Design of a Reinforced Concrete Shear Wall and Coupling Beam

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March 21, 2021

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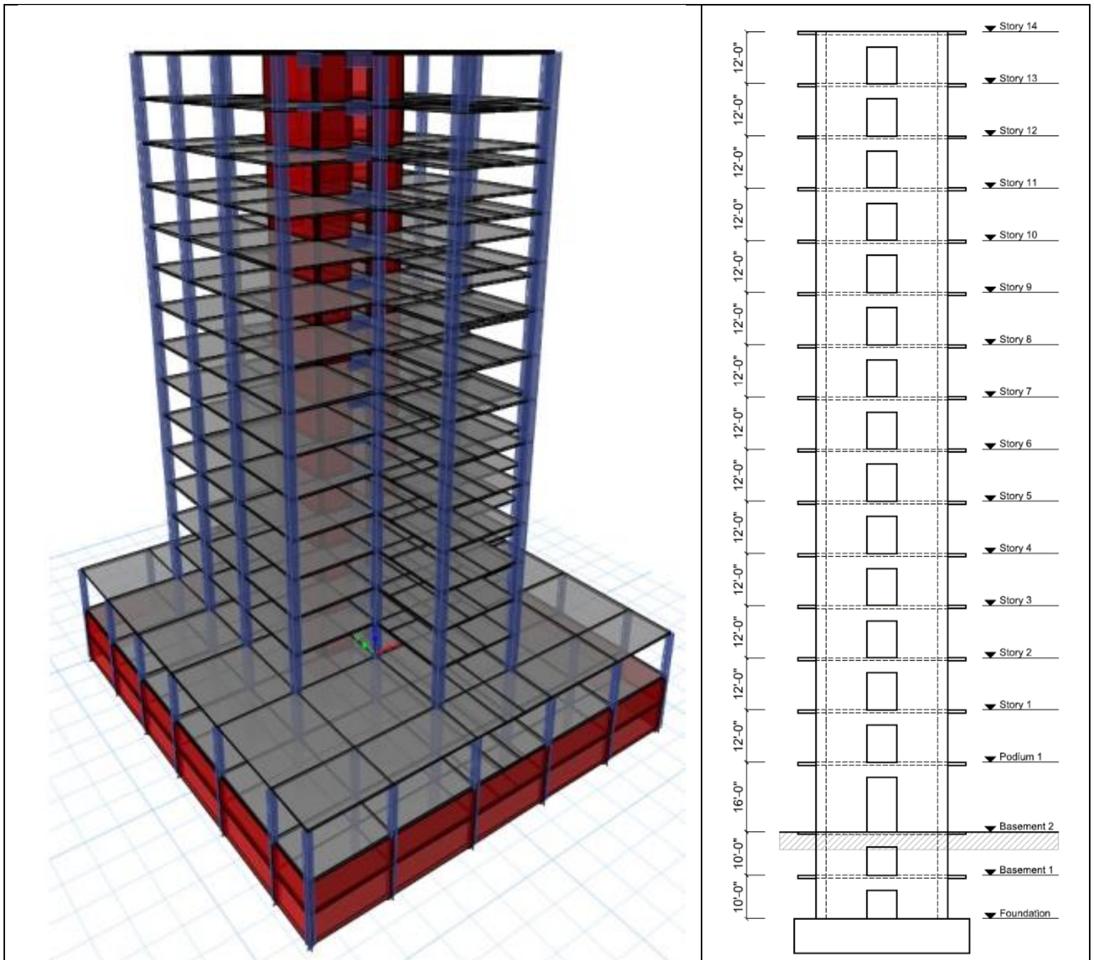
Project Description

A residential building is being constructed in downtown Los Angles, California and requires preliminary assessment and design of its concrete core wall. This report undertook the flexure and shear design of the core wall as well as the detailing of the reinforcement.

Building Description

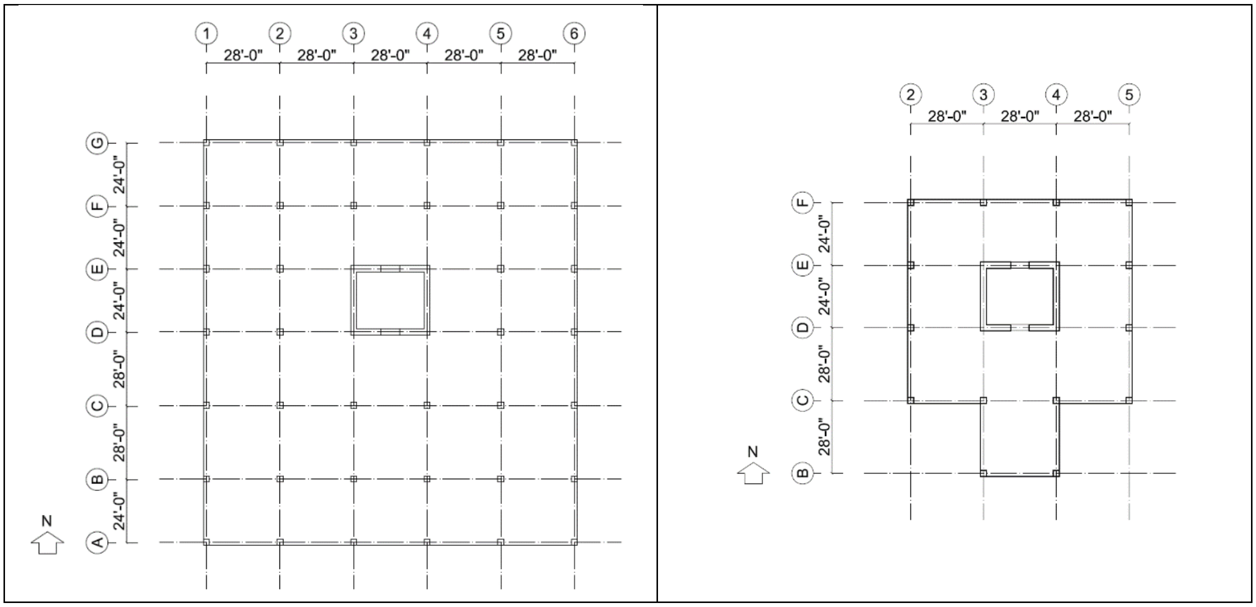
The project is a 17 story residential building in downtown LA, California. It will consist of two basement levels (10’-0” each), one podium level (16’-0”), and 14 typical levels (12’-0” each) above the podium, see Figure 1(b). The main lateral force resisting system is the concrete core wall which has been prescribed by the architect to enclose stairwells and elevators shafts, shown in red. Openings in the structural wall are provided in NS direction for functionality purposes with a coupling beam connecting the two walls.



(a) 3D Skeleton of Building (b) NS Elevation of Core Wall

Figure 1 Elevation view of building

The building has 5 bays in the East-West direction and 6 bays in the North-South direction, see Figure 2(a). The typical tower floor above the podium, see Figure 2(b), is asymmetrical about the EW direction. The core wall will be situated in the bay between EW gridlines D and E and NS gridlines 3 and 4. The typical column will be 20”x20” in the base to podium level, and 28”x28” in the tower levels. The slab will be 8” in the basement, 12” in the transfer slab, and 8” for all stories above the base. The columns will be assumed not to contribute to the lateral force resisting system of the building.



(a) Typical Podium Floor Bay Layout (b) Typical Tower Floor Bay Layout

Figure 2 Typical Floor Layouts in Building

The core wall has a uniform thickness of 28” with coupling beams connecting the two C-shaped parts. The centerline length of the core wall will be 24’-0” NS and 10’-6” in the EW direction as shown in Figure 3. The coupling beams are 7’-0” in length and have cross sections of 28”x30” for the two basement levels and 28”x42” in the rest of the building. The core wall and coupling beam will use normal weight concrete with a specified strength of 7 ksi and ASTM A706 Gr. 60 steel rebars.

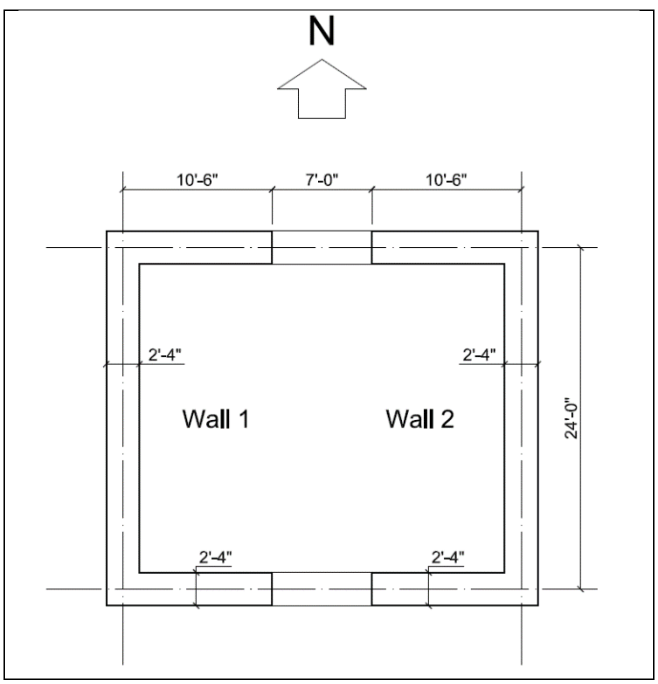


Figure 3 Core Wall Plan View

Earthquake Assessment

The site is located at 34.0497° N, 118.2556° W, with a site soil class D. The building is a considered a non-essential facility with an importance factor of 1.0. The elevation of the site is 288ft above sea level. Using ATC Hazard by Location website, provided by the United States Geological Survey, the mapped acceleration parameters are provided in Table 1. The core wall is considered a special reinforced concrete shear wall and will thus be designed using a response modification coefficient, R, of 6, overstrength factor, Ω_o, of 2.5, and a deflection amplification factor, C_d, of 5. The design spectral response curve is given in Figure 4.

Table 1 Mapped Acceleration Parameters

Notation	**Description**	**Value (g)**
S_s	Maximum Considered Earthquake Record (Period = 0.2s)	1.966
S_1	MCER ground motion (Period = 1.0s)	0.700
S_{MS}	Site-modified spectral acceleration value	1.966
S_{DS}	Site-modified spectral acceleration value	1.190
S_{D1}	Numeric seismic design value at 0.2s SA	1.311
S_{D1}	Numeric seismic design value at 1s SA	0.793

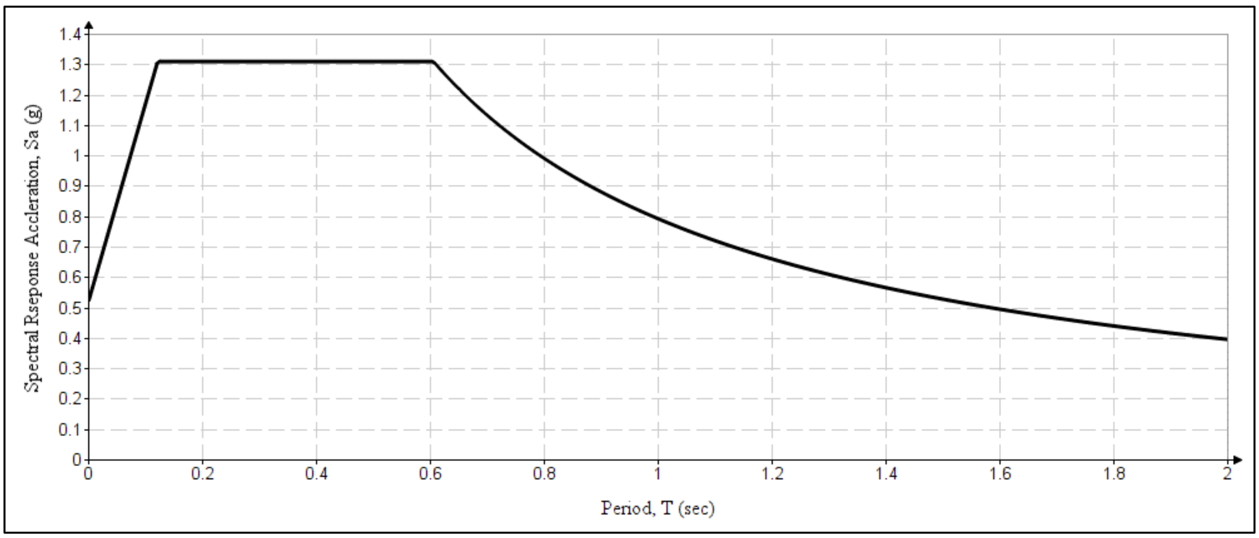


Figure 4 Design Response Spectrum

Determining Seismic Force

The core wall will be subjected to superimposed floor dead load as well as its own self weight. The surface area superimposed dead floor loads are listed in Table 2. There is an additional 10 plf around the perimeter of the building that accounts for the weight of the façade.

Table 2 Superimposed Floor Dead Load

	Typical Stories and Podium [psf]	Basement [psf]
Covering	1.0	1.0
Built in Partition	10.0	0.0
Ceiling	3.0	3.0
MEP	3.0	3.0
Miscellaneous	3.0	3.0
Sum	**20.0**	**10.0**

Using ETABS, the total seismic weight of the building was found to be 22,847 kips, considering a redundancy factor $\rho = 1.3$. The fundamental period was found in the EW direction to be 1.65 seconds and in the NS direction to be 1.74 seconds. According to the equivalent lateral force (ELF) procedure in ASCE 7-16 §12.8, the seismic response coefficient C_s was found to be 0.095g in the EW direction and 0.095g in the NS direction. This gives a seismic base shear, $V = \rho C_s W$, of 2807 kips in both directions.

Additionally, modal analysis was performed under ASCE 7-16 §12.9 in order to compare lateral forces between the two methods. According to §12.9.1.1, the analysis should include a minimum number of nodes in order to obtain at least 90% modal mass participation in each direction. The cumulative mass participation was plotted against the fundamental period of the structure shown in Figure 5. The lateral forces, which have been reduced by a factor of R, was found to be 1508 kips, V_{tx}, in the EW direction and 1639 kips, V_{ty}, in the NS direction. When compared, the lateral forces from the ELF method were amplified by 1.86 and 1.71 in the EW and NS direction respectively, calculated as $\frac{\rho C_s W}{V_t}$ for each direction.

