Chapter 4 Ductile Coupled Reinforced Concrete Shear Walls

2020 NEHRP Provisions Training Materials

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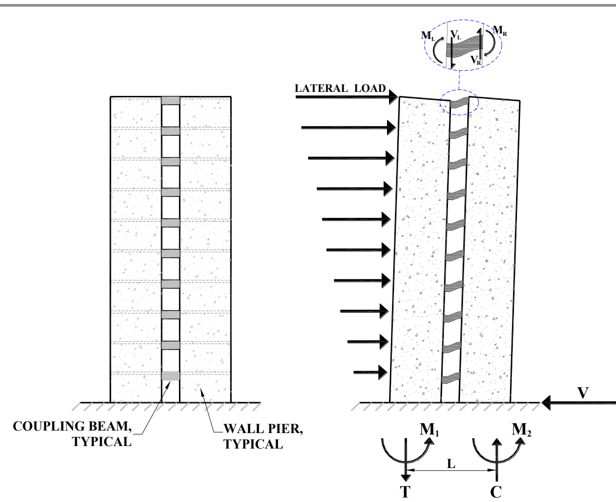
Coupled Walls



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Coupled Walls

Courtesy: Cary Kopczynski & Company, Bellevue, WA

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Coupled Walls

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Coupled Walls

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Coupled Walls

Coupled shear wall systems are recognized as distinct from isolated shear wall systems in Canadian and New Zealand codes; they are also accorded higher response modification factors in view of their superior seismic performance. ASCE/SEI 7, prior to its 2022 edition, made no such distinction.

Ductile Coupled Shear Walls

Bertero wrote in 1977: "Use of coupled walls in seismic-resistant design seems to have great potential. To realize this potential it would be necessary to prove that it is possible to design and construct "ductile coupling girders" and "ductile walls" that can SUPPLY the required strength, stiffness, and stability and dissipate significant amounts of energy through stable hysteretic behavior of their critical regions."

Thus, discussion needs to focus not on just coupled walls, but **ductile coupled walls** consisting of **ductile shear walls** and **ductile coupling beams.**

Energy Dissipation in Coupling Beams

MKA Study:

Non-linear response history analyses were conducted using spectrally matched ground motion records on a variety of coupled shear wall archetypes. Archetypes ranged from 5 to 50 stories in height and contained a range of longitudinal reinforcement ratios in the coupling beams as well as the shear walls.

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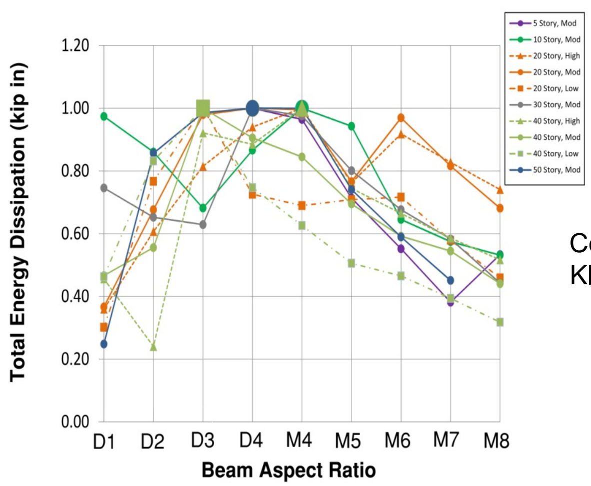
Energy Dissipation in Coupling Beams



Courtesy: Magnusson Klemencic Associates

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ACI 318-19 18.10.9 Ductile Coupled Walls

18.10.9.1 Ductile coupled walls shall satisfy the requirements of this section.

18.10.9.2 Individual walls shall satisfy $h_{wcs}/\ell_w \geq 2$ and the applicable provisions of 18.10 for special structural walls.

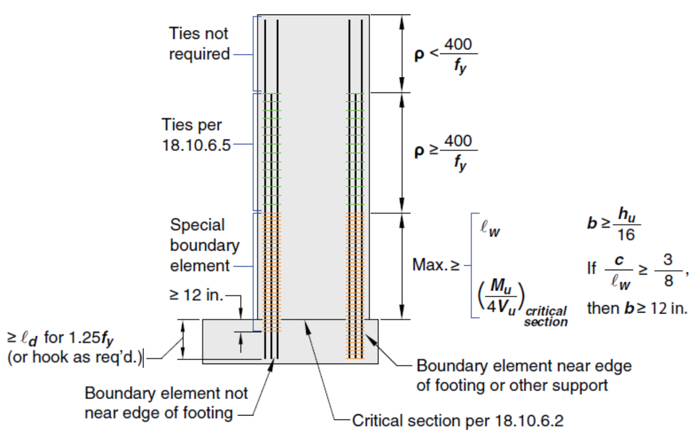
18.10.9.3 Coupling beams shall satisfy 18.10.7 [Coupling beams] and (a) through (c) in the direction considered.

