Design Example 5
Reinforced Concrete Wall with Coupling Beams

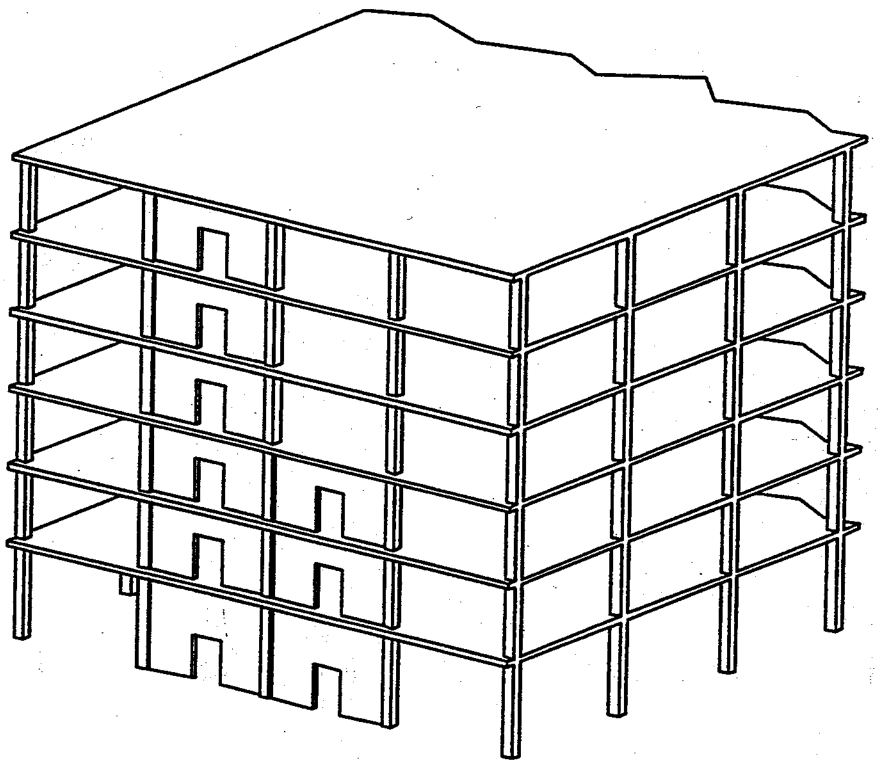


Figure 5-1. Six-story concrete office building (partial view)

Overview

The structure in this Design Example is a 6-story office building with reinforced concrete walls (shear walls) as its lateral force resisting system. The example focuses on the design and detailing of one of the reinforced concrete walls. This is a coupled wall running in the transverse building direction and is shown in Figure 5-1. The example assumes that design lateral forces have already been determined for the building, and that the seismic moments, shears, and axial loads on each of the wall components, from the computer analysis, are given.

The purpose of this Design Example is to illustrate the design of coupling beams and other aspects of reinforced concrete walls that have openings. Research on the behavior of coupling beams for concrete walls has been carried out in New Zealand, the United States, and elsewhere since the late 1960s. The code provisions of the UBC derive from this research.

Outline

This Design Example illustrates the following parts of the design process:

1. **Load combinations for design.**
2. **Preliminary sizing of shear wall.**
3. **Coupling beam design.**
4. **Design of wall piers for flexure.**
5. **Plastic analysis of flexural mechanism in walls.**
6. **Design of wall piers for shear.**
7. **Boundary zone detailing of wall piers.**
8. **Detailing of coupling beams.**

Given Information

The following information is given:

Seismic zone = 4
Soil profile type = S_D
Near-field = 5 km from seismic source type A
Redundancy/reliability factor, $\rho = 1.0$
Importance factor, $I = 1.0$
Concrete strength, $f'_c = 4000$ psi
Steel yield strength, $f_y = 60$ ksi

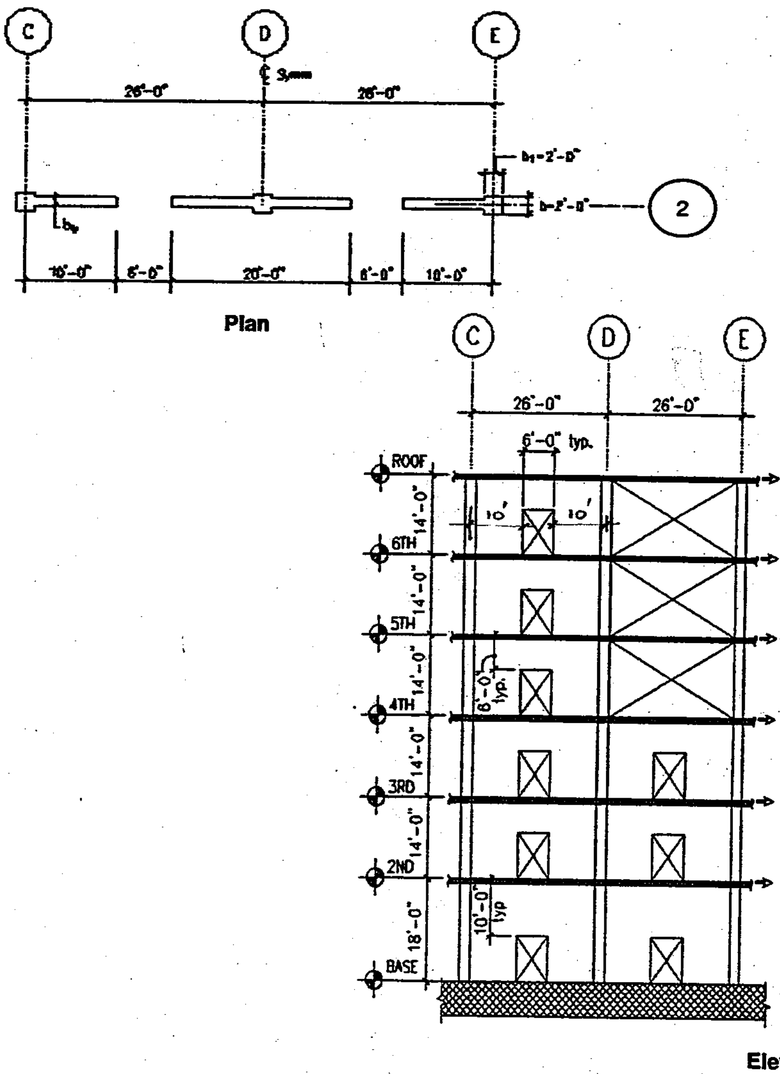
The wall to be designed, designated Wall 3, is one of several shear walls in the building. The wall elevation, a plan section, and the design forces are shown in Figure 5-2. An elastic analysis of the wall for lateral forces, using a computer program, gives the results shown in Figure 5-3, which shows the moments and shear for each coupling beam (i.e., wall spandrel), and the moments, shear and axial forces for each vertical wall segment (i.e., wall pier).

Lateral story displacements, corresponding to gross section properties, are also shown on the figure. Where displacements are used in design they should correspond to effective section properties rather than gross section properties, as indicated in §1633.2.4. Typical practice is to use a percentage of the gross stiffness, e.g., 50 percent, for the effective stiffness. In such a case, the displacements from the gross section model can be uniformly factored up. The displacements for a linear elastic model using 50 percent of I_g will be two times the displacements using the gross section properties. In this Design Example, the displacement output is not used. In an actual building design, the displacements would need to be considered for: 1.) design of elements not part of the lateral-force-resisting system, 2.) building separations, 3.) boundary design by the strain calculation procedure, and 4.) $P\Delta$ analysis. Other recommendations for member stiffness assumptions are given in Section 5.3 of Paulay and Priestley [1992].

AN ABERRATION!

THIS IS ONLY TRUE IF BLDG IS RESPONDING IN THE CONSTANT ACCEL. REGION. NOT LIKELY FOR A 7-STORY BLDG!

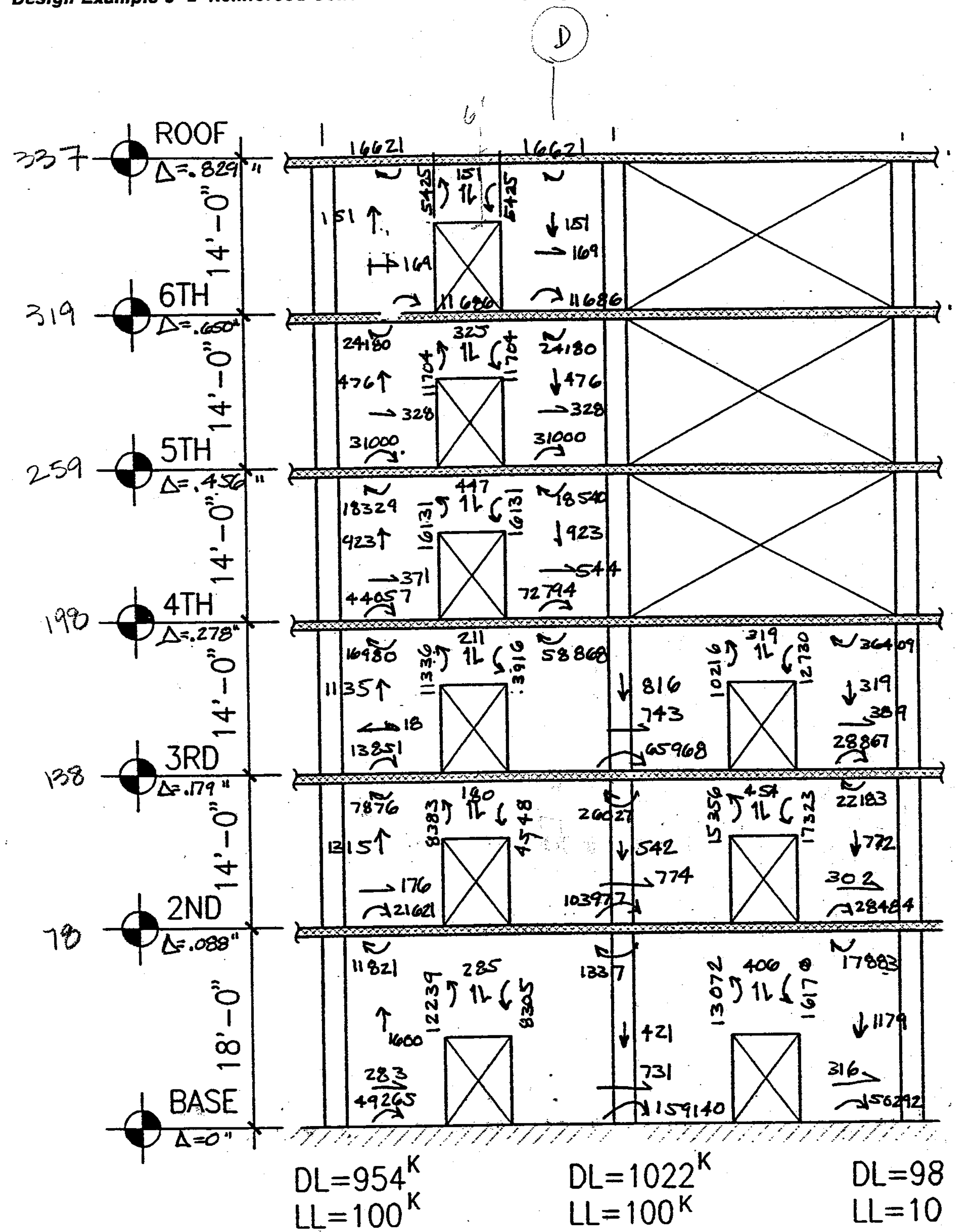
Gravity loads are not included in the computer model. Gravity effects are added separately by hand calculations.



