Design of RC Ductile Coupled Shear Wall System According to ACI 318-19 and ASCE/SEI 7-22 Codes

Course No: S03-031
Credit: 3 PDH

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COURSE CONTENT

A ductile coupled wall system of reinforced concrete is now defined in ACI 318-19 (ACI, 2019), and it is recognized as a distinct seismic force-resisting systems in ASCE/SEI 7-22 Table 12.2-1, **Design Coefficients and Factors for Seismic Force-Resisting Systems.** Three-line items have been added to the table, featuring the ductile coupled wall system of reinforced concrete. The line items are under: A. Bearing Wall Systems, B. Building Frame Systems, and D. Dual Systems with Special Moment Frames. **$R = 8$, $C_d = 8$, and $\Omega_0 = 2.5$** are the design coefficients in all the line items. The height limits are the same as for corresponding uncoupled isolated wall systems. Several important changes made in ACI 318-19 for the design and detailing of special structural walls were implemented in the design of prototypes for the FEMA P695 study supporting the above values.

A few words about terminology may be in order here. "Reinforced Concrete Ductile Coupled Shear Wall System" is the terminology used in the title of this course. An effort has been made to use this terminology consistently throughout the chapter, except that "reinforced concrete" is typically dropped as being redundant or understood. The whole chapter, after all, is about a reinforced concrete system. There are, however, a few obstacles to attaining total consistency. The system discussed in this chapter is defined in ACI 318-19 Section 2.3 as follows: "structural wall, ductile coupled – a seismic-force-resisting-system complying with 18.10.9." Several aspects of this definition ought to be noted. First, "reinforced concrete" is not mentioned; it is understood. Second, shear walls are called structural walls in ACI 318-19. Third and most importantly, a wall, which is a structural member or element, is defined as a system; this is lax usage of terms. Finally, and this may not be important, "seismic-force-resisting-system" in the definition is "seismic force-resisting system" in ASCE/SEI 7-22 as well as in this chapter. In ASCE/SEI 7-22 Section 18.10.9 itself, the terminology used is Ductile Coupled Walls. Where ACI 318-19 is referenced directly, the terminology used here is Ductile Coupled Structural (Shear) Wall System. Where ACI 318-19 text is essentially reproduced (with or without quotation marks), terminology used by ACI Committee 318 is left alone. Finally, in ASCE/SEI 7-22 Table 12.2-1, the walls providing seismic force-resistance as part of the structural system under discussion here are called Reinforced Concrete Walls. In portions of the table reproduced in this chapter; "shear" has not been inserted before "walls."

In addition to the *2020 Provisions*, the following documents are either referred to directly or may serve as useful design aids.

Useful Design Aid Resources

ACI (2019). *Building Code Requirements for Structural Concrete*, ACI 318-19 and *Commentary*, ACI 318R-19, American Concrete Institute, Country Club Hills, MI.

ASCE (2017). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

CSA Group (2014). *Design of Concrete Structures*, A 23.3.14, Mississanga, Ontario, Canada.

FEMA (2009). *Quantification of Building Seismic Performance Factors*, FEMA P695, prepared by the Applied Technology Council for the Federal Emergency Management Agency, Washington, D.C., June.

Los Angeles Tall Buildings Structural Design Council (2017). *An Alternative Procedure for Seismic Analysis and Design of Tall Buildings Located in the Los Angeles Region*, 2017 Edition, Los Angeles, June.

Standards New Zealand (2006). *Concrete Structures Standard*, NZS 3101. 1&2: 2006, Wellington, New Zealand.

1. Introduction

Functional and often structural requirements make the use of shear walls desirable in many buildings. Functionally, shear walls are useful in buildings because they serve as partitions between spaces. Structurally, they make buildings laterally stiff, particularly when used interactively with moment frames, thereby helping to keep lateral deflections within tolerable limits. Often, such walls are pierced by numerous openings for windows, doors, and other purposes. Two or more walls separated by vertical rows of openings, with beams at every floor level between the vertically arranged openings, are referred to as coupled shear walls. When a coupled shear wall system is subject to lateral loads due to wind or earthquake forces, shear forces generated at the ends of the coupling beams accumulate into a tensile force in one of the coupled wall piers and into a compression force in the other wall pier. The couple, due to these tension and compression forces, resists a part of the overturning moment at the base of the wall system, with the remainder of the overturning moment being resisted by the wall piers themselves (Figure 1). The ratio of the overturning moment resisted by the tension-compression couple to the total overturning moment at the base of the coupled wall system is often referred to as the degree of coupling. The shorter and deeper the coupling beams, the higher the degree of coupling. When the degree of coupling is very low (25% or lower), the two wall piers tend to behave like isolated walls, and when the degree of coupling is very high (75% or higher), the entire coupled wall system tends to behave like a shear wall with openings. It should be noted, however, that as inelastic displacements develop in the coupling beams, the degree of coupling tends to lose its significance.

A coupled shear wall system can be designed such that a considerable amount of earthquake energy is dissipated by shear yielding in coupling beams with low span-to-depth ratios or flexural yielding at the ends of coupling beams with higher span-to-depth ratios before flexural hinges form (typically) at the bases of the wall piers (assuming they are slender, with height-to-length ratios larger than or equal to two). Although such coupled wall systems are highly suitable as the seismic force-resisting systems of multistory buildings, they were not recognized as distinct entities in Table 12.2-1 of ASCE/SEI 7-16. Therefore, such systems needed to be designed using *R*-values that essentially ignore the considerable benefits of having the coupling beams, which can dissipate much of the energy generated by earthquake excitation. This course reports on a successful effort to address this situation.

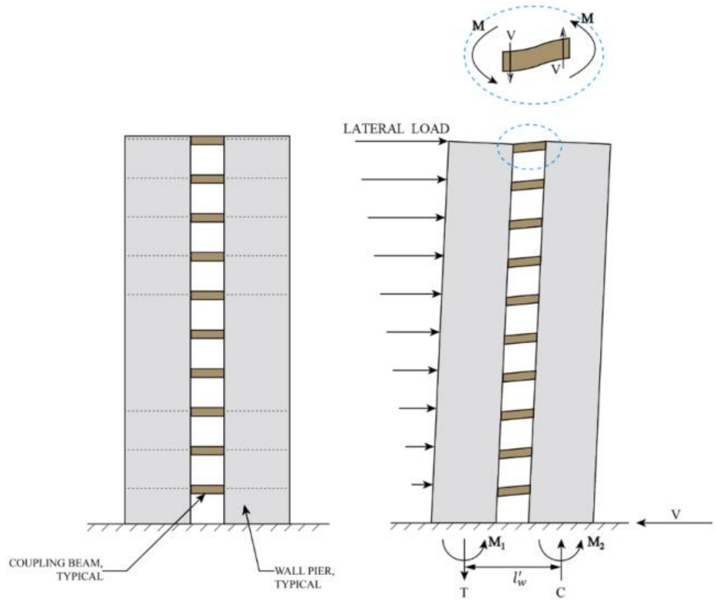


Figure 1. A Coupled Shear Wall System

2. Ductile Coupled Structural (Shear) Wall System of ACI 318-19

To quote from Bertero (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, the discussion in this course needs to focus not on just coupled walls but ductile coupled walls consisting of ductile shear walls and **ductile coupling beams**.

In the 2019 edition of ACI 318, a new system definition has been created to recognize the Ductile Coupled Structural (Shear) Wall (DCSW) system. The shear walls in such a system must be special structural walls in conformance with ACI 318-19 Section 18.10, including the proportioning requirements of Section 18.10.9, and the coupling beams must comply with the detailing requirements in ACI 318-19 Section 18.10.7.

The objective of the ductile coupled shear wall system is for the majority of energy dissipation to occur in the coupling beams. This is analogous to strong column weak beam behavior in moment frames. Studies were conducted at Magnusson Klemencic Associates (MKA) to identify system characteristics that lead to coupling beam energy dissipation of no less than 80% of total system energy dissipation under MCE ground motions. In these studies, nonlinear 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 considered a range of longitudinal reinforcement ratios in the coupling beams as well as the shear walls. The results of these analyses are presented in Figure 2. The x-axis represents the aspect ratio (clear span-to-total depth) of the coupling beams, with D designating a diagonally reinforced beam design and M designating a special moment frame beam design. For example, D4 is a diagonally reinforced coupling beam with an aspect ratio of 4 and M4 is a coupling beam detailed as a special moment frame beam with an aspect ratio of 4. The y-axis is the percentage of total system energy dissipation that occurs in the coupling beams alone. The resulting trend shows an energy "dome" with coupling beams dissipating the majority of system energy between aspect ratios of 2 and 5.

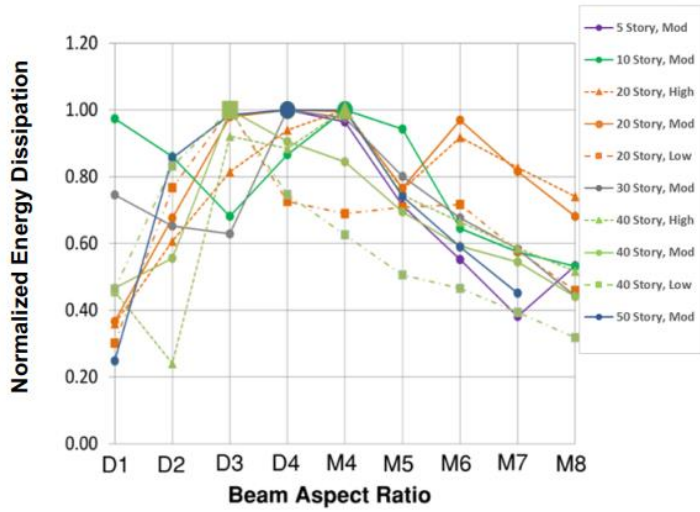


Figure 2. Energy Dissipation in Coupling Beams

The primary characteristics of such a system were found to be governed by geometry. Squat walls were found to be too stiff to allow sufficient story drift for coupling beams to become inelastic. For this reason, shear walls in the Ductile Coupled Shear Wall (DCSW) system need to have a total height to length aspect ratio of no less than 2.0. Squat coupling beams were found to over-couple the seismic force-resisting system and lead to significant energy dissipation in the shear walls. As such, coupling beams in DCSW systems need to have clear span to total depth aspect ratio of no less than 2.0 in all cases. Very slender coupling beams, designated as having an aspect ratio greater than 5.0, are not stiff enough to contribute sufficient hysteretic energy dissipation and are allowed in no more than 10% of the levels of the building. Lastly, coupling beams conforming to these geometric constraints are required to be present at all levels in order to dissipate the intended amount of energy. It has been clarified in ACI 318-19 that longitudinal reinforcement in coupling beams detailed as special moment frame beams and diagonal reinforcement in diagonally reinforced coupling beams must develop 1.25 times f_y of the reinforcement at each end. This last requirement is intended to preclude the use of fixed-pinned coupling beams that are the outcome where insufficient length exists to adequately develop the coupling beam reinforcement into the adjacent shear wall.

Plotted along the y-axis of Figure 2 is energy dissipated by a coupling beam, normalized with respect to the energy dissipated by a coupling beam with an aspect ratio of 4 in a 40-story building. In Figure 2 legend, "High," "Moderate," and "Low" refer to 100%, 75%, and 50%, respectively, of the amount of reinforcement needed to generate $V_n = 10\sqrt{f_c'}A_{cw}$ (see ACI 318-19 Equation 18.10.7.4). For coupling beams detailed as special moment frame beams, "High," "Mod," and "Low" mean 100%, 75%, and 50%, respectively, of the longitudinal reinforcement that would generate an M_{pr} for which $2M_{pr}/\ell_n = 10\sqrt{f_c'}A_{cw}$.
As noted earlier, the requirements of the Ductile Coupled Structural (shear) Wall system are in addition to those required for Special Structural (shear) Walls and Coupling Beams. The final

language of the DCSW definition in ACI 318-19 reflects the input of ACI 318 Subcommittee H as well as Building Seismic Safety Council (BSSC) PUC Issue Team (IT) 4 on Shear Walls.