(a) Coupling beams shall have $\ell_n/h \geq 2$ at all levels of the building.

(b) All coupling beams at a floor level shall have $\ell_n/h \leq 5$ in at least 90 percent of the levels of the building.

(c) The requirements of 18.10.2.5 shall be satisfied at both ends of coupling beams [reinforcement developed for $1.25f_y$].

Special Shear Walls

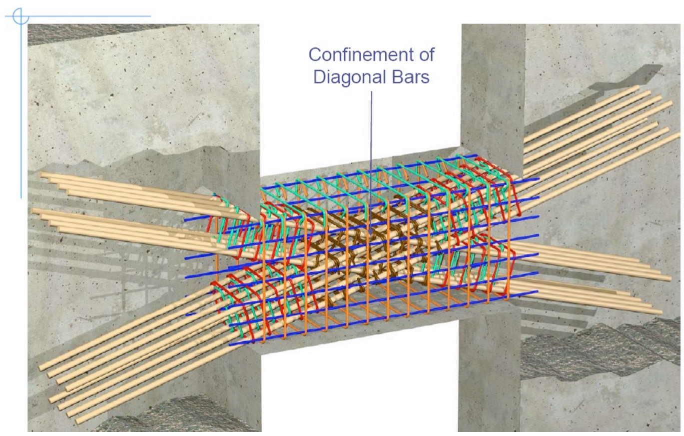


(a) Wall with $h_w/\ell_w \geq 2.0$ and a single critical section controlled by flexure and axial load designed using 18.10.6.2, 18.10.6.4, and 18.10.6.5

Ductile Coupling Beams

- Aspect ratio $\ell_n/h \geq 4$
  - Satisfy requirements of 18.6
- Aspect ratio $\ell_n/h < 4$
  - Permitted to be reinforced with two intersecting groups of diagonal bars
- Aspect ratio $\ell_n/h < 2$ and $V_u > 4\sqrt{f'_c}A_{cw}$
  - Must be reinforced with two intersecting groups of diagonal bars

Ductile Coupling Beams



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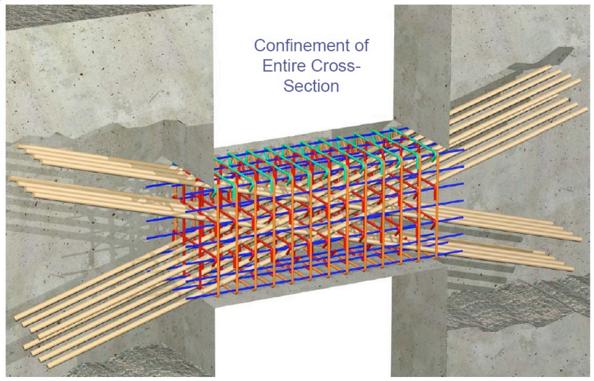
Ductile Coupling Beams

ACI 318-08 Alternate Detail



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2020 *NEHRP Provisions*


NEHRP Recommended Seismic Provisions for New Buildings and Other Structures

Volume I: Part 1 Provisions, Part 2 Commentary

FEMA P-2082-1/ September 2020

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2020 *NEHRP Provisions*

- Part 1: Modifications to ASCE/SEI 7-16
- Part 2: Commentary to the Modifications
- Part 3: Resource Papers

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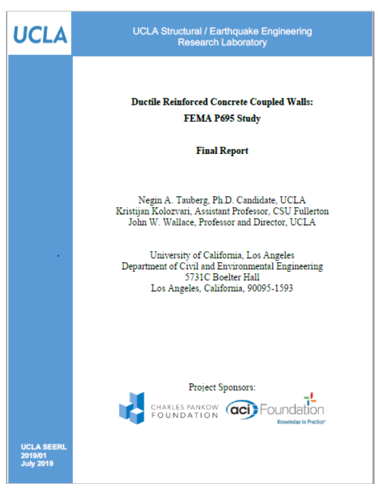
P695 Study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Ductile Reinforced Concrete Coupled Walls: FEMA P695 Study

Final Report

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P-695 Study

Forty-one ductile coupled shear wall buildings were designed using a range of variables expected to influence the collapse margin ratio; the primary variables were building height (i.e., 6, 8, 12, 18, 24, and 30 stories), wall cross section (i.e., planar and flanged walls), coupling beam aspect ratio (ℓ_n/h) ranging from 2.0 to 5.0, and coupling beam reinforcement arrangement (i.e., diagonally and conventionally reinforced).

