UNIVERSITY OF CALIFORNIA, SAN DIEGO

SE140B Professional Issues and Design of Civil Structures II

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Project 03: Seismic Design of a Special Reinforced Concrete Wall System

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1. INTRODUCTION

The objective of this project is to design the lateral force-resisting system, the special reinforced concrete core wall system that resists earthquake loading in both the North-South (N-S) and East-West (E-W) directions for the 8-story building being designed for SLOTH Architects.

1.1 Description, Materials and Preliminary Dimensions

The building has one podium level (Story 1), and seven typical levels (a total of 8 stories). The North-South (N-S) and East-West (E-W) lateral force- resisting system for the building consists of four core walls. The specified centerline length of the walls is 30'-i" in the E-W direction and 30'-j" in the N-S direction. i and j were the third and second to last digits of our student PID number respectively. The compression resistance of concrete was found using the same method, but instead used the last digit of our student PID number. After calculating the compression resistance, it was then used in order to calculate the Young's Modulus.

Table 1 Material Properties and Preliminary Dimensions

Description	Variable	Value
Reinforced Concrete and Steel		
Compression Resistance of Concrete	f_c	9 [ksi]
Young's Modulus	E_c	5407.5 [ksi]
Weight per unit volume	γ_c	0.150 [kip/ft^3]
Reinforcement Steel		
Yield Resistance of Steel bars	f_y	60 [ksi]
EW and NS Core Walls		
Wall's length (EW direction)	l_w	30.5 [ft]
Wall's length (NS direction)	l_w	30.167 [ft]

Table 2 Dimensions of core walls

Level	Core Walls (EW and NS directions)	
	Floor height	Thickness t_w (in)
Podium (Story 1)	18'- 0"	30 [in]
Story 2 to Story 12	12'- 6"	30 [in]

Table 3 Dimensions of slabs and columns

Level	Column Dimension	Slabs
	Square shape (in^2)	Thickness (in)
Podium (Story 1)	30" x 30"	8"
Story 2 to Story 12	30" x 30"	8"

2. STRUCTURAL MODEL

The structural model for the project encompasses a comprehensive analysis of the building's lateral force-resisting system, primarily consisting of a core wall running along the North-South (N-S) and East-West (E-W) directions. This core wall, with specified centerline dimensions and wall thickness, serves the primary element to resist earthquake loads.

2.1 Site Seismicity and Design coefficients

An assumption is made that only walls resist lateral loading, columns and beams are designed for gravity loads only. From ASCE 7-22, section 12.2-1, the design coefficients and factors can be determined. The same coefficients are used for the EW and NS lateral force resisting systems.
Site information is input into the ATC Hazards by location website to obtain the mapped acceleration parameters per ASCE 7-22 (11.4.1).

Table 4 Parameters of site location

Description	Detailed information
Hazard Type	Seismic
Reference document	ASCE 7-22
Risk category	II
Site Class	D

To elaborate the acceleration response spectrum, parameters described in table 4 are used. Using this information, the design response spectrum is obtained based on terms given by ASCE 7-22 (11.4).

Table 5 Acceleration parameters

Notation	Description	Value (g)
S_s	5% damped, spectral response acceleration parameter at short period	1.9
S_1	5% damped, spectral response acceleration parameter at 1s period	0.57
S_{DS}	Design spectral response acceleration parameter at short periods	1.39
S_{D1}	Design spectral response acceleration parameter at 1s periods	0.95

Table 6 Importance factor and Seismic Risk Category

Description	Detailed information
Risk category	II
Importance factor	1.0
Seismic design category based on short periods	D
Seismic design category based on 1s period	D

After obtaining these values, they are used in order to plot the design response spectrum. The spectral response acceleration is defined as a piecewise function of the structural period. For periods less than the lower transition period (T_O), the spectral acceleration is given by the equation $S_a = S_{DS}(0.4 + 0.6\frac{T}{T_S})$. For periods between the lower transition period (T_O) and the short-period transition period (T_S), spectral acceleration is constant at the design spectral response acceleration parameter at short periods. Finally, periods greater than T_S, the acceleration decays proportionally to $\frac{S_{D1}}{T}$.

