gravity and wind loads. The planning and layout of the structure, however, must be undertaken with due consideration of the special requirements for earthquake-resistant design. Thus, modifications in both configuration and proportions to anticipate earthquake-related requirements should be incorporated at the outset into the basic design for gravity and wind. Essential to the finished design is particular attention to details that can often mean the difference between a severely damaged structure and one with only minor, repairable damage.

10.5.2 Example Designs of Elements of a 12-Story Frame-Shear Wall Building

The application of the earthquake-resistant design provisions of IBC-2000 with respect to design loads and those of ACI 318-95[(10-10)] relating to proportioning and detailing of members will be illustrated for representative elements of a 12-story frame—shear wall building located in seismic zone 4. The use of the seismic design load provisions in IBC-2000, is based on the fact that it represents the more advanced version, in the sense of incorporating the latest revisions reflecting current thinking in the earthquake engineering profession.

The typical framing plan and section of the structure considered are shown in Figure 10-48a[c] and b, respectively. The columns and structural walls have constant cross-sections throughout the height of the building. The floor beams and slabs also have the same dimensions at all floor levels. Although the dimensions of the structural elements in this example are within the practical range, the structure itself is hypothetical and has been chosen mainly for illustrative purposes. Other pertinent design data are as follows:

Service loads — vertical:

- Live load:
  Basic, 50 lb/ft^2.
  Additional average uniform load to allow for heavier basic load on corridors, 25 lb/ft^2.
  Total average live load, 75 lb/ft^2.
  Roof live load = 20 lb/ft^2
- Superimposed dead load:
  Average for partitions 20 lb/ft^2.
  Ceiling and mechanical 10 lb/ft^2.
  Total average superimposed dead load, 30 lb/ft^2.

Material properties:

- Concrete:
  $f_c' = 4000$ lb/in.^{2} $w_c = 145$ lb/ft^3.
- Reinforcement:
  $f_y = 60$ ksi.

Determination of design lateral forces

On the basis of the given data and the dimensions shown in Figure 10-48, the weights that may be considered lumped at a floor level (including that of all elements located between two imaginary parallel planes passing through mid-height of the columns above and below the floor considered) and the roof were estimated and are listed in Tables 10-1 and 10-2. The calculation of base shear V, as explained in Chapter 5, for the transverse and longitudinal direction is shown at the bottom of Tables 10-1 and 10-2. For this example, it is assumed that the building is located in Southern California with values of S_s and S_1 of 1.5 and 0.6 respectively. The site is assumed to be class B (Rock) and the corresponding values of F_a and F_b are 1.0. On this basis, the design spectral response acceleration parameters S_{DS} and S_{MI} are 1.0 and 0.4 respectively. At this level of design parameters, the building is classified as Seismic Group D according to IBC-2000. The building consist of moment resisting frame in the longitudinal direction, and dual system consisting of wall and moment resisting frame in the transverse direction. Accordingly, the response modification factor, R, to be used is 8.0 in both directions.

[c] Reproduced, with modifications, from Reference 10-78, with permission from Van Nostrand Reinhold Company.

Calculation of the undamped (elastic) natural periods of vibration of the structure in the transverse direction (N-S)

As shown in Figure 10-49 using the story weights listed in Table 10-1 and member stiffnesses based on gross concrete sections, yielded a value for the fundamental period of 1.17 seconds. The mode shapes and the corresponding periods of the first five modes of vibration of the structure in the transverse direction are shown in Figure 10-49. The fundamental period in the longitudinal (E-W) direction was 1.73 seconds. The mode shapes were calculated using the Computer Program ETABS[(10-66)], based on three dimensional analysis. In the computer model, the floors were assumed to be rigid. Rigid end offsets were assumed at the end of the members to reflect the actual behavior of the structure. The portions of the slab on each side of the beams were considered in the analysis based on the ACI 318-95 provisions. The structure was assumed to be fixed at the base. The two interior walls were modeled as panel elements with end piers (26x26 in.). The corresponding values of the fundamental period determined based on the approximate formula given in IBC-2000 were 0.85 and 1.27 seconds in the N-S and the E-W directions respectively. However, these values can be increased by 20% provided that they do not exceed those determined from analysis. On this basis, the value of T used to calculate the base shears were 1.02 and 1.52 seconds in the N-S and the E-W directions respectively.

The lateral seismic design forces acting at the floor levels, resulting from the distribution of the base shear in each principal direction are also listed in Tables 10-1 and 10-2.

For comparison, the wind forces and story shears corresponding to a basic wind speed of 85 mi/h and Exposure B (urban and suburban areas), computed as prescribed in ASCE 7-95, are shown for each direction in Tables 10-1 and 10-2.

Lateral load analysis of the structure along each principal direction, under the respective seismic and wind loads, based on three dimensional analysis were carried out assuming no torsional effects.

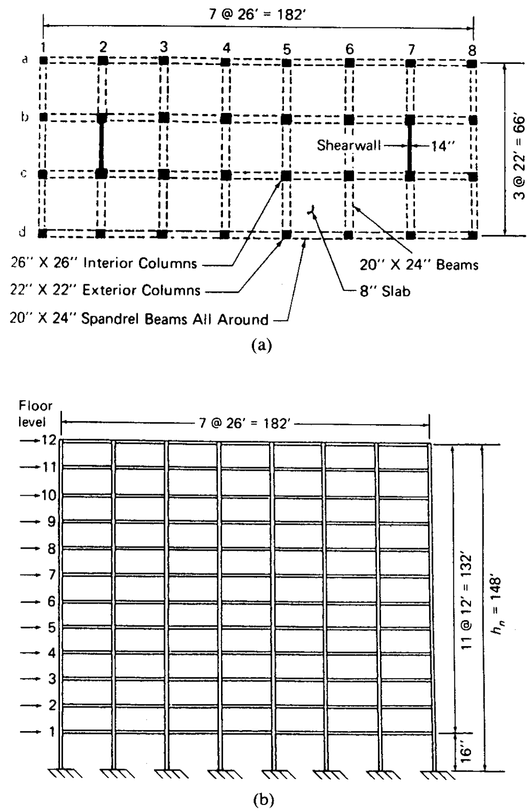


Figure 10-48. Structure considered in design example. (a) Typical floor framing plan. (b) Longitudinal section

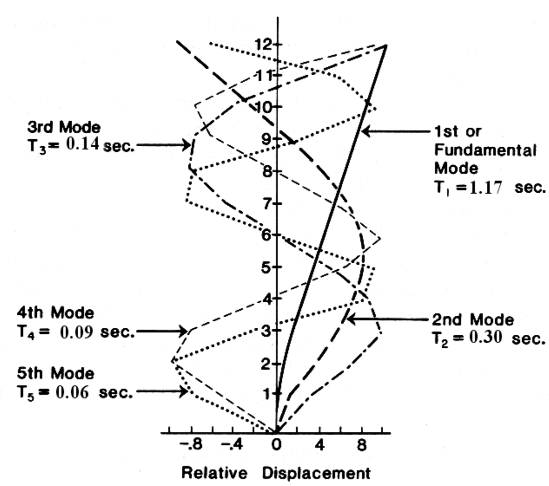


Figure 10-49. Undamped natural modes and periods of vibration of structure in transverse direction

Table 10-1. Design Lateral Forces in Transverse (Short) Direction (Corresponding to Entire Structure).

Floor Level	Height, h_x, ft	h_x^k k=1.26	story weight, w_x, kips	Seismic forces				Wind forces		
				$w_x h_x^k$ ft-kips x10^3	C_{vx}	Lateral force, F_x kips	Story shear ΣF_x, kips	wind pressure lbs/ft^2	lateral force H_x, kips	Story shear ΣH_x, kips
Roof	148	543	2100	1140	0.162	208.8	208.8	21.1	23.0	23.0
11	136	488	2200	1073	0.152	196.0	404.8	20.9	45.6	68.9
10	124	434	2200	955	0.135	174.0	578.8	20.5	44.8	113.4
9	112	382	2200	840	0.120	154.7	733.5	20.2	44.1	157.5
8	100	331	2200	728	0.103	132.8	866.3	19.8	43.2	200.7
7	88	282	2200	620	0.088	113.4	979.7	19.4	42.4	243.1
6	76	234	2200	515	0.073	94.1	1073.8	18.9	41.3	284.4
5	64	189	2200	415	0.059	76.1	1149.9	18.4	40.2	324.6
4	52	145	2200	320	0.045	58.0	1207.9	17.8	38.9	363.5
3	40	104	2200	230	0.033	42.5	1250.4	17.1	37.3	400.8
2	28	67	2200	147	0.021	27.1	1277.5	16.2	35.4	436.2
1	16	33	2200	72	0.010	12.9	1290.4	14.9	38.0	474.2
Total		-	26,300	7055	-	1290.4	-	-	474.2	-

Calculation of Design Base Shear in Transverse (Short) Direction

Base shear, $V = C_S W$ where $0.1\, S_{D1}\, I < C_S = \dfrac{S_{DS}}{R/I} < \dfrac{S_{D1}}{T(R/I)}$

$S_{DS} = 2/3\, S_{MS}$, where $S_{MS} = F_a S_S = 1.0 \times 1.5 = 1.5$ and $S_{D1} = 2/3\, S_{MI}$
where $S_{MI} = F_v S_1 = 1.0 \times 0.6 = 0.6$; $S_{DS} = 1.0$, $S_{D1} = 0.4$; R=8; I=1.0; $T = C_T h_n^{3/4} = 0.02 \times (148)^{3/4} = 0.849$ sec; T can be increased by a factor of 1.2 but should be less than the calculated value (i.e. 1.17 sec). $\therefore T = 0.849 \times 1.2 = 1.018 < 1.17$

$$0.1 \times 0.4 < C_S = \frac{1.0}{8/1} < \frac{0.4}{1.018(8/1)}$$

$0.04 < C_S = 0.125 < 0.0491 \;\therefore$ use $C_S = 0.0491$
<u>V = 0.0491 x 26,300 = 1290.4 kips</u>

Table 10-2. Design Lateral Forces in Longitudinal Direction (Corresponding to Entire Structure).

Floor Level	Height, h_x, ft	h_x^k k=1.51	story weight, w_x, kips	Seismic forces				Wind forces		
				$w_x h_x^k$ ft-kips x10^3	C_{vx}	Lateral force, F_x, kips	Story shear ΣF_x, kips	wind pressure lbs/ft^2	lateral force H_x, kips	Story shear ΣH_x, kips
Roof	148	1893	2100	3975	0.178	154.5	154.5	17.2	6.8	6.8
11	136	1666	2200	3665	0.164	142.4	296.9	17.0	13.5	20.3
10	124	1449	2200	3188	0.142	123.3	420.2	16.6	13.1	33.4
9	112	1243	2200	2734	0.122	105.9	526.1	16.3	12.9	46.3
8	100	1047	2200	2304	0.103	89.4	615.5	15.9	12.6	58.9
7	88	863	2200	1899	0.085	73.8	689.3	15.5	12.3	71.2
6	76	692	2200	1522	0.068	59.0	748.3	15.0	12.0	83.2
5	64	534	2200	1174	0.052	45.1	793.4	14.5	11.5	94.7
4	52	390	2200	858	0.038	33.0	826.4	13.9	11.0	105.7
3	40	263	2200	578	0.026	22.6	849.0	13.2	10.5	116.2
2	28	153	2200	337	0.015	13.0	862.0	12.3	9.7	125.9
1	16	66	2200	145	0.006	5.2	867.2	11.0	10.2	136.1
Total		-	26,300	22,379	-	867.2	-	-	136.1	-

In longitudinal direction, C_t (for reinforced concrete moment resisting frames) = 0.03;
$T = C_t\,(h_n)^{3/4} = (0.03)\,(148) = 1.27$; T can be increased by a factor of 1.2,
$\therefore\; T = 1.2 \times 1.27 = 1.524 < 1.73$

$$0.1 \times 0.4 < C_S = \frac{1.0}{8/1} < \frac{0.4}{1.524(8/1)}$$

$0.04 < C_S = 0.125 < 0.0329 \therefore$ use $C_S = 0.0329$
<u>$V = 0.033 \times 26{,}300 = 867.2$ kips</u>