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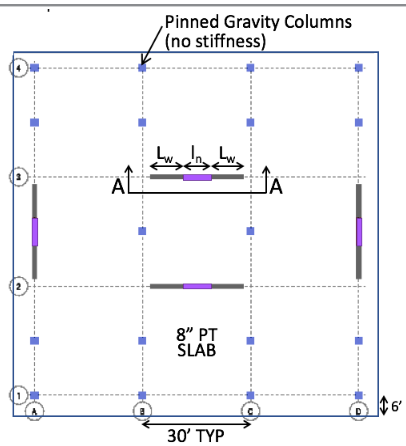
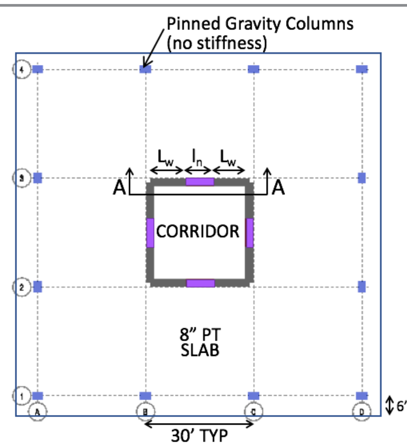
P-695 Study – Plan Views of Archetype Buildings



Typical Plan View



Typical Plan View

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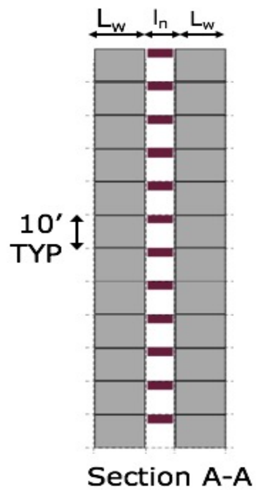
P-695 Study – Elevation View of Archetype Buildings



P-695 Study – Design Parameters

The designs were for Risk Category II structures with an importance factor I_e = 1.0. It incorporated provisions of ASCE/SEI 7-16 and ACI 318-19 as well as the seismic design parameters specified in FEMA P695 (importance factor, redundancy factor, and site class and spectral values). The redundancy factor ρ was taken equal to 1.0. The seismic spectral acceleration values used are summarized below for seismic hazard D_{max} as specified in FEMA P695.

- S_S = 1.5g F_a = 1.0 S_{DS} = 1.00 g
- S_1 = 0.6g F_v = 1.5 S_{D1} = 0.60g

Additional ACI 318-19 Changes in Special Shear Wall Design

There have been four significant ACI 318-19 code changes, all adopted in our FEMA P695 study, to address the flexural-compression wall failure issue.

(1) **18.10.3.1** (shear amplification) - would typically require design shear (required shear strength) V_u to be amplified by a factor of up to 3 (similar to New Zealand, Canada).

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Additional ACI 318-19 Changes in Special Shear Wall Design

(2) **18.10.6.4** - requires improved wall boundary and wall web detailing, i.e, overlapping hoops if the boundary zone dimensions exceed 2:1, crossties with 135-135 hooks on both ends, and 135-135 crossties on web vertical bars.

(3) **18.10.6.2(b)** (Wall drift or deformation capacity check) - requires a low probability of lateral strength loss at MCE level hazard (you can think of it as requiring a minimum wall compression zone thickness), and

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Additional ACI 318-19 Changes in Special Shear Wall Design

(4) **18.10.2.4** - Minimum wall boundary longitudinal reinforcement, to limit the potential of brittle tension failures for walls that are lightly-reinforced.

Shear Amplification: Concrete Shear Walls

18.10.3.1 The design shear force V_e shall be calculated by:

$$V_e = \Omega_v \omega_v V_u \leq 3V_u \qquad (18.10.3.1)$$

where V_u, Ω_v, and ω_v are defined in 18.10.3.1.1, 18.10.3.1.2, and 18.10.3.1.3, respectively.

