CHAPTER 3

RESIDENTIAL BUILDING WITH SHEAR WALL-FRAME INTERACTIVE AND BUILDING FRAME SYSTEMS

3.1 INTRODUCTION

According to IBC Table 1617.6, shear wall-frame interactive systems may be used as the seismic-force-resisting system where structures are assigned to low Seismic Design Categories (SDC), i.e., SDC A or B. In such systems, shear walls and frames resist the design lateral forces in proportion to their lateral rigidities.

Building frame systems may be used as the seismic-force-resisting system for structures assigned to SDC C, D, and F, subject to the limitations in IBC Table 1617.6. This system has an essentially complete space frame that resists the gravity loads and shear walls that resist the lateral forces. It is essential that the deformation compatibility requirements of IBC 1617.6.4.3 be satisfied for building frame systems assigned to SDC D and above. These provisions recognize that members that are not designated to be part of the seismic-force-resisting system deform with the members of the seismic-force-resisting system when subjected to the code-specified earthquake design forces, since all of the components are connected at every floor level through the floor systems. Thus, members that are not of the seismic-force-resisting system must be able to carry reactions from gravity forces when subjected to earthquake-induced lateral displacements.

This chapter illustrates the design and detailing of typical structural members in these two types of systems for structures assigned to SDC A, B, C, D, and E.

3.2 DESIGN FOR SDC A

3.2.1 Design Data

A typical plan and elevation of a 9-story residential building is shown in Figure 3-1. The computation of wind and seismic forces according to the 2000 IBC is illustrated below. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

A shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls may be used for structures assigned to SDC A without any limitations according to IBC Table 1617.6. This type of system is utilized in this example.

It is assumed mainly for simplicity that slabs, columns, and walls have constant cross-sections throughout the height of the building, and that the bases of the lowest story segments are fixed. Although the member dimensions in the following sections are within the practical range, the structure itself is a hypothetical one, and has been chosen mainly for illustrative purposes.