Also as noted earlier, ACI 318-19 Section 2.3 – Terminology defines structural wall, ductile coupled as a seismic force-resisting-system complying with Section 18.10.9.

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 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 all coupling beams.

3. Ductile Coupled Structural (Shear) Wall System in ASCE/SEI 7-22

Building Seismic Safety Council BSSC PUC Issue Team (IT) 4 of the Provisions Update Committee (PUC) of the Building Seismic Safety Council (BSSC) developed a proposal that led to the addition of three line items to ASCE/SEI 7-22 Table 12.2-1, Design Coefficients and Factors for Seismic Force-Resisting Systems, featuring the reinforced concrete ductile coupled shear wall system (Table 1). The line items are under: A. Bearing Wall Systems, B. Building Frame Systems, and D. Dual Systems with Special Moment Frames.

Table 1. Addition of Reinforced Concrete Ductile Coupled Walls to ASCE/SEI 7-16 Table 12.2-1

Seismic Force-Resisting System	ASCE/SEI 7 Section Where Detailing Requirements Are Specified	R	Ω_0	C_d	Structural System Limitations Including Structural Height, h_n (ft) Limits[d]				
					Seismic Design Category				
					B	C	D	E	F
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Reinforced concrete ductile coupled walls	14.2	8	2½	8	NL	NL	160	160	100
3. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
B. BUILDING FRAME SYSTEMS									
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Reinforced concrete ductile coupled walls	14.2	8	2½	8	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
...									
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES...									
7. Special reinforced concrete shear walls[g,h]	14.2	7	2½	5½	NL	NL	NL	NL	NL
8. Reinforced concrete ductile coupled walls	14.2	8	2½	8	NL	NL	NL	NL	NL
9. Ordinary reinforced concrete shear walls[g]	14.2	6	2½	5	NL	NL	NP	NP	NP

Based on a FEMA P695 study, R = 8, C_d = 8, and Ω_o = 2.5 have been proposed in all the line items. The height limits are the same as for corresponding uncoupled isolated wall systems. It is possible to increase the 160-ft height limit to 240 ft for buildings without significant torsion because ASCE/SEI 7-22 Section 12.2.5.4 has been made applicable to these systems. A minimum height limit of 60 ft has been imposed on seismic force-resisting systems featuring the reinforced concrete ductile coupled walls because, in shorter buildings, there may not be enough coupling beams to absorb sufficient energy to merit an R-value of 8.

4. FEMA P695 Studies Involving Ductile Coupled Structural (Shear) Walls

The proposed response modification factors for seismic force-resisting systems featuring reinforced concrete ductile coupled shear walls were validated (Tauberg et al. 2019) using the FEMA P695 methodology (FEMA, 2009). A series of 37 ductile coupled shear wall buildings, as summarized in Table 2, were designed using a range of variables expected to influence the collapse margin ratio, with primary variables of 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). The period domain in Table 2 is defined by the number of stories.

There have been four significant ACI 318-19 code changes, all adopted in the FEMA P695 study (Tauberg et al. 2019), to address the flexural-compression wall failure issue.

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).

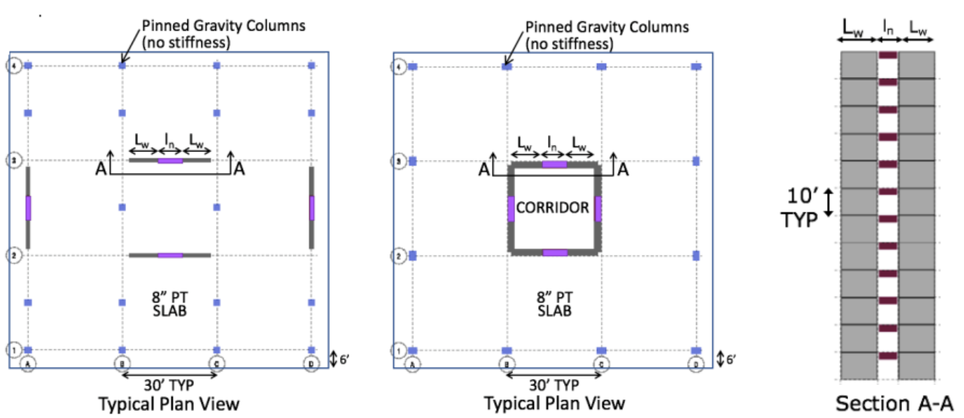
18.10.6.4 - requires improved wall boundary and wall web detailing, i.e, overlapping hoops if the boundary zone dimensions exceed 2:1, crossties with 135-135 degree hooks on both ends, and 135-135 degree crossties on web vertical bars.

18.10.6.2(b) (Check on mean top-of-wall drift capacity at 20% loss of lateral strength) - requires a low probability of lateral strength loss at MCE level hazard, and

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

For details on these important changes, reference can be made to Ghosh, Taylor (2021a, 2021b).

The range of variables was chosen considering those used to define a Ductile Coupled Structural (Shear) Wall system in ACI 318-19. The resulting designs have the minimum wall area (length and thickness) required, which is governed by shear amplification and the requirement that walls sharing a common shear force not exceed a shear stress of $8\sqrt{f'_c}A_{cv}$. Typical floor plans and a wall elevation view are presented in Figure 3.



(a) Planar Walls (6, 8, 12 Story) (b) Flanged Walls (18, 24, 30 Story) (c) Elevation View

Figure 3. Archetype Floor Plans and Typical Wall Elevation View

Table 2. Coupled Wall Archetype Design Information

Performance Group Summary					
Group No.	**Grouping Criteria**				**Number of Archetypes**
	Basic Configuration	**Design Load Level**		**Period Domain**	
		Gravity	**Seismic**		
PG-1	Planar walls, diagonally reinforced coupling beams with ℓ_n/h = 2.0,2.4, 3.0, 3.3	Typical	SDC D_{max}	Short	9 (6, 8, 12 story)
PG-2	Planar walls, conventional reinforced coupling beams with ℓ_n/h = 3.3, 4.0, 5.0	Typical	SDC D_{max}	Short	7 (6, 8, 12 story)
PG-3	Flanged walls, diagonally reinforced coupling beams with ℓ_n/h = 2.0,2.4, 3.0, 3.3	Typical	SDC D_{max}	Short	4 (18 story)
PG-4				Long	8 (24 and 30 story)
PG-5	Flanged walls, conventional reinforced coupling beams with ℓ_n/h = 3.3, 4.0, 5.0	Typical	SDC D_{max}	Short	3 (18 story)
PG-6				Long	6 (24 and 30 story)

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 mentioned earlier, 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 since the use of a larger value would increase seismic design forces (and strengths) and produce more conservative designs. 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

Seismic design forces were determined using the Modal Response Spectrum Analysis (MRSA) method of ASCE/SEI 7, subject to scaling the base shear to 100% of the Equivalent Lateral Force base shear of ASCE/SEI 7-16 for a period $T = C_u T_a$. Modal damping ratio was assumed to be 5 percent, and the Complete Quadratic Combination (CQC) method was used to combine modal responses. The story heights were taken equal to 10 feet for all designs. Building stories, the fundamental period T_1, the design period $T = C_u T_a$, the design coefficient C_s, and the design base shear V are summarized in Table 4-3 for a subset of six of the 37 archetype designs in Table 2. In the archetype numbering, 6H, 8H, 12H, 18H, 24H, and 30H indicate total heights of 60, 80, 120, 180, 240, and 300 ft, respectively; DR indicates diagonally reinforced; and the last number, 2 or 3, indicates the clear span to total depth ratio of the coupling beams.

Table 3. Coupled Wall Archetype Design Information

Archetype	Total Height	T_1 (s)	$T = C_u T_a$ (s)	V (kips)	DOC	Wall Pier Dimensions	$P_{u,1}$[1] $/A_g f'_c$	$P_{u,2}$[2] $/A_g f'_c$	$P_{u,3}$[3] $/A_s f_y$
6H-DR-2 (planar wall)	60'	0.83	0.604	1,062 $C_s = 0.124$	0.60	l_w = 8.0' $t_{w,L1\text{-}4}$ = 14" $t_{w,L5\text{-}6}$ = 10"	0.16	0.28	-0.31
8H-DR-3 (planar wall)	80'	1.27	0.749	1,201 $C_s = 0.100$	0.61	l_w = 8.5' $t_{w,L1\text{-}4}$ = 14" $t_{w,L5}$ = 12" $t_{w,L6\text{-}8}$ = 10"	0.14	0.27	-0.27
12H-DR-3 (planar wall)	120'	2.14	1.015	1,360 $C_s = 0.074$	0.66	l_w = 9.25' $t_{w,L1\text{-}4}$ = 16" $t_{w,L5\text{-}12}$ = 12"	0.13	0.30	-0.33
18H-DR-3 (flanged wall)	180'	3.14	1.376	1,489.5 $C_s = 0.0545$	0.66	l_w = 9.0' $t_{w,L1\text{-}5}$ = 24" $t_{w,L6\text{-}8}$ = 20" $t_{w,L5\text{-}12}$ = 16"	0.13	0.19	-0.07
24H-DR-3 (flanged wall)	240'	3.39	1.707	1,654 $C_s = 0.044$	0.69	l_w = 10.0' $t_{w,L1\text{-}18}$ = 24" $t_{w,L19\text{-}24}$ = 18"	0.17	0.23	-0.03
30H-DR-3 (flanged wall)	300'	3.62	2.018	2,112 $C_s = 0.044$	0.68	l_w = 11.25' $t_{w,L1\text{-}10}$ = 30" $t_{w,L11\text{-}20}$ = 24" $t_{w,L21\text{-}30}$ = 18"	0.13	0.20	-0.06

[1] $P_{u,1}$ is the gravity axial stress under load combination $1.2D+1.6L$.
[2] $P_{u,2}$ is the maximum axial stress under load combination $(1.2+0.2S_{DS})D+0.5L+1.0E$.
[3] $P_{u,3}$ is the minimum (net tensile) axial stress under load combination $(0.9\text{-}0.2S_{DS})D+1.0E$.

It should be noted that incorporating wall shear amplification in the design was necessary because preliminary analysis results using $R = C_d = 8$ and with walls designed in compliance with ACI 318-14 shear provisions did not meet the FEMA P695 acceptability criteria due to a high number of shear failures experienced during incremental dynamic analysis. The wall shear amplification requirement per the new code provision of ACI 318-19 amplifies the code level shear force (V_u) by a flexural overstrength factor (Ω_v) and a dynamic shear amplification factor (ω_v) that accounts for higher mode effects. The dynamic shear amplification factor (ω_v) depends on the number of stories (n_s). The overstrength factor (Ω_v) is the ratio of probable moment strength M_{pr} to code required strength M_u, which shall not be taken less than 1.5 per ACI 318-19. In this study, the ratio of M_{pr} to M_u was set equal to 1.5 for all designs so that the walls would not be overdesigned for shear strength and represent the governing case for collapse analysis.

Two-dimensional nonlinear models were created for each design using the structural analysis software Open Systems for Earthquake Engineering Simulation (OpenSees). Nonlinear static pushover (NSP) analyses were used to compute the system overstrength factor (Ω_0) and the period-based ductility (μ_T), while incremental dynamic analyses (IDA) were performed in accordance with FEMA P695 to assess collapse. Per FEMA P695, the period-based ductility (μ_T) is obtained by dividing the ultimate roof displacement (δ_u) by the effective yield displacement ($\delta_{y,eff}$). Effective yield displacement is defined by Eq. (5-3) in (Tauberg et al. 2019).

The dynamic analyses were conducted at Design Earthquake (DE) and Maximum Considered Earthquake (MCE) hazard levels, as well as at ground motion intensities representative of the collapse capacity of each design. For the IDAs, a set of twenty-two pairs of far-field horizontal ground motion records of FEMA P695 were used. The collapse capacity was determined by incrementally increasing the intensity of the 22 pairs of scaled far-field ground motions of FEMA P695 Appendix A until just less than half of the records caused collapse of the archetypical building as represented by the established failure modes of the model.