18.10.3.1.1 V_u is the shear force obtained from code lateral load analysis with factored load combinations.

Shear Amplification: Concrete Shear Walls

18.10.3.1.2 Ω_v shall be in accordance with Table 18.10.3.1.2.

Table 18.10.3.1.2—Overstrength factor Ω_v at critical section

Condition	Ω_v	
$h_{wcs}/\ell_w > 1.5$	Greater of	M_{pr}/M_u[1]
		1.5[2]
$h_{wcs}/\ell_w \leq 1.5$	1.0	

[1] For the load combination producing the largest value of Ω_v.

[2] Unless a more detailed analysis demonstrated a smaller value, but not less than 1.0.

Shear Amplification: Concrete Shear Walls

18.10.3.1.3 For walls with $h_{wcs}/\ell_w < 2.0$, ω_v shall be taken as 1.0. Otherwise, ω_v shall be calculated as:

$$\omega_v = 0.9 + \frac{n_s}{10} \qquad n_s \leq 6$$

$$\omega_v = 1.3 + \frac{n_s}{30} \leq 1.8 \qquad n_s > 6 \qquad (18.10.3.1.3)$$

where n_s shall not be taken less than the quantity $0.007h_{wcs}$.

Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

Basic Seismic Force-resisting System	Detailing Reference Section	R	Ω_0	C_d	System Limitations And Building Height Limitations (Ft) By Seismic Design Category				
					B	C	D	E	F
A. Bearing Wall System									
1. Special reinforced concrete shear walls	14.2	5	$2^1/_2$	5	NL	NL	160	160	100
2. Ductile Coupled reinforced concrete shear walls[q]	14.2	8	$2^1/_2$	8	NL	NL	160	160	100
3. Ordinary reinforced concrete shear walls	14.2	4	$2^1/_2$	4	NL	NL	NP	NP	NP

[q] Structural height, h_n, shall not be less than 60 ft (18.3 m).

Minimum height is intended to ensure adequate degree of coupling and significant energy dissipation provided by the coupling beams.

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Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

Basic Seismic Force-resisting System	Detailing Reference Section	R	Ω_0	C_d	System Limitations And Building Height Limitations (Ft) By Seismic Design Category				
					B	C	D	E	F
B. Building Frame System									
4. Special reinforced concrete shear walls	14.2	6	$2^1/_2$	5	NL	NL	160	160	100
5. Ductile Coupled reinforced concrete shear walls[q]	14.2	8	$2^1/_2$	8	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls	14.2	5	$2^1/_2$	$4^1/_2$	NL	NL	NP	NP	NP

[q] Structural height, h_n, shall not be less than 60 ft (18.3 m).

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Earthquake Force-Resisting Structural Systems of Concrete — ASCE/SEI 7-22

Basic Seismic Force-resisting System	Detailing Reference Section	*R*	Ω_0	C_d	System Limitations And Building Height Limitations (Ft) By Seismic Design Category				
					B	C	D	E	F
D. Dual Systems with Special Moment Frames									
3. Special reinforced concrete shear walls	14.2	7	$2^1/_2$	$5^1/_2$	NL	NL	NL	NL	NL
4. Ductile Coupled reinforced concrete shear walls[q]	14.2	8	$2^1/_2$	8	NL	NL	NL	NL	NL
5. Ordinary reinforced concrete shear walls	14.2	6	$2^1/_2$	5	NL	NL	NP	NP	NP

[q] Structural height, h_n, shall not be less than 60 ft (18.3 m).

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Example Problem

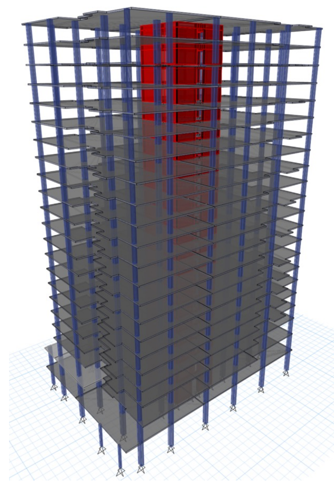
Design of a Special Reinforced Concrete Ductile Coupled Wall

Introduction

A 22-story reinforced concrete residential building is designed following the requirements of ASCE/SEI 7-22, and ACI 318-19. The building consists of a flat plate-column gravity system with a central core, formed by four reinforced concrete coupled structural walls, which acts as the seismic force-resisting system. The structural walls are designed as Ductile Coupled Reinforced Concrete Shear (Structural) Walls.

A computer rendering of the building framing is shown on the next two slides. The plan view of the building changes from one floor to another. A plan view of the second floor of the building is shown.