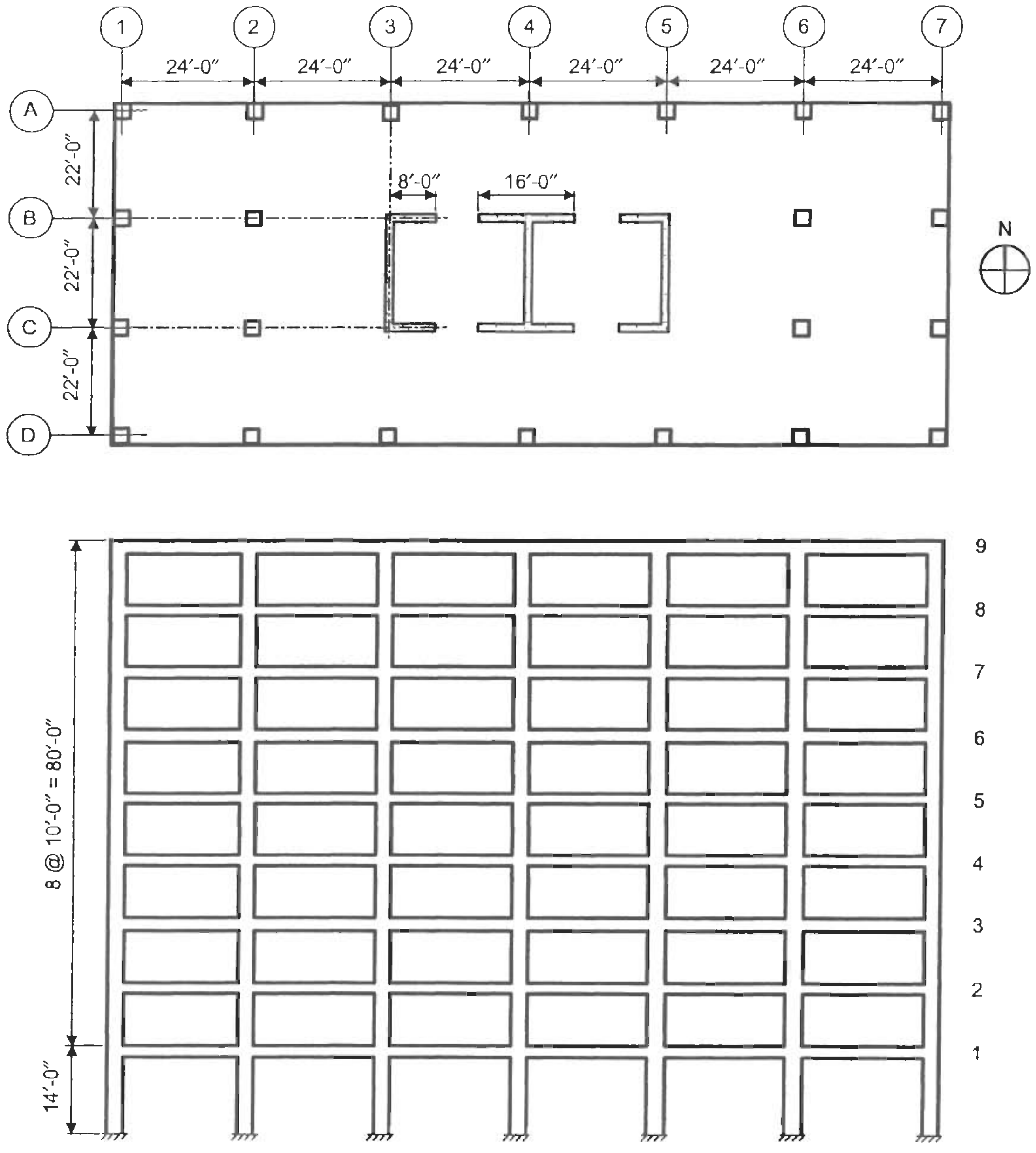

Figure 3-1 Typical Plan and Elevation of Example Building (SDC A)

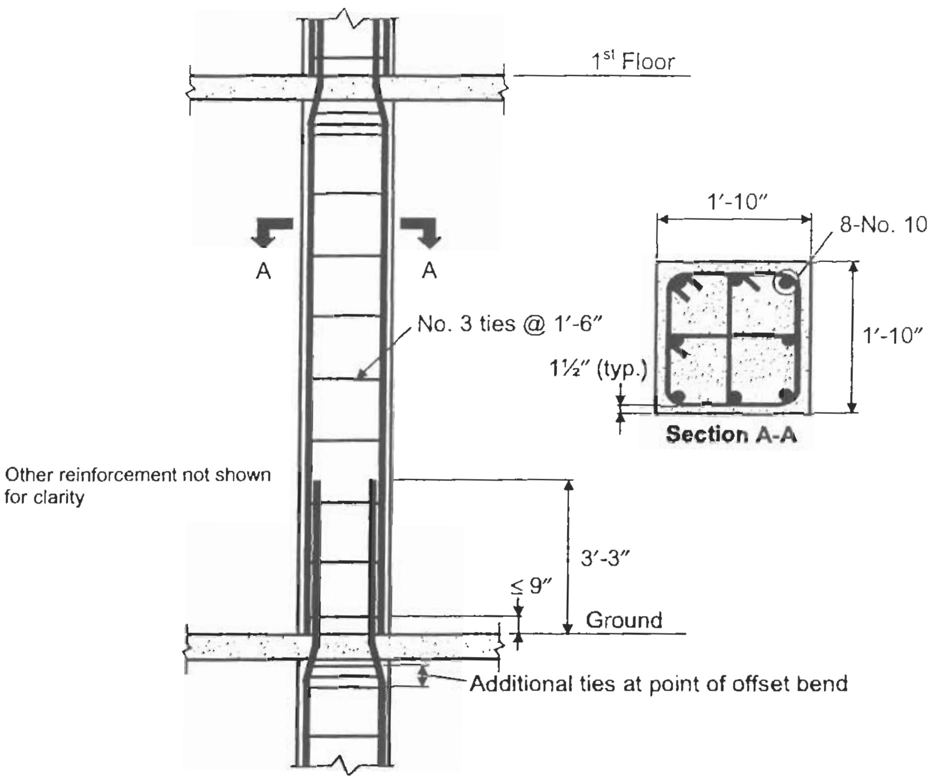

Figure 3-7 Reinforcement Details for Column B2 Supporting the 1st Floor Level (SDC A)

3.2.4.4 Design of Shear Wall on Line 4

Table 3-12 contains a summary of the design axial forces, bending, moments, and shear forces at the base of the wall. Note that wind in the N-S direction causes appreciable reactions in this member; thus, the governing reactions in Table 3-12 are for wind in that direction.

Table 3-12 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC A)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	1,549	0	0
Live (L)	195	0	0
Wind (W)	0	± 12,641	± 222
Load Combinations			
$1.4D + 1.7L$	2,500	0	0
$0.75(1.4D + 1.7L + 1.7W)$	1,875	16,117	283
$0.9D + 1.3W$	1,394	-16,433	-289

Design for shear.