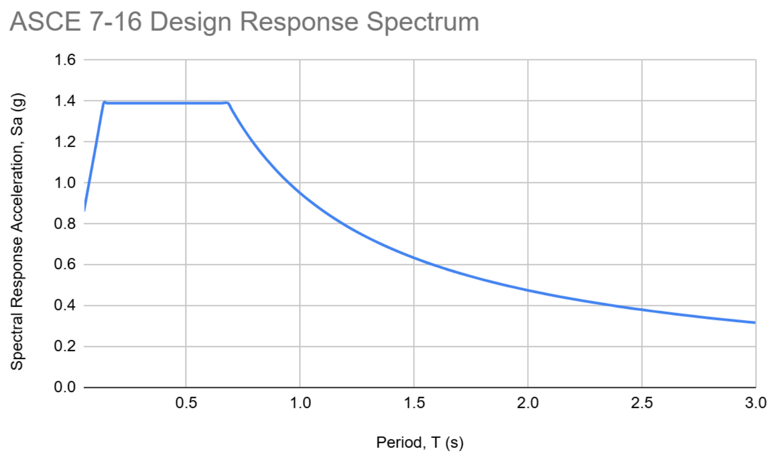


Figure 2.1: Design Response Spectrum

2.2 Effective Seismic Weight

The core walls will be subjected to gravity loading due to various factors. These include external dead loads, self-weight, and a superimposed dead load and live load acting on the floors. In total, the seismic weight was composed of the weight of the concrete walls, slabs, columns, and beams and the superimposed dead and live load. It's important to note that different dead loads (self-weight) are assigned to the first story compared to the roof and typical stories. The seismic weight does not take into account the normal live load, but instead a seismic live load which is 25% of the gravity live load. This comes out to 12.5 psf for the typical floor and 5 psf for the roof. To convert the area loads into a point load, the area of the building was found and multiplied by the area loads of the self-weights of the components and imposed loads. To find the load of the wall, the area load was multiplied by the total length of the walls and the tributary height of the wall, which was half the story height of the story below and above the specified floor. The total weights of all the components are tabulated in Table 7. Table 8 shows the effective seismic weight of each floor, including the cumulative height of the floor from the base of the building.

Table 7 Seismic Weight calculation (kip)

Level	Slab	Walls	Beams	Columns	Dead Load	Live Load $0.25L_o$	Total
Roof	3103 [kip]	1137.5 [kip]	2213 [kip]	187.5 [kip]	931 [kip]	155 [kip]	7727 [kip]
Typical Floor	3103 [kip]	2275 [kip]	2213 [kip]	375 [kip]	931 [kip]	388 [kip]	9284 [kip]
Story 1	3103 [kip]	2775.5 [kip]	2213 [kip]	457.5 [kip]	931 [kip]	388 [kip]	9867 [kip]

Table 8 Effective Seismic Weight

Level	Cumulative Height from Base, h_x [ft]	Effective Seismic Weight, w_x [kip]
8	105.5	7727
7	93	9284
6	80.5	9284
5	68	9284
4	55.5	9284
3	43	9284
2	30.5	9284
1	18	9867
Total	---	73300

2.3 Seismic Response Coefficient

From ASCE 7-22, Table 12.2-1, an appropriate response modification factor R and deflection amplification factor C_d shall be used in determining the base shear, element design forces and design story drift. The type of wall used in this building is a special reinforced concrete shear wall, giving a response modification factor R and deflection amplification factor C_d value of 5 for both, as shown in Table 9. These values will eventually be used to calculate the seismic base shear.

Table 9 Seismic force resisting parameters

Seismic Force	Response Modification Factor, R	Deflection Amplification Factor, C_d
Special reinforced concrete shear walls	5	5

Determination of seismic forces at the base :

The seismic base shear was assumed to be the same in the x- and y-directions. These are calculated using the equivalent lateral force procedure outlined in ASCE 7-22 Section 12.8, considering a redundancy factor, ρ, of 1.0. The fundamental period of the building was calculated per ASCE Section 12.8.2.1, using the variables defined in Table 1, given a C_t and x value of 0.02 and 0.75 respectively. This is due to the fact that the shear wall is classified as “all other structural systems." Using the building height of 105.5 ft, the fundamental period came out to be 0.658 seconds in both directions, as shown in Table 10. Using the fundamental period, the maximum allowable seismic response coefficient was calculated. The seismic response coefficient of the building was also calculated and checked against the maximum value. Since it was less than the max, that value was used. To calculate the seismic base shear, the total seismic weight of the building was multiplied by the seismic response coefficient, shown in Table 11.