Example Building Configuration



3D View

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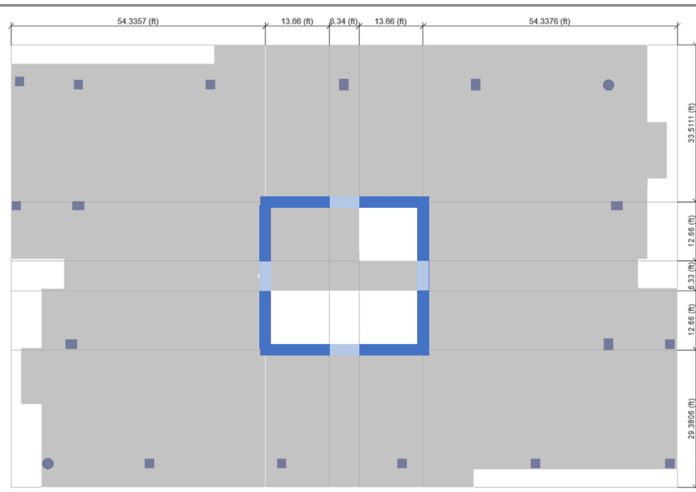
Example Building Configuration



Second Floor Plan View

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Design Criteria

- Member Sizes:
  - Shear walls: 26 in. thick
  - Slabs (2nd and 3rd floors): 8 in. thick
    (4th floor and higher): 7.5 in. thick
  - Gravity columns: Various sizes
- Material properties:
  - Concrete (used in structural walls and columns): f_c' = 8000 psi (all floors)
  - Concrete (used in slabs): f_c' = 6000 psi (floors)
  - All members are constructed of normal weight concrete (w_c = 150 pcf)
  - Reinforcement (used in all structural members): f_y = 60,000 psi

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Design Criteria

- Service Loads:
  - Superimposed dead load: 25 psf (includes SDL on the floor plus the weight of cladding distributed over the floor slab.)
  - Live load: Based on the 40 psf live load prescribed in ASCE/SEI 7-22 Table 4.3-1 for residential buildings (private rooms and corridors serving them), a reduced live load of 20 psf is used in the example.
  - Reduced roof Live load: 20 psf
- Seismic Design Data:
  - Risk Category: II
  - Seismic importance factor, I_e = 1.0
  - Site Class: D

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Design Criteria

- Seismic Design Data (contd.):
  - The maximum considered earthquake spectral response acceleration:
    At short periods, S_S = 1.65g, and
    At 1-sec period, S_1 = 0.65g.
  - The maximum considered earthquake spectral response acceleration (site modified):
    At short periods, S_{MS} = 1.65g, and
    At 1-sec period, S_{M1} = 0.98g.
  - Design Spectral Response Acceleration Parameters (at 5% damping):
    At short periods: $S_{DS} = 2/3\ S_{MS}/g = 2/3 \times 1.65 = 1.10$
    At 1-sec period: $S_{D1} = 2/3\ S_{M1}/g = 2/3 \times 0.98 = 0.65$

Design Criteria

- Seismic Design Data (contd.):
  - Long-period transition period, T_L = 8 sec
  - Ductile Coupled Reinforced Concrete Structural Walls ... R = 8; C_d = 8.0, Ω_0 = 2.5 (ASCE/SEI 7-22 Table 12.2-1)
  - Seismic Design Category: Based on both S_{DS} (ASCE/SEI 7-22 Table 11.6-1) and S_{D1} (ASCE/SEI 7-22 Table 11.6-2), the Seismic Design Category (SDC) for the example building is D.

Design Procedure

Although ASCE/SEI 7-22 permits the Equivalent Lateral Force procedure to be used in all situations, the modal response spectrum analysis (MRSA) procedure (ASCE/SEI 7-22 Section 12.9.1) is used in this example. However, as part of the MRSA procedure, base shear is also determined using Equivalent Lateral Force (ELF) procedure. This is because ASCE/SEI 7-22 requires that the base shear obtained from MRSA be scaled up to match the ELF base shear.

The building was modeled in ETABS 2016, and the total seismic weight was obtained from the program as **43,099 kips**.

Modal Response Spectrum Analysis

A 3-D modal response spectrum analysis (MRSA) is performed using ETABS (v2016).