The shear strength of the concrete is determined in accordance with ACI 11.10.5 for walls subjected to axial compression:

$$V_c = 2\sqrt{f_c'}hd$$

$$= 2\sqrt{4{,}000} \times 8 \times 220.8 / 1{,}000 = 223.4 \text{ kips}$$

where d is permitted to be taken equal to $0.8\ell_w = 0.8 \times 276 = 220.8$ in. (ACI 11.10.4). The maximum factored shear force is 289 kips from the third load combination (see Table 3-12). Since $\phi V_c = 0.85 \times 223.4 = 189.9 \text{ kips} < V_u = 289 \text{ kips}$, horizontal shear reinforcement shall be provided in accordance with ACI 11.10.9. With 2 layers of No. 4 bars in the web, the required bar spacing is determined by Eq. 11-33:

$$s_2 = \frac{A_v f_y d}{V_s} = \frac{(2 \times 0.20) \times 60 \times 220.8}{(289/0.85) - 223.4} = 45.5 \text{ in.} > 18 \text{ in.}$$

where, according to ACI 11.10.9.3, maximum spacing of horizontal reinforcement shall not exceed $\ell_w/5 = 276/5 = 55.2$ in., $3h = 3 \times 8 = 24$ in., or 18 in. (governs). Try 2-No. 4 horizontal bars @ 18 in.

Ratio of horizontal shear reinforcement ρ_h shall not be less than 0.0025 (ACI 11.10.9.2). For 2-No. 4 horizontal bars spaced at 18 in., the ratio ρ_h of horizontal shear reinforcement area to gross concrete area of vertical section is

$$\rho_h = \frac{2 \times 0.20}{8 \times 18} = 0.0028 > 0.0025 \quad \text{O.K.}$$

Use 2-No. 4 horizontal bars @ 18 in.

The shear strength V_n at any horizontal section must be less than or equal to $10\sqrt{f_c'}hd$ = 1,117 kips (ACI 11.10.3). In this case,

$$V_n = V_c + V_s = 223.4 + \frac{(2 \times 0.20) \times 60 \times 220.8}{18} = 517.8 \text{ kips} < 1{,}117 \text{ kips} \quad \text{O.K.}$$

The ratio of vertical shear reinforcement area to gross concrete area of horizontal section shall not be less than 0.0025 nor the value obtained from Eq. 11-34 (ACI 11.10.9.4):

$$\rho_n = 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(\rho_h - 0.0025)$$

$$= 0.0025 + 0.5\left(2.5 - \frac{94}{23}\right)(0.0028 - 0.0025) = 0.0023 < 0.0025$$

Thus, $\rho_n = 0.0025$.

According to ACI 11.10.9.5, spacing of vertical shear reinforcement shall not exceed $\ell_w/3 = 276/3 = 92.0$ in., $3h = 3 \times 8 = 24$ in., or 18 in. (governs). For 2-No. 4 vertical bars spaced at 18 in.:

$$\rho_n = \frac{2 \times 0.20}{8 \times 18} = 0.0028 > 0.0025 \quad \text{O.K.}$$

Use 2-No. 4 vertical bars @ 18 in.

The provided vertical and horizontal reinforcement satisfy the requirements of ACI 14.3.2 and 14.3.3 for minimum ratio of vertical and horizontal reinforcement to gross concrete area, respectively, and ACI 14.3.5 for maximum bar spacing.

Design for axial force and bending.

ACI 14.4 requires that walls subjected to axial load or combined flexure and axial load shall be designed as compression members in accordance with ACI 10.2, 10.3, 10.10 through 10.14, 10.17, 14.2, and 14.3 unless the empirical design method of ACI 14.5 or the alternative design method of ACI 14.8 can be used. Clearly, both of these methods cannot be applied in this case, and the wall is designed in accordance with ACI 14.4.

Preliminary design indicates that 2-No. 4 vertical bars @ 18 in. in the web is not sufficient for the load combinations in Table 3-12. Figure 3-8 contains the interaction diagram of the wall reinforced with 2-No. 4 vertical bars @ 12 in. and 4-No. 10 bars at each end of the wall. As seen from the figure, the wall is adequate for the load combinations in Table 3-12.

Splice length of reinforcement.

Class B lap splices are utilized for the vertical bars in the wall. No splices are required for the No. 4 horizontal bars, since full length bars weigh approximately $0.668 \times 23 = 15.4$ lbs. and are easily installed.