(a) Lateral displacements due to seismic and wind effects: The lateral displacements due to both seismic and wind forces listed in Tables 10-1 and 10-2 are shown in Figure 10-50 . Although the seismic forces used to obtain the curves of Figure 10-50 are approximate, the results shown still serve to draw the distinction between wind and seismic forces, that is, the fact that the former are external forces the magnitudes of which are proportional to the exposed surface, while the latter represent inertial forces depending primarily on the mass and stiffness properties of the structure. Thus, while the ratio of the total wind force in the transverse direction to that in the longitudinal direction (see Tables 10-1 and 10-2) is about 3.5, the corresponding ratio for the seismic forces is only 1.5. As a result of this and the smaller lateral stiffness of the structure in the longitudinal direction, the displacement due to seismic forces in the longitudinal direction is significantly greater than that in the transverse direction. By comparison, the displacements due to wind are about the same for both directions. The typical deflected shapes associated with predominantly cantilever or flexure structures (as in the transverse direction) and shear (open-frame) buildings (as in the longitudinal direction) are evident in Figure 10-50. The average deflection indices, that is, the ratios of the lateral displacement at the top to the total height of the structure, are 1/5220 for wind and 1/730 for seismic

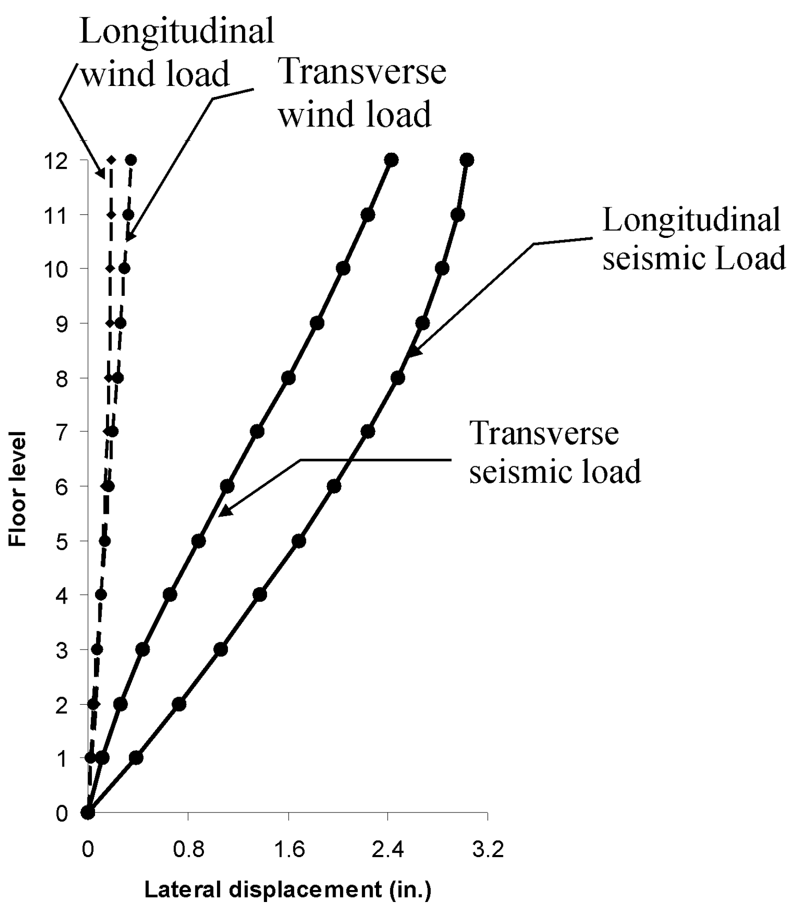


Figure 10-50. Lateral displacements under seismic and wind loads.

loads in the transverse direction. The corresponding values in the longitudinal direction are 1/9350 for wind and 1/590 for seismic loads. It should be noted that the analysis for wind was based on uncracked sections whereas that for seismic was based on cracked sections. The use of cracked section moment of inertia is a requirement by IBC-2000 for calculation of drift due to earthquake loading. However, under wind loading, the stresses within the structure in this particular example are within the elastic range as can also be observed from the amount of lateral deflections. As a result, the amount of cracking within the members is expected to be insignificant. However, for the case of seismic loading, the members are expected to deform well into inelastic range of response under the design base shear. To consider the effects of cracked sections due to seismic loads, the moments of inertia of beams, columns and walls were assumed to be 0.5, 0.7 and 0.5 of the gross concrete sections respectively.

(b) Drift requirements: IBC-2000 requires that the design story drift shall not exceed the allowable limits. In calculating the drift limits, the effect of accidental torsion was considered in the analysis. On this basis, the mass at each floor level was assumed to displace from the calculated center of mass a distance equal to 5% of the building dimension in each direction. Table 10-3 shows the calculated displacements and the corresponding story drifts in both E-W and N-S directions. To determine the actual story drift, the calculated drifts were amplified using the C_d factor of 6.5 according to IBC-2000. These increased drifts account for the total anticipated drifts including the inelastic effects. The allowable drift limit based on IBC-2000 is 0.025 times the story height which corresponds to 3.6 in. and 4.8 in. at a typical floor and first floor respectively. The calculated values of drift are less than these limiting values. It is to be noted that using IBC-2000 provisions, it is permissible to use the computed fundamental period of the structure without the upper bound limitation when determining the story drifts limits. However, the drift values shown are based on the calculated values of the fundamental period based on the code limits. Since the calculated drifts are less than the allowable values, further analysis based on the adjusted value of period was not necessary. In addition, the P-Δ effect need not to be considered in the analysis when the stability coefficient as defined by IBC-2000 is less than a limiting value. For the 12-story structure, the effect of P-Δ was found to be insignificant.

(c) Load Combinations: For design and detailing of structural components, IBC-2000 requires that the effect of seismic loads to be combined with dead and live loads. The loading combinations to be used are those prescribed in ASCE-95 as illustrated in Equation (10-2) except that the effect of seismic loads are according to IBC-2000 as defined in Equation (10-3).

To consider the extent of structural redundancy inherent in the lateral-force-resisting system, the reliability factor, ρ, is defined as follows for structures in seismic design category D as defined by IBC-2000:

$$\rho = 2 - \frac{20}{r_{\max}\sqrt{A_x}}$$

where

r_{max} = the ratio of the design story shear resisted by the single element carrying the most shear force in the story to the total story shear, for a given direction of loading. For shear walls, r_{max} is defined as the shear in the most heavily loaded wall multiplied by $10/l_w$, divided by the story shear (l_w is the wall length)

A_x = the floor area in square feet of the diaphragm level immediately above the story

Table 10-3. Lateral displacements and Inerstory drifts Due to Seismic Loads (in.).

	E-W Direction			N-S Direction		
Story Level	displacement	drift	drift × C_d^*	displacement	drift	drift × C_d^*
Roof	3.03	0.07	0.45	2.43	0.19	1.24
11	2.96	0.12	0.78	2.24	0.20	1.30
10	2.84	0.16	1.04	2.04	0.21	1.37
9	2.68	0.20	1.30	1.83	0.23	1.50
8	2.48	0.24	1.56	1.60	0.24	1.56
7	2.24	0.27	1.76	1.36	0.24	1.56
6	1.97	0.28	1.82	1.12	0.23	1.50
5	1.69	0.31	2.02	0.89	0.23	1.50
4	1.38	0.32	2.08	0.66	0.22	1.43
3	1.06	0.33	2.15	0.44	0.18	1.17
2	0.73	0.34	2.21	0.26	0.15	0.98
1	0.39	0.39	2.54	0.11	0.11	0.72

* $C_d = 6.5$

When calculating the reliability factor for dual systems such as the frame wall structure in the N-S direction, it can be reduced to 80 percent of the calculated value determined as above. However, this value can not be less that 1.0.

In the N-S direction, the most heavily single element for shear is the shear wall. Table 10-4 shows the calculated values for *r* over the 2/3 height of the structure. The maximum value of *r* occurs at the base of the structure where the shear walls carry most of the shear in the N-S direction. On this basis, the maximum value of ρ determined was 1.0.

The load combinations used for the design based on ρ= 1.0 and S_{DS}=1.0 by combining

*Table 10-4.*Element story shear ratios for redundancy factor in N-S direction.

Story Level	V_i = shear force in wall	V_i x $10/L_w$	story shear	r_i
8	189	78	886	0.09
7	234	97	980	0.10
6	275	114	1074	0.11
5	317	131	1150	0.11
4	359	149	1208	0.12
3	408	169	1250	0.14
2	448	185	1278	0.15
1	570	236	1290	0.18

$$\rho = 2 - \frac{20}{r_{max}\sqrt{A_x}}$$

$$\rho = 2 - \frac{20}{0.18 \times \sqrt{66 \times 182}} = 0.99 \quad \text{but} \quad \rho_{min} = 1.0$$

equations (10-2) and (10-3) are as follows:

$$U = \begin{cases} 1.2\,\mathrm{D} + 1.6\,\mathrm{L} + 0.5\,\mathrm{L_r} \\ 1.4\,\mathrm{D} \pm 1.0\,\mathrm{Q_E} + 0.5\,\mathrm{L} \\ 0.7\,\mathrm{D} \pm 1.0\,\mathrm{Q_E} \end{cases} \tag{10-8}$$

The 3-D structure was analyzed using the above load combinations. The dead and live loads were applied to the beams based on tributary areas as shown in Figure 10-51. The effect of accidental torsion was also considered in the analysis.

To protect the building against collapse, IBC-2000 requires that in dual systems, the moment resisting frames be capable to resist at least 25% of prescribed seismic forces. For this reason, the building in the N-S direction was also subjected to 25% of the lateral forces described above without including the shear walls.

An idea of the distribution of lateral loads among the different frames making up the structure in the transverse direction may be obtained from Table 10-5, which lists the portion of the total story shear at each level resisted by each of the three groups of frames. The four interior frames along lines 3, 4,5, and 6 are referred to as Frame T-1, while the Frame T-2 represents the two exterior frames along lines 1 and 8. The third frame, T-3 represents the two identical frame-shear- wall systems along lines 2 and 7. Note that at the top (12th floor level), the lumped frame T-1 takes 126% of the total story shear. This reflects the fact that in frame-shear-wall systems of average proportions, interaction between frame and wall under lateral loads results in the frame "supporting" the wall at the top, while at the base most of the horizontal shear is resisted by

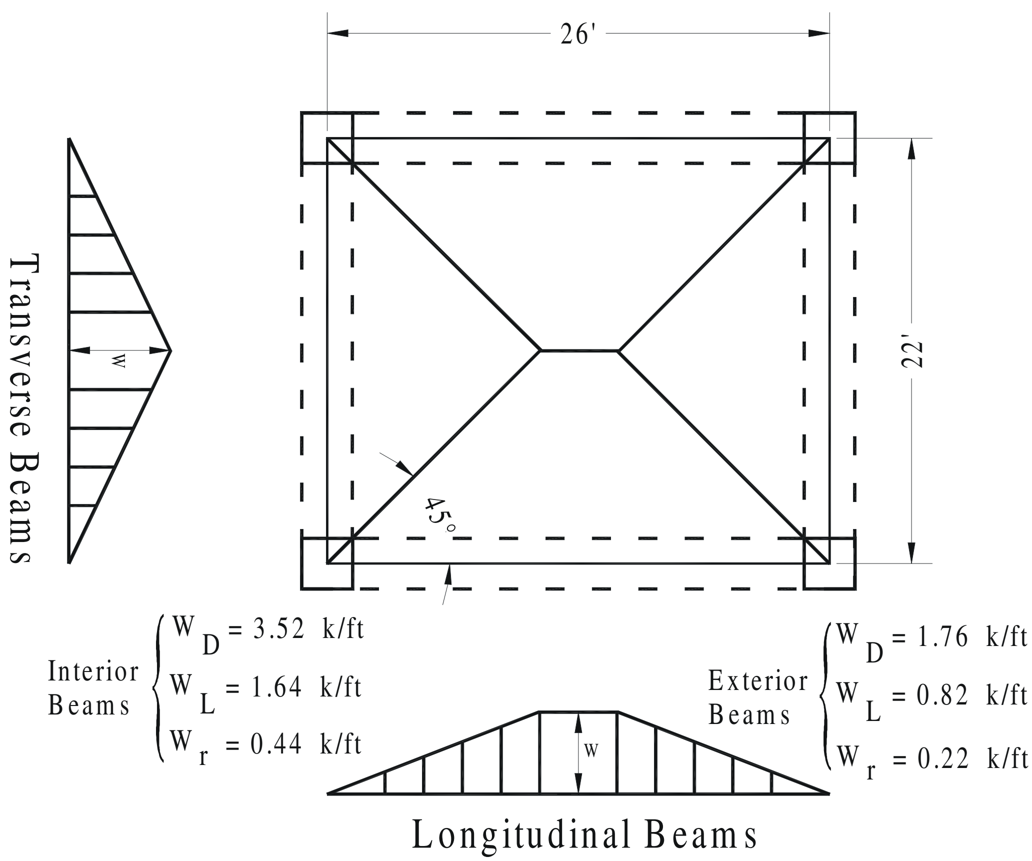


Figure 10-51. Tributary area for beam loading.