- Semi-rigid diaphragms are assigned at each level.
- The effective cracked member stiffnesses used in the analyses are as follows:
  - Columns and shear walls, $I_{eff} = 0.7I_g$
  - Coupling beams, $I_{eff} = 0.25I_g$
  - Gravity columns, $I_{eff} = 0.1I_g$ (with pinned connections at the base)
- Adequate number of modes are considered in the modal analysis to incorporate **100%** of the modal mass in each of x- and y-directions. Also, appropriate scale factors are applied to the base shears calculated in the x- and y-directions to amplify them to those calculated in the ELF procedure.

Floor Forces from MRSA

Story	Elevation (ft)	X-Dir (kip)	Y-Dir (kip)
L23	234.25	195.35	195.35
L22	224.25	206.00	206.00
L21	214.25	192.84	192.84
L20	204.25	177.63	177.63
L19	194.25	165.00	165.00
L18	184.25	150.81	150.81
L17	174.25	138.90	138.90
L16	164.25	125.66	125.66
L15	154.25	114.43	114.43
L14	144.25	102.24	102.24
L13	134.25	91.78	91.78

Story	Elevation (ft)	X-Dir (kip)	Y-Dir (kip)
L12	124.25	80.68	80.68
L11	114.25	71.00	71.00
L10	104.25	61.10	61.10
L09	94.25	52.34	52.34
L08	84.25	43.80	43.80
L07	74.25	35.84	35.84
L06	64.25	28.48	28.48
L05	54.25	21.77	21.77
L04	44.25	16.22	16.22
L03	31.25	12.95	12.95
L02	16.25	3.78	3.78

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Story Drifts from MRSA (X-Direction)

Story	Story Ht. (ft)	δ_{xe} (in.)	C_d	δ_x (in.)	Relative Drift (%)
L23	10	3.36	8	26.84	1.00
L22	10	3.21	8	25.65	1.02
L21	10	3.05	8	24.43	1.03
L20	10	2.90	8	23.18	1.05
L19	10	2.74	8	21.92	1.07
L18	10	2.58	8	20.64	1.08
L17	10	2.42	8	19.33	1.10
L16	10	2.25	8	18.02	1.10
L15	10	2.09	8	16.69	1.11
L14	10	1.92	8	15.36	1.11
L13	10	1.75	8	14.03	1.11

Story	Story Ht. (ft)	δ_{xe} (in.)	C_d	δ_x (in.)	Relative Drift (%)
L12	10	1.59	8	12.70	1.10
L11	10	1.42	8	11.38	1.09
L10	10	1.26	8	10.07	1.08
L09	10	1.10	8	8.77	1.06
L08	10	0.94	8	7.50	1.04
L07	10	0.78	8	6.25	1.00
L06	10	0.63	8	5.06	0.95
L05	10	0.49	8	3.91	0.89
L04	13	0.36	8	2.85	0.80
L03	15	0.20	8	1.60	0.59
L02	16.25	0.07	8	0.55	0.28

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Story Drifts from MRSA (Y-Direction)

Story	Story Ht. (ft)	δ_{xe} (in.)	C_d	δ_x (in.)	Relative Drift (%)
L23	10	3.64	8	29.11	1.07
L22	10	3.48	8	27.83	1.09
L21	10	3.32	8	26.52	1.11
L20	10	3.15	8	25.19	1.13
L19	10	2.98	8	23.83	1.15
L18	10	2.81	8	22.45	1.17
L17	10	2.63	8	21.05	1.18
L16	10	2.45	8	19.63	1.19
L15	10	2.28	8	18.21	1.19
L14	10	2.10	8	16.77	1.20
L13	10	1.92	8	15.34	1.19

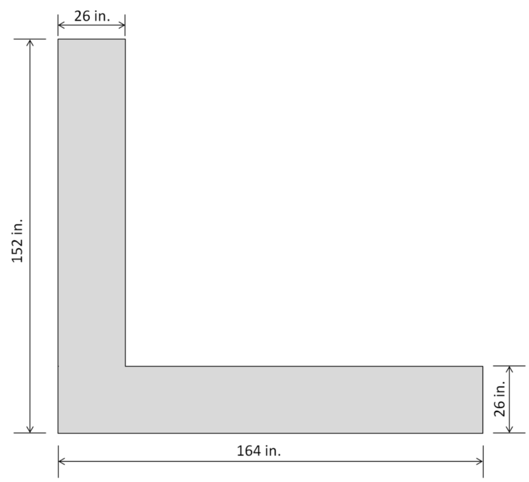
Story	Story Ht. (ft)	δ_{xe} (in.)	C_d	δ_x (in.)	Relative Drift (%)
L12	10	1.74	8	13.91	1.19
L11	10	1.56	8	12.48	1.18
L10	10	1.38	8	11.06	1.17
L09	10	1.21	8	9.65	1.15
L08	10	1.03	8	8.27	1.13
L07	10	0.86	8	6.92	1.09
L06	10	0.70	8	5.60	1.04
L05	10	0.54	8	4.35	0.98
L04	13	0.40	8	3.18	0.87
L03	15	0.23	8	1.82	0.67
L02	16.25	0.08	8	0.62	0.32