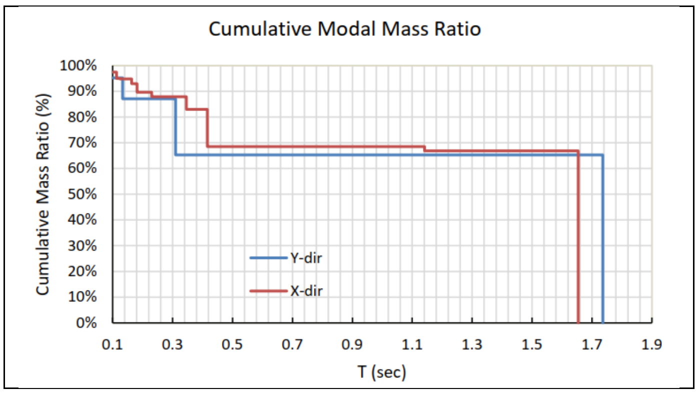


Figure 5 Cumulative Modal Mass Participation per Natural Period of the Structure

The first three modes are shown in Figure 6. The first mode is predominantly in the NS direction while the second mode is predominantly in the EW direction. Both modes demonstrate some level of eccentricity due to the eccentricity of the building. The third mode is primarily a single-twist, torsional mode with a slight translation in the EW direction.

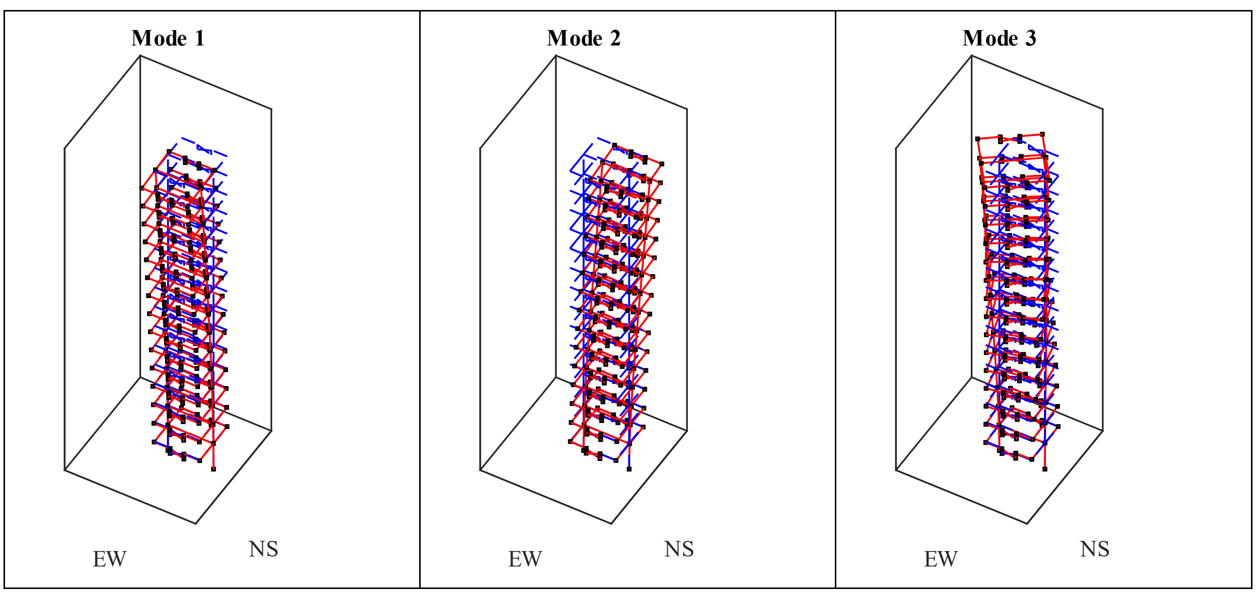


Figure 6 3D View of the First 3 Modes of the Structure

Seismic Load Effects and Combinations

The seismic load effects as defined in ASCE 7-16 §12.4.2 shall be determined by $E = E_h + E_v$ (12.4-1) or $E = E_h - E_v$ (12.4-2) depending on the critical condition where E_h is horizontal seismic load effect and E_v is the vertical seismic load effects. E_h is defined by §12.4.2.1 as ρQ_E where ρ is the redundancy factor and Q_E is the effect of the horizontal seismic forces. E_v is defined by §12.4.2.2 as $0.2S_{DS}D$ where S_{DS} is the design spectral response acceleration parameter and D is the effect of the dead load. As per §2.4.3, the basic combinations considered with seismic load effect are load combinations 6 and 7, given as $1.2D + 1.0E + 0.5L$ and $0.9D + 1.0E$. By substituting in the values of E_v and E_h, the equations become $(1.2 + 0.2S_{DS})D + 0.5L \pm 1.0E$ and $(0.9 - 0.2S_{DS})D \pm 1.0E$.

<u>Analysis Results</u>

The analysis was ran with a stiffness modifier of $0.5E_C I_g$ for both C-shaped walls according to suggested values from ACI 318-19 §6.6.3.1.2. The coupling beams used a stiffness modifier equal to $0.14E_C I_g$ as recommended by LATBSDC 2020.

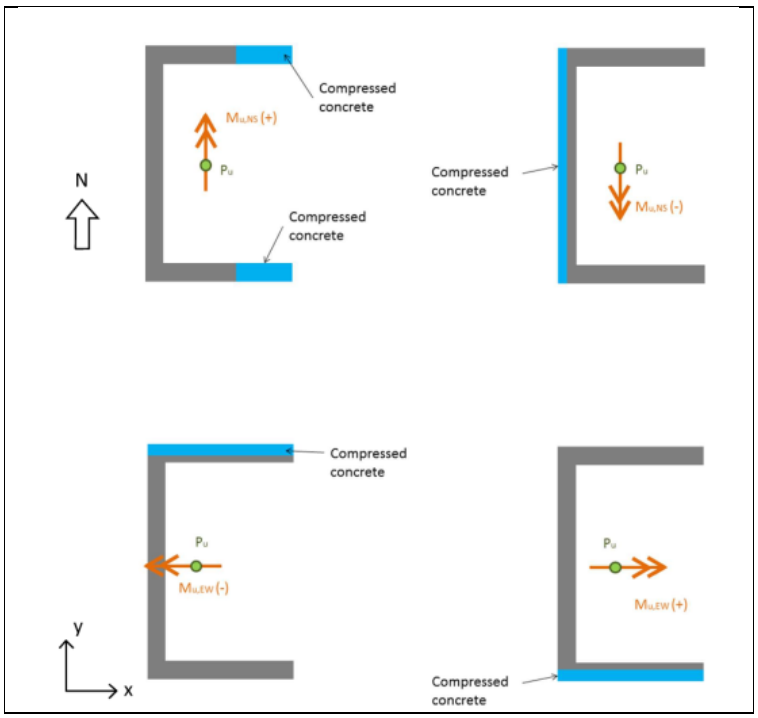


Figure 7 Sign Convention for Moments

The sign convention is shown in Figure 7. The results for the core walls are presented in Table 3 and Table 4 while the results for the coupling beam are shown in Table 5. It is noted that the core wall will have asymmetrical shear distribution due to the torsion in the building. The shear force for the core walls is shown only for the south flange which has the maximum magnitude of shear of the two flanges. Similarly, only the maximum forces are shown for the coupling beam.

Table 3 Analysis Results of West Core Wall Forces for Earthquake in the EW Direction

Load Combination	P_u [Kips]	$M_{u,NS}$ [Kip-ft]	$V_{u,NS}$ [Kips]	δ_u [in]	$\frac{\delta_u}{h_{wcs}}$ -
$0.9D - 0.2S_{DS} + 1.0\,E_{QX}$	6199	27322	898	16.7	0.0076
$0.9D - 0.2S_{DS} - 1.0\,E_{QX}$	-13256	-27460	-898	16.7	0.0076
$1.2D + 0.2S_{DS} + 0.5L + 1.0\,E_{QX}$	1006	27243	891	16.7	0.0076
$1.2D + 0.2S_{DS} + 0.5L - 1.0\,E_{QX}$	-18449	-27540	-891	16.7	0.0076

Table 4 Analysis Results of West Core Wall Forces for Earthquake in the NS Direction

Load Combination	P_u [Kips]	$M_{u,EW}$ [Kip-ft]	$V_{u,EW}$ [Kips]	δ_u [in]	$\frac{\delta_u}{h_{wcs}}$ -
$0.9D - 0.2S_{DS} + 1.0\,E_{QY}$	-3531	-131790	1473	17.1	0.0077
$0.9D - 0.2S_{DS} - 1.0\,E_{QY}$	-3526	130537	-1470	17.1	0.0077
$1.2D + 0.2S_{DS} + 0.5L + 1.0\,E_{QY}$	-8724	-132810	1477	17.1	0.0077
$1.2D + 0.2S_{DS} + 0.5L - 1.0\,E_{QY}$	-8719	129516	-1467	17.1	0.0077

Table 5 Analysis Results of Coupling Beam End Forces

Load Combination	South Coupling Beam Forces at Podium 1		North Coupling Beam Forces at Podium 1	
	M_u [Kip-ft]	V_u [Kips]	M_u [Kip-ft]	V_u [Kips]
$0.9D - 0.2S_{DS} + 1.0\,E_{QY}$	2229	640	1417	408
$1.2D + 0.2S_{DS} + 0.5L + 1.0\,E_{QY}$	2242	649	1430	416

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Core Wall Design (BE Transverse).xlsx

Example Drawings Core wall 1.pdf

Example Report.pdf

Interaction Diagram Example.pdf

Interaction diagram students (ACI318-14).xlsx

Interaction Diagram Theory.pdf (ACI318-14).pdf

SE211 HW5 Core Wall Design Example – Longitudinal Reinforcement and Boundary Elements r1.1.pdf

SE211 HW5 Coupling Beam Design Example r3.0.pdf

SE211 W21 HW5 Appendix r1.5.pdf

SE211 W21 HW5 r3.0.pdf

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Design

Summary

For the flexural design of the core wall, a uniaxial-moment interaction diagram was created for a single core wall using various reinforcement ratios. After selecting a steel ratio for the longitudinal rebars that met the demand, the maximum probable moment capacity of the wall was calculated in order to design the shear reinforcement in the transverse direction. The coupling beam was reinforced with two intersecting groups of diagonally placed bars symmetric about the midspan according the ACI 318-19 §18.10.7. Special boundary elements were required by analysis and were designed according to ACI 318-19 §18.10.6.

Core Wall Pier Design

Flexure Design

The axial moment diagram is formulated using elastic, perfectly-plastic constitutive model for the steel rebar and an elastic model for the concrete up to a compressive crushing strain of -0.003. The core wall was discretized into 5 sections as shown in Figure 8 with the steel rebar lumped at the centroid of each section. The moment interaction diagram is formed by performed a section equilibrium for a given curvature over the entire section. For a given curvature and assuming plane sections remain plane, the strain for the lumped steel can be found which is then related to stress using the elastic, perfectly-plastic constitutive steel model. The stress in the concrete is found by using the equivalent stress block method which equates the stress profile of the concrete in compression with an equivalent stress block modified by β_1 and γ factors which accounts for the equivalent neutral axis height and the equivalent peak stress of the stress block, respectively. β_1 is taken as a maximum of 0.85 at strengths $\leq$ 4 ksi and 0.65 for concrete strength $\geq$ 8 ksi, and linearly interpolated in between. The stresses in the concrete and steel are then converted to a force and equilibrium is found for the internally developed forces for the given curvature to find the applied axial load as well as the moment capacity of the section. A sample calculation is attached. For design, ACI 318-19 includes a flexural capacity factor ϕ that penalizes compression-controlled flexural failures. This is found by considering the strain at the extreme tensile fiber of the concrete and how close it is to the tensile yielding strain of the rebar.

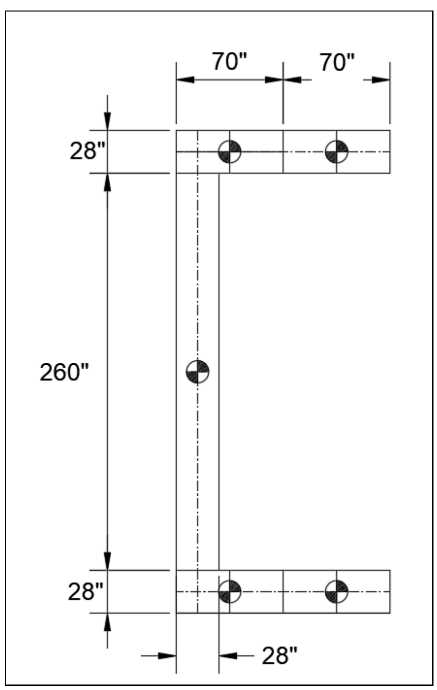


Figure 8 Discretization of Core Wall

PM-Diagrams

The PM diagrams code described above was implement in MATLAB and the code is attached. Rebar ratios were selected for each lump rebar area and a ϕPM diagram was generated using the MATLAB code. Using the simplified lump steel rebar approach above is adequate for design although it does not consider the account strain diagram or actual stress-strain relationship of concrete and steel. From the analysis above, the core wall required a minimum of 1.2% reinforcing ratio in the walls.