Table 10-5. Distribution of Horizontal Seismic Story Shears among the Three Transverse Frames.

Story Level	Frame T-1 (4 interior frames)		Frame T-2 (2 exterior frames)		Frame T-3 (2 interior frames with shear walls)		Total story shear, kips
	Story shear	% of total	Story shear	% of total	Story shear	% of Total	
Roof	263.6	126	102.1	49	-156.9	-75	208.8
11	228.5	56	90.3	22	86.0	21	404.8
10	259.9	45	101.9	18	216.8	37	578.8
9	282.5	39	110.4	15	340.6	46	733.5
8	303.6	35	117.3	14	445.4	51	866.3
7	317.3	32	123.6	13	538.8	55	979.7
6	324.0	30	125.6	12	624.2	58	1073.8
5	320.0	28	124.0	11	705.9	61	1149.9
4	303.2	25	117.9	10	786.8	65	1207.9
3	269.6	22	104.4	8	876.4	70	1250.4
2	225.1	18	86.4	7	966.0	75	1277.5
1	96.0	7	34.8	3	1159.6	90	1290.4

the wall. Table 10-5 indicates that for the structure considered, the two frames with walls take 90% of the shear at the base in the transverse direction.

To illustrate the design of two typical beams on the sixth floor of an interior frame, the results of the analysis in the transverse direction under seismic loads have been combined, using Equation 10-8, with results from a gravity-load analysis . The results are listed in Table 10-6. Similar values for typical exterior and interior columns on the second floor of the same interior frame are shown in Table 10-7. Corresponding design values for the structural wall section at the first floor of frame on line 3 (see Figure 10-48) are listed in Table 10-8. The last column in Table 10-8 lists the axial load on the boundary elements (the 26 × 26-in, columns forming the flanges of the structural walls) calculated according to the ACI requirement that these be designed to carry all factored loads on the walls, including self-weight, gravity loads, and vertical forces due to earthquake-induced overturning moments. The loading condition associated with this requirement is illustrated in Figure 10-45. In both Tables 10-7 and 10-8, the additional forces due to the effects of horizontal torsional moments corresponding to the minimum IBC-2000 -prescribed eccentricity of 5% of the building dimension perpendicular to the direction of the applied forces have been included.

Table 10-6. Summary of design moments for typical beams on sixth floor of interior transverse frames along lines 3 through 6 (Figure 10-48a).

$$U = \begin{cases} 1.2D + 1.6L + 0.5L_r & (9-8a) \\ 1.4D + 0.5L \pm 1.0Q_E & (9-8b) \\ 0.7D \pm 1.0Q_E & (9-8c) \end{cases}$$

BEAM AB		Design moment, ft-kips A	Midspan of AB	B
9-8 a		-76	$\boxed{+100}$	-202
9-8 b	Sides way to right	+91	+83	$\boxed{-326}$
	Sides way to left	$\boxed{-213}$	+85	-19
9-8 c	Sides way to right	$\boxed{+127}$	+35	-229
	Sides way to left	-177	+37	$\boxed{+79}$
BEAM BC		**Design moment, ft-kips B**	**Midspan of BC**	**C**
9-8 a		-144	$\boxed{+92}$	-144
9-8 b	Sides way to right	-41	+77	$\boxed{-282}$
	Sides way to left	$\boxed{-282}$	+77	-41
9-8 c	Sides way to right	$\boxed{+110}$	+33	-213
	Sides way to left	-213	+33	$\boxed{+110}$

It is pointed out that for buildings located in seismic zones 3 and 4 (i.e., high-seismic-risk areas), the detailing requirements for ductility prescribed in ACI Chapter 21 have to be met even when the design of a member is governed by wind loading rather than seismic loads.

2. *Design of flexural member AB.* The aim is to determine the flexural and shear reinforcement for the beam *AB* on the sixth floor of a typical interior transverse frame. The critical design (factored) moments are shown circled in Table 10-6. The beam has dimensions b = 20 in. and d = 21.5 in. The slab is 8 in. thick, $f'_c = 4000$ lb/in.^{2} and $f_y = 60{,}000$ lb/in.^{2}

In the following solution, the boxed-in section numbers at the right-hand margin correspond to those in ACI 318-95 .

(a) Check satisfaction of limitations on section dimensions:

$$\frac{width}{depth} = \frac{20}{21.5}$$

$$= 0.93 > 0.3 \text{ O.K } \boxed{21.3.1.3}$$

$$\boxed{21.3.1.4}$$

$$width = 20\,in. \begin{cases} \geq 10 \text{ in.} \qquad \text{O.K.} \\ \leq \text{(width of suuporting column} \\ \quad + 1.5 \times \text{depth of beam} \\ = 26 + 1.5(21.5) = 58.25 \text{ in.} \quad \text{O.K.} \end{cases}$$

Table 10-7. Summary of design moments and axial loads for typical columns on second floor of interior transverse frames along lines 3 through 6 (Figure 10-48a).

$$U = \begin{cases} 1.2D + 1.6L + 0.5L_r & (9-8a) \\ 1.4D + 0.5L \pm 1.0Q_E & (9-8b) \\ 0.7D \pm 1.0Q_E & (9-8c) \end{cases}$$

		Exterior Column A			Interior Column B		
		Axial load,	Moment, ft-kips		Axial load,	Moment, ft-kips	
		kips	Top	Kips	kips	Top	Bottom
9-8 a		$\boxed{-1076}$	-84	+94	$\boxed{-1907}$	+6	-12
9-8 b	Sides way to right	-806	-33	+25	-1630	+73	-108
	Sides way to left	-1070	$\boxed{-110}$	$\boxed{+134}$	-1693	$\boxed{-94}$	$\boxed{+119}$
9-8 c	Sides way to right	-280	+8	-20	-698	+79	-111
	Sides way to left	-544	-69	+88	-760	-88	+116

Table 10-8. Summary of design loads on structural wall section at first floor level of transverse frame along line 2 (or 7) (Figure *10*-48a).

$$U = \begin{cases} 1.2D + 1.6\,L + 0.5L_r & (9-8a) \\ 1.4D + 0.5L \pm 1.0\,Q_E & (9-8b) \\ 0.7\,D\ \pm 1.0\,Q_E & (9-8c) \end{cases}$$

	Design forces acting on entire structural wall			Axial load# on boundary element, kips
	Axial Load, kips	Bending (overturning) Moment, ft-kips	Horizontal shear, kips	
9-8 a	$\boxed{-5767}$	Nominal	Nominal	-2884
9-8 b	-5157	$\boxed{30469}$	$\boxed{651}$	$\boxed{-3963}$
9-8 c	-2293	30469	651	-2531

Based on loading condition illustrated in Figure 10-45 @ bending moment at base of wall

(b) Determine required flexural reinforcement:

(1) Negative moment reinforcement at support *B:* Since the negative flexural reinforcement for both beams *AB* and *BC* at joint *B* will be provided by the same continuous bars, the larger negative moment at joint *B* will be used. In the following calculations, the effect of any compressive reinforcement will be neglected. From $C = 0.85f_c'ba = T = A_sf_y$,

$$a = \frac{A_s}{0.85f_c'b} = \frac{60A_s}{(0.85)(4)(20)} = 0.882A_s$$

$$M_u \le \phi M_n = \phi A_s f_y (d - a/2)$$

$$-(326)(12) = (0.90)(60)A_s \times [21.5 - (0.5)(0.882A_s)]$$

$$A_s^2 - 48.76A_s + 164.3 = 0$$

or

$$A_s = 3.64\ \text{in.}^2$$

Alternatively, convenient use may be made of design charts for singly reinforced flexural members with rectangular cross-sections, given in standard references.[10-79] *Use five No. 8 bars,* A_s*=3.95 in.*^{2} This gives a negative moment capacity at support *B* of ϕM_n = 351 ft-kips.

Check satisfaction of limitations on reinforcement ratio:

$$\rho = \frac{A_s}{bd} = \frac{3.95}{(20)(21.5)} \qquad \boxed{21.3.2.1}$$

$$= 0.0092$$

$$> \rho_{\min} = \frac{200}{f_y} = 0.0033$$

$$> \rho_{\min} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4000}}{60{,}000} = 0.0032$$

and $<\rho_{\max} = 0.025$ O.K.

(2) Negative moment reinforcement at support *A:*

M_u = 213 ft-kips

As at support *B*, $a = 0.882A_s$.

Substitution into

$$M_u = \phi A_s f_y (d - a/2)$$

yields A_s = 2.31 in.^{2}. *Use three No. 8 bars,* A_s = 2.37 *in.*^{2} This gives a negative moment capacity at support *A* of ϕM_n = 218 ft-kips.

(3)Positive moment reinforcement at supports: A positive moment capacity at the supports equal to at least 50% of the corresponding negative moment capacity is required, i.e., $\boxed{21.3.2.2}$

$$\min M_u \text{(at support A)} = \frac{218}{2} = 109\, ft - kips$$

which is less than M^+_{max} = 127 ft-kips at *A* (see Table 10-6), but greater than the required M_u^+ near midspan of AB (=100 ft-kips).

min M_u^+ (at support *B* for both spans *AB* and *BC)* $= \dfrac{351}{2} = 176\, ft - kips$

Note that the above required capacity is greater than the design positive moments near the mid-spans of both beams *AB* and *BC*.

Minimum positive/negative moment capacity at any section along beam AB or BC = 351/4 =87.8 ft-kips.

(4) Positive moment reinforcement at mid-span of beam AB- to be made continuous to supports: (with an effective T-beam section flange width = 52 in.)

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{60 A_s}{(0.85)(4)(52)} = 0.339 A_s$$

Substituting into

$$M_u = (127)(12) = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

yields A_s (required) = 1.35 in.^{2}. Similarly, corresponding to the required capacity at support *B*, M_u^+ = 163 ft-kips, we have A_s (required) =1.74 in.^{2}. *Use three No. 7 bars continuous through both spans. A_s = 1.80 in.^{2}* This provides a positive moment capacity of 172 ft-kips.

Check:

$$\rho = \frac{1.8}{(20)(21.5)} = 0.0042$$

$$> \rho_{\min} = \frac{200}{f_y} = 0.0033 \quad \text{O.K.} \quad \boxed{10.5.1}$$

$$> \rho_{\min} = \frac{3\sqrt{f_c'}}{f_y} = \frac{3\sqrt{4000}}{60{,}000}$$

(c) Calculate required length of anchorage of flexural reinforcement in exterior column:

Development length $l_{dh} \geq \begin{cases} f_y d_b / 65\sqrt{f_c'} \\ 8 d_b \\ 6 in. \end{cases}$ $\boxed{21.5.4.1}$

(plus standard 90° hook located in confined region of column). For the No. 8 (top) bars (bend radius, measured on inside of bar, $\geq 3d_b$ = 3.0 in.),

$$l_{dh} \geq \begin{cases} \dfrac{(60{,}000)(1.0)}{65\sqrt{4000}} = 15\, in. \\ (8)(1.0) = 8.0\, in \\ 6\, in. \end{cases}$$

For the No. 7 bottom bars (bend radius $\geq 3d_b$ = 2.7 in.), l_{dh} = <u>13 in.</u>

Figure 10-52 shows the detail of flexural reinforcement anchorage in the exterior column. Note that the development length l_{dh} is measured from the near face of the column to the far edge of the vertical 12-bar-diameter extension (see Figure 10-35).