LATERAL FORCES F_x	GRAVITY LOADS EACH COLUMN	
	P_D	P_L
337^k	133	10
319^k	281	25
259^k	428	44
198^k	576	63
138^k	742	82
78^k	874	100
Σ 1329		

P_D = EXCLUDE WALL WEIGHT

Figure 5-2. Wall elevation, plan section, and design forces of Wall 3



Units:
P=kips beam moment at edge of wall piers
V=kips pier moments at floor levels
M=kips-inch

Figure 5-3. Results of ETABS computer analysis for Wall 3

Calculations and Discussion | **Code Reference**

1. Load combinations for design.

Load combinations for reinforced concrete are discussed in detail in Part 1 of Design Example 4. As in that example, we assume here that the presiding building department has indicated approval of the SEAOC recommended revisions to the UBC load combinations. Thus the governing load combinations become:

$$(1.2 \pm 0.5C_a I)D \pm \rho E_h + (f_1 L + f_2 S)$$ Blue Book §101.7.2.1

$$0.9D \pm \rho E_h$$ Blue Book §C403.11

Since the given structure is an office building, $f_1 = 0.5$. And since there is no snow load, $S = 0$.

The same seismic zone, soil profile, near-field, redundancy, and importance factors are assumed as for Design Example 4, thus $C_a = 0.484$. With $I = 1.0$ and $\rho = 1.0$, the governing load combinations for this Design Example are:

$$0.9D \pm E_h$$

$$[1.2 \pm 0.5(0.484)]D \pm E_h + L \quad \begin{cases} = 1.44D \pm E_h + 0.5L \\ = 0.958D \pm E_h + 0.5L \quad \textit{does not govern} \end{cases}$$

The forces shown in Figure 5-3 correspond to E_h.

2. Preliminary sizing of shear wall.

For walls with diagonally reinforced coupling beams, the required wall thickness is often dictated by the layering of the reinforcement in the coupling beam. Typically, a wall thickness of 15 inches or larger is required for diagonally reinforced coupling beams conforming to the 1997 UBC.

For the subject wall, a wall thickness, b_w, of 16 inches will be tried.

3. Coupling beam design.

3a. Requirement for diagonal reinforcement.

Code requirements for the diagonal reinforcement of coupling beams (§1921.6.10.2) are based on the clear-length to depth ratio for the coupling beam, l_n/d, and on the level of shear stress in the coupling beam.

For the wall in this Design Example, it will be assumed that d equals 0.8 times the overall depth, so that $l_n/d = 72''/(0.8 \times 72'') = 1.25$ for the typical coupling beam, and $l_n/d = 72''/(0.8 \times 120'') = 0.75$ for the coupling beams at the second floor.

As shown in Table 5-1 (6th column), for five of the nine coupling beams the shear exceeds $4\sqrt{f'_c}b_w d$. For these coupling beams, diagonal reinforcement is required.

For the four coupling beams that have lower shear stress, diagonal reinforcement is not required by the UBC. Designing these 4 coupling beams without diagonal reinforcement, using horizontal reinforcement to resist flexure and vertical stirrups to resist shear, might lead to cost savings in the labor to place the reinforcing steel.

In this Design Example, however, diagonal reinforcement is used in all of the coupling beams of the wall because: 1.) it can simplify design and construction to have all coupling beams detailed similarly, and 2.) research results show that diagonal reinforcement improves coupling beam performance, even at lower shear stress levels, as discussed in §C407.7 of the SEAOC Blue Book.

Table 5-1. Coupling beam forces and diagonal reinforcement

Grid Line	Level	V_u (kips)	h (in.)	d (in.)	$V_u/b_w d\sqrt{f'_c}$ (1)	Diagonal Bars	A_d (in.²)	α (degrees)	ϕV_n (kips)	$\phi V_n/V_u$
C-D	Roof	151	72	57.6	2.6	4-#8	3.16	37.9	198	1.31
C-D	6th	325	72	57.6	5.6	4-#10	5.08	37.9	318	0.98
C-D	5th	447	72	57.6	7.7	6-#10	7.62	36.0	456	1.02
C-D	4th	211	72	57.6	3.6	4-#9	4.00	37.9	251	1.19
C-D	3rd	180	72	57.6	3.1	4-#9	4.00	37.9	251	1.39
C-D	2nd	285	120	96.0	2.9	4-#9	4.00	53.1	326	1.14
D-E	4th	319	72	57.6	5.5	6-#9	6.00	36.0	359	1.13
D-E	3rd	454	72	57.6	7.8	6-#10	7.62	36.0	456	1.00
D-E	2nd	406	120	96.0	4.2	4-#10	5.08	53.1	414	1.02

Note: Diagonal bars are required when this ratio exceeds 4.

$\tan 37.9 = \frac{h_{eff}}{72}$ $h_{eff_1} = 56.05$ A

$h_{eff_2} = 52.311$ 6

3b. Design of diagonal reinforcement.

Diagonal reinforcement is provided in the coupling beams according to Equation (21-1) of §1921.6.10.2:

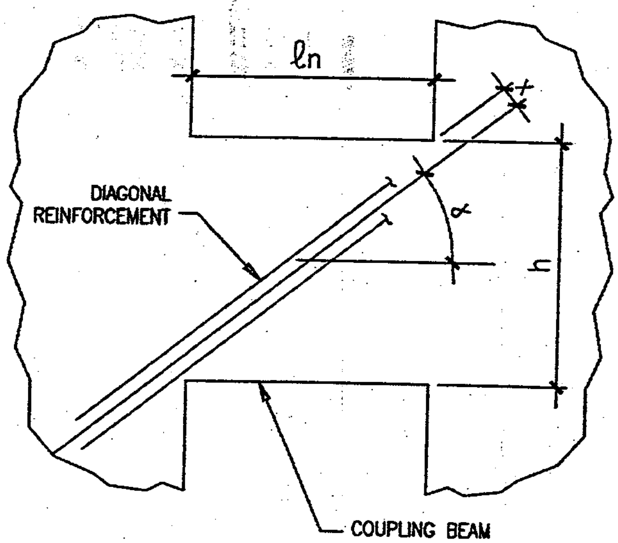
(21-11)
$$\phi V_n = 2\phi f_y \sin\alpha A_{vd} \qquad \phi = 0.85 \tag{21-1}$$

EXAMPLE CALC: GRID C-D LEV. 6TH $\phi V_n = 2(0.85)60 \sin(37.9)\, 4\times1.27 = 318.^k$ ✓

Each group of diagonal bars must consist of at least 4 bars (§1921.6.10.2). The calculation of the required diagonal reinforcement is shown in Table 5-1. For coupling beams with higher shear stresses, 6 bars are needed in each group, as shown in Table 5-1.

The angle α of the diagonal bars is calculated based on the geometry of the reinforcement layout, as shown in Figure 5-4. The value of α depends somewhat on overall dimension of the diagonal bar group and on the clearance between the diagonal bar group and the corner of the wall opening. This affects the dimension x shown in Figure 5-4 and results in a slightly different value of α for a group of 6 bars compared to that for a group of 4 bars, as shown in Table 5-1.

The provided diagonal bars are shown in Figure 5-5.