Table 10 Fundamental period of the building, T

Description	Variable	Value
EW Direction		
Hand Calculation (Eq.12.8-7, ASCE 7-22)	T_x	0.658 [s]
NS Direction		
Hand Calculation (Eq.12.8-7, ASCE 7-22)	T_y	0.658 [s]

Table 11 Lateral force at the base (Seismic Base Shear), V

Parameter	Variable	Value
Total seismic weight	W	73300 [kip]
Seismic response coefficient	C_{sx}	0.278 [g]
Seismic base shear (EW)	$V_x = \rho C_{sx} W$	20377 [kip]
Seismic response coefficient	C_{sy}	0.278 [g]
Seismic base shear (NS)	$V_y = \rho C_{sy} W_{tot}$	20377 [kip]

Distribution of lateral seismic forces :

The lateral forces for building analysis are determined following section 12.8.3 of ASCE 7-22. Base shears V_x and V_y obtained from equivalent lateral force procedure have already been reduced by a factor R (response modification factor, see Table 9). The lateral seismic forces induced at any level were determined using ASCE Eq. 12.8-11. To find the distribution exponent k, the fundamental period lies between 0.5 and 2.5 seconds, therefore linear interpolation was used. Using the equation $\frac{1.5+T_a}{2}$, the k value was determined to be 1.079. After calculating the lateral seismic forces on each floor, the story shear and overturning moments could be calculated. The story shear is the sum of the all lateral forces above the given level, and the overturning moment is the sum of the moments caused by the forces about that level. These values are tabulated in Table 12. Because there are no lateral forces applied above the roof, the story shear of the roof is equal to the lateral seismic force and no moments are generated at the roof. The distribution of the lateral force, story shears, and overturning moments can be seen in Figure 1.

Table 12 Distribution of Lateral Seismic Forces obtained from the Equivalent Analysis

Level	h_x (ft)	w_x (kip)	$h_x w_x^k$ (kip.ft)	F_x (kip)	Story Shear (kip)	Story Overturning Moment (kip.ft)
8	105.5	7727	815164	3877	3877	0
7	93	9284	863445	4066	7944	48468
6	80.5	9284	747391	3480	11423	147765
5	68	9284	631336	2900	14324	290559
4	55.5	9284	515282	2328	16653	469608
3	43	9284	399227	1769	18422	677775
2	30.5	9284	283173	1221	19643	908052
1	18	9867	177612	734	20377	1153590