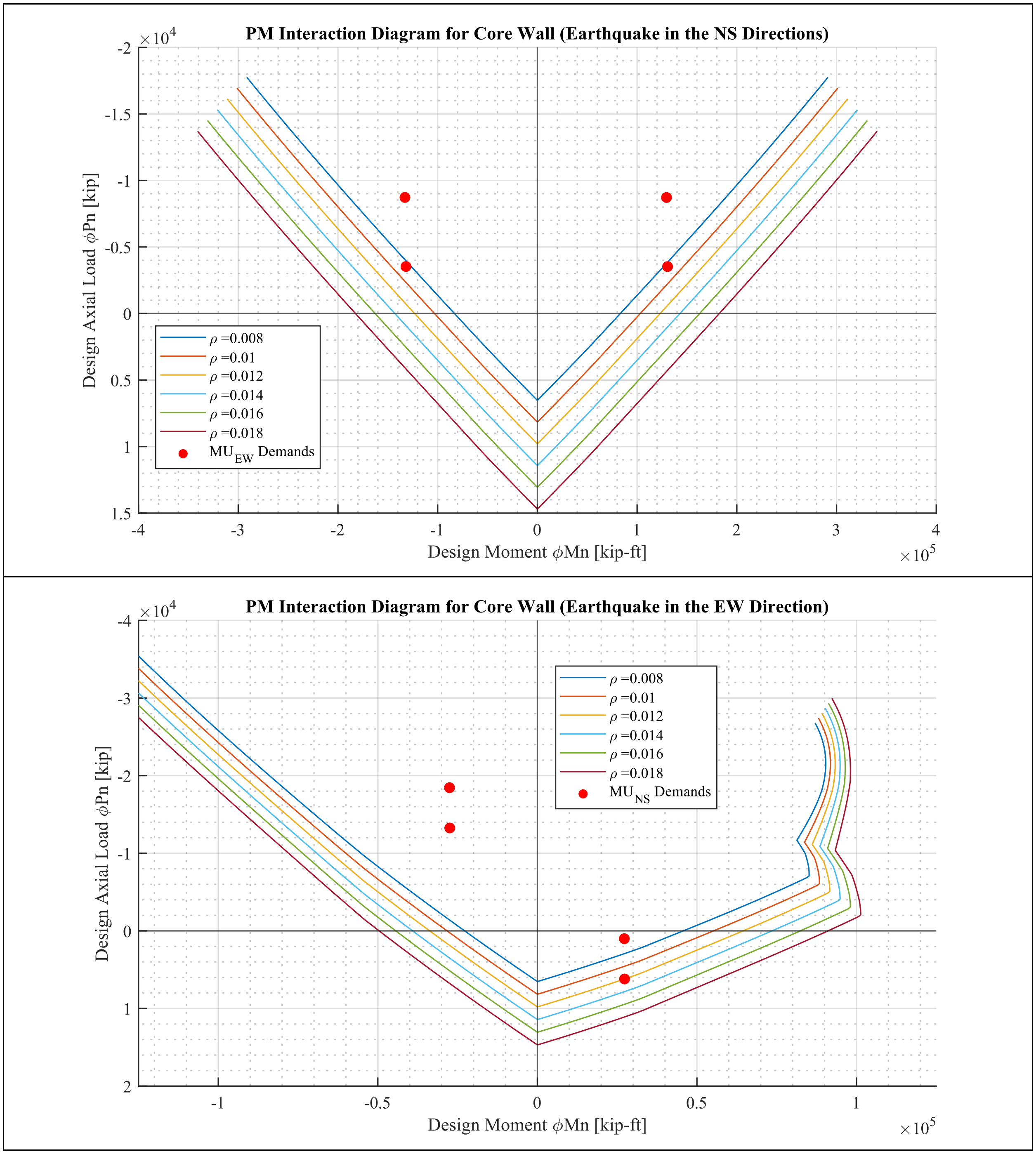


Figure 9 Factored Axial-Moment Diagrams for the Core Wall in Both Directions

Boundary Element

The special boundary element is required per ACI 318-19 §18.10.6.2:

$$1.5\frac{\delta_u}{h_{wcs}} \geq \frac{l_w}{600c}$$

where c is the largest neutral axis depth for the axial-force and nominal moment strength curve consistent with the direction of displacement δ_u. The ratio $\frac{\delta_u}{h_w}$ shall not be taken less than 0.005 per ACI 318-19 §18.10.6.2(a) and were provided. Thus, the requirement of the special boundary element was necessary when the neutral axis in the core wall exceed the limits found below.

$$c_{NS} = 45.60"$$

$$c_{EW} = 20.47"$$

The maximum neutral axis for the earthquake motion in the NS and the EW direction were found to be

$$c_{NS} = 27.45"$$

$$c_{EW} = 11.25"$$

Although it was unnecessary to provide a special boundary element, the walls were designed to include a minimum boundary element equal to 15% the length of the wall as given by ACI 318-19 §18.10.2.4(a). The final dimensions of the boundary elements were 28"x28" with 1½" covers. 16 #9 bars were used for longitudinal reinforcement, for a steel ratio of 2.04%. 1 #5 rebar was used as the perimeter transverse reinforcement and 3 #4 rebar, hooked at 135°, were placed vertically and horizontally for the remaining transverse reinforcement.

Additionally, even though the corners of the core wall did not require boundary elements, under bi-directional seismic loading, results from nonlinear analysis may show that the wall may require it. Thus, a corner element, detailed as a boundary element was specified with dimensions of 51 $\frac{7}{8}$"x28" with 1½" covers, located at both ends of the NS web of the core wall. There were 32 #9 bars spaced at 4" on center in the corner element with 1 #5 perimeter transverse reinforcement, 11 #4 rebar for ties in the horizontal direction and 2 #4 rebars ties in the vertical direction. The rebars were placed in 2 main curtains with 2 intermediary curtains each with 4 rebar.

Detailing of Longitudinal Reinforcement

The longitudinal reinforcement was distributed by first accounting for the reinforcement necessary in the special boundary elements and corner element. After the necessary longitudinal bars were placed, the remaining number of bars to fulfill the required steel ratio was found and then the required spacing was determined.

According to the preliminary design, 1.25% steel ratio was required, which for the wall area of 15,150 in^2, meant an area of 189 in^2 needed to be provided in steel. The area in the two boundary element and two corner elements provided 100 in^2 of steel reinforcement, thus on 89 in^2 needed to be divided among the rest of the wall. Using #9 bars, this would require 89 rebars. Using two curtains of #9 with 8" spacing, the rebars used outside of the corner and boundary elements totaled 92, just a little bit over the required 89 rebar. This gave a final steel ratio of 1.27% in the entire wall.

Shear Design of the Critical Region

The shear reinforcement in the boundary element used the greater of Table §18.10.6.4(g) equations (a) and (b) for the rectilinear hoop. This gave a required steel area of 1.086 in^2 for both directions. 3 #4 rebars and 1 #5 perimeter rebar were provided for a steel area of 1.22 in^2. In the corner element, the same equations were used and the required area in the x direction was 2.063 in^2 and 0.988 in^2 in y direction. 11 #4 rebars were used and 1#5 perimeter rebar for the x direction for a provided area of 2.82 in^2. 2 #4 rebars were used and 1#5 perimeter rebar for the y direction for a provided area of 1.02 in^2. Spacing of all transverse reinforcement was no more than 6 inches as checked per §18.7.5.3.

Shear design of the core wall

The expected shear demand was calculated per §18.10.3. This section is important since the maximum moment developed by the wall under extreme loads may be greater than the designed for strength. The shear in wall needs to be designed to consider the overstrength factors as well as dynamic amplification factors, but no more than 3 times the shear capacity of the wall as the wall may become shear dominated. The expected shear capacity was calculated per §18.10.4. In the design 4 #5 rebars at 4" spacing were specified.

Other Transverse Reinforcement

The longitudinal reinforcement detailed away from the boundary element has to be braced against premature buckling.

According to ACI 318-19 §18.7.5.3, spacing of transverse reinforcement of boundary elements shall not exceed the least of three below:

i) One fourth of the minimum column dimension = 7 in.

ii) For Grade 60, 6db of the smallest longitudinal bar = 6.768 in

iii) $S_o = 4 + \left(\frac{14-h_x}{3}\right)$ = 6.417 in (§18.7.5.3)

The calculated minimum spacing from the three conditions is 6 in. And the designed spacing of transverse reinforcement of boundary element is 4 in. Therefore, the required spacing of the transverse reinforcement is satisfied.

According to ACI 318-19 §11.7.2.4, spacing of transverse reinforcement in cast-in-place walls shall not exceed the lesser of 3h and 18 in., where h is the wall thickness.

i) 3 x h = 3 x 28 in = 84 in

ii) 18 in

The calculated minimum spacing for the two conditions is 18 in. And the designed spacing of transverse reinforcement of the core wall is 4 in for the shear reinforcement and 8 in for the ties. Therefore, the required spacing of the transverse reinforcement is satisfied.

Coupling Beam Design

Structural walls coupled by coupling beams provide excellent lateral forces especially in case of tall building. Coupling beams reduce lateral deflections and inter-story drifts, especially in the upper story levels of tall buildings.

The core wall of this building is coupled by coupling beams at every story. The dimensions of the coupling beams:

Height (h) = 42 in, Base width (bw) = 28 in, Length (ln) = 7 ft

The applied force:

North coupling beam: $V_n = 416\ kips,\ \ Mu = 1430\ kip * ft$

South coupling beam: $V_n = 649\ kips,\ \ Mu = 2242\ kip * ft$

<u>Preliminary checks</u>

According to ACI 318-19 §18.10.7.2, coupling beams with $\left(\frac{l_n}{h}\right) < \ 2$ and with $V_u\ \geq 4\,\lambda\,\sqrt{f_c'}A_{cw}$ needed to be reinforced with two intersecting groups of diagonally placed bars symmetric about the midspan.

Both north and south coupling beams falls into this category and needs to be diagonally reinforced.

<u>Diagonal Reinforcement</u>

The diagonal reinforcement resists the shear force, and it is calculated by:

$\phi V_n \geq Vu,\ \ \phi = 0.85$

$V_n = 2\,A_{vd} f_y sin\alpha \leq 10\,\sqrt{f_c'}A_{cw}$ (§18.10.7.2)

α is the angle between the diagonal bars and the longitudinal axis of the coupling beam.

Diagonal reinforcement was designed by using 10 #11 for the north coupling beams and 12 #11 + 4 #9 for the south coupling beams.

<u>Transverse Reinforcement</u>

Transverse reinforcement shall be provided for the entire beam cross section. The reinforcement should be satisfying $A_{sh,required} < A_{sh,provided}$

The maximum reinforcement ($A_{sh,required}$) is calculated by the maximum of following two equations.

i) $0.09\ s\ b_c \frac{f_c'}{f_{yt}}$ (§18.10.7.4)

ii) $0.3\ s\ b_c\ \left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}}$ (§18.10.7.4)

North coupling beam provided 1 #5 stirrup and 3 #5 ties for horizontal reinforcement and provided 1 #5 stirrup and 5 #5 ties for vertical reinforcement. And south coupling beam provided the same amount of transverse reinforcement as the north.

<u>Development Length</u>

The development length of diagonal bars is required as the maximum of the three followings.

(a) $\left(\frac{f_y\psi_e\psi_p\psi_o\psi_c}{75\sqrt{f_c'}}\right)d_b^{1.5}$ (§25.4.2.3)

(b) 8 db

(c) 6 in

The required development length is 82.157 in for both north and south coupling beams.

Longitudinal Reinforcement

#4 bars are used for the longitudinal reinforcement and it is extended 6 in. into the walls. Reinforcement other than diagonal bars shall not be extended more than 6 in. because excessive longitudinal reinforcement results in increase in bending moment which lead to increase in shear force.

Appendix A Coupling Beam Calculations

Coupling beam parameters

The dimensions of the beam are

Height of beam	$h := 4\ ft$
Width of beam	$b := 2.5\ ft$
Length of beam	$l_n := 8.0\ ft$

The material property of the concrete and rebar are

Yielding strength	$f_y := 60\ ksi$
Compressive strength	$f'_c := 7\ ksi$

The design demands are

Maximum shear	$V_u := 626\ kip$
Maximum moment	$M_u := 2504\ kip \cdot ft$

Preliminary Checks

Following ACI 318-19 provisions, 18.10.7 Coupling Beams, the coupling beams will be diagonally reinforced.

Area of concrete	$A_{cw} := h \cdot b = 1440\ in^2$	
Ratio of length to height of beam	$\dfrac{l_n}{h} = 2$	§(18.10.7.2)
Concrete parameter	$\lambda := 1.0$	
Check that shear capacity is required	$4.0 \cdot \lambda \cdot \sqrt{\dfrac{f'_c}{psi}}\ psi \cdot A_{cw} = 481.916\ kip$	§(18.10.7.2)

Determine the angle of diagonal reinforcement

The initial guess angle of the diagonal reinforcements considers #9 diagonal rebars @8" O.C. and #5 vertical hoops.

First order approximation	$\alpha := \operatorname{atan}\left(\dfrac{h}{l_n}\right) = 26.565\ °$
#9 Diagonal bars	$d_b := 1.128\ in$
#5 Vertical hoops	$d_{b\_hoop} := 0.625\ in$
Estimated Cover	$d_{cover} := 1.5\ in$
Spacing between diagonal bars	$s := 8\ in$
Total spacing between the two bars	$d_{total} := \left(\dfrac{s - d_b}{2} \cdot \dfrac{1}{\cos(\alpha)}\right) + \left(\dfrac{d_b}{\cos(\alpha)}\right) + d_{b\_hoop} + d_{cover} = 7.228\ in$
Adjusted angle of the rebar	$\alpha := \operatorname{atan}\left(\dfrac{h - 2 \cdot d_{total}}{l_n}\right) = 19.261\ °$

Diagonal Reinforcement Using the Catalogue Method

Shear capacity factor	$\phi := 0.85$	§(21.2.4.4)
Number of diagonal bars	$n_{bars} := 20$	
Total area of reabr	$A_{vd} := n_{bars} \cdot d_b^{\ 2} \cdot \frac{\pi}{4} = 19.987\ in^2$	
Max shear capacity	$V_{n\_max} := 10 \cdot \sqrt{\frac{f'_c}{psi}}\ psi \cdot A_{cw} = 1204.79\ kip$	§(18.10.7.4)
Shear capacity	$V_n := min\left(2 \cdot A_{vd} \cdot f_y \cdot \sin(\alpha), V_{n\_max}\right) = 791.144\ kip$	§(18.10.7.4)
Factored Shear capacity	$\phi \cdot V_n = 672.472\ kip$	
Check	$\phi \cdot V_n > V_u = 1$	§(9.5.1)

Check Angle of Reinforcement

Since there are four layers of reinforcement, the spacing between the first and fourth row is the sum of 2 bar diameters and 3 clear spacing. In the design, the diagonal are #9 bars and the clear spacing is 2". This gives a clear spacing between the first and fourth row as 8.25".

Clear spacing	$d_{clear} := 2\ in$
Total distance	$d_{total} := \left(\frac{2\ d_b + 3\ d_{clear}}{2} \cdot \frac{1}{\cos(\alpha)}\right) + \left(\frac{d_b}{\cos(\alpha)}\right) + d_{b\_hoop} + d_{cover} = 7.693\ in$
True angle	$\alpha := \operatorname{atan}\left(\frac{h - 2 \cdot d_{total}}{l_n}\right) = 18.765\ °$

Development length of the Reinforcement

The development of the diagonal rebars will be straight.