To assess collapse, three primary failure modes were considered to capture lateral strength loss and failure:

1***. flexural failure (crushing of concrete, buckling of reinforcing bar, tensile fracture of longitudinal reinforcement***) was assessed using a statistical drift capacity model developed based on an extensive database of wall tests,

2. ***shear failure (diagonal tension/compression)*** assessment was based on the relationship between wall shear force and tensile strain of wall longitudinal reinforcement, following Los Angeles Tall Buildings Seismic Design Council or LATBSDC (2017) recommendations, and

3. ***axial failure*** was estimated using a shear friction model.

It may be natural to wonder why axial failure is associated with shear friction. Explanation is provided in the following excerpt from (Tauberg et al., 2019). For references cited and the figure mentioned in the excerpt, refer to (Tauberg et al., 2019); the reference listings and the table are not reproduced here.

"Wall axial failure is defined using the lateral drift capacity model proposed by Wallace et al. (2008) which defines the lateral drift capacity at axial failure using an assumed critical shear crack angle and a shear friction model as shown in Figure 4. The initial model for the limit state of axial collapse is based on the column model proposed by Elwood and Moehle (2003, 2005), and modified for application to walls (Wallace et al., 2008). The model is based on equilibrium for an assumed shear friction relation, assuming the critical crack plane extends along the main diagonal of the wall pier (or over a single story). Axial failure results along the critical crack plane when the shear demand exceeds the shear friction capacity."

For the study, ***collapse was defined as being associated with either flexure or shear***, meaning that the axial failure model did not govern because the lateral drift values at axial failure (generally greater than 5%) exceeded drifts at which flexural failure occurred; the axial failure model also has not been verified (although collapse of buildings with reinforced concrete walls has rarely been reported following strong earthquakes). Because amplified wall shear demands were used in design, shear failures were mostly suppressed, and flexure-related collapse was typically defined by the drift capacity model for most archetypes. Overall, the criteria used for collapse assessment in the study were conservative since the failure models predict the onset of strength loss (a 20% drop in lateral strength) and not necessarily collapse. The approach is conservative because loss of axial load carrying capacity typically does not occur until lateral strength drops to near zero. In some studies, axial failure has been assumed to occur at a specified roof drift ratio, which has been typically defined as 4 to 5% (NIST GCR-10-917-8), whereas, in this study, the conservative approach used to assess collapse resulted in drift ratios at failure that were typically about 3%.

Results from the incremental dynamic analyses are used to obtain the median collapse capacity intensity (S_{CT}) and the collapse margin ratio (CMR) for each archetype. The median collapse intensity (S_{CT}) is established by determining the 5%-damped spectral acceleration at which half of the ground motions cause the structure to collapse using the project failure criteria, i.e., with the drift capacity model or the shear failure model for this study. The collapse margin ratio is then computed to characterize the collapse safety of the archetype as the ratio of the median collapse spectral intensity S_{CT} to S_{MT}, where S_{MT} is the intensity of the Maximum Considered Earthquake (MCE) obtained from the response spectrum of MCE ground motions at the fundamental period (T) of the building.

$$\text{CMR} = S_{CT}/S_{MT}$$

$$\text{ACMR} = \text{SSF} \times \text{CMR}$$

The collapse margin ratio is adjusted by the period and ductility dependent spectral shape factors (SSFs) prescribed in FEMA P695 Section 7.2.2 in order to account for the effects of the frequency content (spectral shape) of the ground motion record set. For this study, the acceptable adjusted collapse margin ratios (ACMRs) were established as 1.96 and 1.56 for the 10% and 20% collapse probability scenarios, respectively. Once results from incremental dynamic analyses are obtained, each archetype is assessed for conformance with the FEMA P695 acceptability criteria by comparing its ACMR to the acceptable ACMR based on the system collapse uncertainty (β_{TOT}). For a given archetype, if the building ACMR is greater than the acceptable ACMR at 20% collapse probability ($\text{ACMR}_{20\%}$), then the archetype passes the performance criteria. The average of the ACMRs of the archetypes in each performance group must also be compared to the acceptable ACMR at 10% collapse probability ($\text{ACMR}_{10\%}$) to assess whether the performance group as a whole passes the FEMA P695 performance criteria.

A summary of the analysis results for all archetypes is presented in Table 4. The results show that all 6-story to 30-story archetypes pass the FEMA P695 collapse acceptability criteria, thus validating the use of R = 8 for ductile coupled shear wall systems that are designed in conformance with ASCE/SEI 7-16 and ACI 318-19 provisions.

Table 4. Summary of Collapse Results for Ductile RC Coupled Wall Archetypes

Archetype ID	Pushover Results		IDA Results			Acceptability			
	Static Ω [a]	μ_T [b]	S_{MT} [c] [T]	S_{CT} [d] [T]	CMR [e]	SSF [f]	ACMR [g]	Accep. ACMR	Pass /Fail
6H-DR-2.0	2.28	7.54	1.49	2.68	1.80	1.35	2.42	1.56	Pass
8H-DR-2.0	2.11	5.43	1.20	1.90	1.58	1.29	2.04		Pass
8H-DR-2.4	2.09	6.06		1.95	1.62	1.32	2.13		Pass
8H-DR-3.0	2.25	5.26		1.95	1.62	1.29	2.08		Pass
8H-DR-3.3	2.12	5.60		1.95	1.62	1.30	2.10		Pass
Mean:	**2.17**	**5.98**				**Mean:**	**2.16**	**1.96**	**Pass**
6H-CR-5.0	1.73	7.97	1.49	2.55	1.71	1.36	2.32	1.56	Pass
8H-CR-3.3	1.63	8.03	1.20	1.85	1.54	1.38	2.23		Pass
8H-CR-4.0	1.59	7.93		1.85	1.54	1.38	2.29		Pass
8H-CR-5.0	1.45	7.57		1.85	1.54	1.37	2.22		Pass
Mean:	**1.60**	**7.87**				**Mean:**	**2.27**	**1.96**	**Pass**
12H-DR-2.0	1.36	6.65	0.89	1.33	1.50	1.41	2.12	1.56	Pass
12H-DR-2.4	1.40	6.07		1.33	1.50	1.39	2.08		Pass
12H-DR-3.0	1.54	5.50		1.39	1.57	1.36	2.14		Pass
12H-DR-3.3	1.59	6.15		1.39	1.57	1.39	2.18		Pass
Mean:	**1.47**	**6.09**				**Mean:**	**2.13**	**1.96**	**Pass**
12H-CR-3.3	1.35	8.06	0.89	1.22	1.38	1.46	2.01	1.56	Pass
12H-CR-4.0	1.36	7.76		1.29	1.46	1.46	2.13		Pass
12H-CR-5.0	1.31	7.71		1.29	1.46	1.46	2.12		Pass
Mean:	**1.34**	**7.84**				**Mean:**	**2.09**	**1.96**	**Pass**
18H-DR-2.0	2.06	6.24	0.65	0.97	1.48	1.49	2.21	1.56	Pass
18H-DR-2.4	2.01	5.35		0.97	1.48	1.44	2.14		Pass
18H-DR-3.0	1.98	5.11		1.02	1.55	1.43	2.21		Pass
18H-DR-3.3	2.04	4.40		1.02	1.55	1.38	2.15		Pass
Mean:	**2.02**	5.28				**Mean:**	**2.18**	**1.96**	**Pass**

Table 4. Summary of Collapse Results for Ductile RC Coupled Wall Archetypes (Continued)

Archetype ID	Pushover Results		IDA Results			Acceptability			
	Static Ω [a]	**μ_T [b]**	***S_{MT}* [c] *[T]***	***S_{CT}* [d] *[T]***	**CMR [e]**	**SSF [f]**	**ACMR [g]**	**Accep. ACMR**	**Pass /Fail**
18H-CR-3.3	1.59	6.09		0.91	1.39	1.48	2.06		Pass
18H-CR-4.0	1.58	5.85	0.65	0.97	1.48	1.47	2.18	1.56	Pass
18H-CR-5.0	1.65	5.42		0.97	1.48	1.44	2.14		Pass
Mean:	**1.61**	5.79				**Mean:**	**2.13**	**1.96**	**Pass**
24H-DR-2.0	1.63	11.94		0.76	1.443	1.61	2.32		Pass
24H-DR-2.4	1.65	9.70	0.53	0.76	1.443	1.61	2.32	1.56	Pass
24H-DR-3.0	1.80	7.10		0.77	1.455	1.57	2.28		Pass
24H-DR-3.3	1.83	7.07		0.77	1.455	1.57	2.28		Pass
Mean:	**1.73**	8.95				**Mean:**	**2.30**	**1.96**	**Pass**
24H-CR-3.3	1.390	10.04		0.75	1.43	1.61	2.30		Pass
24H-CR-4.0	1.391	9.40	0.53	0.75	1.43	1.61	2.30	1.56	Pass
24H-CR-5.0	1.513	8.16		0.76	1.44	1.61	2.32		Pass
Mean:	**1.43**	9.20				**Mean:**	**2.31**	**1.96**	**Pass**
30H-DR-2.0	1.21	14.61		0.79	1.77	1.61	2.85		Pass
30H-DR-2.4	1.25	13.66	0.45	0.79	1.77	1.61	2.85	1.56	Pass
30H-DR-3.0	1.43	10.30		0.82	1.84	1.61	2.96		Pass
30H-DR-3.3	1.62	8.32		0.82	1.84	1.61	2.96		Pass
Mean:	**1.38**	11.72				**Mean:**	**2.91**	**1.96**	**Pass**
30H-CR-3.3	1.24	14.44		0.76	1.70	1.61	2.74		Pass
30H-CR-4.0	1.29	10.13	0.45	0.76	1.70	1.61	2.74	1.56	Pass
30H-CR-5.0	1.44	10.03		0.75	1.68	1.61	2.71		Pass
Mean:	**1.32**	11.53				**Mean:**	**2.73**	**1.96**	**Pass**

Note:

a) Ω : Overstrength factor
b) μ_T : Period-based Ductility
c) S_{MT}: Intensity of the Maximum Considered Earthquake (MCE)
d) S_{CT}: Medial Collapse Capacity Intensity
e) CMR: Collapse Margin Ratio
f) SSF: Spectral Shape Factors
g) ACMR: Adjusted Collapse Margin Ratio

The C_d factor for the Reinforced Concrete Ductile Coupled Shear Wall System was assessed using the ratio of a median value of nonlinear inelastic roof drifts (δ) from 44 records at DE level shaking to the design level drifts (δ_E/R). The design drifts in this study were obtained using a wall flexural effective stiffness $I_{eff} = 0.75I_g$ based on input from the advisory panel for the study for effective stiffness values commonly used in practice for RC coupled walls. This effective stiffness assumption results in lower design drifts than if, for example, $I_{eff} = 0.5I_g$ were used in design. However, since the archetypes have been designed for amplified shear demands and conform to the drift capacity check per the new provisions of ACI 318-19, the designs were not drift-governed, and the wall piers were thicker and stiffer than they would have been if designed per ACI 318-14. The maximum design drifts observed at the center of mass among any of the archetypes was less than 1.6% when using $I_{eff} = 0.75I_g$ and less than 2% when using $I_{eff} = 0.5I_g$ (per ACI 318-14, where this value is permitted to compute drifts).

Table 5 summarizes the drifts and resulting C_d values for a subset of archetypes. The computed C_d values for these archetypes result in a median value of C_d = 8.8 (coefficient of variation = 0.13). For the subset of archetypes listed in Table 5, adjusting the nonlinear response history analysis roof drift values for 5% damping results in a median value of C_d = 8.4. Therefore, a deflection amplification factor of $C_d = R = 8$ was proposed.