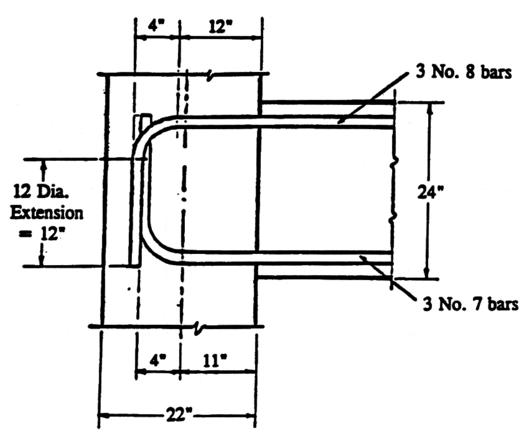


Figure 10-52. Detail of anchorage of flexural reinforcement in exterior column

(d) Determine shear-reinforcement requirements: Design for shears corresponding to end moments obtained by assuming the stress in the tensile flexural reinforcement equal to $1.25f_y$ and a strength reduction factor ϕ = 1.0, plus factored gravity loads (see Figure 10-16). Table 10-9 shows values of design end shears corresponding to the two loading cases to be considered. In the table,

$W_U = 1.2\ W_D + 1.6\ W_L = 1.2 \times 3.52 + 1.6 \times 1.64 = 6.85$ kips/ft

ACI Chapter 21 requires that the contribution of concrete to shear resistance, V_c, be neglected if the earthquake-induced shear force (corresponding to the probable flexural strengths at beam ends calculated using $1.25f_y$ instead of f_y and ϕ = 1.0) is greater than one-half the total design shear and the axial compressive force including earthquake effects is less than $A_g f'_c/20$. $\boxed{21.3.4.2}$

For sidesway to the right, the shear at end *B* due to the plastic end moments in the beam (see Table 10-9) is

$$V_b = \frac{230+477}{20} = 35.4\ kips.$$

which is approximately 50% of the total design shear, V_u = 69.6 kips. Therefore, the contribution of concrete to shear resistance can be considered in determining shear reinforcement requirements.
At right end *B*, V_u = 69.6 kips. Using

$$V_c = 2\sqrt{f'_c}\,b_w d = \frac{2\sqrt{4000}(20)(21.5)}{1000} = 54.4 kips$$

we have

$$\phi V_s = V_u - \phi V_c = 69.6 - 0.85 \times 54.4 = 23.4 kips \qquad \boxed{11.1.1}$$

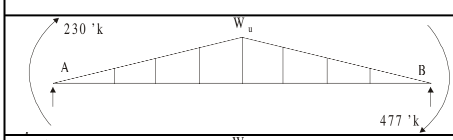
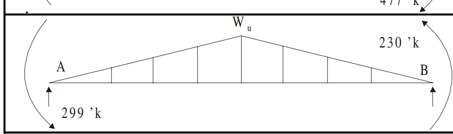
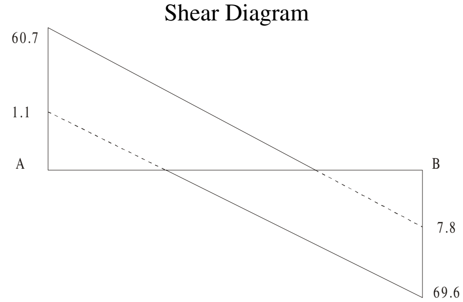
$$V_s = 27.5 kips$$

Required spacing of No. 3 closed stirrups (hoops), since A_v (2 legs) = 0.22 in.^{2}:

$$s = \frac{A_v f_y d}{V_s} = \frac{(0.22)(60)(21.5)}{27.5} = 10.3\text{ in.} \qquad \boxed{11.5.6.2}$$

Maximum allowable hoop spacing within distance 2d = 2(21.5) = 43 in. from faces of supports:

Table 10-9. Determination of Design Shears for Beam AB.

<table>
<tr><td rowspan="2">Loading</td><td colspan="2">$V_u = \frac{M_{pr}^A + M_{pr}^B}{l} \pm \frac{w_u l}{2}, (kips)$</td></tr>
<tr><td>A</td><td>B</td></tr>
<tr><td>
</td><td>1.1</td><td>69.6</td></tr>
<tr><td>
</td><td>60.7</td><td>7.8</td></tr>
<tr><td colspan="3">Shear Diagram

</td></tr>
</table>

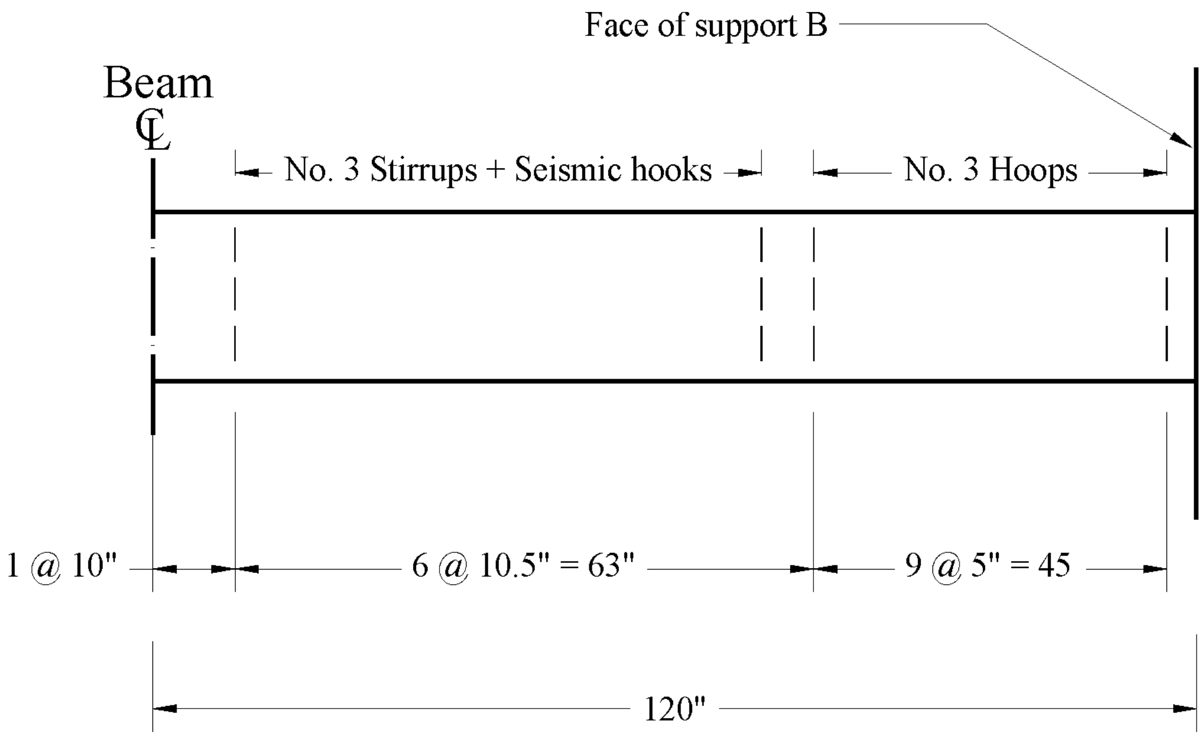


Figure 10-53. Spacing of hoops and stirrups in right half of beam AB

$$s_{\max} = \begin{cases} d/4 = 21.5/4 = 5.4 \; in. \\ 8 \times (\text{dia. of smallest long. bar}) \\ = 8(0.875) = 7 \text{ in.} \\ 24 \times (\text{dia. of hoop bars}) = 24(0.375) = 9 \text{ in.} \\ 12 \text{ in.} \end{cases}$$

$\boxed{21.3.3.2}$

Beyond distance 2d from the supports, maximum spacing of stirrups:

$$s_{\max} = d/2 = 10.75 \; in. \qquad \boxed{21.3.3.4}$$

Use No. 3 hoops/stirrups spaced as shown in Figure 10-53. The same spacing, turned around, may be used for the left half of beam AB.

Where the loading is such that inelastic deformation may occur at intermediate points within the span (e.g., due to concentrated loads at or near mid-span), the spacing of hoops will have to be determined in a manner similar to that used above for regions near supports. In the present example, the maximum positive moment near mid-span (i.e., 100 ft-kips, see Table 10-6) is much less than the positive moment capacity provided by the three No. 7 continuous bars (172 ft-kips). $\boxed{21.3.3.1}$

(e) Negative-reinforcement cut-off points: For the purpose of determining cutoff points for the negative reinforcement, a moment diagram corresponding to plastic end moments and 0.9 times the dead load will be used. The cut-off point for two of the five No. 8 bars at the top, near support *B* of beam *AB,* will be determined.

With the negative moment capacity of a section with three No. 8 top bars equal to 218 ft-kips (calculated using $f_s = f_y$ = 60 ksi and ϕ = 0.9), the distance from the face of the right support *B* to where the moment under the loading considered equals 218 ft-kips is readily obtained by summing moments about section *a—a* in Figure *10-*54 and equating these to -218 ft-kips. Thus,

$$51.8x - 477 - 3.2\frac{x^3}{60} = -218$$

Solution of the above equation gives x = 5.1 ft. Hence, two of the five No. 8 bars near support *B* may be cut off (noting that d = 21.5 in.> $12d_b = 12 \times 1.0$=12 in.) at 12.10.3

$$x + d = 5.1 + \frac{21.5}{12} = 6.9\,ft \quad \text{say } 7.0 \text{ ft}$$

from the face of the right support *B*. With l_{dh} (see figure 10-35) for a No. 8 top bar equal to 14.6 in., the required development length for such a bar with respect to the tensile force associated with the negative moment at support *B* is $l_d = 3.5\ l_{dh} = 3.5 \times 14.6/12$ = 4.3 ft < 7.0 ft. Thus, *the two No. 8 bars may be cut off 7.0 ft from the face of the interior support B.* 21.5.4.2

At end *A,* one of the three No. 8 bars may also be cut off at a similarly computed distance of 4.5 ft from the (inner) face of the exterior support *A. Two bars are required to run continuously along the top of the beam.* *21.3.2.3*

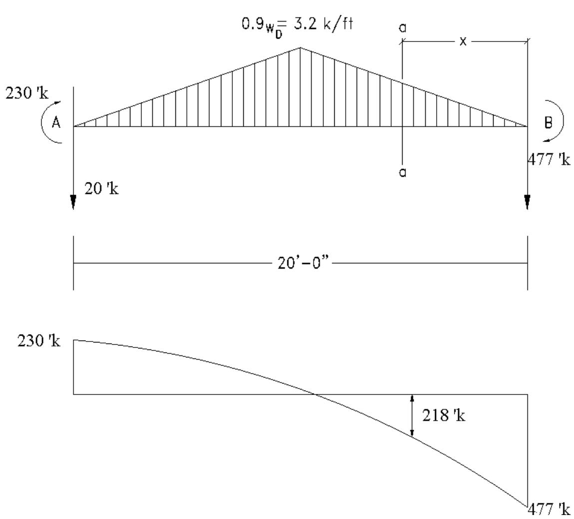


Figure 10-54. Moment diagram for beam AB

(f) Flexural reinforcement splices: Lap splices of flexural reinforcement should not be placed within a joint, within a distance 2d from faces of supports, or at locations of potential plastic hinging. Note that all lap splices have to be confined by hoops or spirals with a maximum spacing or pitch of *d*/4, or 4 in., over the length of the lap. 21.3.2.3

(1) Bottom bars, No. 7: The bottom bars along most of the length of the beam may be subjected to maximum stress. Steel area required to resist the maximum positive moment near midspan of 100 ft-kips (see Table 10-6), A_s = 1.05 in.^{2} Area provided by the three No. 7 bars = 3 (0.60) = 1.80 in.^{2}, so that

$$\frac{A_{s(provided)}}{A_{s(required)}} = \frac{1.80}{1.05} = 1.71 < 2.0$$

Since all of the bottom bars will be spliced near midspan, use a class B splice. 12.15.2

Required length of splice = 1.3 $l_d \geq$ 12 in.
where

$$l_d = \frac{3}{40}\frac{d_b f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\dfrac{c + k_{tr}}{d_b}\right)} \qquad 12.2.3$$

where
α = 1.0 (reinforcement location factor)
β = 1.0 (coating factor)
γ = 1.0 (reinforcement size factor)
λ = 1.0 (normal weight concrete)

$$c = 1.5 + 0.375 + \frac{0.875}{2} = 2.31 \qquad \text{(governs)}$$

(side cover, bottom bars)

or

$$c = \frac{1}{2}\left[\frac{20 - 2(1.5 + 0.375) - 0.875}{2}\right] = 3.84 \text{ in.}$$

(half the center to center spacing of bars)

$$k_{tr} = \frac{A_{tr} f_{yt}}{1500 sn}$$

where

A_{tr} = total area of hoops within the spacing s and which crosses the potential plane of splitting through the reinforcement being developed (ie. for 3#3 bars)
f_{yt} = specified yield strength of hoops = 60,000 psi
s = maximum spacing of hoops = 4 in.
n = number of bars being developed along the plane of splitting = 3

$$k_{tr} = \frac{(3 \times 0.11)60{,}000}{1500 \times 4.0 \times 3} = 1.1$$

$$\frac{c + k_{tr}}{d_b} = \left(\frac{2.31 + 1.1}{0.875}\right) = 3.90 > 2.5 \text{, use 2.5}$$

$$\therefore l_d = \frac{3}{40}\frac{0.875 \times 60{,}000}{\sqrt{4000}}\frac{1}{2.5} = 24.9 \text{ in.}$$

Required length of class B splice = 1.3 × 24.9 = <u>32.0 in.</u>

(2) Top bars, No. 8: Since the mid-span portion of the beam is always subject to a positive bending moment (see Table 10-6), splices in the top bars should be located at or near midspan. Required length of class A splice = 1.0 l_d.