Casting Position (other)	$\psi_t := 1.3$	§(25.4.2.5)
Epoxy Coefficient (Uncoated)	$\psi_e := 1.0$	§(25.4.2.5)
Reinforcement Grade (60)	$\psi_g := 1.0$	§(25.4.2.5)
Normal Weight	$\lambda := 1.0$	§(25.4.2.5)
Straight bar development length (No. 7 and Larger)	$l_d(d_b) := 1.25 \left(\frac{f_y \cdot \psi_t \cdot \psi_e \cdot \psi_g}{20\ \lambda \cdot \sqrt{\frac{f'_c}{psi}}\ psi}\right) \cdot d_b$	§(25.4.2.3)
#9 Rebar Development length	$l_{req} := l_d(1.128\ in) \cdot \cos(\alpha) = 62.232\ in$	
Available Length	$l_{wall} := 8\ ft$	
Check (Fudge length 6" is used)	$l_{wall} - 6\ in \geq l_{req} = 1$	

Hoops and Cross-Ties Confinement

The required vertical and horizontal rebars are calculated

Area of hoops	$A_b := d_{b\_hoop}{}^2 \cdot \dfrac{\pi}{4} = 0.307\ in^2$	
Spacing of diagonals	$s := 3\ in$	
Max spacing	$s_{max} := min\left(6\ in, 6 \cdot d_b\right) = 6\ in$	
Width assuming clear cover	$b_{c1} := b - 2 \cdot d_{cover} = 27\ in$	
Height assuming clear cover	$b_{c2} := h - 2 \cdot d_{cover} = 45\ in$	
Area of confined concrete	$A_{ch} := b_{c1} \cdot b_{c2} = 1215\ in^2$	
Total area of beam	$A_g := b \cdot h = 1440\ in^2$	
Equation 1	$eq\_i(b) := 0.09 \cdot s \cdot b \cdot \left(\dfrac{f'_c}{f_y}\right)$	§(18.10.7.4 (c&d i))
Equation 2	$eq\_ii(b) := 0.3 \cdot s \cdot b \cdot \left(\dfrac{A_g}{A_{ch}} - 1\right) \cdot \left(\dfrac{f'_c}{f_y}\right)$	§(18.10.7.4 (c&d ii))
Required hor. rebar	$A_{sh\_hor\_req} := \max\left(eq\_i\left(b_{c1}\right), eq\_ii\left(b_{c1}\right)\right) = 0.851\ in^2$	
Required ver. rebar	$A_{sh\_ver\_req} := \max\left(eq\_i\left(b_{c2}\right), eq\_ii\left(b_{c2}\right)\right) = 1.418\ in^2$	

Detail Hoops and Cross-Ties Confinement for Entire Beam

There will be one #5 bar that goes all the way around the entire section. This is in addition to #4 provided in the vertical and horizontal direction. The required area of steel reinforcement was $A_{sh\_hor\_req} = 0.851\ in^2$ in the horizontal direction and $A_{sh\_ver\_req} = 1.418\ in^2$ in the vertical direction.

(3)#4 bars will be provided in the hor. direction and (5)#4 bars will be provided in the vert. direction.

Number of bars in the hor. dir.	$n_{hor} := 3$
Area of #4 Bars	$A_{b\#4} := 0.2\ in^2$
Area of #5 Bars	$A_{b\#5} := 0.31\ in^2$
Rebar area provided horizontally	$A_{sh\_hor} := n_{hor} \cdot A_{b\#4} + 2\ A_{b\#5} = 1.22\ in^2$
Check	$A_{sh\_hor} > A_{sh\_hor\_req} = 1$
Number of bars in the hor. dir.	$n_{ver} := 5$
Rebar area provided vertically	$A_{sh\_ver} := n_{ver} \cdot A_{b\#4} + 2\ A_{b\#5} = 1.62\ in^2$
Check	$A_{sh\_ver} > A_{sh\_ver\_req} = 1$

Coupling Beam Design [ACI 318-19] Kyoungyeon Lee

-North Coupling Beam-

Parameters	**Applied Forces**	**Material Properties**
$h := 42\ in$	$V_u := 416\ kip$	$f_y := 60\ ksi$
$b_w := 28\ in$	$M_u := 1430\ kip \cdot ft$	$f'_c := 7\ ksi$
$l_n := 7\ ft$		$\lambda := 1.0$ (normal weight)

Step 1 - Preliminary checks

$A_{cw} := h \cdot b_w = \left(1.176 \cdot 10^3\right)\ in^2$

$\dfrac{l_n}{h} = 2$ §(18.10.7.2)

$4.0 \cdot \lambda \cdot \sqrt{\dfrac{f'_c}{psi}}\ psi \cdot A_{cw} = 393.565\ kip$ §(18.10.7.2)

18.10.7.2 Coupling beams with $(\ell_n/h) < 2$ and with $V_u \geq 4\lambda\sqrt{f'_c}A_{cw}$ shall be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan, unless it can be shown that loss of stiffness and strength of the coupling beams will not impair the vertical load-carrying ability of the structure, the egress from the structure, or the integrity of nonstructural components and their connections to the structure.

Therefore, the coupling beam needs to be reinforced with two intersecting groups of diagonally placed bars symmetrical about the midspan.

Step 2 - Determine the angle of the diagonal reinforcement to the horizontal

First order approximation $\alpha_{approx} := \operatorname{atan}\left(\dfrac{h}{l_n}\right) = 26.565\ °$

Exposure: Not exposed to weather or in contact with the ground $d_{cover} := 1.5\ in$ §(20.5.1.3.1)

Diagonal reinforcement: #11 $d_b := 1.41\ in$

Hoop: #5 $d_{b\_hoop} := 0.625\ in$

distance from cover to the centerline of diagonal reinforcement

$$cover_{center} := \frac{\left(8\ in - d_b\right)}{2 \cdot \cos\left(\alpha_{approx}\right)} + \frac{d_b}{\cos\left(\alpha_{approx}\right)} + d_{b\_hoop} + d_{cover} = 7.385\ in$$

$$\alpha := \operatorname{atan}\left(\frac{\left(h - 2 \cdot cover_{center}\right)}{l_n}\right) = 17.961\ °$$

Step 3 - Diagonal reinforcement using the Catalogue Method

$\phi := 0.85$ §(21.2.4.4)

$V_{n\_max} := 10 \cdot \sqrt{\frac{f'_c}{psi}}\ psi \cdot A_{cw} = 983.912\ kip$ §(18.10.7.4)

$n_{bars} := 10$

$A_{vd} := n_{bars} \cdot {d_b}^2 \cdot \frac{\pi}{4} = 15.615\ in^2$

$V_n := min\left(2 \cdot A_{vd} \cdot f_y \cdot \sin(\alpha), V_{n\_max}\right) = 577.79\ kip$ §(18.10.7.4)

$\phi \cdot V_n = 491.121\ kip$

$\phi \cdot V_n > V_u = 1$ §(9.5.1)

Using 10 #11 bars,

$V_u = 416\ kip$ $\phi \cdot V_n = 491.121\ kip$

$\phi \cdot V_n > V_u = 1$ Therefore, OK.

Step 4 - Check Angle α

$d_{clearspacing} := 2\ in$

$n_{layers} := 3$

$d_{clear} := \left(n_{layers} - 2\right) \cdot d_b + \left(n_{layers} - 1\right) \cdot d_{clearspacing} = 5.41\ in$

$d_{total} := \left(\frac{d_{clear}}{2} \cdot \frac{1}{\cos(\alpha)}\right) + \left(\frac{d_b}{\cos(\alpha)}\right) + d_{b\_hoop} + d_{cover} = 6.451\ in$

$\alpha_{check} := \operatorname{atan}\left(\frac{h - 2 \cdot d_{total}}{l_n}\right) = 19.107\ °$

The difference between the initial guess α and α_{check} is less than 2.
Therefore, OK.

Step 5 - Development of the Reinforcement

Straight bar development.

$\lambda = 1$ (normal weight) §(25.4.2.5)

$\psi_t := 1.3$ (over 12" of fresh concrete)

$\psi_e := 1.0$ (uncoated)

$\psi_g := 1.0$ (Grade 60)

$$l_d\left(d_{b\_input}\right) := 1.25 \cdot \left(\frac{f_y \cdot \psi_t \cdot \psi_e \cdot \psi_g}{20 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}}\ psi} \right) \cdot d_{b\_input}$$

§(25.4.2.3)

§(18.10.2.5)

Using #11 bars, the required development length is

$l_{req} := l_d\left(d_b\right) = 82.157\ in$

Check if the straight bars developed into the wall piers fit.

$l_{req} \cdot \cos(\alpha) = 78.153\ in$

$l_{wall} := 7.5\ ft$

$l_{wall} - 6\ in = 84\ in$ 6 inches: to ensure bars do not encroach too close to the wall pier's edge.

$l_{wall} - 6\ in \geq l_{req} \cdot \cos(\alpha) = 1$

Therefore, OK.

Diagonal reinforcement can be developed into the wall piers with straight bars. Headed reinforcement is not needed.

Step 6 - Hoops and Cross-Ties Confinement

Using #5 for hoop with 4 in spacing,

$A_b := {d_{b\_hoop}}^2 \cdot \frac{\pi}{4} = 0.307\ in^2$

$s := 4\ in$ (spacing)

Check maximum spacing:

$s_{max} := min\left(6\ in, 6 \cdot d_b\right) = 6\ in$

Calculate required reinforcement

$b_{c1} := b_w - 2 \cdot d_{cover} = 25\ in$ (width)

$b_{c2} := h - 2 \cdot d_{cover} = 39\ in$ (height)

$A_{ch} := b_{c1} \cdot b_{c2} = 975\ in^2$ (area of confined concrete) §(18.10.6.4)

$A_g := b_w \cdot h = \left(1.176 \cdot 10^3\right)\ in^2$ (total area of beam)

$eq\_i\left(b_{input}\right) := 0.09 \cdot s \cdot b_{input} \cdot \left(\frac{f'_c}{f_y}\right)$

$eq\_ii\left(b_{input}\right) := 0.3 \cdot s \cdot b_{input} \cdot \left(\frac{A_g}{A_{ch}} - 1\right) \cdot \left(\frac{f'_c}{f_y}\right)$

Required Reinforcement for Horizontal Ties

$A_{sh\_hor\_req} := \max\left(eq\_i\left(b_{c1}\right), eq\_ii\left(b_{c1}\right)\right) = 1.05\ in^2$ §(18.10.7.4 (d))

Required Reinforcement for Vertical Ties

$A_{sh\_ver\_req} := \max\left(eq\_i\left(b_{c2}\right), eq\_ii\left(b_{c2}\right)\right) = 1.638\ in^2$ §(18.10.7.4 (d))

Step 7 - Detail Hoops and Cross-Ties Confinement for Entire Beam

$A_{b\#4} := 0.2\ in^2$

$A_{b\#5} := 0.31\ in^2$

Required Reinforcement for Horizontal Ties

$A_{sh\_hor\_req} = 1.05\ in^2$

Provide : 3 - #5 Horizontal ties and 1 - #5 ext. stirrup = 5 legs

$n_{hor} := 3$

$A_{sh\_hor} := n_{hor} \cdot A_{b\#5} + 2 \cdot A_{b\#5} = 1.55\ in^2$

$A_{sh\_hor} > A_{sh\_hor\_req} = 1$ Therefore, OK.

Required Reinforcement for Vertical Ties

$A_{sh\_ver\_req} = 1.638\ in^2$

Provide : 5 - #5 Vertical ties and 1 - #5 ext. stirrup = 7 legs

$n_{ver} := 5$

$A_{sh\_ver} := n_{ver} \cdot A_{b\#5} + 2 \cdot A_{b\#5} = 2.17\ in^2$

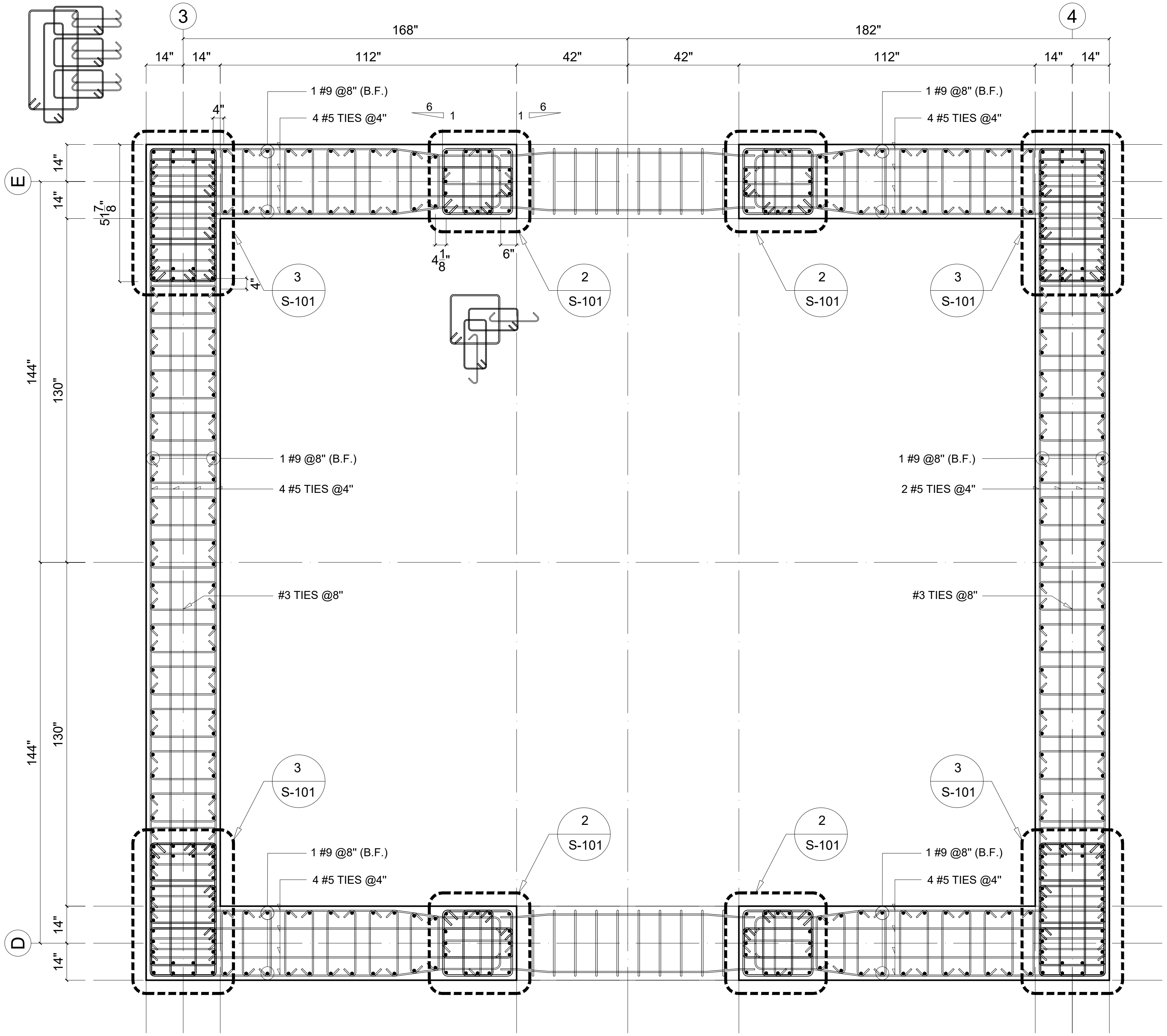
$A_{sh\_ver} > A_{sh\_ver\_req} = 1$ Therefore, OK.