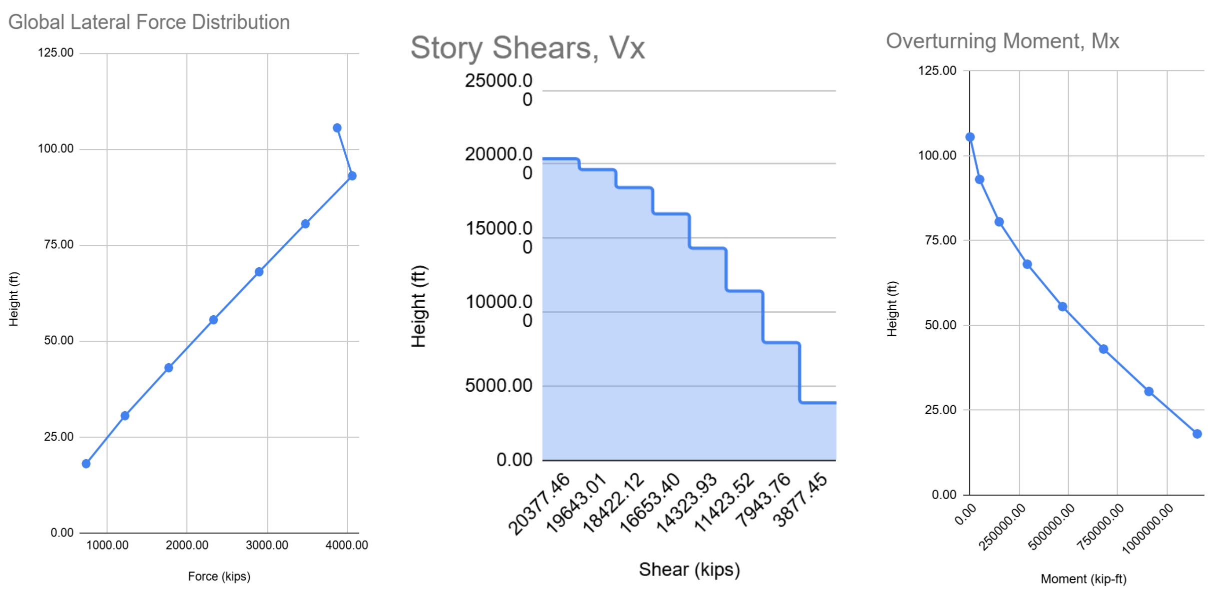


Figure 2.3: (a) Lateral force, (b) Story shear, and (c) Overturning Moment Distribution

2.4 Analysis results

The analysis is carried out with stiffness modifiers equal to $0.5E_c I_g$, in both directions of core walls, according with the suggestions for ACI 318-19, section 6.6.3.1.2. This is to account for the moment of inertia of a wall when it is cracked, resulting in a reduced stiffness. The analysis is carried out of the following two load combinations:

$$\left(0.9 - 0.2S_{DS}\right)D + \rho Q_E$$ - (lateral seismic load + downward vertical acceleration)

$$\left(1.2 + 0.2S_{DS}\right)D + 0.5L + \rho Q_E$$ – (lateral seismic load + upward vertical acceleration)

Story Drift Check :

Story drift at each floor level is calculated based on the most conservative case among the load combinations for both the EW and NS direction and checked against the limit from ASCE 7-22

Table 12.12-1. Deflection at a given floor is a combination of flexure and shear. The flexural deformation occurs below or at the load, bending like a cantilever column that is subjected to a given load. However, the shear deformation occurs above the load, where the wall above the load remains straight but rotates according to the slope at that height. Since there is a deflection contribution from each floor at every given floor, a matrix was set up, then summed to determine the total elastic deflection at each floor. These were then multiplied by the Cd factor and divided by the importance factor to calculate the inelastic deflection, resulting in a maximum deflection of 16.114 inches. The inter-story drift was calculated by taking a given floor's inelastic deflection and subtracting it by the inelastic deflection of the floor below. The maximum drift was 2.58 inches, well within the 3-inch requirement for a 12.5 ft-tall story, mandated by the 2% drift limit for buildings of this height. The drift requirements were met for both the East-West and North-South directions.

Table 13 Lateral Deflections and Inter-Storey Drift check along EW direction

Level	h_{sx} (in)	δ_{xe} (in)	δ_x (in)	Δ_x (in)	Δ_a (in)	Story Drift Ratio (%)	Status
8	1266 [in]	3.223 [in]	16.114 [in]	2.58 [in]	3 [in]	1.72 [%]	Pass
7	1116 [in]	2.707 [in]	13.535 [in]	2.567 [in]	3 [in]	1.71 [%]	Pass
6	966 [in]	2.194 [in]	10.968 [in]	2.502 [in]	3 [in]	1.67 [%]	Pass
5	816 [in]	1.693 [in]	8.466 [in]	2.362 [in]	3 [in]	1.57 [%]	Pass
4	666 [in]	1.221 [in]	6.104 [in]	2.130 [in]	3 [in]	1.42 [%]	Pass
3	516 [in]	0.795 [in]	3.974 [in]	1.79 [in]	3 [in]	1.19 [%]	Pass
2	366 [in]	0.437 [in]	2.185 [in]	1.331 [in]	3 [in]	0.89 [%]	Pass
1	216 [in]	0.171 [in]	0.854 [in]	0.854 [in]	4.32 [in]	0.40 [%]	Pass