For No. 8 bars,

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

where α = 1.3 (top bars), β = 1.0, γ = 1.0, and λ = 1.0

$$c = 1.5 + 0.375 + \frac{1.0}{2} = 2.375 \text{ in. (governs)}$$

$$c = \frac{1}{2}\left[\frac{20 - 2(1.5 + 0.375) - 1.0}{2}\right] = 3.81 \text{ in.}$$

k_{tr}= 1.1

$$\frac{c + k_{tr}}{d_b} = \frac{2.375 + 1.1}{1.0} = 3.5 > 2.5 \text{ use 2.5}$$

$$\therefore l_d = \frac{3}{40}\frac{1.0x60000}{\sqrt{4000}}\frac{1.3}{2.5} = 37.0 \text{ in.}$$

Required length of splice = 1.0 l_d = <u>37.0 in.</u>

(g) Detail of beam. See Figure 10-55.

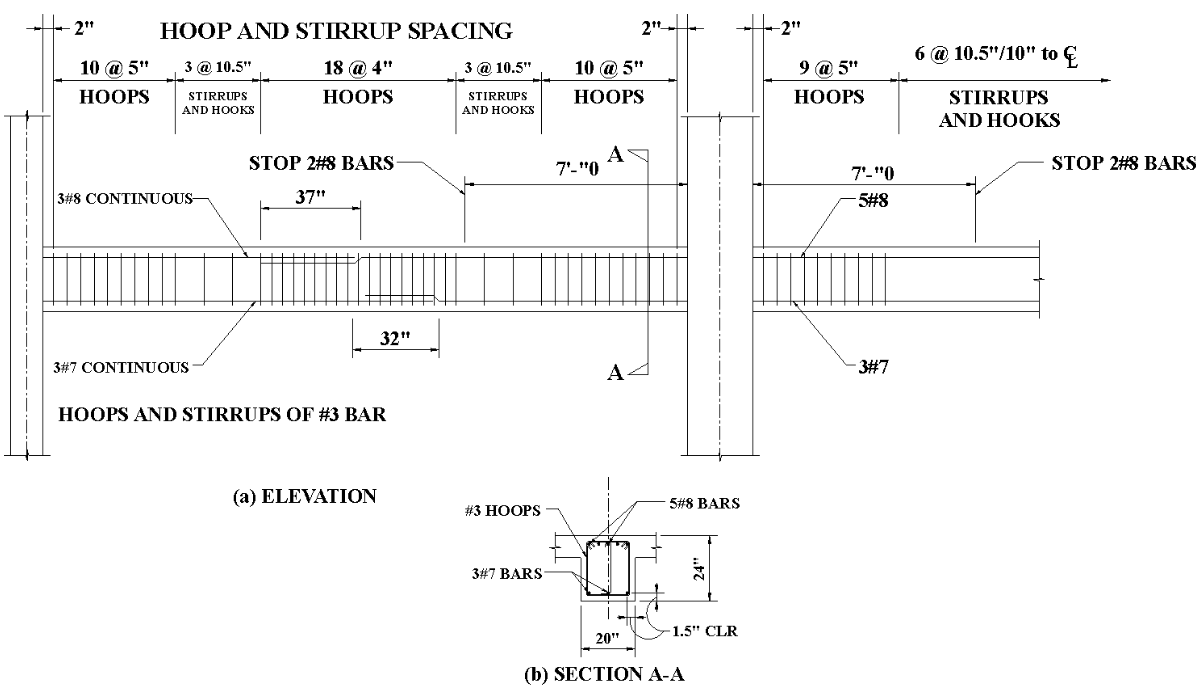


Figure 10-55. Detail of reinforcement for beam AB.

3. *Design of frame column A.* The aim here is to design the transverse reinforcement for the exterior tied column on the second floor of a typical transverse interior frame, that is, one of the frames in frame T-1 of Figure 10-48. The column dimension has been established as 22 in. square and, on the basis of the different combinations of axial load and bending moment corresponding to the three loading conditions listed in Table 10-7, *eight No. 9 bars arranged in a symmetrical pattern* have been found adequate.(10-80,10-81) Assume the same beam section framing into the column as considered in the preceding section. $f_c' = 4000 lb/in.^2$ *and* $f_y = 60{,}000 lb/in.^2$

From Table 10-7, P_u(max) = 1076 kips:

$$P_u(\max) = 1076 kips > \frac{A_g f_c'}{10} = \frac{(22)^2(4)}{10} = 194 kips$$

Thus, ACI Chapter 21 provisions governing members subjected to bending and axial load apply. $\boxed{21.4.1}$

(a) Check satisfaction of vertical reinforcement limitations and moment capacity requirements:

(1) Reinforcement ratio:

$0.01 \le \rho \le 0.06$

$$\rho = \frac{A_{st}}{A_g} = \frac{8(1.0)}{(22)(22)} = 0.0165 \qquad \text{O.K.}$$

$\boxed{21.4.3.1}$

(2) Moment strength of columns relative to that of framing beam in transverse direction (see Figure 10-56)

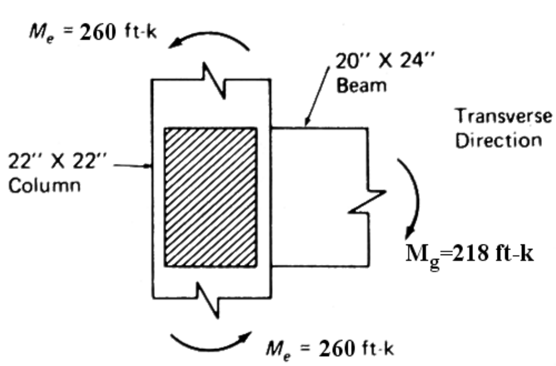


Figure 10-56. Relative flexural strength of beam and columns at exterior joint— transverse direction.

$$M_e(columns) \ge \frac{6}{5} M_g(beams) \qquad \boxed{21.4.2.2}$$

From Section 10.5.2, item 2, ϕM_n^- of the beam at A is 218 ft-kips, which may be mobilized during a sidesway to the left of the frame. From Table 10-7, the maximum axial load on column A at the second floor level for sidesway to the left is P_u = 1070 kips. Using the *P-M* interaction charts given in ACI SP-17A,(10-81) the moment capacity of the column section corresponding to $P_u = \phi P_n$ = 1070kips, f_c' = 4 ksi, f_y = 60 ksi, γ = 0.75 (γ = ratio of distance between centroids of outer rows of bars to dimension of cross-section in the direction of bending, and ρ = 0.0165 is obtained as $\phi M_n = M_e$ = 260 ft-kips). With the same size column above and below the beam, total moment capacity of columns = 2(260) = 520 ft-kips. Thus,

$$\sum M_e = 520 > \frac{6}{5} M_g = \frac{(6)(218)}{5}$$

= 262 ft-kips O.K.

(3) Moment strength of columns relative to that of framing beams in longitudinal direction (see Figure 10-57): Since the columns considered here are located in the center portion of the exterior longitudinal frames, the axial forces due to seismic loads in the longitudinal direction are negligible. (Analysis of the longitudinal frames under seismic loads indicated practically zero axial forces in the exterior columns of the four transverse frames represented by frame on line 1 in Figure 10-48) Under an axial load of 1.2 D + 1.6 L + 0.5 L_r = 1076 kips, the moment capacity of the column section with eight No. 9 bars is obtained as $\phi M_n = M_e$ = 258 ft-kips. If we assume a ratio for the negative moment reinforcement of about 0.0075 in the beams of the exterior longitudinal frames (b_w = 20 in., d = 21.5 in.), then

$$A_s = \rho b_w d \approx (0.0075)(20)(21.5)$$

= 3.23 in.^{2}

Assume four No. 8 bars, A_s = 3.16 in. Negative moment capacity of beam:

$$a = \frac{A_s f_y}{0.85 f_c' b_w} = \frac{(3.16)(60)}{(0.85)(4)(20)} = 2.79\, in$$

$$\phi M_n^- = M_g^- = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$= (0.90)(3.16)(60)(21.5 - 1.39)/12$$

= 286 ft-kips

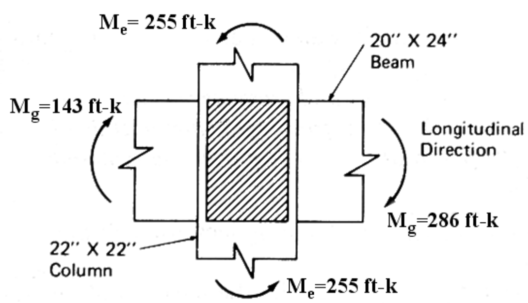


Figure 10-57. Relative flexural strength of beam and columns at exterior joint— longitudinal direction.

Assume a positive moment capacity of the beam on the opposite side of the column equal to one-half the negative moment capacity calculated above, or 143 ft-kips. Total moment capacity of beams framing into joint in longitudinal direction, for sidesway in either direction:

$$\sum M_g = 286 + 143 = 429\, ft - kips$$

$$\sum M_e = 2(258) = 516\, ft - kips$$

$$> \frac{6}{5} \sum M_g = \frac{6}{5}(429) = 515\, ft - kips$$

O.K. $\boxed{21.4.2.2}$

(b) Orthogonal effects: According to IBC-2000, the design seismic forces are permitted to be applied separately in each of the two orthogonal directions and the orthogonal effects can be neglected.

(c) Determine transverse reinforcement requirements:

(1) Confinement reinforcement (see Figure 10-38). Transverse reinforcement for confinement is required over a distance l_0 from column ends, where

$$l_0 \geq \begin{cases} depth\, of\ member = 22\, in.\ (governs) \\ \frac{1}{6}(clear\, height) = \frac{10 \times 12}{6} = 20\, in. \\ 18\, in. \end{cases} \qquad \boxed{21.4.4.4}$$

Maximum allowable spacing of rectangular hoops:

$$s_{\max} = \begin{cases} \frac{1}{4}(\text{smallest dimension of column}) \\ \quad = \frac{22}{4} = 5.5\, in. \\ 4\, in.\ (governs) \end{cases}$$

$\boxed{21.4.4.2}$

Required cross-sectional area of confinement reinforcement in the form of hoops:

$$A_{sh} \geq \begin{cases} 0.09 s h_c \frac{f_c'}{f_{yh}} \\ 0.3 s h_c \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f_c'}{f_{yh}} \end{cases}$$

$\boxed{21.4.4.1}$

where the terms are as defined for Equation 10-6 and 10-7. For a hoop spacing of 4 in., f_{yh} = 60,000 lb/in.^{2}, and tentatively assuming No. 4 bar hoops (for the purpose of estimating h_c and A_{ch})' the required cross-sectional area is

$$A_{sh} \geq \begin{cases} \frac{(0.09)(4)(18.5)(4000)}{60{,}000} \\ = 0.44\ in^2 \\ (0.3)(4)(18.5)\left(\frac{484}{361} - 1 \right) \frac{4000}{60{,}000} \\ = 0.50\, in^2 \text{ (governs)} \end{cases} \qquad \boxed{21.4.4.3}$$

No. 4 hoops with one crosstie, as shown in Figure 10-58, provide A_{sh} = 3(0.20) = 0.60 in.^{2}

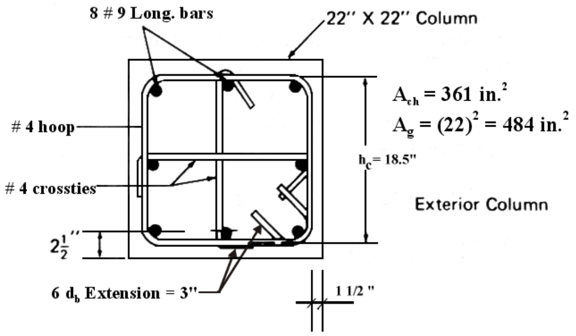


Figure 10-58. Detail of column transverse reinforcement.