$\alpha \sim \text{ATAN} \dfrac{h-14''}{\ell_n}$

$\text{ATAN} \dfrac{72-14}{72} = 38.9^\circ$ Vs $36.^\circ$

$\text{ATAN} \dfrac{120-14}{72} = 55.8^\circ$ Vs 53.1°

Error $\sim \dfrac{\sin 38.9}{\sin 36} = 1.07$

$\dfrac{\sin 55.8}{\sin 53.1} = 1.03$

Figure 5-4. Geometry of coupling beam diagonal bars

$\dfrac{16}{4} = 4$

$\dfrac{19}{6} = 3$

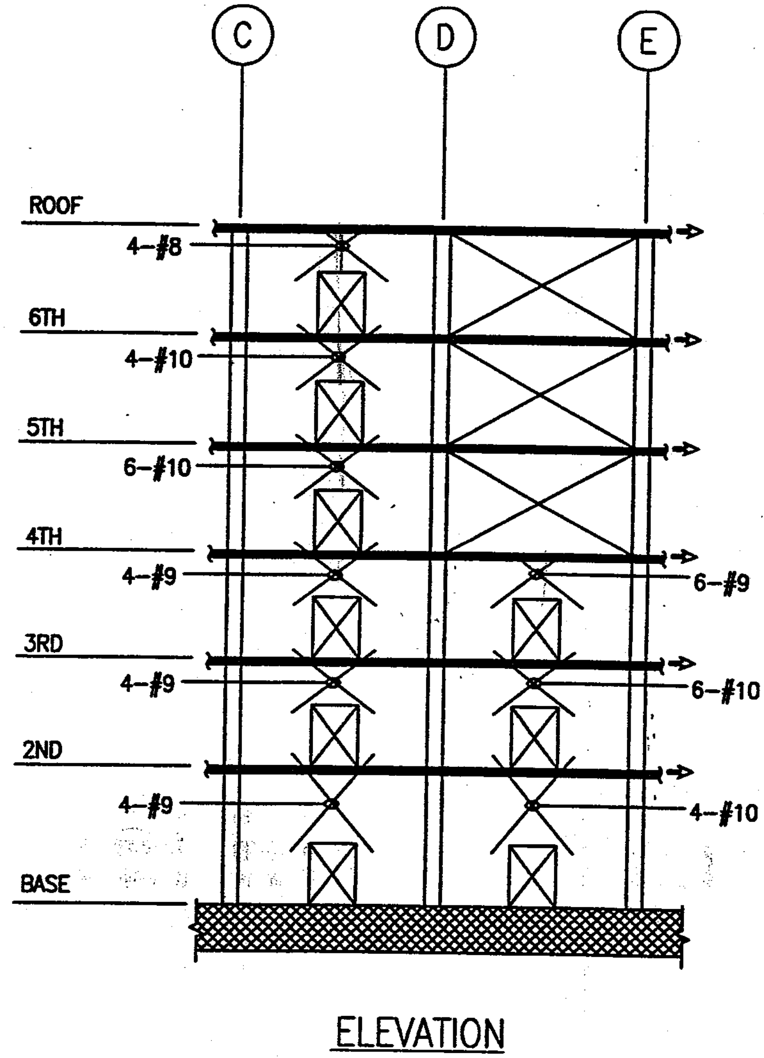


Figure 5-5. Diagonal bars provided in coupling beams

4. Design of wall piers for flexure.

The design of the vertical wall segments for flexure is carried out following the procedures and recommendations given for conventional "solid" walls. This is shown in Part 3 of Design Example 4. From Figure 5-3, the critical wall segments (i.e., those with the highest moments or earthquake axial forces) include the wall pier at the 4th floor on Line D, and the wall piers at the base on Lines C and E. The 20-foot long wall pier on Line D at the base is also checked.

4a. Critical moments and axial forces.

As can be seen from Figure 5-2, the gravity loads on each wall pier are not concentric with the wall pier centroid. Therefore, gravity load moments must be considered in the design of flexural reinforcement. The dead and live loads (except wall self-weight shown in Table 5-2) in Figure 5-2 act at the column grid lines, and have an eccentricity, e_{DF}, with respect to the section centroid, as given in Table 5-3 (**Note:** The calculation of weights, section centroids, e_{DF}, *and* e_{DW} is not shown). The wall self-weight provides additional dead load at each level, equal to the values given in Table 5-2.

Table 5-2. Dead load from wall self-weight

	Line C		Line D		Line E	
Level	Sum of Wall Weight (kips)	Eccentricity, e_{DW} (ft) [1]	Sum of Wall Weight (kips)	Eccentricity, e_{DW} (ft) [1]	Sum of Wall Weight (kips)	Eccentricity, e_{DW} (ft) [1]
Above 6th	26	2.06	26	-2.06	0	
Above 5th	53	2.06	53	-2.06	0	
Above 4th	79	2.06	79	-2.06	0	
Above 3rd	106	2.06	132	-3.71	26	-2.06
Above 2nd	132	2.06	185	-2.65	53	-2.06
At base	166	2.03	252	-1.94	86	-2.00

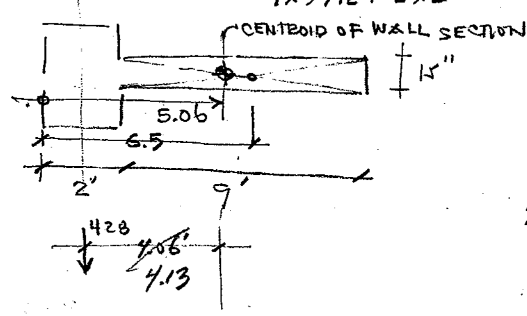
Note:
1. e_{DW} = distance between centroid of weight and centroid of wall section.

The calculation of the factored forces on the critical wall piers is shown in Table 5-3. In this table, gravity moments are calculated about the section centroid, using the gravity loads acting at the column centerline, P_{DF} and P_L, plus the dead load from wall self-weight, P_{DW}. Earthquake moments, M_E, are taken from Figure 5-3.

Loads are factored according to the combinations discussed in Part 1 of this Design Example, giving two cases for each wall pier: minimum axial load and maximum axial load. The minimum axial load case is based on the combination of E_h with $0.9D$, and the maximum axial load case is based on the combination of E_h with $1.44D+0.5L$.

Considering that larger axial compression generally increases moment strength, potentially governing combinations are shown as shaded areas in Table 5-3.

$$x_{cg} = \frac{9' \times 15/12 \times (6.5) + 2 \times 2 \times 1}{9 \times 15/12 + 2 \times 2} = \cancel{5.06} \quad 5.125$$



428×4.13 − 79×2.06

44×4.13

$P_E + 0.9(P_{DF} + P_{DW})$

Table 5-3. Calculation of factored axial forces and moments on critical wall piers

Level	Line	P_{DF} (kips)	e_{DR} (ft)	P_{DW} (kips)	e_{DW} (ft)	P_L (kips)	Direction of force	P_E (kips)	M_E (k-ft)	M_D (k-ft)	M_L (k-ft)	Minimum Axial P_U	Minimum Axial M_U	Maximum Axial P_U	Maximum Axial M_U
4th	D	428	-4.13	79	-2.06	44	west	-923	-6,070	1,603	182	-467	-4,628	-171	-3,671
4th	D	428	-4.13	79	-2.06	44	east	923	6,070	1,603	182	1,379	7,512	1,675	8,469
1st	C	874	4.13	166	2.03	100	west	1,600	-4,105	-3,268	-413	2,536	-7,047	3,148	-9,018
1st	C	874	4.13	166	2.03	100	east	-1,600	4,105	-3,268	-413	-664	1,164	-52	-807
1st	E	874	-4.13	86	-2.00	100	west	-1,179	-4,191	3,433	413	-315	-1,101	253	959
1st	E	874	-4.13	86	-2.00	100	east	1,179	4,191	3,433	413	2,043	7,281	2,611	9,341
1st	D	874	0	252	-1.94	100	west	-421	-13,250	-489	0	592	-13,690	1,250	-13,954

Notes:

P_{DF} = dead load distributed over floor area, which acts at the column line.

e_{DF} = distance between P_{DF} and centroid of wall section.

P_{DW} = dead load from wall self-weight.

e_{DW} = distance between P_{DW} and centroid of wall section.

4b. Vertical reinforcement.

The program PCACOL [PCA, 1999] is used to design the reinforcement in each wall pier. Figure 5-6 shows a wall section with the typical layout of vertical reinforcement. Typical reinforcement in the "column" portion of the wall piers is 8-#9 and typical vertical reinforcement in the wall web is #7@12. The PCACOL results of Figure 5-7a, 5-7b, and 5-7c show that this reinforcement is adequate in all locations except Line D at the 4th floor where 8-#10 are required instead of 8-#9. Figure 5-7d shows that the typical reinforcement provides adequate moment strength to the 20-foot long wall pier on Line D.

Figure 5-8 shows the vertical reinforcement provided in the wall piers to satisfy moment strength requirements. Note that the vertical reinforcement in the column portion of the 4th floor piers is increased to 8-#11 (from 8-#9 used at the lower levels), and that at the 5th and 6th floors is increased to 8-#10. The reasons for this will be discussed in Part 5 of this Design Example.