Table 5. Assessment of C_d Based on Drifts from a Subset of Archetypes

Archetype	**Number of stories**	**h_n (ft)**	**Design $(\delta_E/R)/h_n$ (%)**	**Median RHA δ / h_n (%)**	**C_d $\delta / (\delta_E/R)$**
30H-DR-3	30	300	0.145	1.14	**7.9**
18H-DR-3	18	180	0.111	1.22	**10.9**
12H-DR-3	12	120	0.161	1.58	**9.8**
8H-DR-3	8	80	0.127	1.14	**9.0**
6H-DR-2	6	60	0.109	0.94	**8.6**
6H-CR-5	6	60	0.130	1.05	**8.1**

As a result of this study, a system over strength factor of Ω_0 = 2.5 was proposed based on nonlinear static pushover analysis results indicating that mean over strength values of the performance groups range from 1.3 and 2.2. The proposed response modification factor R = 8 was validated based on incremental dynamic analysis results indicating that mean Adjusted Collapse Margin Ratio values of the performance groups range from 2.09 to 2.91, corresponding to collapse probabilities of less than ten percent based on using a conservative definition of collapse as noted in the prior paragraph. The deflection amplification factor of C_d = 8 was proposed based on damping considerations, and the assessment of median roof drift responses from DE level shaking compared to design roof drifts. A minimum height limit of 60 feet was recommended for the Ductile Coupled Shear Wall System with the proposed seismic response parameters to be adopted in ASCE/SEI 7; in shorter buildings, there may not be enough coupling beams to absorb sufficient energy to merit an R value of 8. Overall, the results of this study suggested that an overstrength factor of Ω_0 = 2.5, a response modification factor R = 8, and a deflection amplification factor of C_d = 8 are appropriate seismic design parameters for reinforced concrete Ductile Coupled Wall systems that are designed per ASCE/SEI 7-22 and ACI 318-19 provisions.

5. DESIGN EXAMPLE OF A SPECIAL REINFORCED CONCRETE DUCTILE COUPLED WALL

5.1 INTRODUCTION

5.1.1. GENERAL

A 22-story reinforced concrete residential building is designed following the requirements of ASCE/SEI 7-22, and ACI 318-19. A computer rendering of the building framing is shown in Figure 4(a). The plan view of the building changes from one floor to another. A plan view of the second floor of the building is shown in Figure 4(b). Story elevations above the base and story heights can be seen in Table 6.

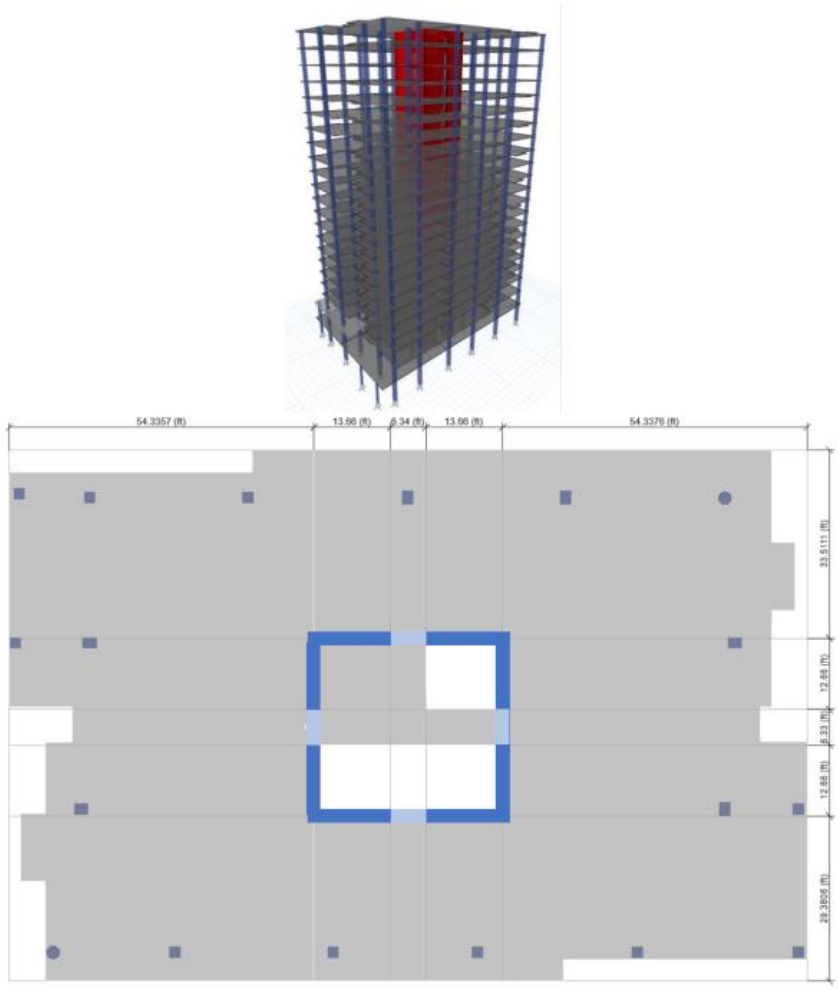


Figure 4. (a) A 3D View and (b) a Second-floor Plan View of the Example Building

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 Reinforced Concrete Ductile Coupled Structural (Shear) Walls. ***The advantage of this new system is a higher value of the response reduction factor, R, which is 8 for this system.*** Isolated or non-coupled special structural walls are assigned an *R*-value of 6 when designed as part of a building frame system and an *R*-value of 5 when designed as part of a bearing wall system. However, the higher *R*-value, and, consequently, a lower seismic base shear comes with some restrictions on the wall geometry as well as an added detailing requirement, as shown below:

▪ Individual walls need to satisfy $h_{wcs}/\lambda_w \geq 2.0$, where h_{wcs} is the height of the entire structural wall above the critical section for flexural and axial loads, and λ_w is the wall length. In this example, h_{wcs} = 2,811 in. (234.25 ft) and λ_w = 164 in. (13.67 ft) along the x-axis of the building and 152 in. (12.67 ft) along the y-axis of the building. So, the minimum value of $h_{wcs}/\lambda_w = 17.1 > 2.0$. Thus, the building satisfies the first condition.

▪ Coupling beams need to satisfy $\lambda_n/h \geq 2.0$ at all levels of the building, where λ_n is the length of the clear span of the beam, and h is the height of the beam. In this example, λ_n = 76 in. (6.33 ft) and h = 28 in. (2.33 ft). So, $\lambda_n/h = 2.71 > 2.0$. Thus, the coupling beams satisfy this condition.

▪ In at least 90 percent of the floors, all coupling beams need to have $\lambda_n/h \leq 5.0$. In the example building, all coupling beams on all floor levels are the same dimensions. So, this condition is also satisfied.

▪ The last condition requires the provisions related to the development of the beam reinforcement to be in accordance with ACI 318 Section 18.10.2.5. This will be satisfied at the detailing stage of the beam.

So, the structural walls in the example building satisfy all the conditions to qualify to be designed as ductile coupled shear walls

5.1.2 DESIGN CRITERIA

The member sizes for the structure are chosen as follows:

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

Other relevant design data are as follows:

Material properties

▪ Concrete (used in structural walls and columns): f_c' = 8,000 psi (all stories)
▪ 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

Service loads

▪ Superimposed dead load: 25 psf (includes superimposed dead load on the floor plus the weight of cladding distributed over the floor slab.)

▪ **Floor live load**: Based on the 40 psf live load prescribed in ASCE/SEI 7 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

ASCE/SEI 7-22 requires structures in U.S. locations to be designed using multi-period spectra from the USGS Seismic Design Geodatabase. The example was done using the two-period design response spectrum of Section 11.4.5.2 (ASCE/SEI 7-22), using the following ground motion parameters.

The maximum considered earthquake spectral response acceleration:

At short periods, $S_S = 1.65$g, and
At 1-sec period, $S_1 = 0.65$g.

The maximum considered earthquake spectral response acceleration (site modified):
At short periods, $S_{MS} = 1.65$g, and
At 1-sec period, $S_{M1} = 0.975$g.

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: $S_{D1} = 2/3\ S_{M1}/g = 2/3 \times 0.975 = 0.65$

Long-period transition period, $T_L = 8$ sec

Reinforced Concrete Ductile Coupled Structural (Shear) Walls ... $R = 8$; $C_d = 8.0$, $\Omega_0 = 2.5$
(ASCE/SEI Table 12.2-1)

Seismic Design Category: Based on both S_{DS} (ASCE/SEI Table 11.6-1) and S_{D1} (ASCE/SEI Table 11.6-2), the seismic design category (SDC) for the example building is D.

5.1.3 DESIGN BASIS

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 Section 12.9.1) is used in this example. However, as part of the MRSA procedure, base shear is also determined using the Equivalent Lateral Force (ELF) procedure. This is because ASCE/SEI 7 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.

5.1.4 LOAD COMBINATIONS FOR DESIGN

The following load combinations are used in the strength design method for concrete.
a. $U = 1.4D$
b. $U = 1.2D + 1.6L$
c. $U = 1.2D + 0.5L + 1.0E$
d. $U = 0.9D + 1.0E$

where: D = dead load effect
L = live load effect
$E = \rho Q_E + 0.2S_{DS}D$ when the effects of gravity and seismic loads are additive
$E = \rho Q_E - 0.2S_{DS}D$ when the effects of gravity and seismic loads are counteractive
Q_E = the effect of horizontal seismic forces
ρ = redundancy factor (discussed below)

5.1.5 SYSTEM IRREGULARITY AND ACCIDENTAL TORSION

ASCE/SEI 7 requires consideration of accidental torsion in seismic analysis in a given direction when the structure has a horizontal irregularity Type 1a or 1b (torsional irregularity and extreme torsional irregularity, respectively) when subjected to seismic forces in the same direction. This was first investigated by performing a preliminary analysis of the structure by applying the seismic forces separately along the x- and y-axes of the structure and by incorporating a load eccentricity equal to 5% of the floor width. Presence of torsional irregularity was determined by checking if the ratio of maximum to average story drift at any floor equals or exceeds 1.2.

It was found that, for seismic forces acting along the x-axis of the structure, no torsional eccentricity is present in the structure. Thus, for the final seismic analysis of the structure, no accidental eccentricity was considered.

For seismic forces acting along the y-axis of the structure, torsional irregularity was found to be present. As a result, a 5% accidental eccentricity, as described above, was included in the final analysis. Additionally, because the structure is assigned to SDC D, the accidental torsion was required to be amplified in the first four floor levels as the ratio of maximum to average story drift at those floors exceeded 1.2.

5.1.6 REDUNDANCY FACTOR, ρ

A check was made to see if the seismic force-resisting system of the structure can be considered redundant or not. A structure is considered redundant if removal of a shear wall or wall pier with a height-to-length ratio greater than 1.0 within any story would not result in more than a 33% reduction in story strength; nor would the resulting system have an extreme torsional irregularity (horizontal structural irregularity Type 1b).

In the example building, the height-to-length ratio of the shear walls in the first story is greater than one. There are four such walls of equal length in either direction. So, removal of one such wall would result in the reduction of the shear resistance of the building by approximately 25%.

After performing seismic analysis separately along the x- and y-axes of the building with the wall removed, it was seen that the maximum ratio of maximum to average story drift was 1.39, which is less than the 1.6 threshold for defining an extreme torsional irregularity in ASCE/SEI 7-22.

In view of the above, the seismic force-resisting system of the building can be considered redundant, and the value of ρ can be taken as 1.0.

5.1.7 ANALYSIS BY EQUIVALENT LATERAL FORCE PROCEDURE

Structural period calculation

Coefficient, C_t [ASCE/SEI Table 12.8-2]= 0.02
Coefficient, x [ASCE/SEI Table 12.8-2] x = 0.75
Structure height above base, h_n = 234.25 ft
Approximate period, $T_a = 0.02h_n^x = 0.02 \times 234.25^{0.75} = 1.2$ sec.
Fundamental period calculated by modal analysis in ETABS, T = 2.58 sec (along x-axis)
Fundamental period calculated by modal analysis in ETABS, T = 2.26 sec (along y-axis)

Calculated period is larger than the approximate period. However, the fundamental period cannot exceed C_uT_a.

