resistance factor, 0.90 for tension and 0.85 for compression;

P_n = nominal axial strength;

M_u = required flexural strength;

ϕ_b = resistance factor for flexure, 0.90;

M_n = nominal flexural strength.

Details for calculating member strength demands can be found in Appendix A.

2.2.4 Strong-column-weak-beam criterion

Plastic zones formed in column members will cause significant problems to the structural system. Retrofitting or replacement of damaged column elements is much more difficult than doing so for other structural members. A story mechanism will possibly result if a number of columns yield within the same story level, leading to potential collapse. Therefore, beam and column member sizes should be proportioned, at the design force level, such that inelastic deformations are expected to concentrate in beams and/or panel zones while columns basically remain elastic or suffer very minor hinging if not completely avoidable.

According to 2000 NEHRP, all structures shall be assigned to a Seismic Design Category based on their Seismic Use Group and the design spectral response acceleration coefficients, S_{D1}

and S_{DS}. Seismic Design Category determines possible structural systems, limitations on height and irregularity, those components of the structure that must be designed for seismic resistance, and types of lateral force analysis that must be performed. For steel SMRF designs that are classified in Seismic Design Category D and higher, the following criterion shall be satisfied at beam-to-column connections to ensure a strong-column-weak-beam mechanism:

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0 \tag{2.11}$$

where

$\sum M^*_{pc}$ = sum of the projections of the nominal flexural strengths of the columns above and below the joint to the beam centerline with a reduction for the axial force in the column; it is permitted to take $\sum M^*_{pc} = Z_c(F_{yc} - P_{uc} / A_g)$, with A_g = gross area of column, F_{yc} = specified minimum yield strength of column, P_{uc} = required column axial compressive strength, Z_c = plastic section modulus of column.

$\sum M^*_{pb}$ = sum of the projections of the expected beam flexural strength(s) at the plastic hinge location(s) to the column centerline; it is permitted to take $\sum M^*_{pb} = \sum(1.1R_y M_{pb} + M_{vb})$, where R_y is the ratio of the expected yield strength to the minimum specified yield strength, and is 1.5, 1.3, and 1.1 for ASTM A36, A572 Grade 42, and all other grades, respectively; $M_{pb} = Z_b F_{yb}$ is the nominal plastic flexural strength; $M_{vb} = S_b F_{yb}$ is the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, with F_{yb} = minimum yield strength of beam, S_b and Z_b = elastic and plastic section modulus of beam, respectively.

2.2.5 Width-thickness ratio limits

Limiting width-thickness ratios for web and flange elements of beams shall comply with λ_{ps} in Table 2.1. When the ratio in Equation 2.11 is less than or equal to 2.0, columns shall comply with λ_{ps} in Table 2.1 (AISC 2000); otherwise, columns shall comply with λ_p in Table 2.1 (AISC 1997). For the notations, *b* = width of compression element; *t* = thickness of element; h_c = web depth; t_w = web thickness; F_y = specified minimum yield stress; P_u = required axial strength of column, P_y = nominal axial yield strength.

Table 2.1 Width-thickness ratio limits (AISC 1994, 1997, 2000)

Description of element	Width-thickness ratio	Limiting width-thickness ratio	
		AISC-LRFD provisions λ_p	AISC seismic provisions λ_{ps}
Flanges of I-shaped rolled beams in flexure	*b/t*	$65\big/\sqrt{F_y}$	$52\big/\sqrt{F_y}$
Webs in combined flexural and axial compression	h_c / t_w	For $P_u\big/\phi_b P_y \le 0.125$: $\frac{640}{\sqrt{F_y}}\left[1-2.75\frac{P_u}{\phi_b P_y}\right]$	For $P_u\big/\phi_b P_y \le 0.125$: $\frac{520}{\sqrt{F_y}}\left[1-1.54\frac{P_u}{\phi_b P_y}\right]$
		For $P_u\big/\phi_b P_y > 0.125$: $\frac{191}{\sqrt{F_y}}\left[2.33-\frac{P_u}{\phi_b P_y}\right] \ge \frac{253}{\sqrt{F_y}}$	