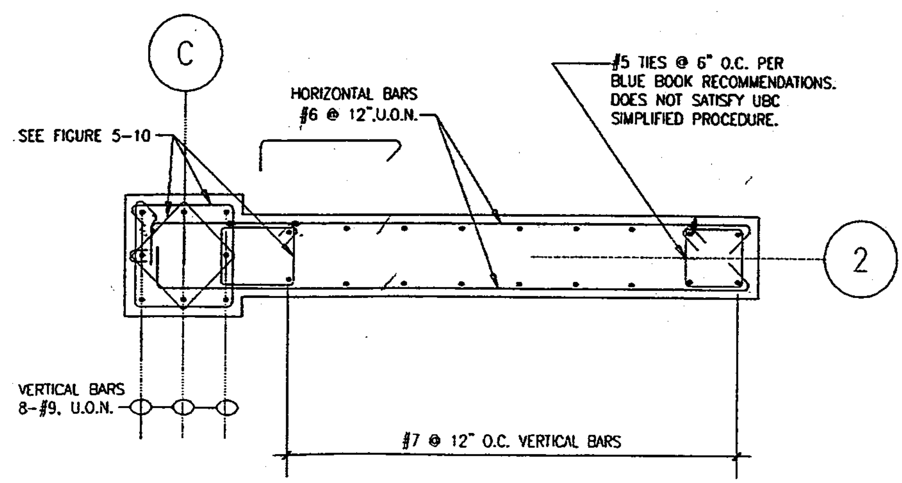


Figure 5-6. Section through wall pier in vicinity of Line C

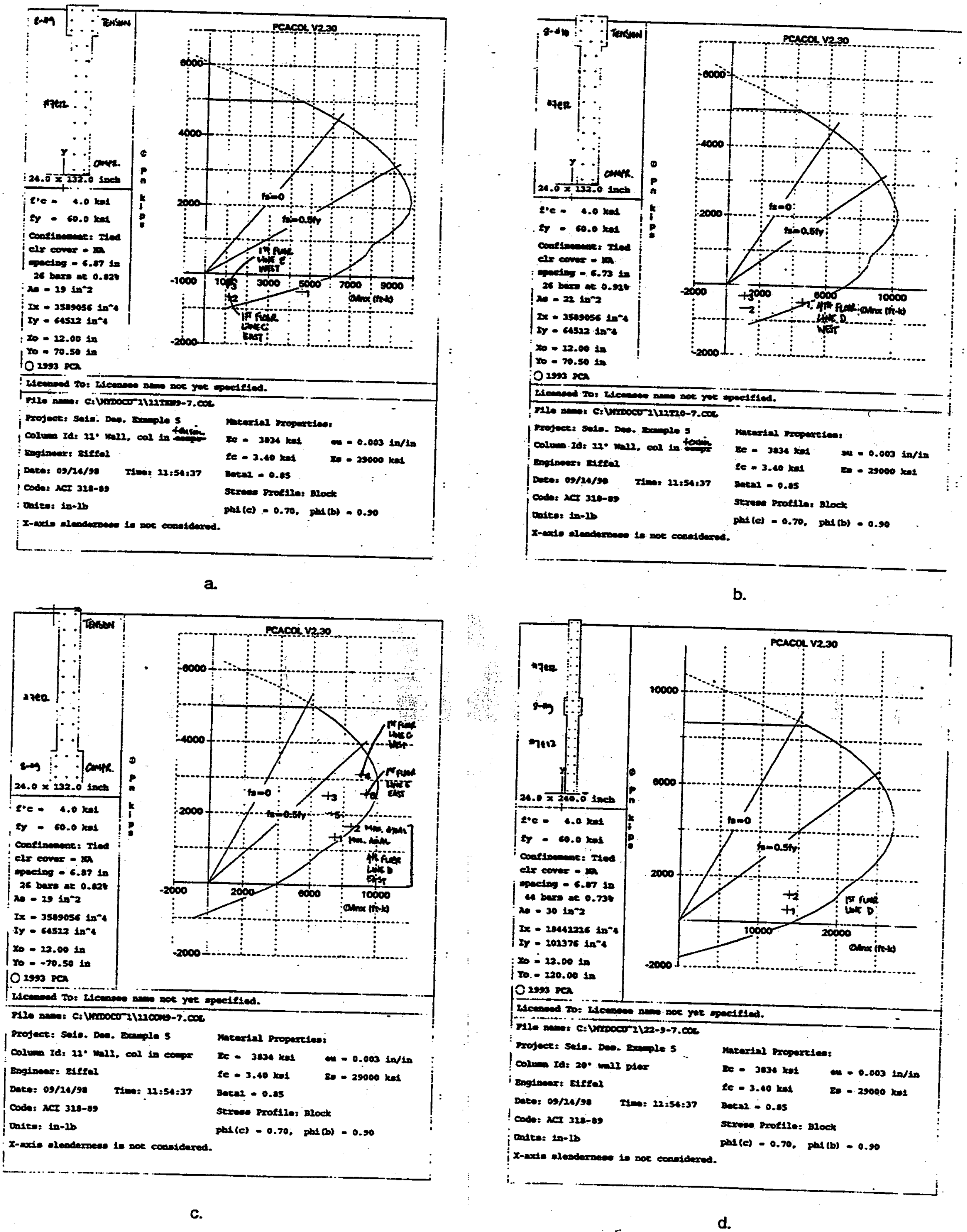


Figure 5-7. PCACOL results for design of vertical reinforcement

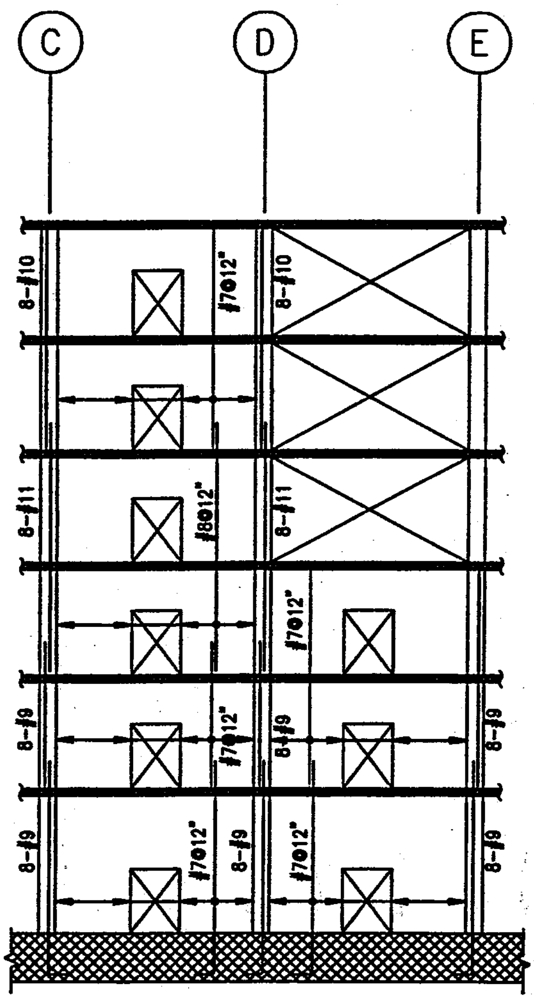


Figure 5-8. Elevation of vertical wall reinforcement

4c. Lap splice locations.

In general, lap splices should be avoided in potential plastic hinge regions of concrete structures. This is discussed in Part 4b of Design Example 4 and in Blue Book §C404.3. For this example wall, plastic hinging is expected (and desired) at the base of each wall pier and in the coupling beams. Plastic hinging may also be possible above the wall setback, in the 4th floor wall piers. (This will be investigated in more detail in Part 5 of this Design Example.)

Lap splices of the vertical wall reinforcement are located to avoid the potential plastic hinge regions in first floor and fourth floor wall piers, as shown in Figures 5-10 and 5-11 and in Tables 5-5 and 5-6 in Part 5B, below.

5. Plastic analysis of flexural mechanism in walls.

Blue Book §C402.8, C407.5.5.2

This part of the Design Example presents a plastic analysis methodology that is not a code requirement. It is included to assist the reader in understanding the post-elastic behavior of coupled shear walls and how they can be analyzed for seismic forces when elements of the wall are yielding.

Plastic analyses are not required by the UBC, but they are recommended in the SEAOC Blue Book: 1.) to establish shear demand corresponding to flexural strength, and 2.) to identify potential plastic hinge regions where special boundary and splicing requirements may be necessary. With the trend toward nonlinear static analysis (pushover) procedures, as called for in performance-based structural engineering guidelines [FEMA-273, 1997 and ATC-40, 1996], the ability to use plastic analyses will become increasingly important. The first three chapters of the textbook *Plastic Design in Steel* [ASCE, 1971] summarize the basic principles and methods of plastic design, and these are recommended reading for the interested reader.

Given below is an illustration of plastic analysis for the reinforced concrete walls and coupling beams of this Design Example.

5a. Probable moment strength.

The "probable flexural strength," M_{pr}, will be determined in calculating shear demands, according to the Blue Book recommendations. As defined in §1921.0, M_{pr} is calculated assuming a tensile stress in the longitudinal bars of $1.25 f_y$, and a strength reduction factor, ϕ, of 1.0. For the purposes of this plastic analysis, we will neglect earthquake axial forces E_v in calculating M_{pr} for each wall pier and assume an axial load of $1.2P_D + 0.5P_L$. In reality, the wall pier with earthquake axial tension will have a decreased moment strength, while the wall pier with earthquake axial compression will have an increased moment strength. These effects tend to cancel out so that our plastic analysis will give a good estimate of 1.) the governing mechanism of response, and 2.) the shear corresponding to the development of a mechanism at probable flexural strength. Table 5-4 shows M_{pr} values for the critical wall piers, based on the PCACOL results shown in Figure 5-9.