For S_{D1} = 0.65, C_u = 1.4
$C_uT_a = 1.4 \times 1.2 = 1.68$ sec

Thus, T used in design = 1.68 sec < T_L (= 8 sec)

Base shear calculation

$V = C_SW$ (ASCE/SEI Eq. 12.8-1)

where:

$$C_S = \frac{S_{DS}I_e}{R} = \frac{1.10\times1.0}{8} = 0.138 \qquad \text{(ASCE/SEI Eq. 12.8-2)}$$

$$\le \frac{S_{D1}I_e}{RT} = \frac{0.65\times1.0}{8\times1.68} = 0.0485 \quad (\text{for } T \le T_L) \qquad \text{(ASCE/SEI Eq. 12.8-3)}$$

$$\ge 0.044S_{DS}I_e = 0.044 \times 0.65 \times 1.0 = 0.029 \qquad \text{(ASCE/SEI Eq. 12.8-5)}$$

$$\ge 0.01 \qquad \text{(ASCE/SEI Eq. 12.8-5)}$$

$$\ge \frac{0.5S_1I_e}{R} = \frac{0.5\times0.65\times1.0}{8} = 0.041 \quad (\text{where } S_1 \ge 0.6\text{g}) \qquad \text{(ASCE/SEI Eq. 12.8-6)}$$

Governing C_S = 0.048
Seismic weight, W = 43,099 kips
Base shear, $V = C_SW = 0.048 \times 43{,}099 = 2{,}090$ kips

5.1.8 MODAL RESPONSE SPECTRUM ANALYSIS

A three-dimensional analysis of the structure is performed using modal response spectrum analysis using ETABS (Version 2016) computer program. In the ETABS model, ***semi-rigid diaphragms*** are assigned at each level. Accidental torsion is addressed in the way described in Section 5.1.5 above.

According to ASCE/SEI Section 12.7.3, the mathematical model must consider cracked section properties. The stiffnesses of members used in the analyses are as follows:

For columns and shear walls, $I_{eff} = 0.7I_g$
For coupling beams, $I_{eff} = 0.25I_g$

For gravity columns, in order to minimize their contribution to the lateral stiffness of the structure, I_{eff} is taken as 0.1I_g. In addition, the columns are connected at the base by pinned connections. P-Δ effects were considered in the lateral analysis.

An 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 and story drifts obtained from the MRSA are shown in Tables 6 and 7, respectively.

Table 6. Floor Forces from MRSA

Story	Elevation (ft)	Story height (ft)	X-Dir (kip)	Y-Dir (kip)
L23	234.25	10	195.346	195.346
L22	224.25	10	206.044	206.044
L21	214.25	10	192.838	192.838
L20	204.25	10	177.633	177.633
L19	194.25	10	165.042	165.042
L18	184.25	10	150.814	150.814
L17	174.25	10	138.882	138.882
L16	164.25	10	125.656	125.656
L15	154.25	10	114.433	114.433
L14	144.25	10	102.24	102.24
L13	134.25	10	91.783	91.783
L12	124.25	10	80.685	80.685
L11	114.25	10	71.058	71.058
L10	104.25	10	61.056	61.056
L09	94.25	10	52.344	52.344
L08	84.25	10	43.803	43.803
L07	74.25	10	35.838	35.838
L06	64.25	10	28.482	28.482
L05	54.25	10	21.771	21.771
L04	44.25	13	16.224	16.224
L03	31.25	15	12.95	12.95
L02	16.25	16.25	3.78	3.78

Table 7. Story Drifts from MRSA

Story	Story height (ft)	X-Direction				Y-Direction			
		δ_{xe} (in.)	C_d	δ_x (in.)	Relative Drift (%)	δ_{xe} (in.)	C_d	δ_x (in.)	Relative Drift (%)
L23	10	3.36	8	26.84	1.00	3.64	8	29.11	1.07
L22	10	3.21	8	25.65	1.02	3.48	8	27.83	1.09
L21	10	3.05	8	24.43	1.03	3.32	8	26.52	1.11
L20	10	2.90	8	23.18	1.05	3.15	8	25.19	1.13
L19	10	2.74	8	21.92	1.07	2.98	8	23.83	1.15
L18	10	2.58	8	20.64	1.08	2.81	8	22.45	1.17
L17	10	2.42	8	19.33	1.10	2.63	8	21.05	1.18
L16	10	2.25	8	18.02	1.10	2.45	8	19.63	1.19
L15	10	2.09	8	16.69	1.11	2.28	8	18.21	1.19
L14	10	1.92	8	15.36	1.11	2.10	8	16.77	1.20
L13	10	1.75	8	14.03	1.11	1.92	8	15.34	1.19
L12	10	1.59	8	12.70	1.10	1.74	8	13.91	1.19
L11	10	1.42	8	11.38	1.09	1.56	8	12.48	1.18
L10	10	1.26	8	10.07	1.08	1.38	8	11.06	1.17
L09	10	1.10	8	8.77	1.06	1.21	8	9.65	1.15
L08	10	0.94	8	7.50	1.04	1.03	8	8.27	1.13
L07	10	0.78	8	6.25	1.00	0.86	8	6.92	1.09
L06	10	0.63	8	5.06	0.95	0.70	8	5.60	1.04
L05	10	0.49	8	3.91	0.89	0.54	8	4.35	0.98
L04	13	0.36	8	2.85	0.80	0.40	8	3.18	0.87
L03	15	0.20	8	1.60	0.59	0.23	8	1.82	0.67
L02	16.25	0.07	8	0.55	0.28	0.08	8	0.62	0.32

5.1.9 STORY DRIFT LIMITATION

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

5.2 Design of Shear Walls

The design of one of the shear walls at the base of the structure is illustrated in this example. Similar procedures may be followed to design the shear wall at the other floor levels. The design of shear walls is performed in accordance with the provisions of ACI 318-19.

Each L-shaped segment of the shear wall core is designed as a flanged wall in each direction. (Figure 5). Per ASCE/SEI 7 Section 12.5.4, "any column or wall that forms part of two or more intersecting seismic force-resisting systems and is subjected to axial load due to seismic forces acting along either principal plan axis equaling or exceeding 20% of the axial design strength of the column or wall" needs to be designed considering the orthogonal combination of seismic forces along x- and yaxes of the structure in accordance with one of the procedures specified in ASCE/SEI 7 Section 12.5.3.1. The maximum axial compression on this flanged wall, when subjected to seismic forces along the x-axis, is 3,385 kips, and that when subjected to seismic forces along the y-axis is 3,586 kips. Both these values are less than 20% of the axial design strength of the wall, which is 29,547 kips (shown later). Thus, it is not required to consider orthogonal combinations of the seismic forces, and the wall is designed for the seismic forces along x- and y-axes separately.

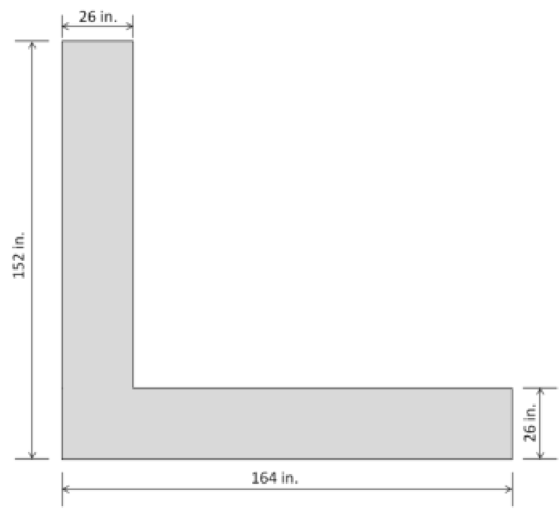


Figure 5. L-Shaped Wall Designed in the Example

5.2.1 DESIGN LOADS

Table 8 shows a summary of the axial force, shear force, and bending moment at the base of the example shear wall based on different load combinations. Seismic forces acting along the x-axis are considered in this design example. The design calculations for the seismic forces acting along the yaxis are similar and are not shown. However, Figure 7 shows the wall in its final configuration after considering seismic forces in both directions.

Table 8. Summary of Design Axial Force, Shear Force, and Bending Moment for Shear Wall between Floor 1 and Floor 2 When Subjected to Seismic Forces along x-Axis

	Load Combinations	Axial Force, P_u (kips)	Shear Force, V_u (kips)	Bending Moment, M_u (kip-ft)
1	$1.4D$	6,335	0	0
2	$1.2D+1.6L+0.5L_r$	6,071	0	0
3	$(1.2+0.2S_{DS})D+\rho Q_E+0.5L$	10,015	576	24,976
4	$(0.9D - 0.2S_{DS}D) + \rho Q_E$	6,460	573	24,585
5	$(0.9D - 0.2S_{DS}D) - \rho Q_E$	-378	573	24,585

5.2.2 DESIGN FOR SHEAR

Height of the shear wall, h_{wcs} = 2,811 in. (234.25 ft)
Length of the shear wall, λ_w = 164 in. (13.67 ft)
h_{wcs}/λ_w = 2,811/164 = 17.1

<u>ACI 318-19 Section 18.10.2.2</u>

At least two curtains of reinforcement shall be used if $V_u > 2A_{cv}\lambda$ or $h_{wcs}/\lambda_w \geq 2.0$. In this case,
For normal-weight concrete, $\lambda = 1$
$A_{cv} = \lambda_w \times b_w = 164 \times 26 = 4{,}264$ in.2
$2A_{cv}\lambda = 2 \times 4{,}264 \times 1 \times /1000 = 763$ kips > 576 kips

However, $h_{wcs}/\lambda_w = 17.1 > 2.0$.

So, at least two curtains of reinforcement are required.

ACI 318 Section 11.7.2.3 also stipulates that walls more than 10 in. thick, except single-story basement walls and cantilever retaining walls, are to be provided with two layers of reinforcement.

ACI 318-19 Section 18.10.3.1

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

For walls with $h_{wcs}/\lambda_w > 1.5$, 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, let us assume $\Omega_v = 1.5$ for now. 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 $h_{wcs}/\lambda_w \geq 2.0$ and the number of stories above the 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.

$\omega_v = 1.3 + 22/30 = 2.03 \Rightarrow \omega_v = 1.8$

So, an initial estimate of $\Omega_v\omega_v = 1.5 \times 1.8 = 2.7$. This will be verified once the correct value of Ω_v can be ascertained using M_{pr}.

$V_e = \Omega_v \omega_v V_u = 1.5 \times 1.8 \times 576 = 1{,}555$ kips

ACI 318-19 Section 18.10.4.4

Before starting to determine the required shear reinforcement, it is good to check if Ve exceeds the maximum shear strength allowed for this section. In that case, wall thickness may need to be increased.

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

$$10A_{cv}\sqrt{f_c'} = 10 \times 4{,}264 \times \sqrt{8000}/1000 = 3{,}813 \text{ kips}$$

So, $\phi Vn = 0.75 \times 3{,}813 = 2{,}860$ kips > $Ve = 1{,}555$ kips

The provided wall section size is acceptable

ACI 318-19 Section 18.10.4.1

For $h_w/\lambda_w = 17.1 \geq 2.0$, $\alpha_c = 2$
For normal-weight concrete, $\lambda = 1$

$$V_n = (\alpha_c\lambda\sqrt{f'_c} + \rho_t f_y)\,A_{cv} \geq V_e/\phi \qquad \text{(ACI 318-19 Eq. 18.10.4.1)}$$

ACI 318-19 Section 21.2.4.1 requires a ϕ of 0.6 to be used in the shear design of a member that resists earthquake forces and may fail in shear before it has a chance to fail in flexure. The shear walls in this example have a minimum h_w/λ_w ratio of 17.1, which makes them flexure-controlled, meaning that flexural failure will precede their failure in shear. Thus, the proper ϕ-value to use in their shear design is 0.75.

Required horizontal shear reinforcement ratio:

$$\rho_t = \left[\frac{V_e}{\varphi A_{cv}} - \alpha_c \lambda \sqrt{f_c'}\right] \Big/ f_y$$

$$= \left[\frac{1555 \times 1000}{0.75 \times 4264} - 2 \times 1.0 \times \sqrt{8000}\right] \Big/ 60000$$

$$= 0.0051$$

Two curtains of #7 horizontal shear reinforcement at a vertical spacing of 7 in. are adequate to resist this shear force. However, the 7” spacing is reduced to 5” in order to maintain uniformity with the reinforcement provided in the other leg of the shear wall to resist shear in y direction, leading to a provided $\rho_t = 0.009$. The 5” spacing also matches the vertical spacing of the transverse reinforcement provided in the special boundary element of the wall (shown later), which helps in the construction efficiency.