Table 14 Lateral Deflections and Inter-Storey Drift check along NS direction

Level	h_{sx} (in)	δ_{xe} (in)	δ_x (in)	Δ_x (in)	Δ_a (in)	Story Drift Ratio (%)	Status
8	1266 [in]	3.223 [in]	16.114 [in]	2.443 [in]	3 [in]	1.63 [%]	Pass
7	1116 [in]	2.734 [in]	13.671 [in]	2.500 [in]	3 [in]	1.67 [%]	Pass

Level	h_{sx} (in)	δ_{xe} (in)	δ_x (in)	Δ_x (in)	Δ_a (in)	Story Drift Ratio (%)	Status
6	966 [in]	2.234 [in]	11.171 [in]	2.501 [in]	3 [in]	1.67 [%]	Pass
5	816 [in]	1.734 [in]	8.670 [in]	2.398 [in]	3 [in]	1.60 [%]	Pass
4	666 [in]	1.254 [in]	6.272 [in]	2.181 [in]	3 [in]	1.45 [%]	Pass
3	516 [in]	0.818 [in]	4.091 [in]	1.841 [in]	3 [in]	1.23 [%]	Pass
2	366 [in]	0.450 [in]	2.250 [in]	1.371 [in]	3 [in]	0.91 [%]	Pass
1	216 [in]	0.176 [in]	0.879 [in]	0.879 [in]	4.32 [in]	0.41 [%]	Pass

Seismic Demands :

The following results will be presented in kip and kip.ft for shear and bending moment respectively. A positive axial load represents tension, and a negative axial load is compression. Forces and lateral displacements are tabulated in Table 15 and 16 to design both shear wall directions. The displayed forces are indicated at the base of Story 1, directly above the basement level. The first combination was used to determine the minimum force demands, while the latter combination was used to determine the maximum force demands.

Note that when calculating the total drift of the structure, the deflection at the top floor (16.114 inches) was taken and divided by the structural height (105.5 ft), with unit conversions applied, resulting in a total building drift of 1.27%.

Table 15 Core wall forces (EQ in the EW direction)

Load Combination	P_u (kip)	M_u EW (kip.ft)	V_u (kip)	δ_{total} (in)	$\frac{\delta_{total}}{h_n}$ (in/in)
$(0.9 - 0.2S_{DS})D + \rho Q_E$	1617.42 [kip]	144,198 [kip.ft]	2547.18 [kip]	16.114 [in]	0.0127 [ ]
$(1.2 + 0.2S_{DS})D + 0.5L + \rho Q_E$	4013.54 [kip]	144, 198 [kip.ft]	2547.18 [kip]	16.114 [in]	0.0127 [ ]

Table 16 Core wall forces (EQ in the NS direction)

Load Combination	P_u (kip)	M_u NS (kip.ft)	V_u (kip)	δ_{total} (in)	$\frac{\delta_{total}}{h_n}$ (in/in)
$(0.9 - 0.2S_{DS})D + \rho Q_E$	1511.22 [kip]	144,198 [kip.ft]	2547.18 [kip]	[in]	0.0127 [ ]
$(1.2 + 0.2S_{DS})D + 0.5L + \rho Q_E$	3761.19 [kip]	144,198 [kip.ft]	2547.18 [kip]	16.114 [in]	0.0127 [ ]

3. CORE WALL DESIGN

3.1 P-M interaction diagram

Axial force (P) and bending moment (M) interaction diagrams were used to determine whether the reinforcements are adequate for the wall. The P-M diagram was used to finalize the required steel reinforcement and find the neutral axis depth, c. The diagram was made with the following assumptions:

- Unconfined concrete crushes at ε_c = -0.003.
- The stress-strain relationship for steel reinforcement is assumed to be elastoplastic.
- Perfect bond is assumed between concrete and steel reinforcement.
- The validity of Bernoulli's assumption of plane section is maintained.
- The cross-section is assumed to remain plane throughout the analysis.
- Reinforcing steel is discretized into three areas based on their tributary area.

The capacity reduction factor ϕ is determined based on the calculated strain in the extreme compression fiber, facilitating the computation of capacity parameters for axial force and bending moment, denoted as ϕP_n and ϕM_n respectively. The P-M diagrams are shown in Figure 3.1.