RESULTS MAY BE SENSITIVE TO P INDUCED BY Σ OF V_{CB}

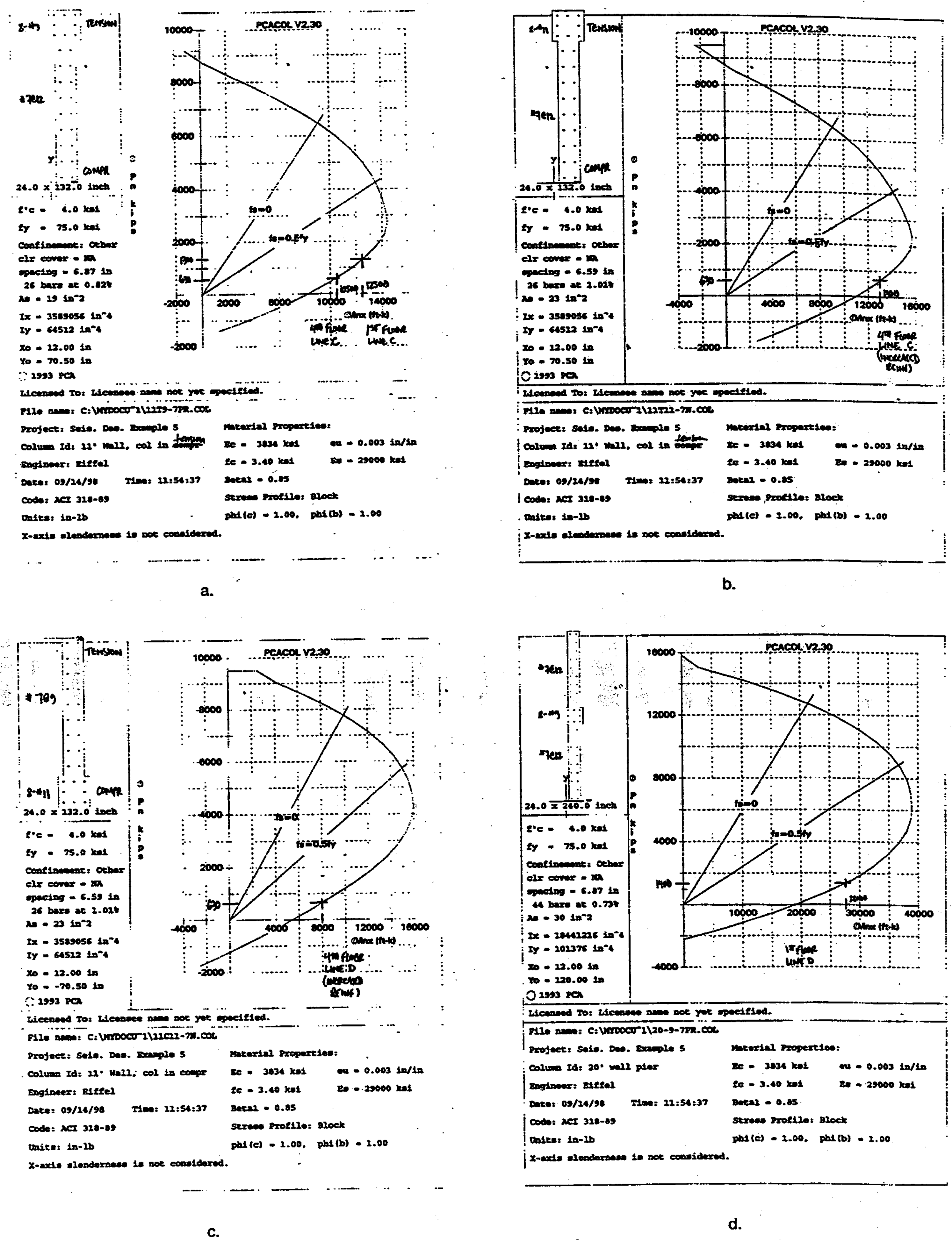


Figure 5-9. PCACOL calculation of probable moment strength $M_{pr}(f_y = 75\text{ksi}, \phi = 1.0)$

Table 5-4. Approximate probable moment strengths of wall piers for plastic analysis

Level	Grid Line	Reinforcement of Column Portion	Axial Load Considered $1.2P_D + 0.5P_L$ (kips)	M_{pr} (k-ft)
4th	C	8-#9	630	10,500
4th	D	8-#10	630	7,500
1st	C	8-#9	1,300	12,500
1st	D	8-#9	1,400	28,000
1st	E	8-#9	1,200	10,000
4th	C	8-#11	630	13,000
4th	D	8-#11	630	8,000

5b. Mechanism with plastic hinging at the base.

The preferred behavior of the wall occurs when plastic hinges occur at the base of the wall piers and in the coupling beams. This produces the desirable situation of flexural yielding, energy dissipation, and avoidance of shear failures.

Table 5-5 shows calculations of the shear strength of the preferred plastic mechanism, which has plastic hinges forming at the base of each wall pier and in each coupling beam. The equivalent plastic hinge length at the pier base, l_p, is taken equal to 5 feet.

The plastic hinge length is used in the calculation of external work shown in Table 5-5. The calculation is not sensitive to the value of l_p assumed, since $l_p/2$ is subtracted from h_i, the height above the base. In this case, the value of 5 feet is taken as one-half the wall length of the external wall piers. Although the central pier is longer, it is assigned the same plastic hinge length. Note that in the strain calculation procedure for wall boundary design, the value used for l_p has a significant effect on the results. This is discussed in Part 7 of Design Example 4.

Plastic lateral story displacements, Δ_i, increase linearly with height above the midpoint of the base plastic hinges. Δ_i is arbitrarily set equal to 1.00 feet at the roof. The external work equals the sum of each lateral story force, f_{xi}, times Δ_i.

The plastic rotation angle of the wall piers, θ, equals the roof displacement divided by the roof height above the midpoint of the plastic hinge. Thus, $\theta = 1.00/85.5$. The plastic rotation angle and internal work of the coupling beams can be calculated as follows:

$$\theta_{cb} = \theta \frac{l_c}{l_n}$$

where:

l_n = clear length of the coupling beam

l_c = distance between centroids of wall pier sections

Internal work $= \Sigma(\theta_{cb} \times M_{pr})$ for each end of each coupling beam

$$= \Sigma(\theta_{cb} \times 1.25V_n l_n/2)$$

$$= \Sigma(\theta \times 1.25V_n l_c/2)$$

$= \Sigma(\theta \times 1.25V_n l_c)$ for each coupling beam (sum of 2 ends)

The internal work of the base plastic hinges equals the sum of M_{pr} times θ for each of the three base plastic hinges. The summation of the internal work is shown in Table 5-5. Equating internal work with external work gives the solution of $V = 2{,}420$ kips.

Table 5-5. Plastic mechanism calculations assuming plastic hinging at base and in all coupling beams[1]

External Work					
Level	h_i *(ft)*	$h_i - l_p/2$ *(ft)*	Δ_i *(ft)*	$\frac{f_{xi}}{V}$	*Work / V (ft)*
R	88	85.5	1.000	0.254	0.254
6th	74	71.5	0.836	0.240	0.201
5th	60	57.5	0.673	0.195	0.131
4th	46	43.5	0.509	0.149	0.076
3rd	32	29.5	0.345	0.104	0.036
2nd	18	15.5	0.181	0.058	0.011
Sum				1.000	0.708
Internal Work, Coupling Beams					
Grid Line	*Level*	$1.25V_n$ *(k)*	l_c *(ft)*	*Work (k-ft)*	
C-D	R	291	21.5	73	
C-D	6th	468	21.5	118	
C-D	5th	671	21.5	169	
C-D	4th	368	21.5	93	
C-D	3rd	368	21.5	93	
C-D	2nd	480	21.5	121	
D-E	4th	528	21.5	133	
D-E	3rd	671	21.5	169	
D-E	2nd	609	21.5	153	
				1,120	
Internal Work, Wall Piers				$\theta = 1.00/85.5$	
Grid Line	*Level*	M_{pr} *(k-ft)*		*Work (k-ft)*	
C	base	12500		146	
D	base	28000		327	
E	base	10000		117	
				591	
V= (1120 + 591)/0.708 = 2,420 kips					

Note:

1. See Figure 5-10 for illustration of hinge locations.

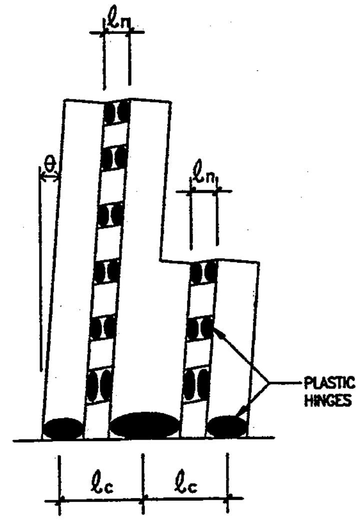


Figure 5-10. Mechanism with plastic hinges at base of wall

5c. Mechanism with plastic hinging at the 4th floor.

Table 5-6 shows calculations of the shear strength of another possible plastic mechanism, which has plastic hinges forming at the 4th floor wall piers and only in the coupling beams at the 5th, 6th, and roof levels. This plastic mechanism is less desirable than a mechanism with hinging at the base, because energy dissipation is concentrated in fewer yielding locations, and because plastic rotations in the wall piers would need to be much greater to achieve the same roof displacement.

As in the previous calculation, plastic lateral story displacements, Δ_i, increase linearly with height above the midpoint of the base plastic hinges, and Δ_i is set equal to 1.00 feet at the roof. For this mechanism, the plastic rotation angle of the wall piers, θ, equals 1.00/39.5. The plastic analysis solution, based on equating internal and external work, gives $V = 2{,}300\,\text{kips}$. Since this is less than 2,420 kips, the mechanism having plastic hinging at the 4th floor governs (i.e., is more likely to form than the preferred base mechanism shown in Figure 5-10).

To help prevent plastic hinging in the 4th floor piers, their flexural strength can be increased. Reinforcement of the column portions of these wall piers is increased to 8-#11. Table 5-6 shows revised internal work calculations. The solution gives $V = 2{,}460\,\text{kips}$. Since this is greater than 2420 kips, the preferred mechanism now governs.

Note that the calculation of the governing plastic limit load, V, depends on the assumed vertical distribution of lateral forces, which in actual seismic response can vary significantly from the inverted triangular pattern assumed. Thus the difference between $V = 2{,}420\,\text{kips}$ and 2,460 kips does not absolutely ensure against plastic hinging in the 4th floor wall piers.

Inelastic dynamic time-history analyses by computer generally show less predictability of yield locations than plastic analyses imply. For the wall of this Design Example, a time-history analysis might show some wall pier yielding both at the base and at the 4th floor. Interaction of the wall with other walls in the structure and with gravity framing can also influence the mechanism of yielding.

Plastic analyses are simpler to carry out and understand than most other analysis methods, particularly inelastic time-history analyses, and they offer valuable insight into the seismic performance of a structure. For this Design Example, the plastic analyses indicate that strengthening the 4th floor piers will protect the upper stories above the setback against high ductility demands, and make it more likely that the preferred mechanism will form.

Table 5-6. Plastic mechanism calculations assuming plastic hinging at 4th floor piers [1]

External Work					
Level	h_i (ft)	$h_i - l_p/2$ (ft)	Δ_i (ft)	$\frac{f_{xi}}{V}$	*Work/V* (ft)
R	42	39.5	1.000	0.254	0.254
6th	28	25.5	0.646	0.240	0.155
5th	14	11.5	0.291	0.195	0.057
4th			0.000	0.149	0.000
3rd			0.000	0.104	0.000
2nd			0.000	0.058	0.000
Sum				**1.000**	**0.466**
Internal Work, Coupling Beams					
Grid Line	*Level*	$1.25V_n$ (k)	l_c (ft)	*Work* (k-ft)	
C-D	R	291	17	125	
C-D	6th	468	17	201	
C-D	5th	671	17	289	
Sum				615	
Internal Work, Wall Piers				θ = 1.00/39.5	
Grid Line	*Level*	M_{pr} (k-ft)		*Work* (k-ft)	
C	4th	10500		266	
D	4th	7500		190	
Sum				456	
V= (615 + 456)/0.466 = 2,300 kips					

Note:

1. See Figure 5-11 for illustration of hinge locations.

Table 5-7. Plastic mechanism calculations assuming plastic hinging at 4th floor piers—revised for stronger piers at 4th floor

Internal Work, Wall Piers				$\theta = 1.00/39.5$	
Grid Line	Level	M_{pr} (k-ft)		Work (k-ft)	
C	4th	13000		329	
D	4th	8000		203	
Sum				532	
V= (615 + 532)/0.466 = 2,460 kips					

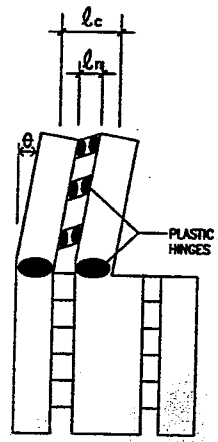


Figure 5-11. Mechanism with plastic hinges at 4th floor wall piers

6. Design of wall piers for shear.

In this part, the wall piers will be designed for shear. Both the UBC and Blue Book approaches will be illustrated. Design for the minimum UBC requirements is given in Part 6a below.