Per ACI 318-19 Section 18.10.2.1, the minimum ρ_t = 0.0025 and maximum reinforcement spacing = 18 in., both of which are satisfied.

ACI 318-19 Section 18.10.4.3

h_w/λ_w exceeds 2.0. Therefore, ρ_λ need not be larger than or equal to ρ_t.

ACI 318-19 Section 18.10.2.1

Longitudinal reinforcement ratio:

$\rho_\lambda \geq 0.0025$ with a maximum spacing of 18 in.

Provided two curtains of #8 vertical reinforcement at 14 in. spacing (ρ_λ = 0.004). This will need to be increased at the end regions of the wall.

ACI 318-19 Section 18.10.2.4

In walls with $h_w/\lambda_w \geq 2.0$ that are effectively continuous from the base of the structure to the top of the wall and are designed to have a single critical section for flexure and axial loads, the longitudinal reinforcement ratio within $0.15\lambda_w$ of the ends of the wall needs to be at least:

$$6\sqrt{f_c'}\ /\ f_y.$$

The end regions in an L-shaped wall where this needs to be provided are shown below in Figure 6. For the wall along the x-axis, the length of this region is 0.15×164 = 24.6 in. For the wall along the y-axis, the length of this region is 0.15×152 = 22.8 in. In the intersection area of the two legs of the wall, the end regions overlap and almost fully cover the intersection area, as shown in Figure 6 below.

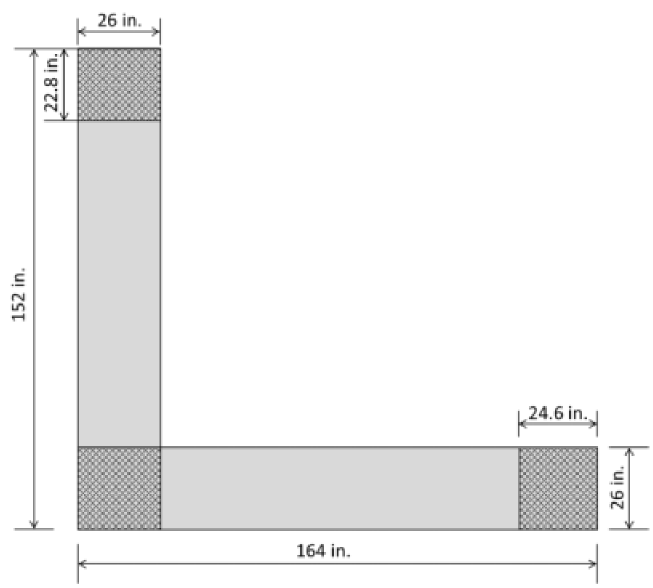


Figure 6. End Regions Requiring Vertical Reinforcement Per ACI 318-19 Section 18.10.2.4(a)

Nine #8 bars are provided in a 3×3 pattern in the wall intersection area.

$$\rho_\ell = 0.01 > 6\sqrt{f'_c}\ /\ f_y\ (= 0.009)\ \ldots\ldots\ldots\ldots\text{OK}$$

This needs to be satisfied at the other ends of the two legs of the wall. However, reinforcement provided there would be governed by special boundary element requirements, which is shown next.

5.2.3 BOUNDARY ELEMENTS OF SPECIAL REINFORCED CONCRETE SHEAR WALLS (ACI 318-19 SECTION 18.10.6)

ACI 318-19 Section 18.10.6.1

The need for special boundary elements at the edges of shear walls is to be evaluated in accordance with ACI 318-19 Section 18.10.6.2 (displacement-based approach) or ACI 318-19 Section 18.10.6.3 (stress-based approach). In this example, the displacement-based approach is used as the wall satisfies the three required conditions:

- $h_{wcs}/\lambda_w \geq 2.0$,
- The wall is continuous from the base of the structure to the top of the wall, and
- The wall has a single critical section for bending and axial loads.

ACI 318-19 Section 18.10.6.2(a): Displacement-based Approach

Compression zones are to be reinforced with special confinement reinforcement where:

$$\frac{1.5\,\delta_u}{h_{wcs}} \geq \frac{\ell_w}{600c} \quad \text{(ACI 318-19 Eq. 18.10.6.2a)}$$

In the expression above, δ_u is the design displacement, which is the total calculated lateral displacement expected for the design earthquake. For seismic forces along the x-axis of the structure, δ_u was determined from the ETABS analysis as 26.84 in (see Table 7).

In addition, c is the largest neutral axis depth of the wall cross-section calculated for the factored axial force and nominal moment strength consistent with the direction of the design displacement. This was determined using the computer program *spColumn* v7.00. Out of the two seismic load combinations considered, the axial compression P_u (= 10,015 kips) from the additive combination (1.2 D + 0.2 $S_{DS}\,D$ + ρ Q_E + 0.5 L) had the highest nominal moment strength, M_n, associated with it. The corresponding depth of the neutral axis was found to be 95 in. when the non-flanged end of the wall is in compression. For this situation,

$$\frac{1.5\delta_u}{h_{wcs}}\left(=\frac{1.5\times 26.84}{2811}=0.0143\right) \geq \frac{\ell_w}{600c}\left(\frac{164}{600\times 95}=0.003\right)$$

Also, δ_u/h_{wcs} cannot be taken less than 0.005. In this example,

δ_u/h_{wcs} = 26.84/2,811 = 0.0095 > 0.005 …..OK

As a result, a special boundary element needs to be provided at the non-flanged end of the wall.

The same check was performed for the flanged end of the wall as well. However, when the flanged end of the wall is under compression, the neutral axis depth is small due to the presence of the flange, and as a result, the above check is not satisfied. So, a special boundary element is not necessary for the flanged end of the wall.

Special boundary element confinement is provided only at the non-flanged end of the wall, as shown below.

ACI 318-19 Section 18.10.6.2(b)(i): Height of special boundary element

The special boundary element reinforcement is to extend vertically from the critical section a distance not less than the larger of λ_w and $M_u/4V_u$.

l_w = *164 in. (13.67 ft)* ... governs

$$\frac{M_u}{4V_u} = \frac{24976}{4 \times 576} = 10.84 \text{ ft}$$

ACI 318-19 Section 18.10.6.2(b)(ii): Width of special boundary element

The width of boundary element, b (= 26 in.) $\geq \sqrt{0.025 c l_w} = \sqrt{0.025 \times 95 \times 164} = 19.3$ in.

Therefore, a more detailed check by ACI 318-19 Section 18.10.6.2(b)(iii) is not necessary.

ACI 318-19 Section 18.10.6.4(a): Length of boundary element

Confined boundary element to extend horizontally from the extreme compression fiber a distance not less than the larger of $c - 0.1l_w$ and $c/2$.

$c - 0.1l_w = 95 - 0.1 \times 164 = 78.6$ in $\approx$ 80 in.…. governs

$c/2 = 95/2 = 47.5$ in.

ACI 318-19 Section 18.10.6.4(b) and (c): Stability check for wall compression zone

Minimum width of the compression zone, b = 26 in., which is required to be at least $h_u/16$, where h_u is the laterally unsupported height (clear height) of the wall (ACI 318-19 Section 18.10.6.4(b)).

h_u = Story height – depth of coupling beam = 158 in.

$h_u/16$ = 9.875 in. < 26 in. …..OK

Also, for this wall, h_{wcs}/l_w = 17.1 > 2.0, and it is effectively continuous from the base of the structure to the top of the wall and designed to have a single critical section for flexure and axial loads. And c/l_w = 95/164 = 0.58 > 3/8. As a result, ACI 318-19 Section 18.10.6.4(c) requires the width of the flexural compression zone b over the length of 80 in. (calculated above) to be greater than or equal to 12 in. This is satisfied as the width of the wall is 26in.

ACI 318-19 Section 18.10.6.4(a): Length of boundary element

Confined boundary element to extend horizontally from the extreme compression fiber a distance not less than the larger of $c - 0.1\, l_w$ and $c/2$.

$c - 0.1\, l_w = 95 - 0.1 \times 164 = 78.6$ in ≈ 80 in.…. governs

$c/2 = 95/2 = 47.5$ in.

ACI 318-19 Section 18.10.6.4(b) and (c): Stability check for wall compression zone

Minimum width of the compression zone, b = 26 in., which is required to be at least $h_u/16$, where h_u is the laterally unsupported height (clear height) of the wall (ACI 318-19 Section 18.10.6.4(b)).

h_u = Story height – depth of coupling beam = 158 in.

$h_u/16$ = 9.875 in. < 26 in. …..OK

Also, for this wall, h_{wcs}/l_w = 17.1 > 2.0, and it is effectively continuous from the base of the structure to the top of the wall and designed to have a single critical section for flexure and axial loads. And c/l_w = 95/164 = 0.58 > 3/8. As a result, ACI 318-19 Section 18.10.6.4(c) requires the width of the flexural compression zone b over the length of 80 in. (calculated above) to be greater than or equal to 12 in. This is satisfied as the width of the wall is 26 in.

ACI 318-19 Section 18.10.6.4(d): Flanged section

It is required that in flanged sections, the boundary element is to include the effective flange width in compression. The boundary element is also required to extend into the web by at least 12 in.

In this example, the flanged side of the wall does not require a special boundary element. So, these requirements do not apply.

ACI 318-19 Section 18.10.6.4(g): Minimum area of transverse reinforcement

$$A_{sh}/sb_c = \text{Greater of } \begin{cases} 0.3\left(\dfrac{A_g}{A_{ch}} - 1\right)\dfrac{f'_c}{f_{yt}} \\ 0.09\dfrac{f'_c}{f_{yt}} \end{cases} \qquad \text{(ACI 318-19 Table 18.10.6.4(g))}$$

Confinement perpendicular to the length of the wall: With a boundary element length of 80 in., width of 26 in. and 1.5 in. clear cover all around the boundary element:

$b_c = 80 - 2 \times 1.5 = 77$ in.

$A_g = 80 \times 26 = 2{,}080$ in.^{2}

$A_{ch} = (80 - 2 \times 1.5) \times (26 - 2 \times 1.5) = 1{,}771$ in.^{2}

$$A_{sh}/sb_c = \text{Greater of } \begin{cases} 0.3\left(\dfrac{2080}{1771} - 1\right)\dfrac{8}{60} \\ 0.09\dfrac{8}{60} \end{cases}$$

$$= 0.012$$

The vertical spacing of the transverse reinforcement, s, is needed to calculate the required A_{sh}. However, determination of s involves the horizontal spacing, h_x, of the laterally supported longitudinal bars, which, in turn, requires knowing A_{sh}. So, an iterative process is needed to ascertain an acceptable value of A_{sh} and its vertical spacing s.

Based on the ACI 318-19 Section 18.10.6.4(e) requirements, a vertical spacing of 6 in. would be acceptable, as shown later. However, in this example, a vertical spacing of 5 in. is provided to keep the required cross-sectional area of transverse reinforcement from being excessive, with s = 5 in., minimum required $A_{sh} = 0.012sb_c = 0.012 \times 5 \times 77 = 4.62$ in.^{2}

Provided 16 #5 bars in the form of hoops and cross-ties with a vertical spacing of 5 in.

A_{sh} provided = $16 \times 0.31 = 4.96$ in.^{2} > 4.62 in.^{2} .... OK

Confinement parallel to the length of the wall: With a wall width of 26 in. and 1.5 in. clear cover all around the boundary element, the width of the boundary element core for confinement along the length of the wall:

$$b_c = 26 - 2 \times 1.5 = 23 \text{ in.}$$

$$A_{sh}/sb_c = \text{Greater of } \begin{cases} 0.3\left(\dfrac{2080}{1771} - 1\right)\dfrac{8}{60} \\ 0.09\dfrac{8}{60} \end{cases}$$

$$= 0.012$$

With s = 5 in., minimum required $A_{sh} = 0.012sb_c = 0.012 \times 5 \times 23 = 1.38$ in.^{2}

Provided 5 #5 bars in the form of a single hoop and two cross-ties with a vertical spacing of 5 in.