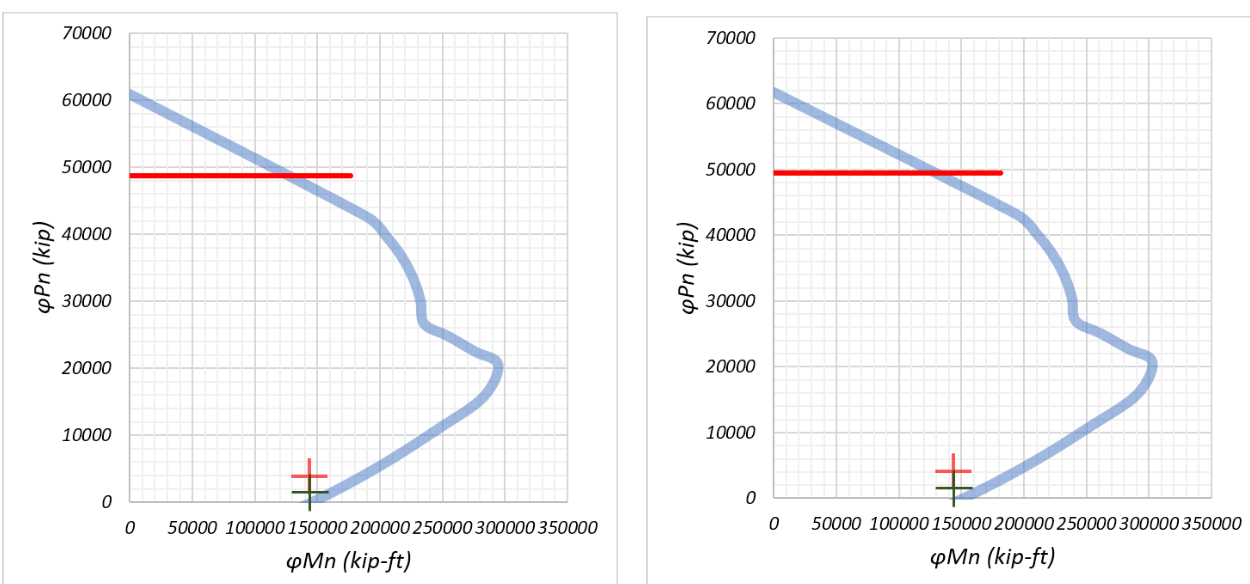


Figure 3.1: (a) P-M diagram for EW direction. (b) P-M diagram for NS direction

3.2 Flexure design and detailing of the critical section

3.2.1. Required longitudinal reinforcement

An initial trial reinforcement was designed for the Special Boundary Elements (SBE) and the web. The reinforcement ratio values were then computed and used to plot P-M diagrams for the walls orthogonal to both the North-South and East-West wall. 36 #9 bars were initially placed in both the SBE areas and two curtains of #6 bars at 12” c.c. were placed in the web for both the NS and EW walls. The P-M diagram excel sheet provided was used to plot the design. The resulting reinforcement ratio was inadequate for the capacity to be greater than the demand on the walls so the reinforcements were redesigned for the demands to fall within the P-M curve’s envelope.

The reinforcements were increased to 40#9 bars in each SBE and 2 curtains of #11 bars at 6” c.c in the web for both the NS and ES walls. This creates a much higher overall reinforcement ratio that allowed for the demands to fit in the P-M diagram’s curve. The reinforcements in the SBE were added while remaining under the 2.5% constructability limit. Though the ratio is larger than the ideal ratio of less than 1.8%, it was still under the required criteria of 2.5%. The P-M diagrams used for the East-West and North-South directions are displayed below in Figure 3.1.a. and Figure 3.1.b.

The initial step looks for assessing the aspect ratio of the wall by referencing table R18.10.1 in the ACI 318R-19 standard. When the ratio $\frac{h_w}{l_w}$ falls below 2, shear forces predominantly govern the design. Conversely, when the ratio $\frac{h_w}{l_w}$ equals or exceeds 2, flexural forces govern, and we design the wall pier with specified column design requirements. In our design, the confinement column thickness matches the wall thickness (**b = a;** see figure 2).