Transverse reinforcement spacing shall not exceed 8" in either direction §(18.10.7.4 (d))

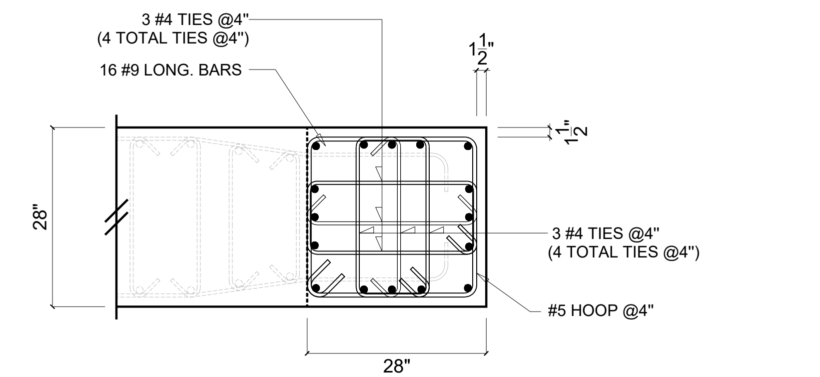
Step 8 - Longitudinal Reinforcement

At least one-fourth of the maximum positive moment reinforcement shall extend along the beam bottom into the support <u>at least 6 in.</u> §(9.7.3.8.2)

Appendix B Drawings



1
S-101
CORE WALL CROSS-SECTION
SCALE: 1 / 50



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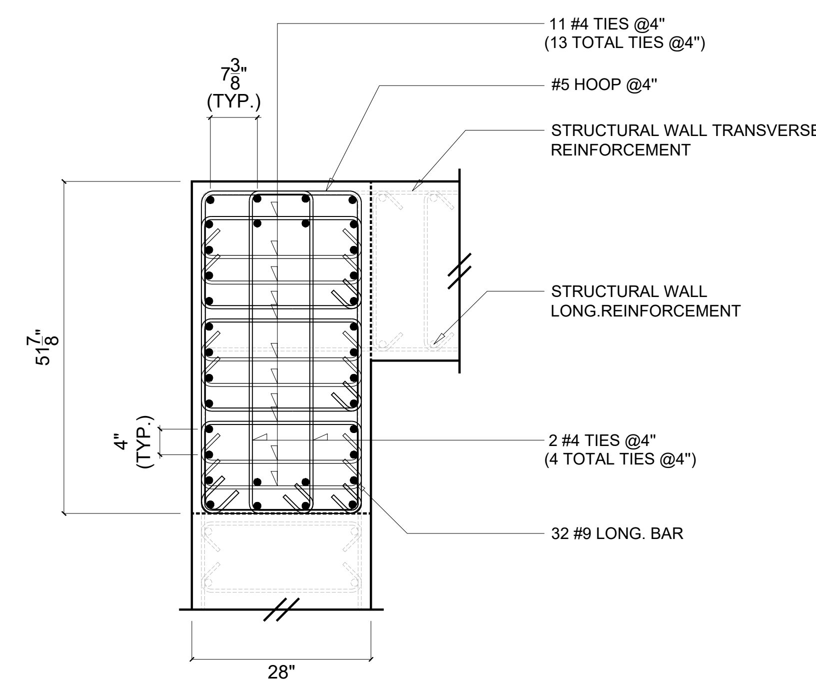
S-101
BOUNDARY ELEMENT
SCALE: 1 / 30



3
S-101
CORNER ELEMENT
SCALE: 1 / 30

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2		
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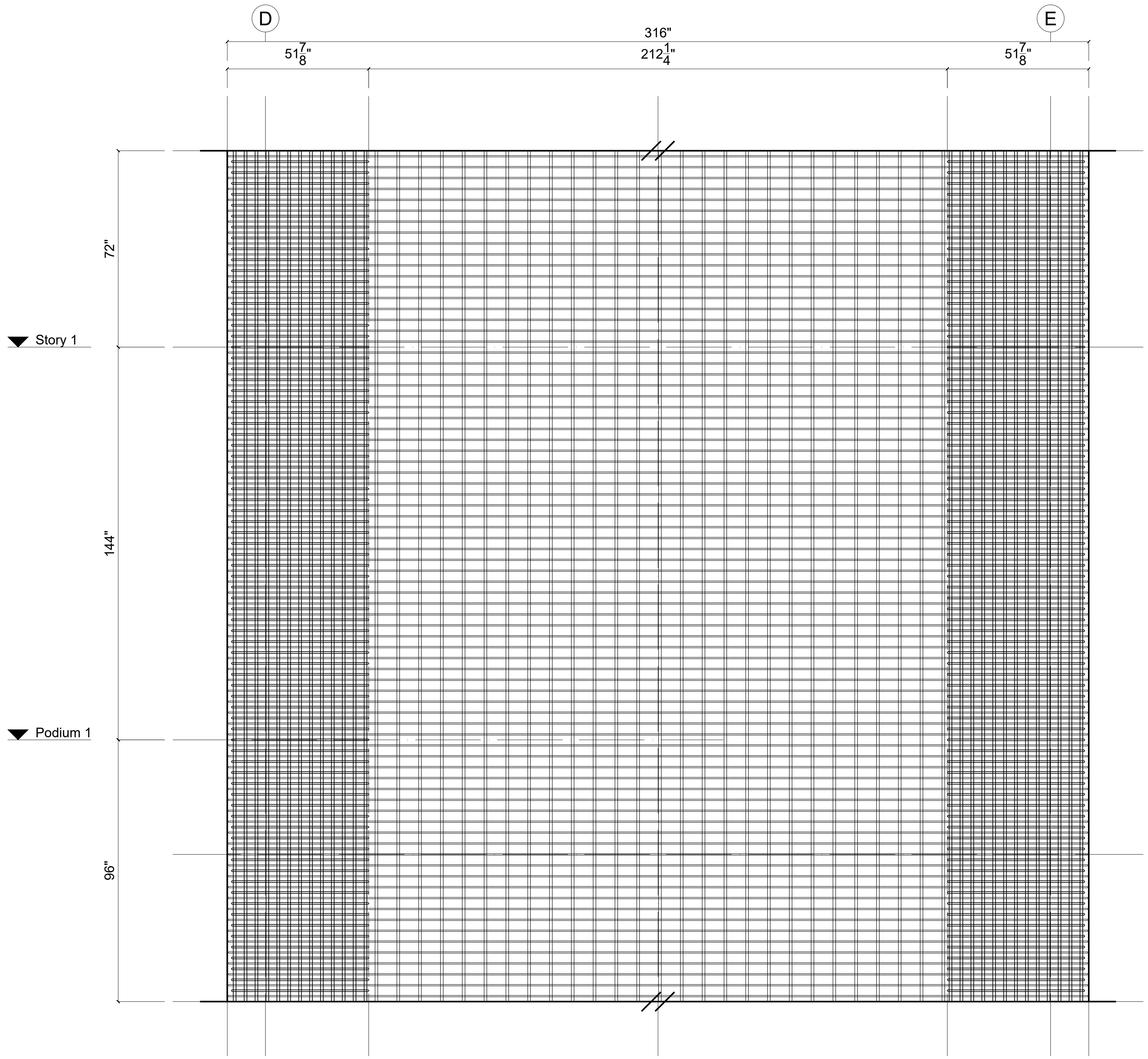
DRAWING TITLE

CORE WALL
CROSS-SECTION

SCALE	1 / 50	DRAWING NO.	
DATE	19 Mar 2021		S-101



1
S-103
CORE WALL EAST ELEVATION
SCALE: 1 / 50

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HOMEWORK #4

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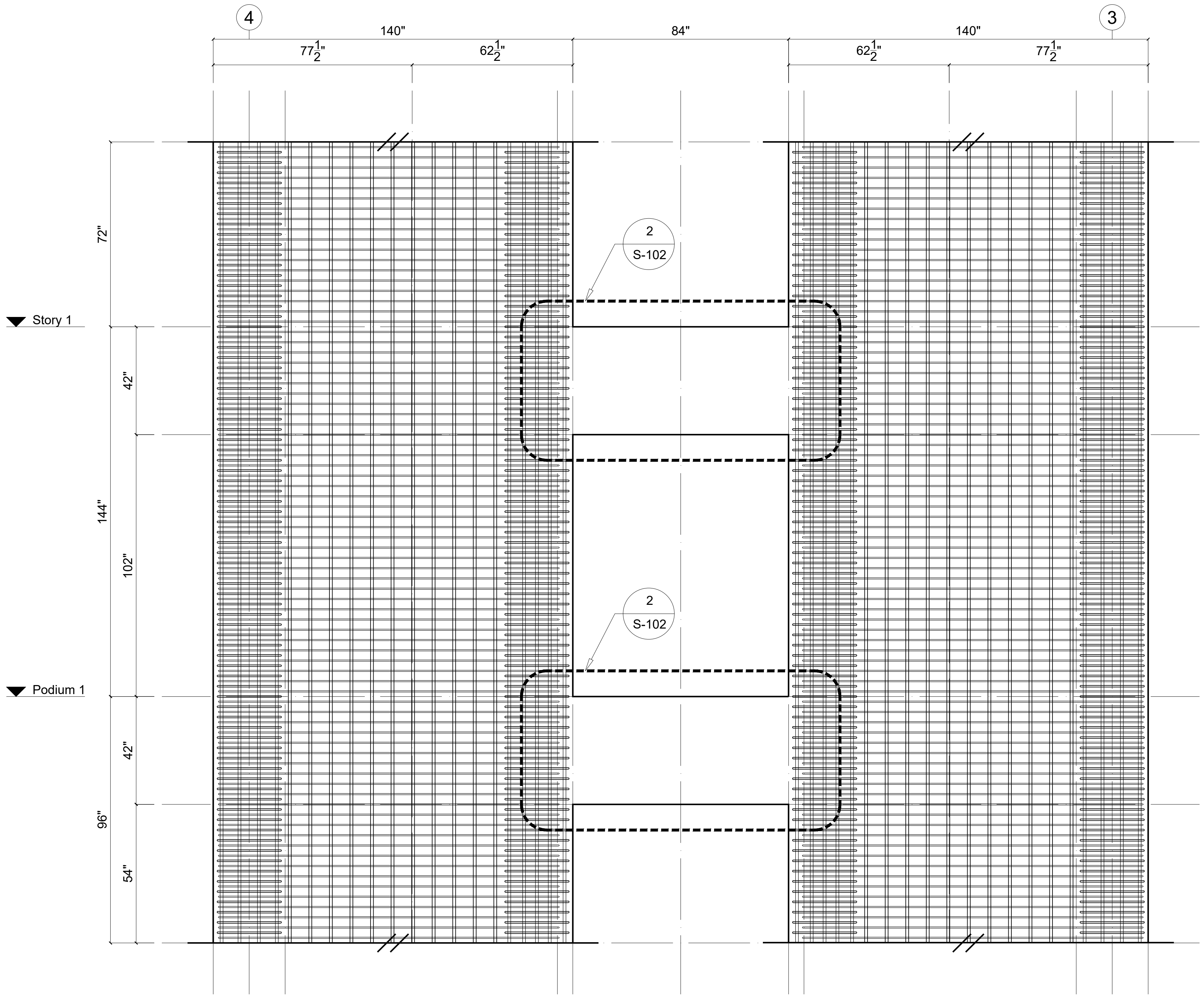
0		