As discussed in Part 5 of Design Example 4, the SEAOC Blue Book contains more restrictive requirements than does the UBC for the shear design of reinforced concrete walls. The SEAOC approach, in Part 6b of this Design Example, is recommended for the reasons given in Design Example 4.

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6a. Design under UBC requirements.

Shear demand.

If designing to the minimum requirements of the UBC, the shear demand is taken directly from the design forces, factored by the load combinations discussed in Part 1. For the example wall, all of the significant shear on the wall piers results from earthquake forces, thus $V_u = V_E$, where the values V_E are those shown in Figure 5-3. The highest shears are at the 4th floor, Line D, with $V_E = 544\text{kips}$ in an 11-foot-long wall pier (48.5 k/ft), and at the 1st floor, Line D, with $V_E = 731\text{kips}$ in a 20-foot long wall pier (36.6 k/ft).

Shear capacity. §1921.6.5

UBC §1911.10 gives shear provisions for walls designed for *nonseismic* lateral forces such as wind or earth pressure. Section 1921.6.5 gives shear strength provisions for walls designed for *seismic* forces.

In Equation (21-7), wall shear strength depends on α_c, which depends on the ratio h_w/l_w.

$$V_n = A_{cv}\left(\alpha_c\sqrt{f'_c} + \rho_n f_y\right) \qquad (21\text{-}7)$$

Per §1921.6.5.4 the ratio h_w/l_w is taken as the larger of that for the individual wall pier and for the entire wall.

Overall wall	$h_w/l_w = 88'/54'$	$= 1.63$
11' long by 8' clear-height pier	$h_w/l_w = 8'/11'$	$= 0.73$
20' long by 8' clear-height pier	$h_w/l_w = 8'/20'$	$= 0.40$

Thus the value $h_w/l_w = 1.63$ governs for all wall piers. The coefficient α_c varies linearly from 3.0 for $h_w/l_w = 1.5$ to 2.0 for $h_w/l_w = 2.0$.

$$\alpha_c = 3.0 - 1.0(1.63 - 1.5)/(2.0 - 1.5) = 2.74$$

As prescribed in §1909.3.4.1, the shear strength reduction factor, ϕ, shall be 0.6 for the design of walls if their nominal shear strength is less than the shear corresponding to development of their nominal flexural strength. For the 11-foot long wall piers: §1921.6.5.3

$\ell_w(1.66 + 576\rho)$ [K-in]

$$\phi V_n = 0.6(16")l_w\left[2.74\sqrt{4{,}000} + \rho_n(60{,}000\text{psi})\right] = l_w(1.66\text{k}-\text{in.}+576\text{k}-\text{in. }\rho_n)$$

For the wall sections with highest shear, the amount of horizontal shear reinforcement is given in Table 5-8.

2#4 @10" EA. FACE

0.0025×10×16 = 0.4

132(1.66 + 576×0.0025) = 409

Table 5-8. Design for shear by the UBC

Level	*Grid Line*	*l_w (in.)*	*V_E (kips)*	*Horizontal Reinforcement*	*ρ_n*	*ϕV_n (kips)*	*$V_u/\phi A_{cv}\sqrt{f'_c}$* (1)
4th	C	132	371	#4@10" E.F.	0.00250	409	4.63
4th	D	132	544	#6@10" E.F.	0.00550	637	6.79
1st	C	132	283	#4@10" E.F.	0.00250	409	3.53
1st	D	240	731	#4@10" E.F.	0.00250	744	5.02
1st	E	132	316	#4@10" E.F.	0.00250	409	3.95

Note:

1. Under §1921.6.5.6, the value of $V_u/\phi A_{cv}\sqrt{f'_c}$ shall not exceed 10 for any wall pier, or 8 for an entire wall section.

As shown above, for all wall pier locations except the 4th floor at Line D, the minimum reinforcement ratio of 0.0025 (required under §1921.6.2.1) is sufficient to meet UBC shear strength requirements.

6b. Design using Blue Book recommendations.

Shear demand. **SEAOC 402.8, C402.8**

To comply with the Blue Book requirement of providing shear strength in excess of the shear corresponding to wall flexural strength, an amplified shear demand must be considered. For this Design Example, shear strength in excess of that corresponding to the development of probable flexural strength will be provided. This has been calculated by the plastic analysis in Part 5 of this Design Example as $V = 2{,}420\text{kips}$ at the base of the wall.

Section C402.8 of the Blue Book Commentary gives the following equation for the shear amplification factor, ω_v, that accounts for inelastic dynamic effects. For application to designs according to the UBC, the amplification factor recommended by Paulay and Priestley [1992] can be reduced by a factor of 0.85, because the Paulay and Priestley recommendations use a different strength reduction factor, ϕ, than does the UBC.

ω_v $= 0.85(0.9 + n / 10)$, for buildings up to 6 stories, where n = number of stories

$= 0.85(0.9 + 6 / 10) = 1.28$

As indicated in the Blue Book, the ω_v factor is derived for analysis using inverted triangular distributions of lateral forces. If a response spectrum analysis is carried out, a slightly lower ω_v factor can be justified in some cases.

At the base of the wall, the magnified shear demand $V_u{}^*$ is calculated as follows:

$$V_u{}^* = \omega_v \left(M_{pr}/M_u\right)\left(V_E\right) = \left(\omega_v 2{,}420\,\text{kips}\right) = 1.28(2{,}420) = 3{,}100\,\text{kips}$$

In the plastic analysis, the amplification effect considered by ω_v can instead be considered by using a different vertical distribution of the lateral forces, f_{xi}. Rather than using the inverted triangular distribution, a vertical distribution with a resultant located lower in the building, such as a uniform distribution pattern, could be used in the plastic analysis to give shear forces.

Shear capacity.

Since we are designing for the nominal shear strength to exceed the shear corresponding to flexural strength, a strength reduction factor, ϕ, of 0.85 can be used. As before, UBC Equation (21-6) is used to calculate shear capacity:

$$V_n = A_{cv}\left(\alpha_c \sqrt{f'_c} + \rho_n f_y\right) \tag{21-7}$$

$$\phi V_n = 0.85(16")\,l_w\left[2.74\sqrt{4{,}000} + \rho_n(60{,}000\text{psi})\right] = l_w(2.36\text{k} - \text{in.} + 816\text{k} - \text{in.}\rho_n)$$

For the shear demand of 3100 k over the net wall length of 42 feet (504 inches) at the first floor, the required amount of horizontal reinforcement is calculated:

$$\phi V_n = 504(2.36 + 816\rho_n) = 1{,}190 + 411{,}000\rho_n \geq 3{,}100$$

$$\rho_n = (3{,}100\text{k} - 1{,}190\text{k})/411{,}000 = 0.00464$$

Try #6 @ 12" o.c. each face

$$\rho_n = 2\left(0.44\text{in.}^2\right)/(12" \times 16") = 0.00458 \quad \textit{o.k.}$$

For the other stories of the building, the shear demands are magnified from the analysis results by the same proportion as for the first floor. The recommended amount of horizontal reinforcement can be calculated as shown in the Table 5-9.

Table 5-9. Design for shear by the Blue Book recommendations

Level	V_E (kips)	V_u * (kips) (1)	l_w net (in.)	Horizontal Reinforcement	ρ_n	ϕV_n (kips)
6th	338	788	264	#5@12" E.F.	0.00323	1,320
5th	656	1,530	264	#6@12" E.F.	0.00458	1,610
4th	915	2,130	264	#6@8" E.F.	0.00688	2,100
3rd	1,150	2,680	504	#6@12" E.F.	0.00458	3,070
2nd	1,250	2,920	504	#6@12" E.F.	0.00458	3,070
1st	1,310	3,100	504	#6@12" E.F.	0.00458	3,070

Note:

1. V_u * = magnified shear demand.

At the 4th floor wall piers, the vertical reinforcement must be increased from #7@12" to #8@12" to provide $\rho_v \geq \rho_n$, per §1921.6.55.5. The Blue Book deletes this requirement for the reasons given in Blue Book §C402.9. However, in this case, the increase in flexural strength of the 4th floor wall piers is desirable, as discussed in Part 5C, above.

6c. **Recommended shear reinforcement.**

A comparison of the Tables 5-8 and 5-9 shows that the Blue Book recommendations for ensuring that shear strength exceeds flexural capacity results in increased horizontal reinforcement compared to that required by the UBC. The Blue Book approach is recommended, as it leads to more ductile wall behavior.

7. **Boundary zone detailing of wall piers.**

The UBC gives two alternatives for determining whether or not boundary zone detailing needs to be provided: a simplified procedure (§1921.6.6.4), and a strain calculation procedure (§1921.6.6.5). For this Design Example, the simplified procedure will be used, and for comparison the Blue Book recommendations for the strain calculation procedure will be checked. For an illustration of the UBC strain calculation procedure, see Design Example 4.