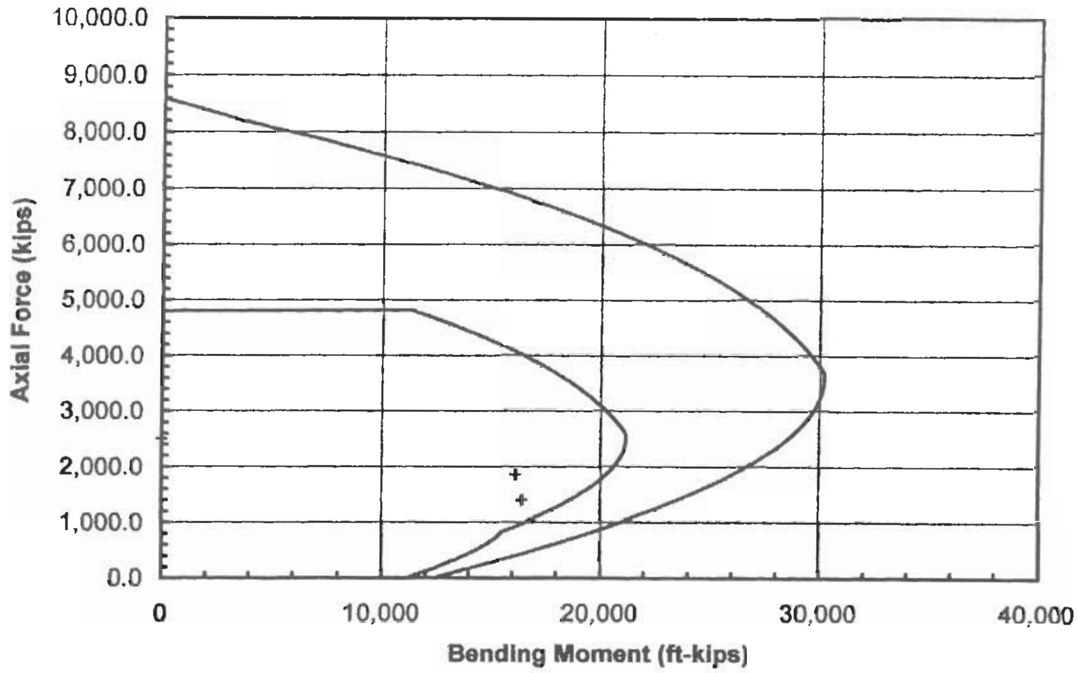

Figure 3-8 Design and Nominal Strength Interaction Diagrams for the Shear Wall Along Line 4 (SDC A)

For the No. 10 vertical bars:

$$\frac{\ell_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 1.0 for No. 7 and larger bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$= \begin{cases} 0.75 + 0.5 + \dfrac{1.27}{2} = 1.9 \text{ in.} \quad \text{(governs)} \\ \dfrac{1}{2} \times 5.2 = 2.6 \text{ in.} \end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b}=\frac{1.9+0}{1.27}=1.5<2.5$$

Therefore,

$$\frac{\ell_d}{d_b}=\frac{3}{40}\times\frac{60{,}000}{\sqrt{4{,}000}}\times\frac{1.0\times1.0\times1.0\times1.0}{1.5}=47.4$$

$$\ell_d=47.4\times1.27=60.2 \text{ in.}=5.0 \text{ ft}$$

Class B splice length = $1.3\ell_d$ = 6.5 ft

Use a 6 ft-6 in. splice length for the No. 10 bars. In lieu of lap splices for these large bars, mechanical or welded splices can be used (ACI 12.14.3).

For the No. 4 bars:

$$\frac{\ell_d}{d_b}=\frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c+K_{tr}}{d_b}\right)}$$

where α = reinforcement location factor = 1.0 for other than top bars

β = coating factor = 1.0 for uncoated reinforcement

γ = reinforcement size factor = 0.8 for No. 6 and smaller bars

λ = lightweight aggregate concrete factor = 1.0 for normal weight concrete

c = spacing or cover dimension

$$=\begin{cases}0.75+0.5+\dfrac{0.5}{2}=1.5 \text{ in.} & \text{(governs)}\\[2ex] \dfrac{1}{2}\times12=6.0 \text{ in.}\end{cases}$$

K_{tr} = transverse reinforcement index = 0

$$\frac{c+K_{tr}}{d_b}=\frac{1.5+0}{0.5}=3.0>2.5, \text{ use } 2.5$$

Therefore,

$$\frac{\ell_d}{d_b} = \frac{3}{40} \times \frac{60{,}000}{\sqrt{4{,}000}} \times \frac{1.0 \times 1.0 \times 0.8 \times 1.0}{2.5} = 22.8$$

$$\ell_d = 22.8 \times 0.5 = 11.4 \text{ in.} < 12 \text{ in., use } 12 \text{ in.}$$

$$\text{Class B splice length} = 1.3\ell_d = 1.3 \text{ ft}$$

Use a 1 ft-4 in. splice length for the No. 4 bars.