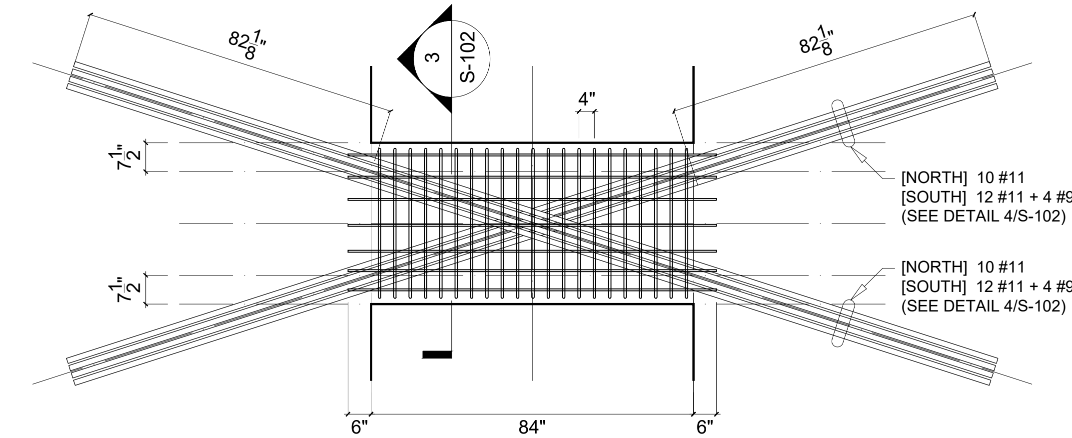
DRAWING TITLE
CORE WALL EAST ELEVATION

SCALE 1 / 50
DATE 19 Mar 2021
DRAWING NO. S-103



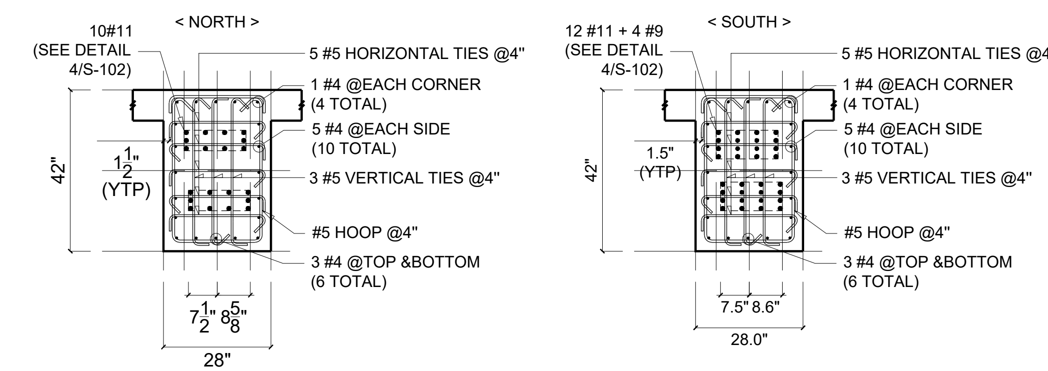
1 S-102 CORE WALL NORTH ELEVATION

SCALE: 1 / 50



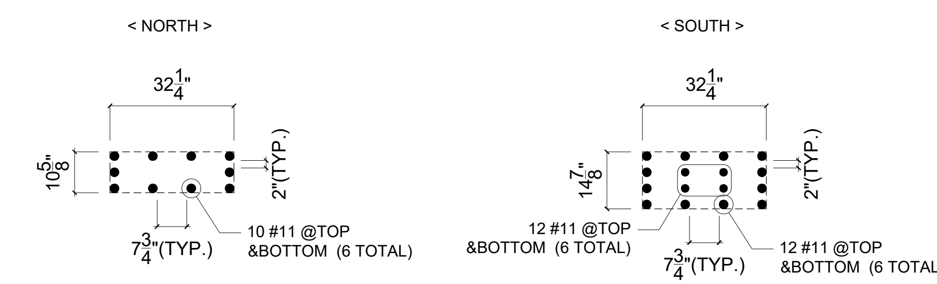
2 S-102 COUPLING BEAM DETAIL

SCALE: 1 / 50



3 S-102 COUPLING BEAM CROSS SECTION

SCALE: 1 / 50



4 S-102 DIAGONAL REINFORCEMENT DETAIL

SCALE: 1 / 50

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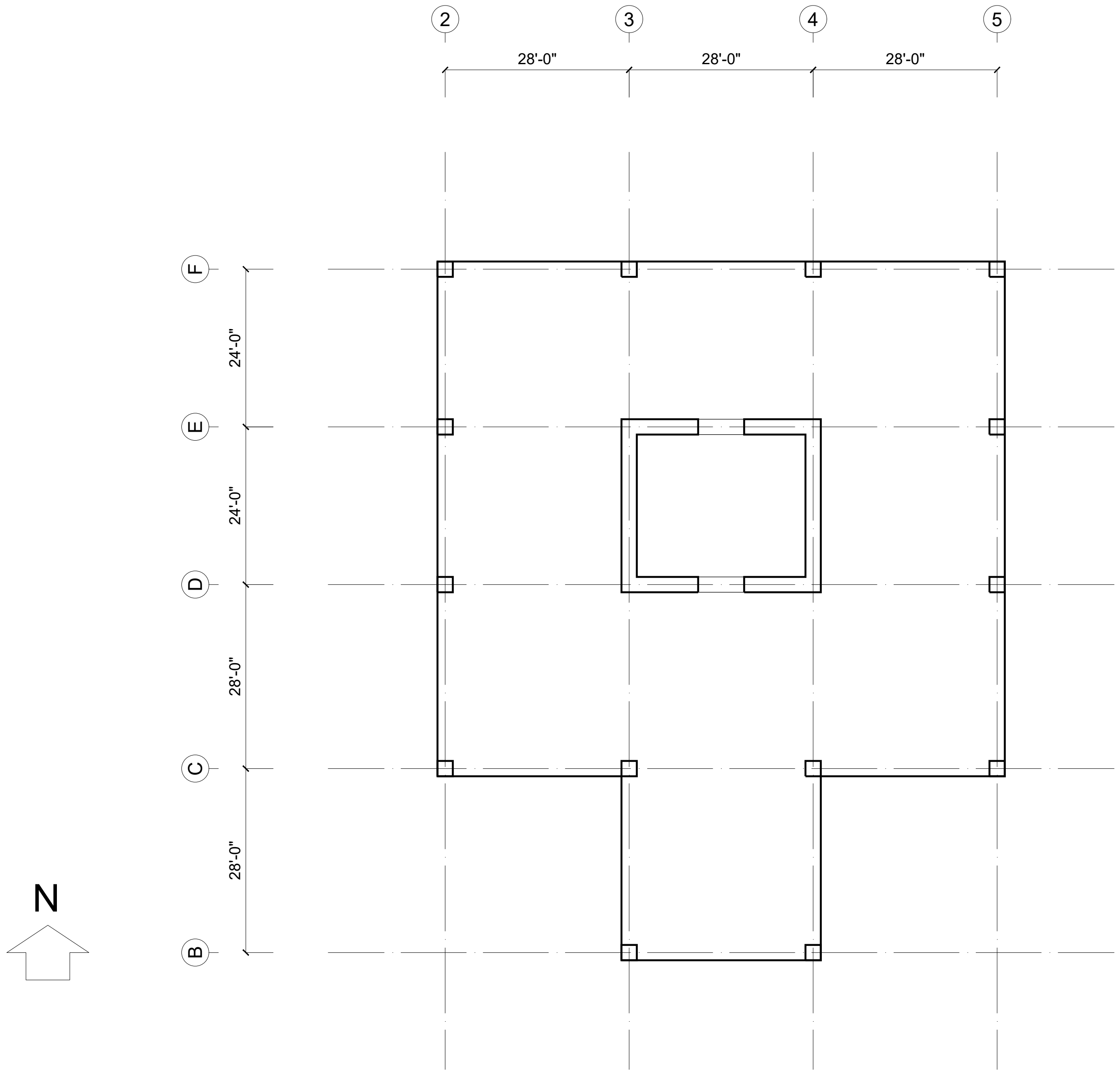
DRAWING TITLE

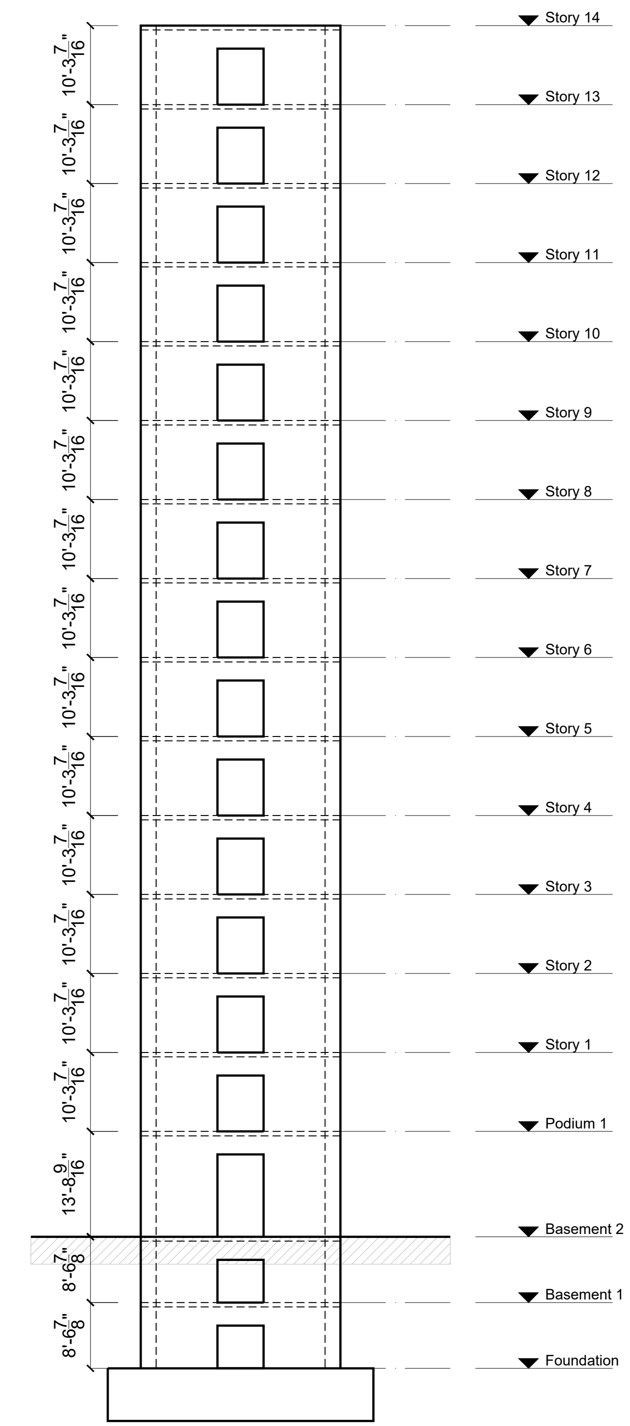
CORE WALL NORTH ELEVATION

SCALE	1 / 50
DATE	19 Mar 2021
DRAWING NO.	S-102



1 S-104 TYPICAL STORY CROSS-SECTION

SCALE: 1 / 300



2 S-104 CORE WALL NORTH ELEVATION

SCALE: 1 / 350

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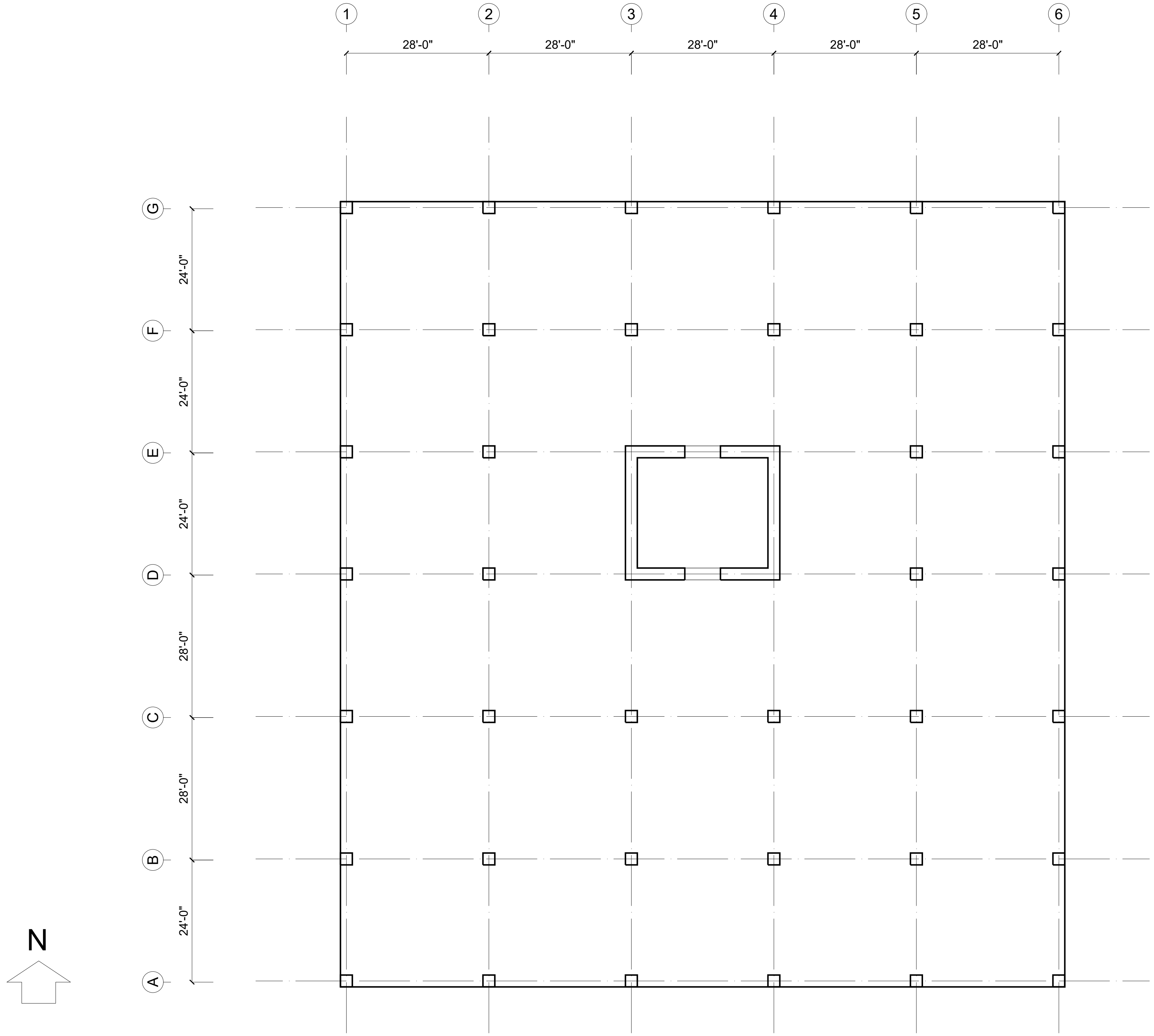
DRAWING TITLE

TYPICAL STORY PLAN & CORE WALL NORTH ELEVATION

SCALE	1 / 300	DRAWING NO.
DATE	19 Mar 2021	S-104



1 S-105 BASEMENT CROSS-SECTION

SCALE: 1 / 300

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CHECK LIST

REV. NO.	ISSUE & DATE	CHECK
1		
2		
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0		
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DRAWING TITLE

BASEMENT PLAN

SCALE	1 / 300	DRAWING NO.
DATE	19 Mar 2021	S-105

Appendix C Calculations

Hazards by Location

Search Information

Address: 550 S Hope St Los Angeles CA 90071 USA

Coordinates: 34.0497206, -118.2556317

Elevation: 288 ft

Timestamp: 2021-03-16T20:39:59.077Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	1.966	MCE_R ground motion (period=0.2s)
S_1	0.7	MCE_R ground motion (period=1.0s)
S_{MS}	1.966	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.311	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.899	Coefficient of risk (0.2s)
CR_1	0.898	Coefficient of risk (1.0s)
PGA	0.842	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.926	Site modified peak ground acceleration

T_L	8	Long-period transition period (s)
SsRT	1.966	Probabilistic risk-targeted ground motion (0.2s)
SsUH	2.187	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.465	Factored deterministic acceleration value (0.2s)
S1RT	0.7	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.779	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.788	Factored deterministic acceleration value (1.0s)
PGAd	0.998	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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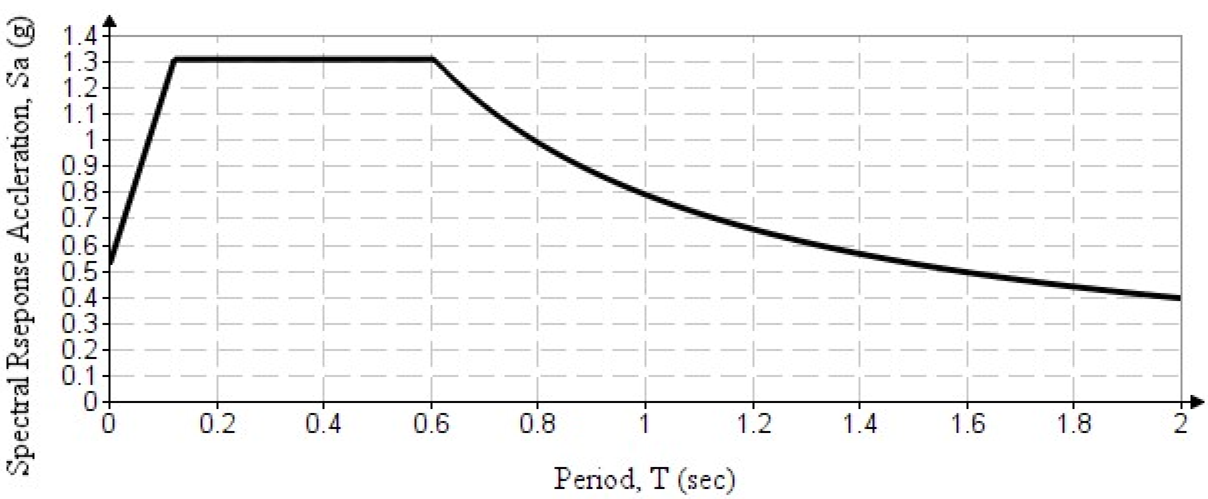
Design Response Spectrum