(2) Transverse reinforcement for shear: As in the design of shear reinforcement for beams, the design shear in columns is based not on the factored shear forces obtained from a lateral-load analysis, but rather on the maximum probable flexural strength, M_{pr} (with ϕ = 1.0 and $f_s = 1.25\ f_y$), of the member associated with the range of factored axial loads on the member. However, the member shears need not exceed those associated with the probable moment strengths of the beams framing into the column.

If we assume that an axial force close to P = 740 kips (ϕ = 1.0 and tensile reinforcement stress of 1.25 f_y, corresponding to the "balanced point' on the P-M interaction diagram for the column section considered – which would yield close to if not the largest moment strength), then the corresponding M_b = 601 ft-kips. By comparison, the moment induced in the column by the beam framing into it in the transverse direction, with M_{pr} = 299 ft-kips, is 299/2 = 150 ft-kips. In the longitudinal direction, with beams framing on opposite sides of the column, we have (using the same steel areas assumed earlier),
M_{pr} (beams) = M^-_{pr} (beam on one side) + M^+_{pr} (beam on the other side) = 390 + 195 = 585 ft-kips, with the moment induced at each end of the column = 585/2 =293 ft-kips. This is less than M_b = 601 ft-kips and will be used to determine the design shear force on the column. Thus (see Figure 10-42),

$V_u = 2\ M_u/l = 2(293)/10 = 59$ kips

using, for convenience,

$$V_c = 2\sqrt{f_c'}\,bd$$

$$= \frac{2\sqrt{4000}(22)(19.5)}{1000} = 54\ kips$$

Required spacing of No. 4 hoops with A_v = 2(0.20) = 0.40 in.^{2} (neglecting crossties) and $V_s = (V_u - \phi V_c)/\phi = 14.8\,kips$:

$$s = \frac{A_v f_y d}{V_s} = \frac{(2)(2.0)(60)(19.5)}{14.8} = 31.6\,in.$$

11.5.6.2

Thus, the transverse reinforcement spacing over the distance l_0 = 22 in. near the column ends is governed by the requirement for confinement rather than shear.

Maximum allowable spacing of shear reinforcement: $d/2$ = 9.7 in. 11.5.4.1

Use No. 4 hoops and crossties spaced at 4 in. within a distance of 24 in. from the columns ends and No. 4 hoops spaced at 6 in. or less over the remainder of the column.

(d) Minimum length of lap splices for column vertical bars:

ACI Chapter 21 limits the location of lap splices in column bars within the middle portion of the member length, the splices to be designed as tension splices. 21.4.3.2

As in flexural members, transverse reinforcement in the form of hoops spaced at 4 in. (<d/4 =19.5/4 = 4.9 in.) is to be provided over the full length of the splice. 21.3.2.3

Since generally all of the column bars will be spliced at the same location, a Class *B* splice will be required. 12.15.2

The required length of splice is $1.3l_d$ where

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

where $\alpha = 1.0$, $\beta = 1.0$, $\gamma = 1.0$, and $\lambda = 1.0$

$$c = 1.5 + 0.5 + \frac{1.128}{2} = 2.6 \text{ in. (governs)}$$

or $$c = \frac{1}{2}\left[\frac{22 - 2(1.5+0.5) - 1.128}{2}\right] = 4.2 \text{ in.}$$

$$k_{tr} = \frac{A_{tr} f_{yt}}{1500 sn} = \frac{(3\times0.2)\times60{,}000}{1500\times4\times3} = 2.0$$

$$\frac{c+k_{tr}}{d_b} = \frac{2.6+2.0}{1.128} = 4.1 > 2.5 \text{ use } 2.5$$

$$\therefore l_d = \frac{3}{40}\frac{1.128x60{,}000}{\sqrt{4000}}\frac{1.0}{2.5} = 32.1 \text{ in.}$$

Thus, required splice length = 1.3(32.1) = 42 in. *Use 44-in, lap splices.*

(e) Detail of column. See Figure 10-59.

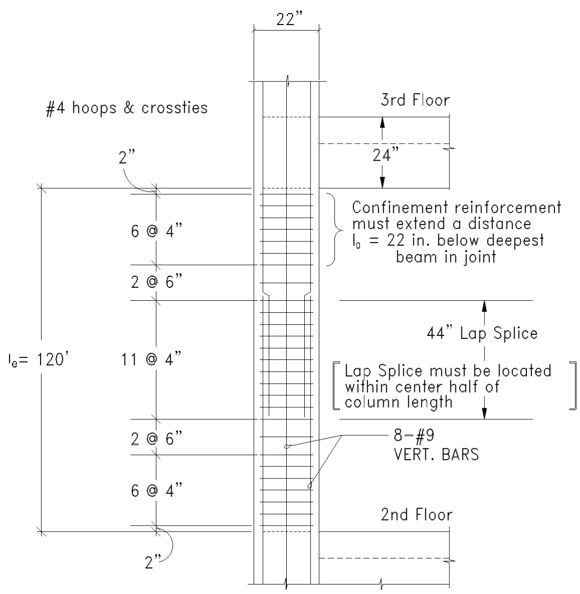


Figure 10-59. Column reinforcement details.

4. *Design of exterior beam—column connection.* The aim is to determine the transverse confinement and shear-reinforcement requirements for the exterior beam-column connection between the beam considered in item 2 above and the column in item 3. Assume the joint to be located at the sixth floor level.

(a) Transverse reinforcement for confinement: ACI Chapter 21 requires the same amount of confinement reinforcement within the joint as for the length l_0 at column ends, unless the joint is confined by beams framing into all vertical faces of the column. In the latter case, only one-half the transverse reinforcement required for unconfined joints need be provided. In addition, the maximum spacing of transverse reinforcement is (minimum dimension of column)/4 or 6 in. (instead of 4 in.).

$\boxed{21.5.2.1}$

$\boxed{21.5.2.2}$

In the case of the beam-column joint considered here, beams frame into only three sides of the column, so that the joint is considered unconfined.

In item 4 above, confinement requirements at column ends were satisfied by No. 4 hoops with crossties, spaced at 4 in.

(b) Check shear strength of joint: The shear across section x-x (see Figure 10-60) of the joint is obtained as the difference between the tensile force at the top flexural reinforcement of the framing beam (stressed to $1.25f_y$) and the horizontal shear from the column above. The tensile force from the beam (three No. 8 bars, $A_s = 2.37$ in.^{2}) is

(2.37)(1.25)(60) = 178 kips

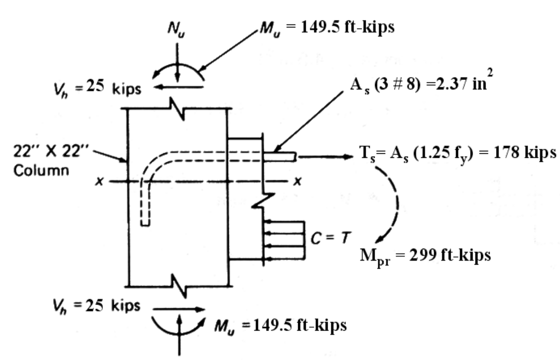


Figure 10-60. Horizontal shear in exterior beam-column joint.

An estimate of the horizontal shear from the column, V_h can be obtained by assuming that the beams in the adjoining floors are also deformed so that plastic hinges form at their junctions with the column, with M_p(beam) = 299 ft-kips (see Table 10-9, for sidesway to left). By further assuming that the plastic moments in the beams are resisted equally by the columns above and below the joint, one obtains for the horizontal shear at the column ends

$$V_h = \frac{M_p(beam)}{story\,height} = \frac{299}{12} = 25\,kips$$

Thus, the net shear at section x-x of joint is 178 -25 = 153 kips. ACI Chapter 21 gives the nominal shear strength of a joint as a function only of the gross area of the joint cross-section, A_j, and the degree of confinement provided by framing beams. For the joint considered here (with beams framing on three sides),

$$\begin{aligned} \phi V_c &= \phi 15\sqrt{f_c'}A_j \\ &= \frac{(0.85)(15)(\sqrt{4000})(22)^2}{1000} \\ &= 390\,kips > V_u = 153\,kips \quad \text{O.K.} \end{aligned}$$

21.5.3.1
9.3.4.1

Note that if the shear strength of the concrete in the joint as calculated above were inadequate, any adjustment would have to take the form (since transverse reinforcement above the minimum required for confinement is considered not to have a significant effect on shear strength) of either an increase in the column cross-section (and hence A_j) or an increase in the beam depth (to reduce the amount of flexural reinforcement required and hence the tensile force *T*).

(c) Detail of joint. See Figure 10-61. (The design should be checked for adequacy in the longitudinal direction.)

Note: The use of crossties within the joint may cause some placement difficulties. To relieve the congestion, No. 6 hoops spaced at 4 in. but without crossties may be considered as an alternative. Although the cross-sectional area of confinement reinforcement provided by No. 6 hoops at 4 in. (A_{sh} = 0.88 in.^{2}) exceeds the required amount (0.59 in.^{2}), the requirement of section 21.4.4.3 of ACI Chapter 21 relating to a maximum spacing of 14 in. between crossties or legs of overlapping hoops (see Figure 10-41) will not be satisfied. However, it is believed that this will not be a serious shortcoming in this case, since the joint is restrained by beams on three sides.

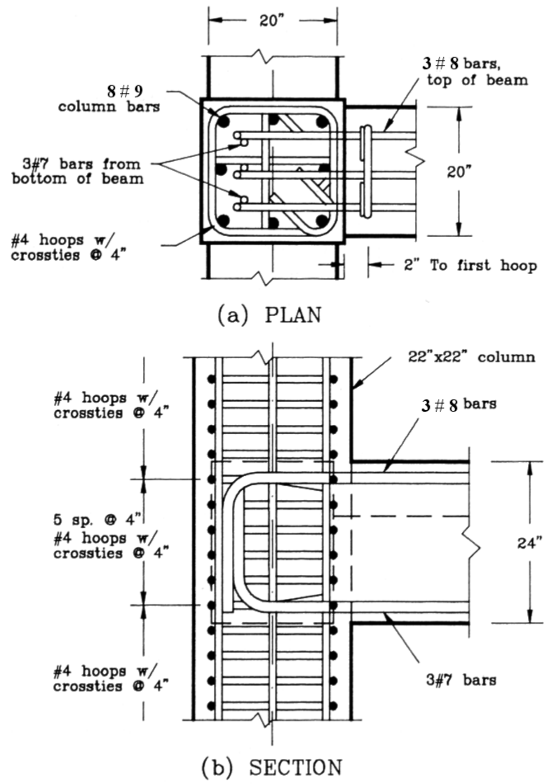


Figure 10-61. Detail of exterior beam-column connection.

5. D*esign of interior beam-column connection.* The objective is to determine the transverse confinement and shear reinforcement requirements for the interior beam-column connection at the sixth floor of the interior transverse frame considered in previous examples. The column is 26 in. square and is reinforced with eight No. 11 bars.

The beams have dimensions b = 20 in. and d = 21.5 in. and are reinforced as noted in Section item 2 above (see Figure 10-55).

(a) Transverse reinforcement requirements (for confinement): Maximum allowable spacing of rectangular hoops,

$$s_{\max} = \begin{cases} \frac{1}{4}(\text{smallest dimension of column}) \\ = 26/4 = 6.5\,in. \\ 6\ in.(governs) \end{cases}$$

$\boxed{21.5.2.2}$
$\boxed{21.4.4.2}$

For the column cross-section considered and assuming No. 4 hoops, h_c = 22.5 in., $A_{ch} = (23)^2$ = 529 in.^{2}, and $A_g = (26)^2$ = 676 in.^{2}. With a hoop spacing of 6 in., the required cross-sectional area of confinement reinforcement in the form of hoops is

$$A_{sh} \geq \begin{cases} 0.09sh_c\frac{f_c'}{f_{yh}} = \frac{(0.09)(6)(22.5)(4000)}{60{,}000} \\ = 0.81in^2 \qquad (governs) \\ 0.3sh_c\left(\frac{A_g}{A_{ch}}-1\right)\frac{f_c'}{f_{yh}} \\ = (0.3)(6)(22.5)\left(\frac{676}{529}-1\right)\frac{4000}{60{,}000} \\ = 0.75in^2 \end{cases}$$

$\boxed{21.4.4.1}$

Since the joint is framed by beams (having widths of 20 in., which is greater than $\frac{3}{4}$ of the width of the column, 19.5 in.) on all four sides, it is considered confined, and a 50% reduction in the amount of confinement reinforcement indicated above is allowed. Thus, A_{sh}(required) ≥ 0.41 in.^{2}.
No. 4 hoops with crossties spaced at 6 in. o.c. provide A_{sh} = 0.60 in.^{2}. (See Note at end of item 4.)