The No. 4 horizontal bars are developed by providing standard 90-degree hooks at the ends of the bars.

Reinforcement details for the shear wall along line 4 are shown in Figure 3-9.

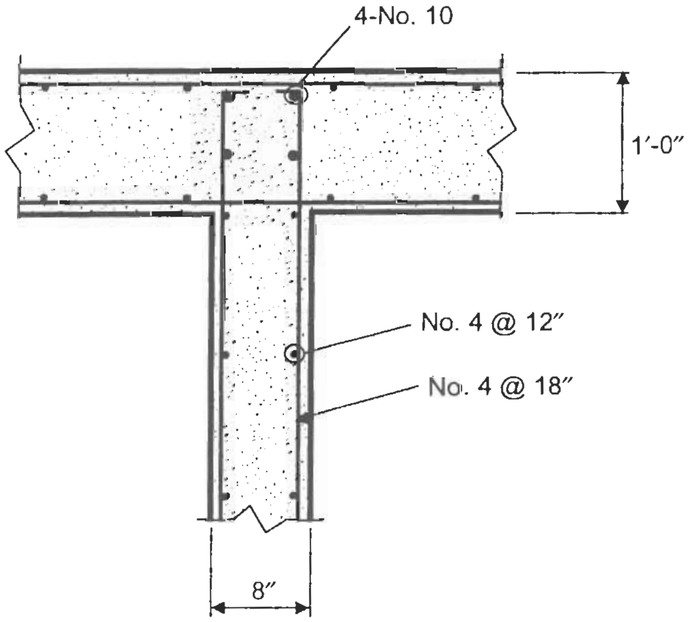

Figure 3-9 Reinforcement Details for Shear Wall Along Line 4 (SDC A)

3.3 DESIGN FOR SDC B

To illustrate the design requirements for Seismic Design Category (SDC) B, the residential building in Figure 3-1 is assumed to be located in Atlanta, GA. Typical structural members are designed and detailed for combined effects of gravity, wind, and seismic forces.

included in the analysis. Also, the provided reinforcement ratio is within the allowable range of 1% and 8% (ACI 10.9.1).

Transverse reinforcement.

Transverse reinforcement requirements must satisfy ACI 7.10.5. The same transverse reinforcement determined in Section 3.2.4.3 for SDC A can be used for SDC B.

Note that since the clear column height to maximum plan dimension of the column is $[(14 \times 12) - 9]/22 = 7.2 > 5$, the column need not be designed for shear in accordance with ACI 21.10.3 (IBC 1910.3.1).

Reinforcement details for the column are the same as for SDC A (see Figure 3-7).

3.3.4.4 Design of Shear Wall on Line 4

Table 3-18 contains a summary of the design axial forces, bending moments, and shear forces at the base of the wall.

Table 3-18 Summary of Design Axial Forces, Bending Moments, and Shear Forces at Base of Shear Wall on Line 4 (SDC B)

Load Case	Axial Force (kips)	Bending Moment (ft-kips)	Shear Force (kips)
Dead (D)	1,549	0	0
Live (L)	195	0	0
Seismic (Q_E)	0	± 14,465	± 204
Load Combinations			
$1.4D + 1.7L$	2,500	0	0
$1.24D + 0.5L + Q_E$	2,018	14,465	204
$0.86D + Q_E$	1,332	-14,465	-204

Design for shear.

Calculations for shear strength for SDC B are similar to those given in Section 3.2.4.4 for SDC A. It is determined that 2-No. 4 horizontal and vertical bars spaced at 18 in. satisfy the shear strength requirements of ACI 11.10 for SDC B, as well as the requirements of ACI 14.3.2 and 14.3.3 for minimum ratio of vertical and horizontal reinforcement to gross concrete area, respectively, and ACI 14.3.5 for maximum bar spacing.

Design for axial force and bending.

Preliminary design indicates that 2-No. 4 vertical bars @ 18 in. in the web is not sufficient for the load combinations in Table 3-18.