Spectral 1s response acceleration	$S_1 := 0.700$
Spectral short response acceleration	$S_s := 1.966$
Short period site coefficient; Site Class D	$F_a := 1.0$
Long period site coefficient; Site Class D	$F_v := 1.7$
Adjusted short response acceleration	$S_{MS} := F_a \cdot S_s = 1.966$
Adjusted short response acceleration	$S_{M1} := F_v \cdot S_1 = 1.190$
Design short response acceleration	$S_{DS} := \frac{2}{3} S_{MS} = 1.3107$
Design short response acceleration	$S_{D1} := \frac{2}{3} S_{M1} = 0.7933$
Short response period	$T_s := \frac{S_{D1}}{S_{DS}} = 0.605$
Seismic Response Factor	$T_o := 0.2 \cdot \left(\frac{S_{D1}}{S_{DS}}\right) = 0.121$
Long Period	$T_L := 8$



Sr. No.	Description	Notation	Units	Value
1	Calulation of longitudinal reinforcement at the base			
1.1	Moment in EW Direction, Shear in NS direction			
	Length of each leg of the wall	L_wall	inch	316
	Width of each leg of the wall	t_wall	inch	28
	Area of each leg of the wall	A_wall	sq.inch	8848
	Curtains on reinforcement (for 30" thickness)	curtains	-	2
	Required reinforcement in walls (more than required, less than 3%)	ρl_req	%	1.25
	Notation of bars proposed to be used	#	-	#9
	Bundled bars	#	-	1
	Diameter of bars proposed to be used	db	inch	1.128
	Area of single bar proposed to be used	Adb	sq.inch	1
	Calculated spacing of bars = Adb*curtains/(t_wall*ρl)	s	inch	5.00
	As per §18.10.2.1 - spacing should be less than 18"	s<18"	Check	OK
1.2	Calculation of shear reinforcement at the base in NS			
	Shear Demand	VuNS	kips	1477
	Dynamic amplification*overstrenght factor	Ωv*ωv	-	3
	Shear Demand	VeNS	Kips	4431
	Compressive strength of concrete	f'c	ksi	7
	Reduction factor for light weight concrete (§17.2.6)	λ	-	1
	Shear Strength of concrete = A_wall * λ * √(f'c)	Vnc	Kips	740.3
	If Ve_NS>Vnc, we required ρ_t > 0.0025	Check	-	REQ.
	Shear strength reduction factor	ϕ	-	0.75
	Strength reduced capacity of concrete = α*ϕ*Vnc	α*ϕ*Vnc	Kips	1665.62
	Balance shear force V_{UNS}-α*ϕ*Vnc	ϕ*Vns	Kips	2765.38
	Shear reinforcement required = ϕVns/(ϕ*A_wall*fy)	ρt_req	%	0.0069
	Reinforcement to be provided, check for minimum	ρt_req	%	0.0069
	Proposing 4 layers of 1#5 bars as transverse reinf.			#5
	Layers of reinforcement	Layers	#	4
	Area of the transverse reinforcement	Layers*Abd	sq.inch	1.24
	Spacing required = layers*Abd/(t_wall*ρt_prov)	s	inch	6.00
	For 60ksi, smax = 7db = 6"	s_prov	inch	4
	Area of transverse reinforcement provided	ρt_prov	%	0.011
		DCR		0.63

Transverse Reinforcement Design [ACI 318-19]

-Boundary Element-

$f'_c := 7\ ksi$

$f_y := 60\ ksi$

$cover := 1.5\ in$

$bar\#_{hoop} := 5$

$d_{b\_hoop} := \dfrac{bar\#_{hoop}}{8}\ in = 0.625\ in$

$A_{s\_hoop} := \pi \cdot \dfrac{{d_{b\_hoop}}^2}{4} = 0.307\ in^2$

$l_x := 28\ in$

$l_y := 28\ in$

$b_{cx} := l_x - cover - 2 \cdot d_{b\_hoop} = 25.25\ in$

$b_{cy} := l_y - 2 \cdot cover - 2 \cdot d_{b\_hoop} = 23.75\ in$

$h_x := 6.75\ in$

$s_a := \dfrac{min\left(l_x, l_y\right)}{4} = 7\ in$

$s_b := 6 \cdot 1.128\ in = 6.768\ in$ (Grade 60)

$s_c := 4\ in + \left(\dfrac{14 - \dfrac{h_x}{in}}{3}\right) in = 6.417\ in$

$s_{max} := min\left(s_a, s_b, min\left(\max\left(s_c, 4\ in\right), 6\ in\right)\right) = 6\ in$ § (18.7.5.3)

18.7.5.3 Spacing of transverse reinforcement shall not exceed the least of (a) through (d):

(a) One-fourth of the minimum column dimension
(b) For Grade 60, $6d_b$ of the smallest longitudinal bar
(c) For Grade 80, $5d_b$ of the smallest longitudinal bar
(d) s_o, as calculated by:

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) \qquad (18.7.5.3)$$

The value of s_o from Eq. (18.7.5.3) shall not exceed 6 in. and need not be taken less than 4 in.

Calculate required reinforcement

$s := 4\ in$

$A_{ch} := b_{cx} \cdot b_{cy} = 599.688\ in^2$

$A_g := l_x \cdot l_y = 784\ in^2$

$eq\_i\left(b_{input}\right) := 0.09 \cdot s \cdot b_{input} \cdot \left(\dfrac{f'_c}{f_y}\right)$ §(18.10.6.4)

$eq\_ii\left(b_{input}\right) := 0.3 \cdot s \cdot b_{input} \cdot \left(\dfrac{A_g}{A_{ch}} - 1\right) \cdot \left(\dfrac{f'_c}{f_y}\right)$ §(18.10.6.4)

Required Reinforcement for local x direction Ties

$A_{sh\_x\_req} := \max\left(eq\_i\left(b_{cx}\right), eq\_ii\left(b_{cx}\right)\right) = 1.086\ in^2$ §(18.10.7.4 (d))

Required Reinforcement for local y direction Ties

$A_{sh\_y\_req} := \max\left(eq\_i\left(b_{cy}\right), eq\_ii\left(b_{cy}\right)\right) = 1.022\ in^2$ §(18.10.7.4 (d))

Table 18.10.6.4(g)—Transverse reinforcement for special boundary elements

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	Greater of	$0.3\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f'_c}{f_{yt}}$	(a)
		$0.09\dfrac{f'_c}{f_{yt}}$	(b)
ρ_s for spiral or circular hoop	Greater of	$0.45\left(\dfrac{A_g}{A_{ch}}-1\right)\dfrac{f'_c}{f_{yt}}$	(c)
		$0.12\dfrac{f'_c}{f_{yt}}$	(d)

Detail Hoops and Cross-Ties

$A_{b\#4} := 0.2\ in^2$

$A_{b\#5} := 0.31\ in^2$

Required Reinforcement for local x direction Ties

$A_{sh\_x\_req} = 1.086\ in^2$

Provide : 3 - #4 Horizontal ties and 1 - #5 ext. stirrup = 5 legs

$n_x := 3$

$A_{sh\_x} := n_x \cdot A_{b\#4} + 2 \cdot A_{b\#5} = 1.22\ in^2$

$A_{sh\_x} > A_{sh\_x\_req} = 1$ Therefore, OK.

Required Reinforcement for local y direction Ties

$A_{sh\_y\_req} = 1.022\ in^2$

Provide : 3 - #4 Vertical ties and 1 - #5 ext. stirrup = 5 legs

$n_y := 3$

$A_{sh\_y} := n_y \cdot A_{b\#4} + 2 \cdot A_{b\#5} = 1.22\ in^2$

$A_{sh\_y} > A_{sh\_y\_req} = 1$ Therefore, OK.

Transverse reinforcement spacing shall not exceed 8" in either direction §(18.10.7.4 (d))

Transverse Reinforcement Design [ACI 318-19]
-Corner Element-

$f'_c := 7\ ksi$

$f_y := 60\ ksi$

$cover := 1.5\ in$

$bar\#_{hoop} := 5$

$d_{b\_hoop} := \frac{bar\#_{hoop}}{8}\ in = 0.625\ in$

$A_{s\_hoop} := \pi \cdot \frac{{d_{b\_hoop}}^2}{4} = 0.307\ in^2$

$l_x := 51.878\ in$

$l_y := 28\ in$

$b_{cx} := l_x - cover - 2 \cdot d_{b\_hoop} = 49.128\ in$

$b_{cy} := l_y - 2 \cdot cover - 2 \cdot d_{b\_hoop} = 23.75\ in$

$h_x := 7.54\ in$

$s_a := \frac{min\left(l_x, l_y\right)}{4} = 7\ in$

$s_b := 6 \cdot 1.128\ in = 6.768\ in$ (Grade 60)

$s_c := 4\ in + \left(\frac{14 - \frac{h_x}{in}}{3}\right) in = 6.153\ in$

18.7.5.3 Spacing of transverse reinforcement shall not exceed the least of (a) through (d):

(a) One-fourth of the minimum column dimension
(b) For Grade 60, $6d_b$ of the smallest longitudinal bar
(c) For Grade 80, $5d_b$ of the smallest longitudinal bar
(d) s_o, as calculated by:

$$s_o = 4 + \left(\frac{14 - h_x}{3}\right) \quad (18.7.5.3)$$

The value of s_o from Eq. (18.7.5.3) shall not exceed 6 in. and need not be taken less than 4 in.

$s_{max} := min\left(s_a, s_b, min\left(\max\left(s_c, 4\ in\right), 6\ in\right)\right) = 6\ in$ § (18.7.5.3)

Calculate required reinforcement

$s := 4\ in$

$A_{ch} := b_{cx} \cdot b_{cy} = (1.167 \cdot 10^3)\ in^2$

$A_g := l_x \cdot l_y = (1.453 \cdot 10^3)\ in^2$

$eq\_i(b_{input}) := 0.09 \cdot s \cdot b_{input} \cdot \left(\frac{f'_c}{f_y}\right)$ §(18.10.6.4)

$eq\_ii(b_{input}) := 0.3 \cdot s \cdot b_{input} \cdot \left(\frac{A_g}{A_{ch}} - 1\right) \cdot \left(\frac{f'_c}{f_y}\right)$ §(18.10.6.4)

Required Reinforcement for local x direction Ties

$A_{sh\_x\_req} := \max(eq\_i(b_{cx}), eq\_ii(b_{cx})) = 2.063\ in^2$ §(18.10.7.4 (d))

Required Reinforcement for local y direction Ties

$A_{sh\_y\_req} := \max(eq\_i(b_{cy}), eq\_ii(b_{cy})) = 0.998\ in^2$ §(18.10.7.4 (d))

Table 18.10.6.4(g)—Transverse reinforcement for special boundary elements

Transverse reinforcement	Applicable expressions		
A_{sh}/sb_c for rectilinear hoop	Greater of	$0.3\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$	(a)
		$0.09\frac{f'_c}{f_{yt}}$	(b)
ρ_s for spiral or circular hoop	Greater of	$0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$	(c)
		$0.12\frac{f'_c}{f_{yt}}$	(d)

Detail Hoops and Cross-Ties

$A_{b\#4} := 0.2\ in^2$

$A_{b\#5} := 0.31\ in^2$

Required Reinforcement for local x direction Ties

$A_{sh\_x\_req} = 2.063\ in^2$

Provide : 11 - #4 Horizontal ties and 1 - #5 ext. stirrup = 13 legs

$n_x := 11$

$A_{sh\_x} := n_x \cdot A_{b\#4} + 2 \cdot A_{b\#5} = 2.82\ in^2$

$A_{sh\_x} > A_{sh\_x\_req} = 1$ Therefore, OK.

Required Reinforcement for local y direction Ties

$A_{sh\_y\_req} = 0.998\ in^2$

Provide : 2 - #4 Vertical ties and 1 - #5 ext. stirrup = 4 legs

$n_y := 2$

$A_{sh\_y} := n_y \cdot A_{b\#4} + 2 \cdot A_{b\#5} = 1.02\ in^2$

$A_{sh\_y} > A_{sh\_y\_req} = 1$ Therefore, OK.

Transverse reinforcement spacing shall not exceed 8" in either direction §(18.10.7.4 (d))

Steel areas for rebar $A_{s\#5} := 0.31\ in^2$ $A_{s\#8} := 0.79\ in^2$ $A_{s\#9} := 1.0\ in^2$

Diameters for rebar $d_{b\#5} := 0.625\ in$ $d_{b\#9} := 1.00\ in$ $d_{b\#9} := 1.128\ in$

Dimension

Thickness of web 1 $t_1 := 28\ in$

Thickness of web 2 $t_2 := 28\ in$

Thickness of web 3 $t_3 := 28\ in$

Length of web 1 $l_1 := 140\ in$

Length of web 2 $l_2 := 260\ in$

Length of web 3 $l_3 := 140\ in$

Total area of wall $A_g := \sum \overrightarrow{t \cdot l} = 15120\ in^2$

Required steel ratio $\rho := 1.25\%$

Required steel area $A_{s\_req} := \rho \cdot A_g = 189\ in^2$

Boundary Element

Boundary element as previously designed.