(b) Check shear strength of joint: Following the same procedure used in item 4, the forces affecting the horizontal shear across a section near mid-depth of the joint shown in Figure 10-62 are obtained:

(Net shear across section x-x) = $T_1 + C_2 - V_h$
=296 + 135 –59
= 372 kips = V_u

Shear strength of joint, noting that joint is confined:

$$\phi V_c = \phi 20\sqrt{f_c'}A_j$$
$$= \frac{(0.85)(20)\sqrt{4000}(26)^2}{1000} \qquad \boxed{21.5.3.1}$$
= 726 kips
> V_u = 372 kips O.K.

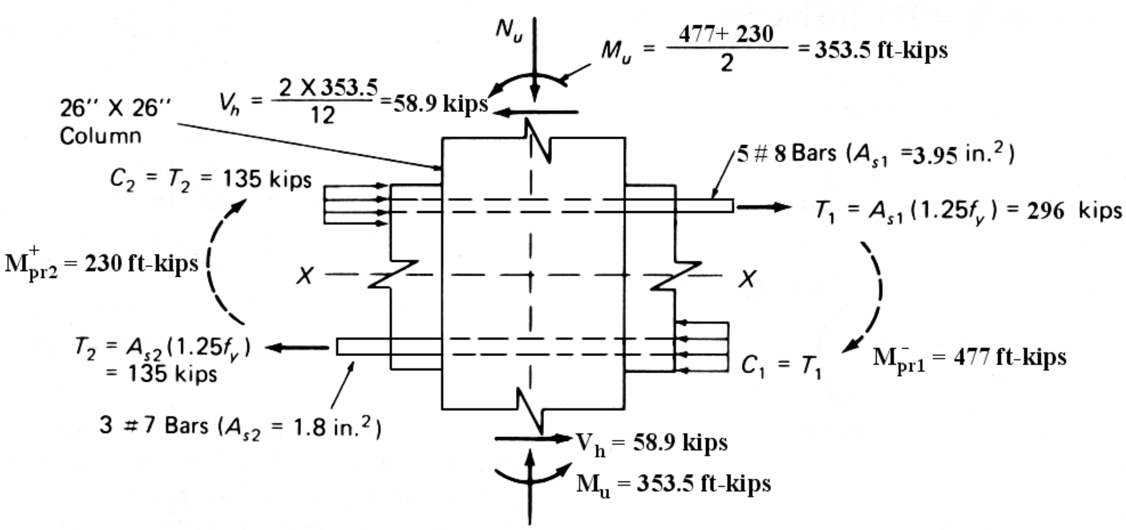


Figure 10-62. Forces acting on interior beam-column joint.

6.*Design of structural wall (shear wall).* The aim is to design the structural wall section at the first floor of one of the identical frame-shear wall systems. The preliminary design, as shown in Figure 10-48, is based on a 14-in.-thick wall with 26-in. -square vertical boundary elements, each of the latter being reinforced with eight No. 11 bars.

Preliminary calculations indicated that the cross-section of the structural wall at the lower floor levels needed to be increased. In the following, a 14-in.-thick wall section with 32 × 50-in. boundary elements reinforced with 24 No. 11 bars is investigated, and other reinforcement requirements determined.

The design forces on the structural wall at the first floor level are listed in Table 10-8. Note that because the axis of the shear wall coincides with the centerline of the transverse frame of which it is a part, lateral loads do not induce any vertical (axial) force on the wall.

The calculation of the maximum axial force on the boundary element corresponding to Equation 10-8b, $1.4\,\mathrm{D} + 0.5\,\mathrm{L} \pm 1.0\mathrm{Q_E}$, P_u = 3963 kips, shown in Table 10-8, involved the following steps: At base of the wall:

Moment due to seismic load (from lateral load analysis for the transverse frames), M_b = 32,860 ft-kips.

Referring to Figure 10-45, and noting the load factors used in Equation 10-8a of Table 10.8,

W = 1.2 D + 1.6 L + 0.5 L_r
= 5767 kips
Ha = 30,469 ft-kips

$$C_v = \frac{W}{2} + \frac{Ha}{d}$$

$$= \frac{5157}{2} + \frac{30{,}469}{22} = 3963 kips$$

(a) Check whether boundary elements are required: ACI Chapter 21 (Section 21.6.2.3) requires boundary elements to be provided if the maximum compressive extreme-fiber stress under factored forces exceeds $0.2 f_c^{'}$, unless the entire wall is reinforced to satisfy Sections 21.4.4.1 through 21.4.4.3 (relating to confinement reinforcement).

It will be assumed that the wall will not be provided with confinement reinforcement over its entire height. For a homogeneous rectangular wall 26.17 ft long (horizontally) and 14 in. (1.17 ft) thick,

$$I_{n.a.} = \frac{(1.17)(26.17)^3}{12} = 1747\, ft^4$$

$$A_g = (1.17)(26.17) = 30.6\, ft^2$$

Extreme-fiber compressive stress under M_u = 30,469 ft-kips and P_u = 5157 kips (see Table 10-8):

$$f_c = \frac{P_u}{A_g} + \frac{M_u h_w / 2}{I_{n.a.}} = \frac{5157}{30.6} + \frac{(30{,}469)(26{,}17)/2}{1747}$$

= 397 ksf = 2.76 ksi > 0.2 $f_c^{'}$ = (0.2)(4)
= 0.8 ksi.

Therefore, *boundary elements are required,* subject to the confinement and special loading requirements specified in ACI Chapter 21.

(b) Determine minimum longitudinal and transverse reinforcement requirements for wall:

(1) Check whether two curtains of reinforcement are required: ACI Chapter 21 requires that two curtains of reinforcement be provided in a wall if the in-plane factored shear force assigned to the wall exceeds $2A_{cv}\sqrt{f_c^{'}}$, where A_{cv} is the cross-sectional area bounded by the web thickness and the length of section in the direction of the shear force considered. $\boxed{21.6.2.2}$

From Table 10-8, the maximum factored shear force on the wall at the first floor level is V_u = 651 kips:

$$2A_{cv}\sqrt{f_c^{'}} = \frac{(2)(14)(26.17 \times 12)\sqrt{4000}}{1000}$$

= 556 kips
< V_u = 651 kips

Therefore, *two curtains of reinforcement are required.*

(2) Required longitudinal and transverse reinforcement in wall:

Minimum required reinforcement ratio,

$$\rho_v = \frac{A_{sv}}{A_{cv}} = \rho_n \geq 0.0025 \qquad \text{(max. spacing = 18 in.)}$$

$\boxed{21.6.2.1}$

With $A_{cv} = (14)(12) = 168$ in.^{2}, (per foot of wall) the required area of reinforcement in each direction per foot of wall is $(0.0025)(168) = 0.42$ in.^{2}/ft. Required spacing of No. 5 bars [in two curtains, $A_s = 2(0.31) = 0.62$ in.^{2}]:

$$s(required) = \frac{2(0.31)}{0.42}(12) = 17.7\ in. < 18\ in.$$

(c) Determine reinforcement requirements for shear. [Refer to discussion of shear strength design for structural walls in Section 10.4.3, under "Code Provisions to Insure Ductility in Reinforced Concrete Members," item 5, paragraph (b).] Assume two curtains of No. 5 bars spaced at 17 in. o.c. both ways. Shear strength of wall ($h_w/l_w = 148/26.17 = 5.66 > 2$):

$$\phi V_n = \phi A_{cv}\left(2\sqrt{f_c^{'}} + \rho_n f_y\right)$$

where

$\phi = 0.60$

$A_{cv} = (14)(26.17\times12) = 4397$ in.^{2}

$$\rho_n = \frac{2(0.31)}{(14)(12)} = 0.0037$$

Thus,

$$\phi V_n = \frac{(0.60)(4397)\left[2\sqrt{4000} + (0.0037)(60{,}000)\right]}{1000}$$

$$= \frac{2638.2[126.5 + 222]}{1000} = 919\,kips$$

$$> V_u = 651\,kips \qquad \text{O.K.}$$

Therefore, *use two curtains of No. 5 bars spaced at 17 in o. c. in both horizontal and vertical directions.* $\boxed{21.7.3.5}$

(d) Check adequacy of boundary element acting as a short column under factored vertical forces due to gravity and lateral loads (see Figure 10-45): From Table 10-8, the maximum compressive axial load on boundary element is $P_u = 3963$ kips.

$\boxed{21.5.3.3}$

With boundary elements having dimensions 32 in.×50 in. and reinforced with 24 No. 11 bars,

$A_g = (32)(50) = 1600$ in.^{2}
$A_{st} = (24)(1.56) = 37.4$ in.^{2}
$\rho_{st} = 37.4/1600 = 0.0234$
$\rho_{min} = 0.01 < \rho_{st} < \rho_{max} = 0.06$ O.K.

$\boxed{21.4.3.1}$

Axial load capacity of a short column:

$$\phi P_n(\max) = 0.80\phi\left[0.85 f_c^{'}\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

$= (0.80)(0.70)[(0.85)(4)(1600 - 37.4) +(60)(37.4)]$

$= (0.56)[5313+ 2244] = 4232$ kips $> P_u = 3963$ kips O.K. $\boxed{10.3.5.2}$

(e) Check adequacy of structural wall section at base under combined axial load and bending in the plane of the wall: From Table 10-8, the following combinations of factored axial load and bending moment at the base of the wall are listed, corresponding to Eqs. 10-8a, b and c:

9-8a: $P_u = 5767$ kips, M_u small
9-8b: $P_u = 5157$ kips, $M_u = 30{,}469$ ft-kips
9-8c: $P_u = 2293$ kips, $M_u = 30{,}469$ ft-kips

Figure 10-63 shows the ϕP_n-ϕM_n interaction diagram (obtained using a computer program for generating *P-M* diagrams) for a structural wall section having a 14-in.-thick web reinforced with two curtains of No. 5 bars spaced at 17 in o.c. both ways and 32 in.×50-in. boundary elements reinforced with 24 No. 11 vertical bars, with $f_c^{'}$ = 4000 lb/in.^{2}, and f_y = 60,000 lb/in.^{2} (see Figure 10-64). The design load combinations listed above are shown plotted in Figure 10-63. The point marked *a* represents the *P-M* combination corresponding to Equation 10-8a, with similar notation used for the other two load combinations.

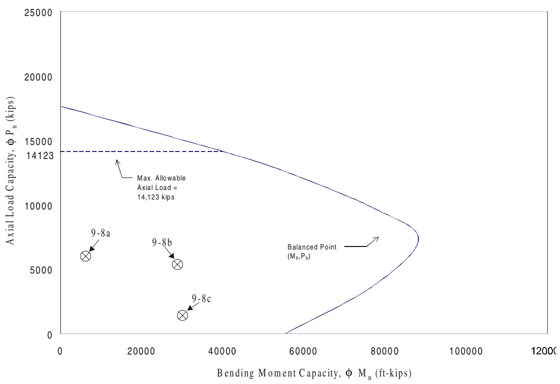


Figure 10-63. Axial load-moment interaction diagram for structural wall section.

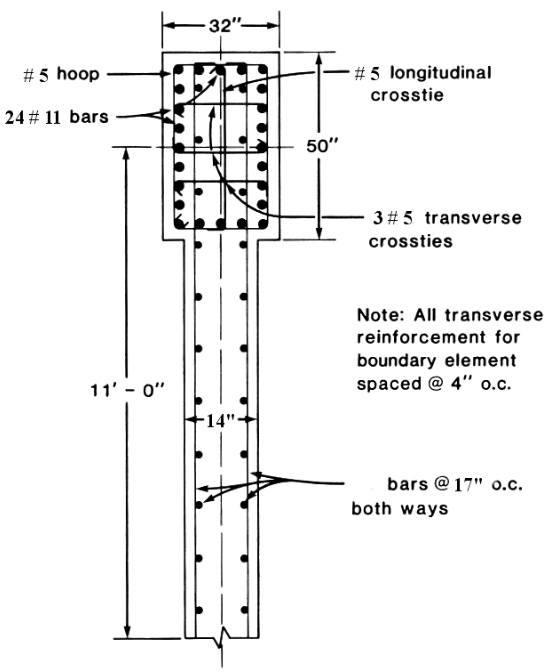


Figure 10-64. Half section of structural wall at base.