7a. UBC simplified procedure. §1921.6.6.4

Under the requirement of §1921.6.6.4, boundary zone detailing need not be provided in the example wall if the following conditions are met:

$$P_u \le 0.10 A_g f'_c \qquad (P_u \le 0.05 A_g f'_c \text{ for unsymmetrical wall sections})$$

and either

$$M_u / (V_u l_w) \le 1.0$$

or

$$V_u \le 3 A_{cv} \sqrt{f'_c}$$

For the critical piers of the example wall, $P_u/A_g f'_c$ calculated as shown in Table 5-10. All of the piers are geometrically unsymmetrical, except for those on Line D at the 1st, 2nd, and 3rd stories. Of the unsymmetrical piers, only those at the 6th floor have $P_u/A_g f'_c \le 0.005$ and $V_u \le 3A_{cv}\sqrt{f'_c}$. All three of the symmetrical piers have $P_u / A_g f'_c \le 0.01$ and $V_u \le 3A_{cv}\sqrt{f'_c}$. Therefore all piers require boundary confinement except those at the 6th floor, and those on Line D at the 1st, 2nd, and 3rd floors.

The required boundary zone length is calculated as a function of $P_u / A_g f'_c$ per §1921.6.6.4. The code requires that shear walls and portions of shear walls not meeting the conditions of §1921.6.6.4 and having $P_u < 0.35 P_o$ shall have boundary zones at each end over a distance that varies linearly from $0.25 l_w$ to $0.15 l_w$ as P_u varies from $0.35 P_o$ to $0.15 P_o$. The boundary zone shall have a minimum length of $0.15 l_w$ and shall be detailed in accordance with §1921.6.6.6. The results of this determination are shown in Table 5-10.

Table 5-10. Boundary zone strength requirement by the UBC simplified procedure

Level	Line	P_u $(1.44P_D + 0.5P_L + P_E)$ (kips)	A_g (in.²)	$\frac{P_u}{A_g f'_c}$	(Required Boundary Length) ÷ l_w	Required Boundary Length (in.)
6th	C,D	388	2,300	0.042	not required	not required
4th	D	1,675	2,300	0.182	0.166	21.9
1st	C	3,148	2,300	0.342	0.246	32.5
1st	E	2,611	2,300	0.284	0.217	28.6
1st	D	1,250	4,030	0.078	not required	not required

At the column end of each wall pier, confining the 8 column bars plus two wall-web bars gives a boundary zone length of 34 inches. At the inside (doorway) end of each wall pier, confining 8 bars give a boundary zone length of 39 inches. The confinement details are shown in Figure 5-12. The required area of boundary ties is calculated according to Equation (21-10):

$$A_{sh} = 0.09 s h_c \, f'_c / f_y \tag{21-10}$$

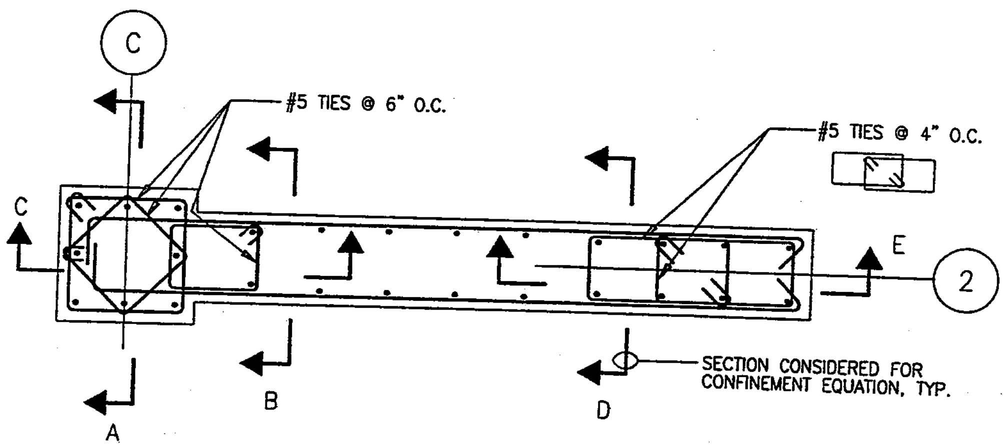

Figure 5-12. Boundary ties required by the UBC simplified procedure

Calculations of A_{sh} are given in Table 5-11, corresponding to section cuts A, B, C, D, and E through the boundary zones as shown in Figure 5-10.

Table 5-11. Required boundary zone ties by the UBC simplified procedure

Section Cut	h_c (in.)	s (in.)	A_{sh} Required (in.^{2})	Tie legs	A_{sh} Provided (in.^{2})
A	20.5	6	0.74	3-#5	0.93
B	12.5	6	0.45	2-#5	0.62
C	32	6	1.12	4-#5	1.24
D	12.5	4	0.45	2-#5	0.62
E	37.5	4	0.90	4-#5	1.24

Note:
1. See Figure 5-12.

7b. **Blue Book recommendations.** SEAOC §402.11

Section 402.11 of the Blue Book contains significant revisions to the UBC provisions for wall boundary confinement. Sections 402.11.1 and 402.11.2 revise definitions used in the strain calculation procedure of §1921.6.6.5. Blue Book §402.11.3 adds the following two exceptions to the UBC procedure:

> ***Exception 1***: Boundary zone details need not be provided where the neutral axis depth c'_u is less than $0.15l_w$.
>
> ***Exception 2***: The length of wall section at the compression boundary over which boundary zone detailing is to be provided may be taken as c_c, where c_c is the larger of $c'_u = 0.1l_w$ or $c'_u/2$.

In applying these recommendations to the example wall, the wall piers with the largest neutral axis depth-to-length ratio, c'_u/l_w, govern the design. The largest neutral axis depth at the column end of a wall pier occurs at the 1st floor at Line C, where a large downward earthquake axial force occurs:

$$P'_u = (1.2P_D + 0.5P_L) + P_E = 1{,}300 \text{ kips} + 1{,}600 \text{ kips} = 2{,}900 \text{ kips}$$

The neutral axis depth, c'_u, for this case is calculated by PCACOL to be 48 inches.

$$c'_u/l_w = 48''/132'' = 0.36 \geq 0.15 \text{ therefore boundary zone detailing is required}$$

$$c_c = c'_u - 0.1l_w = 48'' - 0.1(132'') = 35 \text{ in.} \qquad \textit{governs}$$

$$c_c = c'_u/2 = 48''/2 = 24 \text{ in.} \qquad \textit{does not govern}$$

The calculation of $c_c = 35$ inches can be compared to the required UBC boundary length of 32.5 inches shown in the Table 5-10.

The largest neutral axis depth at the inside (doorway) end of a wall pier occurs at the 1st floor Line E. Compression at this end of the wall pier corresponds to the loading direction that has earthquake axial force acting upward:

$$P'_u = (1.2P_D + 0.5P_L) + P_E = 1{,}200 \text{kips} - 1{,}180 \text{kips} = 20 \text{ kips}$$

The neutral axis depth, c'_u, for this case is calculated by PCACOL to be 20 inches.

$$c'_u/l_w = 20''/132'' = 0.15 \geq 0.15$$

Thus, the requirement for boundary confinement at the inside (doorway) ends of the wall piers is marginal.

$$c_c = c'_u - 0.1l_w = 20" - 0.1(132") = 7 \text{ in.} \qquad \textit{does not govern}$$

$$c_c = c'_u/2 = 20"/2 = 10 \text{ in.} \qquad \textit{governs}$$

The calculation of $c_c = 10"$ can be compared to the required boundary length of 28.6 inches shown in the Table 5-10. Figure 5-6 shows the ties resulting from the Blue Book recommendation, which can be compared to those required by the UBC simplified procedure, shown in Figure 5-12.

8. Detailing of coupling beams.

The detailing of coupling beams may require a number of preliminary design iterations to determine required bar sizes and the lateral dimensions of the diagonal bar group. Preliminary design iterations are not shown in this Design Example.

8a. Layering of reinforcement.

For this Design Example, the recommended layering of reinforcement in the coupling beams is shown in Figure 5-13. The proposed layering corresponds to a clear cover of 1 inch in the coupling beam and 1 3/8 inches in the wall pier.

Section 1921.6.10.3 requires transverse reinforcement around each group of diagonal bars of the coupling beam. Figure 5-13 assumes that these ties are No. 4 in size and extend over the portion of the diagonal bars within the coupling beam length, as shown in Figure 5-14. Thus the diagonal bars, but not the ties around them, must pass between the reinforcement curtains of the wall pier.

The layering shown in Figure 5-13 results in a diagonal bar cage with lateral "core" dimensions of 9.0 inches by 14.8 inches, measured outside-to-outside of the ties. These dimensions conform to the requirement of §1921.6.10.2 that the lateral core dimensions be "not less than $b_w/2$ or 4 inches."

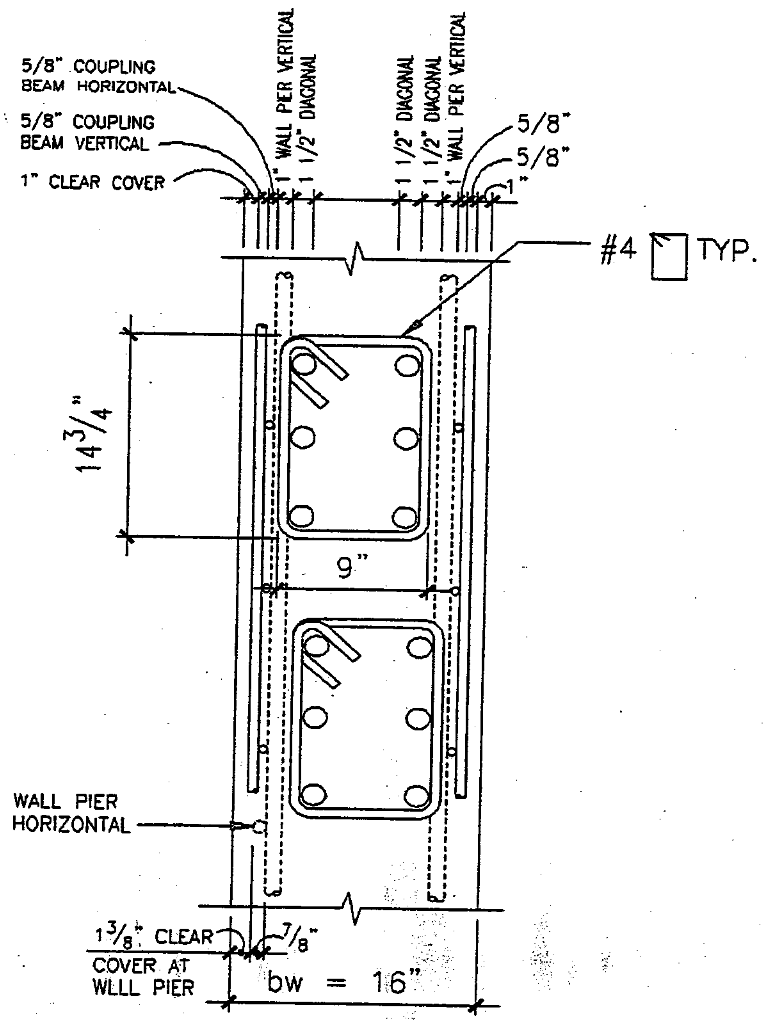

Figure 5-13. Section through coupling beam showing layering of reinforcement

8b. **Ties around diagonal bars.** **§1921.4.4**

Under the requirements of §1921.6.10.3, the required transverse reinforcement around diagonal bars must conform to §1921.4.4.1 through §1921.4.4.3. Section 1921.4.4.2 requires a maximum tie spacing of 4 inches or one-quarter of the minimum member dimension.