Story Drift Limitation

According to ASCE/SEI 7-22 Section 12.12.1, the calculated relative story drift at any story must not exceed 2% (ASCE/SEI 7-22 Table 12.12-1 for all other buildings in Risk Category I and II). As can be seen from the previous slide, this is satisfied in all stories.

Design of Shear Wall

- The design of one of the shear walls at the base of the structure is illustrated in this example in accordance the provisions of ACI 318-19.
- One L-shaped segment of the shear wall core is designed as two flanged walls.
- Orthogonal combination of seismic forces is NOT required as axial loads on the wall from seismic forces are less than 20% of the design axial strength.



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Design of Shear Wall – Design Loads

Seismic forces acting along x-axis are considered in this design example. The design calculations for the seismic forces acting along the y-axis are similar and are not shown. However, the final wall configuration will incorporate effects of seismic forces in both directions.

	Load Combinations	Axial Force, P_u (kips)	Shear Force, V_u (kips)	Bending Moment, M_u (ft-kips)
1	$1.4D$	6335	0	0
2	$1.2D + 1.6L + 0.5L_r$	6071	0	0
3	$(1.2+0.2S_{DS})D + \rho Q_E + 0.5L$	10,015	576	24,976
4	$(0.9D - 0.2S_{DS}D) + \rho Q_E$	6460	573	24,585
5	$(0.9D - 0.2S_{DS}D) - \rho Q_E$	-378	573	24,585

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Design of Shear Wall – Design for Shear

- Height of the shear wall, h_{wcs} = 2811 in. (234.25 ft)
- Length of the shear wall, ℓ_w = 164 in. (13.67 ft)
- h_{wcs}/ℓ_w = 2811/164 = 17.1

ACI 318-19 (hereafter ACI 318) Section 18.10.2.2

At least two curtains of reinforcement shall be used if $V_u > 2A_{cv}\lambda\sqrt{f'_c}$ or $h_{wcs}/\ell_w \geq 2.0$. In this case, h_{wcs}/ℓ_w = 17.1 > 2.0.

So, at least two curtains of reinforcement are required.

Design of Shear Wall – Design for Shear

ACI 318 Section 18.10.3.1

Design shear force, $V_e = \Omega_v \omega_v V_u \leq 3V_u$

- For walls with $h_{wcs}/\ell_w > 1.5$, Ω_v is the greater of M_{pr}/M_u and 1.5. The probable moment strength M_{pr} is unknown at this stage. So, it is assumed that Ω_v = 1.5. This may very well prove to be unconservative. Once the flexural reinforcement has been provided, this will be verified or corrected, if necessary
- For walls with $h_{wcs}/\ell_w \geq 2.0$ and the number of stories above critical section, n_s > 6,
  $\omega_v = 1.3 + n_s/30 \leq 1.8$
  - In this example, n_s = 22. n_s cannot be taken less than the quantity $0.007h_{wcs}$ (= 19.68), which is satisfied.
  - ω_v = 1.3 + 22/30 = 2.03 → ω_v = 1.8

Design of Shear Wall – Design for Shear

<u>ACI 318 Section 18.10.3.1</u>

Design shear force, $V_e = \Omega_v \omega_v V_u \leq 3V_u$

- $V_e = 1.5 \times 1.8 \times 576 = 1555$ kips (governs)
- $V_e = 3V_u = 3 \times 576 = 1728$ kips

<u>ACI 318 Section 18.10.4.4</u>.

The maximum nominal shear strength, V_n, allowed for a wall section is

$$10A_{cv}\sqrt{f'_c} = 10 \times 4264 \times \sqrt{f'_c}\,/1000 = 3813 \text{ kips}$$

So, $\varphi V_n = 0.75 \times 3813 = 2860$ kips > V_e → The provided wall section size is acceptable.