It is seen in Figure 10-63 that the three design loadings represent points inside the interaction diagram for the structural wall section considered. Therefore, *the section is adequate with respect to combined bending and axial load.*

Incidentally, the "balanced point" in Figure 10-63 corresponds to a condition where the compressive strain in the extreme concrete fiber is equal to ε_{cu} = 0.003 and the tensile strain in the row of vertical bars in the boundary element farthest from the neutral axis (see Figure 10-64) is equal to the initial yield strain, ε_y = 0.00207.

(f) Determine lateral (confinement) reinforcement required for boundary elements (see Figure 10-64): The maximum allowable spacing is

$$s_{\max} = \begin{cases} 1/4(\text{smallest dimension} \\ \text{of boundary element}) \\ = 32/4 = 8\,in. \\ 4\,in.\ (\text{governs}) \end{cases}$$

$\boxed{21.6.6.2}$
$\boxed{21.4.4.2}$

(1) Required cross-sectional area of confinement reinforcement in short direction:

$$A_{sh} \geq \begin{cases} 0.09 s h_c \dfrac{f_c'}{f_{yh}} \\ 0.3 s h_c \left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f_c'}{f_{yh}} \end{cases} \qquad \boxed{21.4.4.1}$$

Assuming No. 5 hoops and crossties spaced at 4 in. o.c. and a distance of 3 in. from the center line of the No. 11 vertical bars to the face of the column, we have

h_C = 44 + 1.41 + 0.625 = 46.04 in. (for short direction),

A_{ch}= (46.04 + 0.625)(26 + 1.41 + 1.25) =1337 in.^{2}

$$A_{sh} > \begin{cases} (0.09)(4)(46.04)(4/60) \\ = 1.10\,in^2\ (governs) \\ (0.3)(4)(46.04)(\dfrac{(32)(50)}{1337} - 1)(\dfrac{4}{60}) \\ = 0.72\ in.^2 \end{cases}$$

(required in short direction).

With three crossties (five legs, including outside hoops),

A_{sh} (provided) = 5(0.31) = 1.55 in.^{2} O.K.

(2) Required cross-sectional area of confinement reinforcement in long direction:

h_c = 26 + 1.41 + 0.625 = 28.04 in.

(for long direction),
A_{ch} = 1337 in.^{2}

$$A_{sh} \geq \begin{cases} (0.09)(4)\,(28.04)\,(4/60) \\ = 0.67 \text{ in.}^2 \text{ (governs)} \\ (0.3)(4)(28.04)(1.196 - 1)(4/60) \\ = 0.44 \text{ in.}^2 \end{cases}$$

(required in long direction).

With one crosstie (i.e., three legs, including outside hoop),

A_{sh} (provided) = 3(0.31) = 0.93 in.^{2} O.K.

(g) Determine required development and splice lengths:

ACI Chapter 21 requires that all continuous reinforcement in structural walls be anchored or spliced in accordance with the provisions for reinforcement in tension. 21.6.2.4

(1) Lap splice for No. 11 vertical bars in boundary elements (the use of mechanical connectors may be considered as an alternative to lap splices for these large bars): It may be reasonable to assume that 50% or less of the vertical bars are spliced at any one location. However, an examination of Figure 10-63 suggests that the amount of flexural reinforcement provided–mainly by the vertical bars in the boundary elements–does not represent twice that required by analysis, so that a class B splice will be required. 12.15.2

Required length of splice = 1.3 l_d where l_d = 2.5 l_{dh} 12.15.1

and

$$l_{dh} \geq \begin{cases} f_y d_b / 65\sqrt{f_c'} \\ = \dfrac{(60{,}000)(1.41)}{65\sqrt{4000}} = 21\,in.\,(governs) \\ 8\,d_b = (8)(1.41) = 12\,in. \\ 6\ in. \end{cases}$$

21.5.4.2

Thus the required splice length is (1.3)(2.5)(21) = 68 in.

(2) Lap splice for No. 5 vertical bars in wall "web": Here again a class B splice will be required. Required length of splice = 1.3 l_d, whre l_d = 2.5 l_{dh}, and

$$l_{dh} \geq \begin{cases} f_y d_b / 65\sqrt{f_c'} \\ = \dfrac{(60{,}000)(0.625)}{65\sqrt{4000}} = 9\,in.\,(governs) \\ 8\,d_b = (8)(0.625) = 5.0\,in. \\ 6\ in. \end{cases}$$

Hence, the required length of splice is (1.3)(2.5)(9) = 30 in.

Development length for No. 5 horizontal bars in wall, assuming no hooks are used within the boundary element: Since it is reasonable to assume that the depth of concrete cast in one lift beneath a horizontal bar will be greater than 12 in., the required factor of 3.5 to be applied to the development length, l_{dh}, required for a 90° hooked bar will be used [Section 10.4.3, under "Code Provisions Designed to Insure Ductility in Reinforced-Concrete Members", item 2, paragraph (f)]:

21.5.4.2

l_d = 3.5 l_{dh}, where as indicated above, l_{dh} = 9.0 in. so that the required development length l_d = 3.5(9) = 32 in.

This length can be accommodated within the confined core of the boundary element, so that no hooks are needed, as assumed. However, because of the likelihood of large horizontal cracks developing in the boundary elements, particularly in the potential hinging region near the base of the wall, the horizontal bars will be provided with 90° hooks engaging a vertical bar, as recommended in the Commentary to ACI Chapter 21 and as shown in Figure 10-64.

Required lap splice length for No. 5 horizontal bars, assuming (where necessary) 1.3 l_d = (1.3)(32) = <u>42 in</u>.

(h) Detail of structural wall: See Figure 10-64. It will be noted that the No. 5 vertical-wall "web" reinforcement, required for shear resistance, has been carried into the boundary element. The Commentary to ACI Section 21.6.5 specifically states that the concentrated reinforcement provided at wall edges (i.e. the boundary elements) for bending is not be included in determining shear-reinforcement requirements. The area of vertical shear reinforcement located within the boundary element could, if desired, be considered as contributing to the axial load and bending capacity.

(i) Design of boundary zone using UBC-97 and IBC-2000 Provisions:

Using the procedure discussed in Section 10.4.3 item 5 (f), the boundary zone design and detailing requirements using these provisions will be determined.

(1) Determine if boundary zone details are required:

Shear wall boundary zone detail requirements to be provided unless $P_u \le 0.1A_g f'_c$ and either $M_u/V_u l_u \le 1.0$ or $V_u \le 3\, A_{cv}\sqrt{f'_c}$. Also, shear walls with $P_u > 0.35\, P_0$ (where P_0 is the nominal axial load capacity of the wall at zero eccentricity) are not allowed to resist seismic forces.

Using 26 inch square columns; $0.1A_g f'_c = 0.1 \times (14 \times 19.83 \times 12 + 2 \times 26^2) \times 4 = 1873$ kips $< P_u = 3963$ kips. Using 32 × 50 columns also results in the value of $0.1A_g f'_c$ to be less than P_u. Therefore, boundary zone details are required.

Assume a 14 in. thick wall section with 32 × 50 in. boundary elements reinforced with 24 No. 11 bars as used previously. Also, it was determined that 2#5 bars at 17 in. spacing is needed as vertical reinforcement in the web. On this basis, the nominal axial load capacity of the wall (P_0) at zero eccentricity is:

$$P_0 = 0.85\, f'_c\,(A_g - A_{st}) + f_y A_{st}$$

$$= 0.85 \times 4 \times (6195\text{-}82.68) + (60 \times 82.68) = 25{,}743 \text{ kips}$$

Since $P_u = 3963$ kips $= 0.15\, P_0 < 0.35\, P_0 = 9010$ kips, the wall can be considered to contribute to the calculated strength of the structure for resisting seismic forces.

Therefore, provide boundary zone at each end having a distance of $0.15\, l_w = 0.15 \times 26.17 \times 12 = 47.1$ in. On this basis, a 32×50 boundary zone as assumed is adequate.

Alternatively, the requirements for boundary zone can be determined using the displacement based procedure. As such, boundary zone details are to be provided over the portion of the wall where compressive strains exceed 0.003. The procedure is as follows:

Determine the location of the neutral axis depth, c'_u.

From Table 10-8, $P'_u = 5767$ kips; the nominal moment strength, M'_n, corresponding to P'_u is 89,360 k-ft (see Figure 10-63). For 32 × 50 in. boundary elements reinforced with 24 #11 bars, c'_u is equal to 97.7 in. This value can be determined using the strain compatibility approach.

From the results of analysis, the elastic displacement at the top of the wall, Δ_E is equal to 1.55 in. using gross section properties and the corresponding moment, M'_n, at the base of the wall is 30,469 k-ft (see Table 10-8). From the analysis using the cracked section properties, the total deflection, Δ_t, at top of the wall is 15.8 in. (see Table 10-3, $\Delta_t = 2.43 \times C_d = 2.43 \times 6.5 = 15.8$in.), also $\Delta_y = \Delta_E M'_n/M'_E = 1.55 \times 89{,}360/30{,}469 = 4.55$ in.

The inelastic deflection at the top of the wall is:

$\Delta_i = \Delta_t - \Delta_y = 15.8 - 4.55 = 11.25$in.

Assume $l_p = 0.5\, l_w = 0.5 \times 26.17 \times 12 = 157$ in., the total curvature demand is:

$$\phi_t = \frac{11.25}{(148\times12-157/2)\times157} + \frac{0.003}{26.17\times12}$$
$$= 5.176\times10^{-5}$$

Since ϕ_t is greater than $0.003/c'_u = 0.003/97.7 = 3.07\times10^{-5}$, boundary zone details are required. The maximum compressive strain in the wall is equal to $\phi_t c'_u = 5.176 \times 10^{-5} \times 97.7 = 0.00506$ which is less than the maximum allowable value of 0.015. In this case, boundary zone details are required over the length,

$$\left(97.7 - \frac{0.003}{0.00506}\times 97.7\right) = 39.8in.$$

This is less than the 50 in. length assumed. Therefore, the entire length of the boundary zone will be detailed for ductility.

(2) Detailing requirements:

Minimum thickness:

$$= l_u/16 = \frac{(16\times12)-24}{16} = 10.5\,in. < 32\,in. \quad \text{O.K.}$$

Minimum length = 18 in. < 50 in. O.K.

The minimum area of confinement reinforcement is:

$$A_{sh} = \frac{0.09 s h_c f'_c}{f_{yh}}$$

Using the maximum allowable spacing of $6d_b = 6 \times 1.41 = 8.46$ in. or 6 in. (governs), and assuming #5 hoops and crossties at a distance of 3 in. from the center line of #11vertical bars to the face of the column, we have

$h_c = 44 + 1.41 + 0.625 = 46.04$

$$A_{sh} = \frac{0.09\times6\times46.04\times4}{60} = 1.66\,\text{in.}^2$$

With four crossties (six legs, including outside hoops), A_{sh} provided = 6 (0.31) = 1.86 in.^{2} O.K.

Also, over the splice length of the vertical bars in the boundary zone, the spacing of hoops and crossties must not exceed 4 in. In addition, the minimum area of vertical bars in the boundary zone is $0.005\times32^2 = 5.12$ in.^{2} which is much less than the area provided by 24#11 bars. The reinforcement detail in the boundary zone would be very similar to that shown previously in Figure 10-64.

REFERENCES

The following abbreviations will be used to denote commonly occurring reference sources:

- *Organizations and conferences:*

EBRI Earthquake Engineering Research Institute
WCEE World Conference on Earthquake Engineering
ASCE American Society of Civil Engineers
ACI American Concrete Institute
PCA Portland Cement Association
PCI Prestressed Concrete Institute

- *Publications:*

JEMD Journal of Engineering Mechanics Division, ASCE
JSTR Journal of the Structural Division, ASCE
JACI Journal of the American Concrete Institute

10-1 International Conference of Building Officials, 5360 South Workman Mill Road, Whittier, CA 90601, *Uniform Building Code.* The latest edition of the Code is the 1997 Edition.

10-2 Clough, R. W. and Benuska, K. L., "FHA Study of Seismic Design Criteria for High-Rise Buildings," Report HUD TS-3. Federal Housing Administration, Washington, Aug. 1966.

10-3 Derecho, A. T., Ghosh, S. K., Iqbal, M., Freskakis, G. N., and Fintel, M., "Structural Walls in