Width of the boundary element $L_{BE} := 28\ in$

Height of the boundary element $H_{BE} := 28\ in$

Area of boundary element $A_{c\_BOUNDARY} := L_{BE} \cdot H_{BE} = 784\ in^2$

Reinforcement Area in boundary element $A_{s\_BOUNDARY} := 16 \cdot A_{s\#9}$

Reinforcing ratio $\rho_{BOUNDARY} := \dfrac{A_{s\_BOUNDARY}}{A_{c\_BOUNDARY}} = 2.04\%$

Corner Element

Width of the boundary element $b_{CORNER} := 28\ in$

Height of the boundary element $d_{cover} := 1.5\ in$

Number of curtains $s := 4\ in$

Number of rebars in main curtain $n := 13$

Total number of rebars (+extra) $N_{CORNER} := 2 \cdot n + 8 = 34$

Reinforcement Area in boundary element $A_{s\_CORNER} := N_{CORNER} \cdot A_{s\#9} = 34\ in^2$

Estimating height in curain; taking into account perimeter hoop and spacng

$H_{CORNER} := 2 \cdot d_{b\#5} + d_{b\#9} + (n-1)\ s + d_{cover} = 51.878\ in$

$H_{CORNER} := \text{Round}\left(H_{CORNER}, 1\ in\right) = 52\ in$

Area of boundary element $A_{c\_CORNER} := H_{CORNER} \cdot b_{CORNER} = 1456\ in^2$

Reinforcing ratio $\rho_{BOUNDARY} := \dfrac{A_{s\_CORNER}}{A_{c\_CORNER}} = 2.34\%$

Remaining Rebar

Area left to distbriute $A_{left} := A_{s\_req} - 2 \cdot A_{s\_BOUNDARY} - 2 \cdot A_{s\_CORNER} = 89\ in^2$

Number per bundle and area of each $N_{per\_bundle} := 1 \qquad A_{s\_ea} := A_{s\#9}$

Number of bundles need $N := \text{Ceil}\left(\dfrac{A_{left}}{N_{per\_bundle}\ A_{s\_ea}}, 1\right) = 89$

Number of curtains $N_{curtain} := 2$

Number of bundles/curtain $N_{req\_per\_curatain} := \text{Round}\left(\dfrac{N}{N_{curtain}}, 1\right) = 45$

Spacing of longitudinal bars $s := 8\ in$

Remaining web 1 length for rebar $FLANGE_{remaining} := l_1 - L_{BE} - b_{CORNER} = 84\ in$

Max rebar allowed in web 1 for spacing $N_{req\_flange} := \text{Floor}\left(\dfrac{FLANGE_{remaining}}{s}, 1\right) = 10$

Used number of rebars in flange $N_{flange} := 10$

Remaining web 2 length for rebar $WEB_{remaining} := \left(l_2 + t_1 + t_3\right) - 2 \cdot H_{CORNER} = 212\ in$

Max. rebar allowed in web 2 for spacing $N_{req\_web} := \text{Floor}\left(\dfrac{WEB_{remaining}}{s}, 1\right) = 26$

Used number of rebars in flange $N_{web} := 26$

Total rebar in other parts of wall $N_{design\_per\_curtain} := N_{web} + 2 \cdot N_{flange} = 46$

Area outside of the BE and Corner $A_{ELSEWHERE} := N_{curtain} \cdot N_{design\_per\_curtain} \cdot N_{per\_bundle}\ A_{s\_ea} = 92\ in^2$

Total area of steel $A_{s\_total} := A_{ELSEWHERE} + 2\ A_{s\_CORNER} + 2\ A_{s\_BOUNDARY} = 192\ in^2$

$A_{s\_req} = 189\ in^2$

Final steel ratio $\rho_{final} := \dfrac{A_{s\_total}}{A_g} = 1.27\%$

In each flange, there will be 2 curtains of (10) bundles of 1#9 bars spaced @ 8" o.c.
In the web, there will be 2 curtains of (26) bundles of 1#9 bars spaced @8" o.c.
This will give $N_{design\_per\_curtain} = 46$ bundles with a total area of $A_{ELSEWHERE} = 92\ in^2$.
With the addition of the 2 boundary element areas of steel and two corner elements, this will give a total of $A_{s\_total} = 192\ in^2$ which is greater than the required $A_{s\_req} = 189\ in^2$.
This reinforcement design has a steel ratio of $\rho_{final} = 1.27\%$

Core Wall Shear Design (Earthquake in NS direction)

The expected Moment capacity Mpr is calculated by interpolation from the PM interaction diagram.

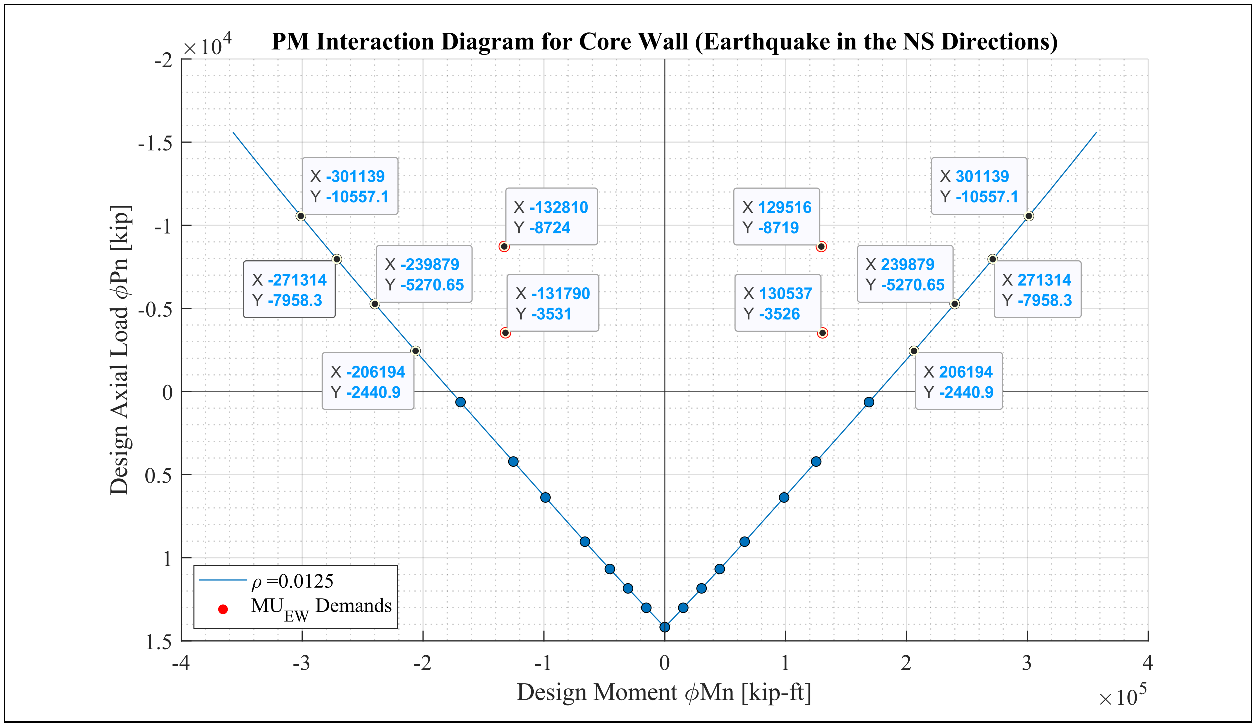


Figure 1 Design Axial Load and Moment Used in Mpr Calculation

Table 1 Calculation of Mpr

LC1		LC2		LC3		LC4	
P	M	P	M	P	M	P	M
Kips	Kip-ft	Kips	Kip-ft	Kips	Kip-ft	Kips	Kip-ft
-5270.65	-239879	-5270.65	239879	-10667.1	-301139	-10557	301139
-3531	**-219170**	-3526	**219110.9**	-8724	**-279745**	-8719	**280044.5**
-2440.9	-206194	-2440.9	206194	-7958.3	-271314	-7958.3	271314

Expected shear demand is calculated as below:

Table 2 Calculation for Ve NS

No.	LC	Pu Kips	Mu,NS Kip-ft	Vu,NS Kips	Mpr Kip-ft	Ωv -	ωv -	Ωv*ωv <=3	Ve,NS Kips
1	0.9D - 0.2SDS + 1.0 Ey Max	-3531	-131790	1473	-219170	1.66	1.8	2.99	**4409.35**
2	0.9D - 0.2SDS + 1.0 Ey Min	-3526	130537	-1470	219111	1.68	1.8	3	**-4410.00**
3	1.2D + 0.2SDS + 0.5L + 1.0 Ey Max	-8724	-132810	1477	-279745	2.11	1.8	3	**4431.00**
4	1.2D + 0.2SDS + 0.5L + 1.0 Ey Min	-8719	129516	-1467	280045	2.16	1.8	3	**-4401.00**

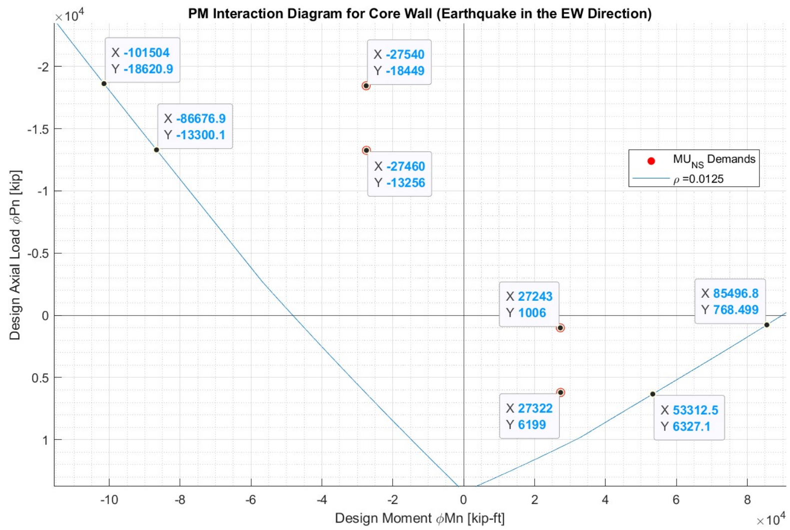
Table 3 Calculation for Wall Shear Reinforcement

Sr. No.	Description	Notation	Units	Value
1	Calculation of longitudinal reinforcement at the base			
1.1	Moment in EW Direction, Shear in NS direction			
	Length of each leg of the wall	L_wall	inch	316
	Width of each leg of the wall	t_wall	inch	28
	Area of each leg of the wall	A_wall	sq.inch	8848
	Curtains on reinforcement (for 30" thickness)	curtains	-	2
	Required reinforcement in walls (more than required, less than 3%)	ρl_req	%	1.25
	Notation of bars proposed to be used	#	-	#9
	Bundled bars	#	-	1
	Diameter of bars proposed to be used	db	inch	1.128
	Area of single bar proposed to be used	Adb	sq.inch	1
	Calculated spacing of bars = Adb*curtains/(t_wall*ρl)	s	inch	5.00
	As per §18.10.2.1 - spacing should be less than 18"	s<18"	Check	OK
1.2	Calculation of shear reinforcement at the base in NS			
	Shear Demand	VuNS	kips	1477
	Dynamic amplification*overstrength factor	Ωv*ωv	-	3
	Shear Demand	VeNS	Kips	4431
	Compressive strength of concrete	f'c	ksi	7
	Reduction factor for light weight concrete (§17.2.6)	λ	-	1
	Shear Strength of concrete = A_wall * λ * √(f'c)	Vnc	Kips	740.3
	If Ve_NS>Vnc, we required ρt > 0.0025	Check	-	REQ.
	Shear strength reduction factor	φ	-	0.75
	Strength reduced capacity of concrete = α*φ*Vnc	α*φ*Vnc	Kips	1665.62
	Balance shear force VuNS-α*φ*Vnc	φ*Vns	Kips	2765.38
	Shear reinforcement required = φVns/(φ*A_wall*fy)	ρt_req	%	0.0069
	Reinforcement to be provided, check for minimum	ρt_req	%	0.0069
	Proposing 4 layers of 1#5 bars as transverse reinf.			#5
	Layers of reinforcement	Layers	#	4
	Area of the transverse reinforcement	Layers*Abd	sq.inch	1.24
	Spacing required = layers*Abd/(t_wall*ρt_prov)	s	inch	6.00
	For 60ksi, smax = 7db = 6"	s_prov	inch	4
	Area of transverse reinforcement provided	ρt_prov	%	0.011
		DCR		0.63

The shear reinforcement of the NS wall is 4 layers of #5 bars spaced at 6 inches.

Demands

$$P_u := \begin{bmatrix} 6199 \\ -13256 \\ 1006 \\ -18449 \end{bmatrix} \textbf{kip} \qquad V_{u\_EW} := \begin{bmatrix} 898 \\ -898 \\ 891 \\ -891 \end{bmatrix} \textbf{kip} \qquad M_u := \begin{bmatrix} 27322 \\ -27460 \\ 27243 \\ -27540 \end{bmatrix} \textbf{kip} \cdot \textbf{ft}$$



Expected moment capacity

Takes into account $(f_y = 1.25\ f_y)$ and $(\phi = 1.0)$

$$M_{pr} := \begin{bmatrix} 53312.5 \\ -86676.9 \\ 85496.8 \\ -101504 \end{bmatrix} \textbf{kip} \cdot \textbf{ft}$$

Expected shear demand §18.10.3

It is taken that $\dfrac{h_{wcs}}{l_w} > 1$

$$\Omega_v := \overrightarrow{maximum\left(\frac{M_{pr}}{M_u}, 1.5\right)} = \begin{bmatrix} 1.951 \\ 3.156 \\ 3.138 \\ 3.686 \end{bmatrix}$$ §18.10.3.1.2

Number of stories $n_s := 14$

Dynamic amplification factor

$$\omega_v := \begin{aligned} &\text{if } n_s \le 6 \\ &\quad \left\| \; 0.9 + \frac{n_s}{10} \right. \\ &\text{else if } n_s > 6 \\ &\quad \left\| \; min\left(1.3 + \frac{n_s}{30}, 1.8\right) \right. \end{aligned} = 1.767$$ §18.10.3.1.3

Expected Shear Demand

$$V_e := \max\left(\overrightarrow{V_{u\_EW} \cdot \overrightarrow{minimum\left(\omega_v \cdot \Omega_v, 3\right)}}\right) = 2694\ \textbf{kip}$$ §18.10.3.1

Capacity §18.10.4

Length of the wall	$L_{wall} := 140\ in$
Thickness of the wall	$t_{wall} := 28\ in$
Area of wall parallel to seismic force	$A_{cv} := L_{wall} \cdot t_{wall} = 3920\ in^2$
Normalweight concret	$\lambda := 1.0$
Taken that $\frac{h_w}{l_w} \geq 2$	$\alpha_c := 2$
Compressive strength of concrete	$f'_c := 7\ ksi$
Tensile strength of steel rebar	$f_y := 60\ ksi$
Shear capacity factor	$\phi_v := 0.75$
Concrete Capacity	$V_{nc} := \phi_v \cdot \left(\alpha_c \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}}\ psi\right) A_{cv} = 491.956\ kip$ §18.10.4.1
Required steel capacity	$V_{s\_req} := V_e - V_{nc} = 2202.044\ kip$
Steel ratio required	$\rho_{req} := \max\left(\frac{V_{s\_req}}{\phi_v \cdot f_y \cdot A_{cv}}, 0.0025\right) = 1.248\%$
Required steel area	$A_{s\_req} := \frac{V_{s\_req}}{\phi_v \cdot f_y} = 48.93\ in^2$
Area of steel rebar used	$A_{s\#6} := 0.44\ in^2$
Number of layers	$N_{layer} := 4$
Spacing of rebar	$s := 5\ in$
Total area of steel provided	$A_s := N_{layer} \cdot A_{s\#6} = 1.76\ in^2$
Steel ratio provided	$\rho_{provided} := \frac{A_s}{s \cdot t_{wall}} = 1.257\%$
Factored Capacity	$\phi_v V_n := \phi_v \cdot \left(\alpha_c \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}}\ psi + \rho_{provided} \cdot f_y\right) A_{cv} = 2709.556\ kip$
Check	$\phi_v V_n \geq V_e = 1$

For shear reinforcement of the EW wall, 4 layers of #6 bars spaced at 5" is adequate and shall be provided.