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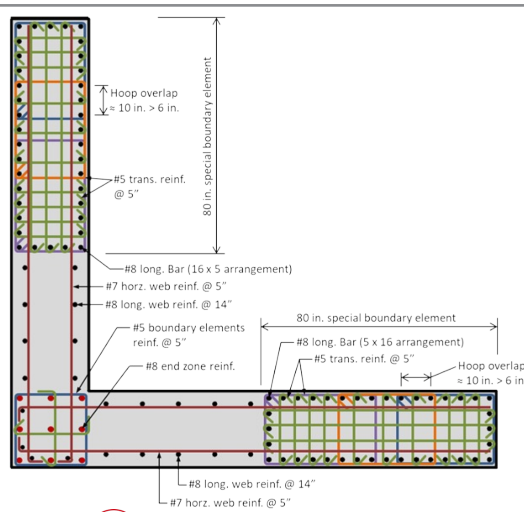
Design of Shear Wall (Grade 60 Reinforcement)



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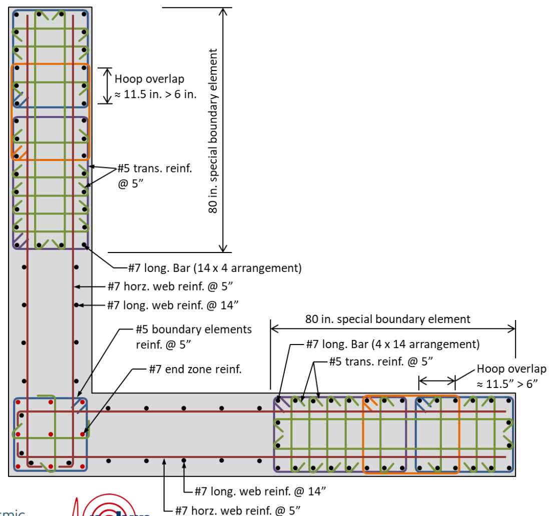
Design of Shear Wall (Grade 80 Reinforcement)



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Design of Shear Wall (Grade 80 Reinforcement)

- Use of Grade 80 steel leads to a considerable reduction in the amount of reinforcement in the wall. In addition to the smaller bar sizes, lesser congestion in the special boundary elements is especially noticeable. However, the vertical spacing of the transverse hoops and cross-ties in the special boundary elements remained 5 in. as that in the Grade 60 design. This is because the maximum value of that spacing is limited to 6 times the diameter of the smallest longitudinal bar. So, smaller bar sizes achieved by higher strength reinforcement ironically led to a tighter spacing compared to what would be necessary for confinement alone. The vertical spacing of the horizontal shear reinforcement is also smaller than what is required for resisting shear so that it matches the spacing of transverse reinforcement in the boundary elements for construction efficiency. Thus, some of the gains achieved by using Grade 80 reinforcement are negated by various other considerations.

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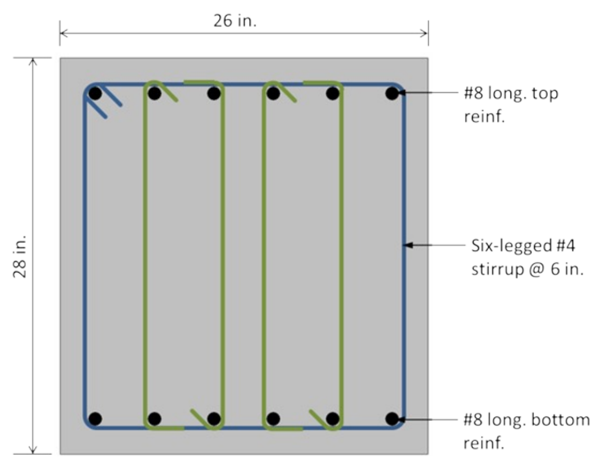
Design of Coupling Beam

A coupling beam oriented along the y-axis of the building at the second floor level is selected for this example. The dimensions of the beam are given below:

- Clear span of the beam, ℓ_n = 76 in. (6.33 ft)
- Height of the beam, h = 28 in. (2.33 ft)
- Width of the beam, b_w = 26 in. (2.17 ft)
- ℓ_n/h = 76/28 = 2.7

Since 2 < ℓ_n/h < 4, per ACI 318 Section 18.10.7.3, this beam can be designed as a deep coupling beam using two intersecting groups of diagonally placed bars, or as a special moment frame flexural member in accordance with the ACI 318 Sections 18.6.3 through 18.6.5. The second option is adopted for this example.

Design of Coupling Beam – Design for Shear



Questions