A_{sh} provided = $5 \times 0.31 = 1.55$ in.^{2} > 1.38 in.^{2} .... OK

ACI 318-19 Section 18.10.6.4(f): Spacing limitation of transverse reinforcement

Transverse reinforcement is to be arranged such that the spacing h_x between laterally supported longitudinal bars around the perimeter of the boundary element does not exceed the lesser of

▪ 14 in. ….Governs

▪ Two-thirds of the boundary element thickness = 2/3 × 23 = 15.33 in. (for transverse reinforcement arranged perpendicular to the wall length)

So, maximum h_{sx} = 14 in.

For transverse reinforcement arranged perpendicular to the wall length, 16 #5 transverse bars are provided over a width of 80 in. Each of these transverse bars engages one #8 longitudinal bar at the perimeter of the boundary element. Assuming an approximately uniform spacing, the distance between laterally supported longitudinal bar at the perimeter:

(80 – 2 × 1.5 – 2 × 0.625 – 1)/(16 – 1) = 4.98 in. < 14 in. ….OK

For transverse reinforcement arranged parallel to the wall length, 5 #5 transverse bars are provided over a width of 26 in. Each of these transverse bars engages one #8 longitudinal bar at the perimeter of the boundary element. Assuming an approximately uniform spacing, the distance between laterally supported longitudinal bar at the perimeter:

(26 – 2 × 1.5 – 2 × 0.625 – 1)/(5 – 1) = 5.18 in. < 14 in. ….OK

Lateral support to the longitudinal bars is required to be provided by a seismic hook of a crosstie or corner of a hoop. The length of a hoop leg (measured as the outside dimension of the hoop leg) cannot exceed two times the boundary element core thickness (2 × 23 = 46 in.), and adjacent hoops need to overlap (measured as the c/c distance of longitudinal bars enclosed by the overlapping hoops) at least the lesser of

▪ 6 in. …..Governs

▪ Two-thirds the boundary element thickness = 2/3 × 26 = 17.33 in.

To meet these requirements, three overlapping hoops with cross-ties are provided, as shown in Figure 7.

ACI 318-19 Section 18.10.6.4(e): Vertical spacing of transverse reinforcement

According to ACI 318-19 Section 18.7.5.3, as revised by ACI 318-19 Section 18.10.6.4(e), the transverse reinforcement is to be vertically spaced at a distance not exceeding

(a) One-third of the least dimension of the boundary element = 23/3 = 7.67 in.

(b) Six times the diameter of the smallest longitudinal reinforcement = 6 × 1.0 = 6.0 in. …..
Governs

(c) so, as defined by ACI 318-19 Eq. (18.7.5.3).

4 in. $s_o = 4 + (14 – h_x)/3$ 6 in. (ACI 318-19 Eq. 18.7.5.3)

4 in. $s_o = 4 + (14 – 5.18)/3$ 6 in.

4 in. $s_o = 6.94$ in. 6 in. => $s_o = 6.0$ in.

The vertical spacing also cannot exceed the maximum value given in ACI 318-19 Table 18.10.6.5(b). For Grade 60 reinforcement within the height of the special boundary element, it is the lesser of

▪ Six times the diameter of the smallest longitudinal reinforcement = 6 × 1.0 = 6 in.

▪ 6 in.

The provided spacing of 5 in. satisfies both these limits.

ACI 318-19 Section 18.10.6.4(h): Concrete in floor system

Concrete within the thickness of the floor system at the special boundary element location is required to have a specified compressive strength of at least $0.7f'_c$. With the slab concrete strength of 6000 psi, this is satisfied.

ACI 318-19 Section 18.10.6.4(i and j)

The special boundary element confinement determined above is to be provided at the non-flanged end of the wall at the base of the shear walls. The confinement needs to extend vertically by at least 12.67 ft above the base (ACI 318-19 Section 18.10.6.2(b)). Below the base, the boundary element transverse reinforcement needs to extend at least 12 in.

ACI 318-19 Section 18.10.6.4(k)

Horizontal web reinforcement is required to be extended to within 6 in. of the wall end. It is also required to be anchored to develop f_y within the confined core of the boundary element.

All horizontal web rebars in the wall have clear cover and clear spacing more than d_b, and the bars are developed within the highly confined special boundary element of the wall. As a result, the provisions of ACI 318-19 Section 25.4.2.3 can be used for the development length calculation.

For No. 7 horizontal bars in the web

$$\ell_d = \left(\frac{f_y \psi_t \psi_e \psi_s}{20\lambda\sqrt{f_c'}}\right) d_b \quad \text{(ACI 318 Table 25.4.2.3)}$$

λ = 1.0 for normalweight concrete
ψ_t = 1.3 (ACI 318-19 Table 25.4.2.5)
ψ_e = 1.0 (ACI 318-19 Table 25.4.2.5)
ψ_g = 1.0 (ACI 318-19 Table 25.4.2.5)

$$\ell_d = \left(\frac{60{,}000 \times 1.3 \times 1.0 \times 1.0}{20 \times 1.0 \times \sqrt{8000}}\right) \times 0.875 = 38.2 \text{ in.}$$

Length of confined core available within the boundary elements
= 80 – 2×1.5 = 77 in. > 38.2 in. ….. OK

Length of boundary element available up to 6 in. from the outside surface

= 80 – 2×1.5 – 6 = 71 in. > 38.2 in. ……OK

Also, for horizontal web reinforcement: $A_v f_y / s$ = 2×0.60×60/5 = 14.4 kips/in.

And for boundary element transverse reinforcement parallel to the web reinforcement:

$A_{sh} f_{yt} / s$ = 5×0.31×60/5 = 18.6 kips/in. > $A_v f_y / s$

So, horizontal web reinforcement can be terminated in the boundary element without a standard hook at 6 in. from the end of the wall.

ACI 318-19 Section 18.10.6.5: Boundary confinement where special boundary element is not required

As mentioned before, when the flanged end of the shear wall is under compression, the depth of neutral axis, *c*, is small due to the presence of a large flange width, and ACI 318-19 Eq. (18.10.6.2a) is not satisfied. As a result, a special boundary element is not required to be provided in the wall flange. However, some minimum ties are still required, as shown below.

$$V_u \geq \lambda \sqrt{f'_c}\, A_{cv}$$

▪ In this example, So, the end of the horizontal web reinforcement that terminates at the edges of the wall flange is required to have a standard hook engaging the edge reinforcement. Alternatively, the edge reinforcement at the flange is required to be enclosed in Ustirrups having the same size and spacing as, and spliced to, the horizontal reinforcement. The first option is utilized for this example, as shown in Figure 7.

Note: Four additional longitudinal bars are shown within the intersection region of the wall in order to anchor the horizontal web reinforcement.

▪ Confinement reinforcement needs to be provided in the flange where the longitudinal reinforcement ratio ρ_λ exceeds $400/f_y$. In this example

$400/f_y = 400/60{,}000 = 0.0067$

In the intersection region of the wall, 9 #8 longitudinal bars are provided within an area of 26 × 26 = 676 in.^{2}

$\rho_l = 9 \times 0.79 / 676 = 0.0105 > 0.0067$

So, in this region, transverse reinforcement needs to be arranged such that the spacing h_x of longitudinal bars laterally supported by the corner of a crosstie or hoop leg does not exceed 14 in. around the perimeter of the region. This is shown in Figure 4-7. The vertical spacing of this transverse reinforcement cannot exceed the maximum value given in ACI 318-19 Table 18.10.6.5(b). For Grade 60 reinforcement within the same height over which the special boundary element is provided (12.67 ft), it is the lesser of

- Six times the diameter of the smallest longitudinal reinforcement = 6 × 1.0 = 6.0 in.
- 6 in.

However, a vertical spacing of 5 in. is provided to match the spacing of the other transverse reinforcement for construction efficiency.

For Grade 60 reinforcement outside of the height over which the special boundary element is provided (13.67 ft), it is the lesser of

- 8 times the diameter of the smallest longitudinal reinforcement = 8 × 1.0 = 8.0 in.
- 8 in.

Outside of the intersection region, the flange is provided with 2 #8 longitudinal bars at 14 in. spacing => $\rho l = 0.0043 < 0.0067$. As a result, no transverse reinforcement is required in this portion of the flange. However, at the end of the flange, a special boundary element will need to be provided based on seismic forces along the y-axis of the structure. This will need to be done in the exact same way as described above. The calculations for the seismic forces acting along y-axis are not shown, but Figure 7 shows the wall in its final configuration after considering seismic forces in both directions.

In addition, for comparison purposes, Figure 4-8 is provided to illustrate what the final configuration of the wall would look like if Grade 80 reinforcement is used instead of Grade 60. While Grade 60 reinforcement has a much wider application in the United States, Grade 80 reinforcement has become popular in high seismic regions of the country. As can be seen from the two figures, 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 of the wall is especially noticeable. However, the vertical spacing of the transverse hoops and crossties in the special boundary elements remained the same (s = 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.

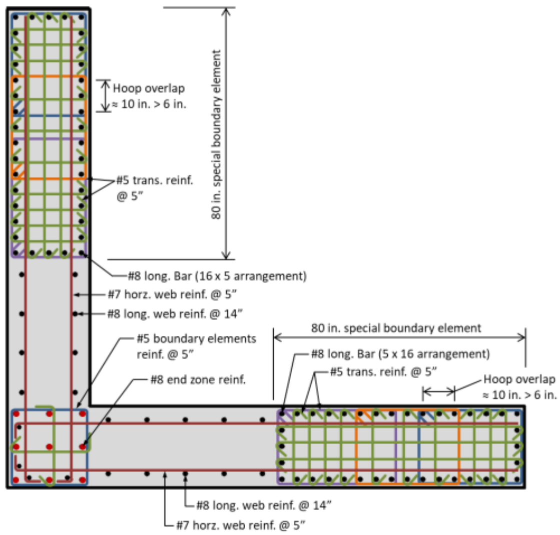


Figure 7. Reinforcement Details at the Critical Section of the Shear Wall Based on Seismic Forces Along x- and y-axes of the Building Using Grade 60 Reinforcement

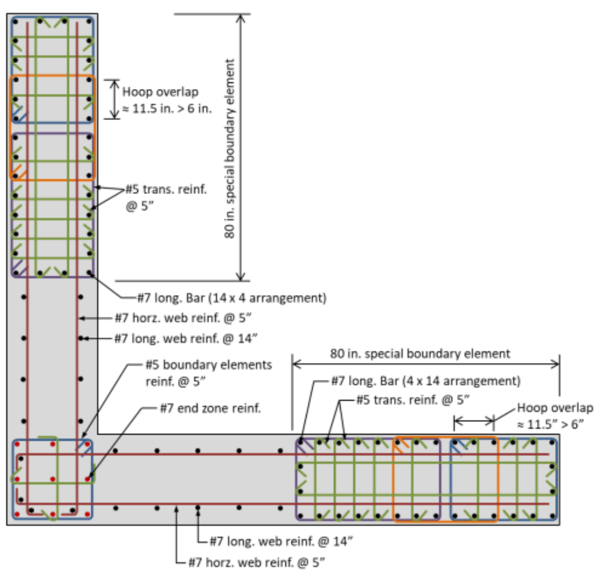


Figure 8. Reinforcement Details at the Critical Section of the Shear Wall Based on Seismic Forces Along x- and y-axes of the Building and Using Grade 80 Reinforcement

5.2.4 CHECK STRENGTH UNDER FLEXURE AND AXIAL LOADS (ACI 318-19 SECTION 18.10.5.1)

Shear walls and portions of such walls subject to combined flexural and axial loads are to be designed in accordance with ACI 318-19 Section 22.4. Boundary elements, as well as the wall web, are to be considered effective.

Figure 9 shows the P-M interaction diagram for example shear wall. As can be seen, all the points representing required strength are within the design strength curve.