Equations (21-3) and (21-4) must be checked in each direction.

$$A_{sh} = 0.3\left(sh_c\, f'_c/f_y\right)\left[\left(A_g/A_{ch}\right)-1\right] \qquad (21\text{-}3)$$

$$A_{sh} = 0.09 sh_c\, f'_c/f_y \qquad (21\text{-}4)$$

The quantity A_g is calculated assuming the minimum cover per §1907.7 around each diagonal bar core. For walls with No. 11 bars and smaller, without exposure to weather, this minimum cover equals ¾ inch. Thus:

$$A_g = [9.0 + 2(0.75)] \times [14.8 + 2(0.75)] = 10.5 \times 16.3 = 171 \text{ in. and}$$

$$A_{ch} = 9.0 \times 14.8 = 133 \text{ in.}$$

Although A_{ch} is based on outside-to-outside of tie dimensions, h_c is based on center-to-center of tie dimensions. Assuming No. 4 ties, $h_c = 9.0 - 0.5 = 8.5$ inches in the horizontal direction, and $h_c = 14.8 - 0.5 = 14.3$ inches in the other lateral dimension. For $h_c = 8.5$:

$$A_{sh} = 0.3(sh_c f'_c / f_{yh})[(A_g / A_{ch}) - 1] \qquad (21\text{-}3)$$

$$= 0.3[(4")(8.5")(4\text{ksi})/60\text{ksi}](171/133 - 1) = 0.194\text{in.}^2$$

$$A_{sh} = 0.09sh_c f'_c / f_{yh} = 0.09(4")(8.5")(4\text{ksi})/(60\text{ksi}) = 0.204\text{in.}^2 \quad \textit{governs} \qquad (21\text{-}4)$$

For $h_c = 14.3$:

$$A_{sh} = 0.3(sh_c f'_c / f_{yh})[(A_g / A_{ch}) - 1] \qquad (21\text{-}3)$$

$$= 0.3[(4")(14.3")(4\text{ksi})/60\text{ksi}](171/133 - 1) = 0.327\text{in.}^2$$

$$A_{sh} = 0.09sh_c f'_c / f_{yh} = 0.09(4")(14.3")(4\text{ksi})/(60\text{ksi}) = 0.343\text{in.}^2 \quad \textit{governs} \qquad (21\text{-}4)$$

A single #4 tie around the six diagonal bars provides two tie legs in each direction and $A_{sh} = 0.40$ in.^{2} A #3 perimeter tie with a #3 crosstie would provide $A_{sh} = 0.22$in.^{2} across the shorter core direction and $A_{sh} = 0.33$in.^{2} across the longer core direction, which would not quite meet the A_{sh} requirement of 0.343 in.^{2}

Per §1921.4.4.3, crossties shall not be spaced more than 14 inches on center. For the heaviest diagonal reinforcement of 6-#10 bars, the center-to-center dimension of the #10 bars is given as 12 inches in Figure 5-14. The center-to-center hoop dimension in this direction thus equals 12 inches plus one diameter of a #10 bar plus one diameter of a #4 tie, equal to 12.0 + 1.27 + 0.5 = 13.8 inches. Since this is less than 14 inches, a crosstie is not needed.

The diagonal bars must be developed for tension into the wall piers. Following the recommendation of Paulay and Priestley [1992], the bars are extended a distance of $1.5l_d$ beyond the face of the supporting wall pier, as shown in Figure 5-14, where l_d is the development length of a straight bar as determined under §1912.2.

Crossties are added at the intersection of the diagonal bars at the center of the coupling beam, and along their development into the wall piers, as shown in Figure 5-14. The crossties are also added in locations where ties around the diagonal bars are not used.

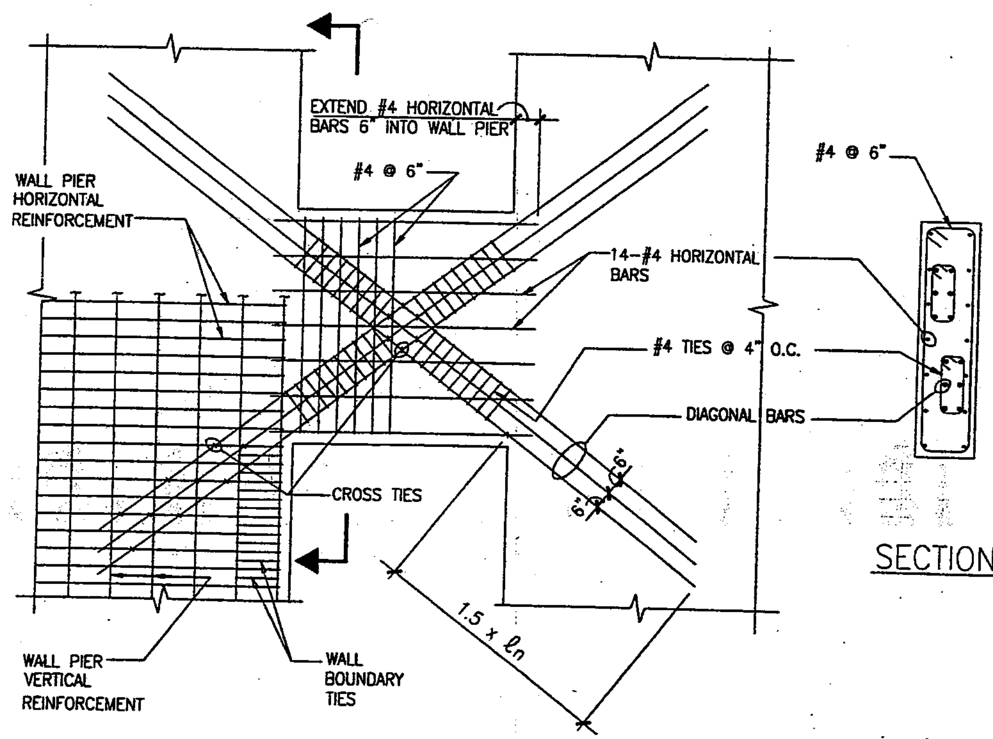

Figure 5-14. Elevation showing detailing of a coupling beam

8c. Reinforcement "parallel and transverse." §1921.6.10.4

Section 1921.6.10.4 requires reinforcement parallel and transverse to the longitudinal axis of the coupling beam, conforming to §1910.5, §1911.8.9, and §1911.8.10. The Blue Book contains less restrictive requirements (in §402.13) for this reinforcement, and the Blue Book Commentary notes that the UBC requirements referenced should not be applied because the diagonal bars, not the parallel and transverse bars, act as the principal flexural and shear reinforcement.

UBC requirements.

By §1911.8.9, for #4@6 transverse (vertical) bars:

$$A_v \geq 0.0015 b_w s = 0.0015(16")(6") = 0.144 \text{in.}^2 \leq 0.40 \text{ in.}^2 \quad o.k.$$

By §1911.8.10, for 14-#4 longitudinal (horizontal) bars:

$$A_{vh} \geq 0.0025 b_w s_2 = 0.0025(16")(72"/7) = 0.41 \text{in.}^2 \cong 0.40 \text{ in.}^2 \quad o.k.$$

By §1910.5.1:

$$A_{s,min} = 200 b_w d / f_y = 200(16")(0.8 \times 72")/60{,}000 \text{psi} = 3.07 \text{ in.}^2 \qquad (10\text{-}3)$$

This requires 7-#6 longitudinal bars ($A_s = 7(0.44 \text{ in.}^2) = 3.08 \text{ in.}^2$) both top and bottom of the coupling beam, or 14-#6 longitudinal bars total. Per the discussion below, these are not recommended by SEAOC to be used, and are not shown in Figure 5-14.

Blue Book recommendations.

Blue Book Commentary §C402.13 cautions against providing excess longitudinal reinforcement in the coupling beam, as required by the application of UBC §1910.5.1. The 1999 ACI code eliminates the requirement of UBC §1910.5.1.

The Blue Book recommends using less longitudinal reinforcement. This can be justified on the basis of UBC §1910.5.3, which states that the requirements of §1910.5.1 need not be applied if the reinforcement provided is "at least one-third greater than that required by analysis." Since the diagonal bars resist the entire flexural tension forces, it could be interpreted that no additional longitudinal reinforcement is required by analysis.

In §402.13 of the Blue Book requires the reinforcement parallel to the longitudinal axis of the beam to be at least No. 3 in size, spaced at not more than 12 inches on center. The reinforcement transverse to the longitudinal axis of the beam must be at least No. 3 in size, spaced at not more than 6 inches on center.

Figure 5-14 shows the recommended parallel and transverse reinforcement: 14-#4 bars longitudinally and #4 ties @ 6" transversely.

Per the Blue Book recommendations of §402.13, the longitudinal reinforcement is extended 6 inches into the wall pier, as shown in Figure 5-14, but is *not* developed for tension.

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