Also, probable moment strength M_{pr} of the final wall configuration, calculated for P_u = 10,015 kips (see Table 8) and using ϕ = 1.0 and $f_u = 1.25f_y$, was found to be 90,828 kip-ft. From Table 4-8, the corresponding M_u = 24,976 kip-ft. So the value Ω_v used for design shear can be recalculated as M_{pr}/M_u = 90,828/24,976 = 3.64

This is larger than the initial value of 1.5 used before (Section 4.5.2.2 above). So, design shear V_e is recalculated as

$V_e = \Omega_v \omega_v V_u$ = 3.64 × 1.8 × 576 = 3,774 kips

However, V_e does not need to be taken greater than $3V_u$ (= 3 × 576 = 1,728 kips).

Thus, the design shear V_e = 1,728 kips. This is only 11% greater than what was initially estimated, and the provided reinforcement is adequate for this increase.

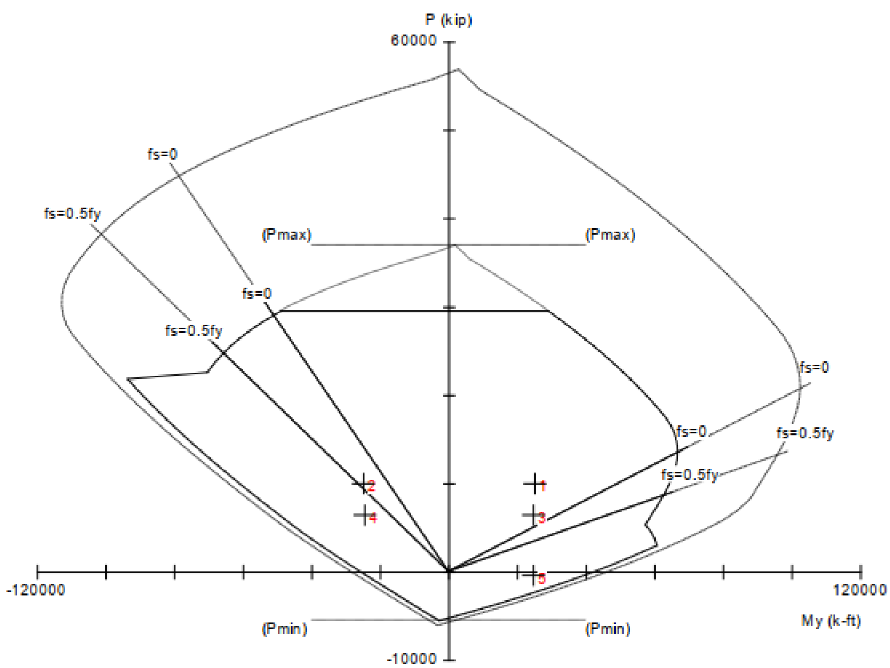


Figure 9. P-M Interaction Diagram for Seismic Forces Along x-axis

5.3 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 the length to height ratio of this beam, 2.7, is less than 4 but greater than 2, per ACI 318-19 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 318-19 Sections 18.6.3 through 18.6.5. The second option is adopted for this example.

5.3.1 DESIGN LOADS

The forces on this beam due to gravity loads are minimal. So, the design shear and moment are determined from the seismic forces alone. The governing forces on this beam come when the seismic forces are acting along the y-axis of the building. Those forces are shown below.

V_u = ±154 kips
M_u = ±488 kip-ft

5.3.2 DESIGN FOR FLEXURE

<u>ACI 318-19 Section 18.6.3.1: Limits on flexural reinforcement</u>

The minimum area of flexural reinforcement required for both top and bottom faces of the beam is shown below. Assuming a 1.5 in. clear cover, No. 8 bars (1 in. dia.) as longitudinal reinforcement and No. 4 bars (0.625 in. dia.) as transverse reinforcement:

Effective depth, d = 28 – 1.5 – 0.5 – 0.5 = 25.5 in.

$$A_{s,min} \geq \frac{3\sqrt{f_c'}}{f_y} b_w d = \frac{3\sqrt{8000}}{60{,}000} \times 26 \times 25.5 \qquad = 2.97 \text{ in.}^2$$

...(ACI 318-19 Section 9.6.1.2(a))

$$\geq \frac{200}{f_y} b_w d = \frac{200}{60{,}000} \times 26 \times 25.5 \quad = 2.21 \text{ in.}^2$$

...(ACI 318-19 Section 9.6.1.2(b))

Also, for Grade 60 steel, the maximum area of flexural reinforcement required for both top and bottom faces of the beam is

$A_{s,max} = 0.025 b_w d = 0.025 \times 26 \times 25.5 = 16.58 \text{ in.}^2$ (ACI 318-19 Section 18.6.3.1)

Also, at least two bars should be continuous at both top and bottom (ACI 318-19 Section 18.6.3.1).

<u>Provided flexural reinforcement and flexural strength</u>

Try the following reinforcement:

6-#8 bars at the bottom => $A_s = 4.74$ in.^{2}
6-#8 bars at the top => $A_s = 4.74$ in.^{2}

Using *spColumn* software, the positive and negative design moment strengths (i.e., ϕM_{n+} and ϕM_{n-}) at all locations of the beam were found to be

$\phi M_n = 526$ ft-kips > 488 kip-ft ….. O.K.

The same reinforcement is continued through the length of the beam. For a beam with a length of 6.33 ft, it is not worth cutting off some of the bars near midspan.

<u>ACI 318-19 Section 18.6.3.2</u>

At the joint face, the positive moment strength must be at least half the negative moment strength. Since the top and bottom reinforcement are the same, this is automatically satisfied.

Additionally, both the negative and the positive moment strength at any section along member length must be at least one-fourth the maximum moment strength provided at the face of either joint. Since no bar is being cut off near midspan, this requirement is also satisfied.

<u>ACI 318 Section 18.10.9.3</u>

In a ductile coupled shear wall, the longitudinal reinforcement needs to be developed at both ends of the beam in accordance with ACI 318-19 Section 18.10.2.5. Item (a) of that section requires that for coupling beams reinforced like a special moment frame beam, the development length of longitudinal reinforcement must be 1.25 times the values calculated for f_y in tension.

All longitudinal rebars in the beam have clear cover and clear spacing more than d_b, and the bars are developed within the highly confined special boundary element of the adjacent walls. As a result, the provisions of ACI 318-19 Section 25.4.2.3 can be used for the development length calculation.

$$\ell_d = \left(\frac{1.25 f_y \psi_t \psi_e \psi_s}{20\lambda\sqrt{f_c'}}\right) d_b$$

λ = 1.0 for normalweight concrete
ψ_t = 1.3 (ACI 318-19 Table 25.4.2.5)
ψ_e = 1.0 (ACI 318-19 Table 25.4.2.5)
ψ_g = 1.0 (ACI 318-19 Table 25.4.2.5)

$$\ell_d = \left(\frac{1.25 \times 60{,}000 \times 1.3 \times 1.0 \times 1.0}{20 \times 1.0 \times \sqrt{8000}}\right) \times 1.0 \approx 55 \text{ in.}$$

Thus, all longitudinal rebars need to be extended into the wall web by a distance of 55 in.

5.3.3 MINIMUM TRANSVERSE REINFORCEMENT REQUIREMENTS

ACI 318-19 Section 18.6.4.1

End regions of the beam would exhibit cyclic inelastic response when the structure is subjected to the design seismic ground motion. As a result, confinement reinforcement is required to be provided in the coupling beam over a length of two times the total depth,

$2h$ = 2 × 28 = 56 in. from both support faces.

The first hoop is to be placed no more than 2 in. from support.

The hoop spacing must not exceed:

- $d/4$ = 25.5/4 = 6.375 in.
- 6 in.
- For Grade 60 reinforcement - six times the diameter of the smallest primary flexural reinforcing bar = 6 × 1.0 = 6 in.

Since this hoop spacing needs to be provided within 56 in. from both supports, and the total length of the coupling beam is 76 in., #4 confinement hoops are provided at 6 in. spacing over the whole length of the beam starting from 2 in. from each wall face.

5.3.4 DESIGN FOR SHEAR

ACI 318 Section 18.6.5.1

$$V_e = \frac{M_{pr}^{-} + M_{pr}^{+}}{\ell_n}$$ (gravity load effects are small on this beam, and are neglected for simplicity)

For calculating the probable flexural strength at the joint faces, the tensile stress in steel should be taken as $1.25f_y$, and the strength reduction factor ϕ is to be taken as 1.0.

With 6-#8 bars at top and bottom:

M_{pr}^{+} and M_{pr}^{-} = 724 kip-ft

As a result,

$$V_e = \frac{M_{pr}^{-} + M_{pr}^{+}}{\ell_n} = \frac{2 \times 724}{6.33} = 229 \text{ kips}$$

ACI 318-19 Section 18.10.4.5

Before starting to determine the required shear reinforcement, it is good to check if V_e exceeds the maximum shear strength allowed for this section.

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

$$10A_{cv}\sqrt{f_c'} = 10 \times 26 \times 28 \times \sqrt{8000}/1000 = 651 \text{ kips}$$

So, $\phi V_n = 0.75 \times 651 = 488$ kips $> V_e$

The provided coupling beam section size is acceptable.

ACI 318-19 Section 18.6.5.2

Transverse reinforcement over a beam length of 56 in. from both supports (as determined in from ACI 318-19 Section 18.6.4.1) to resist shear V_e must be determined assuming $V_c = 0$ if both the following two conditions are met:

(i) Earthquake-induced shear force $> 0.5V_e$
In this example, 100% of the beam shear is earthquake-induced. (Satisfied)

(ii) $P_u = 0$ kip $\leq 0.05\ A_g f_c' = 0.05 \times 34 \times 24 \times 4 = 163.2$ kips (Satisfied)

Since both conditions are met, $V_c = 0$.

The shear reinforcement can be determined as follows:

$$V_s = \frac{V_e}{\varphi} - V_c$$

$$= 229/0.75 - 0 = 305 \text{ kips}$$

Required spacing of six-legged #4 stirrups,

$$s = \frac{A_v f_{yt} d}{V_s} = \frac{6 \times 0.2 \times 60 \times 25.50}{305} = 6.0 \text{ in.}$$

Provided s = 6.0 in. ....OK

Also, the shear force carried by web reinforcement, V_s, cannot exceed $8\sqrt{f_c'}\, b_w d$ (ACI 318-19 Section 22.5.1.2)

$$8\sqrt{f_c'}\, b_w d = 8 \times \sqrt{8000} \times 26 \times 25.5/1000 = 474 \text{ kips} > V_s \text{ (= 305 kips) ....OK}$$

The arrangement of beam reinforcement can be seen in Figure 10.

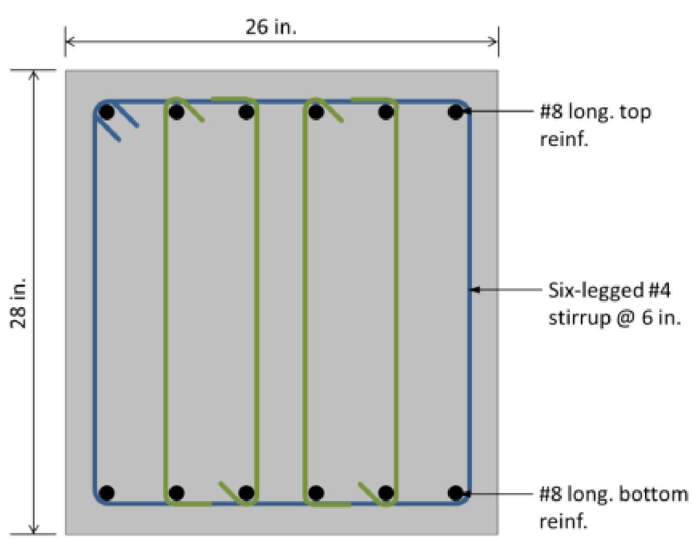


Figure 10. Reinforcement in a Coupling Beam at the Second Floor Level Along the y-axis of the Building

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