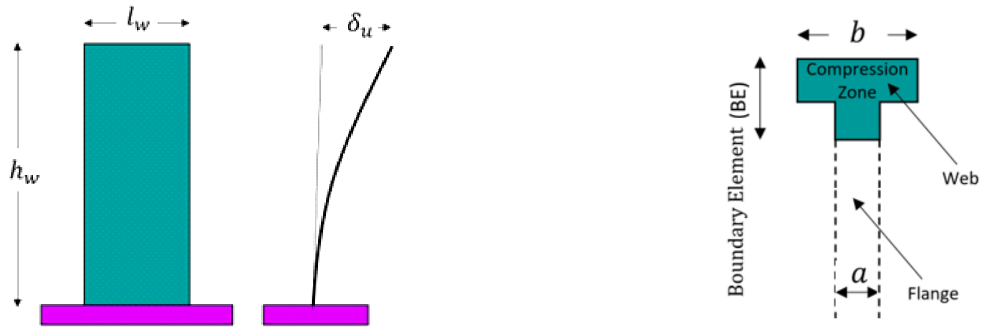


Figure 3.2 : (a) Slenderness of the wall. (b) Compression zone in boundary elements

The distributed web reinforcement ratios, ρ_l and ρ_t, for structural walls must be at least 0.0025 for both, longitudinal and transversal reinforcement steel. It is clear that if the shear force V_u exceed the value of :

$$V_n = 2\lambda\sqrt{f_c}A_{cv} \text{ or } \frac{h_w}{l_w} \geq 2 \quad (1)$$

the wall will need at least 2 curtains of reinforcement. In equation (1), λ takes values of 0.75 and 1.00 for lightweight concrete and normal weight concrete respectively. Thus, two curtains of reinforcement are placed in both the NS and EW walls.

3.2.2. Boundary elements

Special boundary elements are required to reinforce compression zones of structural walls when the flexural and axial capacity is not enough. This ensures that the walls have adequate reinforcement to resist the force and deformation demands. For our design, they were placed at the ends of the walls and span 60” at either end of the wall. Special boundary elements reinforcing the compression zones are required for most critical case given by equations 2 and 3 :

$$\frac{1.5\delta_u}{h_w} \geq \frac{l_w}{600c} \tag{2}$$

where,

$$\frac{\delta_u}{h_w} \leq 0.005 \tag{3}$$

In equation 2, c represents the neutral axis depth calculated for the factored axial load and nominal moment strength, h_w denotes the height of the entire wall from base to top. Data from the P-M curve was used to find c for both the NS and EW walls. The value of c was found on.y for the load combination that produced the greatest demand: $(1.2 + 0.2S_{DS})D + 0.5L + 1.0E_x$. The load combination $(0.9 - 0.2S_{DS})D + 1.0E_x$ was not considered. Both the NS and EW wall require SBE as they do not pass the check. This is seen in Table 17 and Table 18.

Table 17 Estimation of neutral axis depth (EQ in the EW direction)

Load Combination	$\frac{\delta_u}{h_w}$ (eq. 3)	c (in) [P-M curve]	c (in) [eq. 2]	Special boundary
$(1.2 + 0.2S_{DS})D + 0.5L + 1.0E_x$	0.0127	59.762	31.6	YES

Table 18 Estimation of neutral axis depth (EQ in the NS direction)

Load Combination	$\frac{\delta_u}{h_w}$ (eq. 3)	c (in) [P-M curve]	c (in) [eq. 2]	Special boundary
$(1.2 + 0.2S_{DS})D + 0.5L + 1.0E_y$	0.0127	58.745	31.9	YES

The length of the SBE was designed to be 60 in. The required minimum length was calculated to be 29.37 in. for the NS direction and 29.86 in. for hte EW direction so the initial design is adequate.
Transverse reinforcement in the boundary elements is supplied through hoops and crossties. The vertical spacing between each reinforcement is set as S_{max} and is the minimum of 6", a third of the boundary element thickness, or six times the diameter of the smallest longitudinal bars. Both the NS and EW direction result in a spacing of 4.5 in.

The area of transversal reinforcement, A_{sh}, is found to determine how many transverse bars are needed in the boundary element area. The area is divided by the area of a single bar to find the required amount of reinforcements needed to span the SBE area. Both NS and EW directions requried 4 legs of #6 bars. The resulting area of reinforcement steel is 1.76 in^2.

3.3 Shear design

To define the shear force, we use the proposed formulation by ACI318-19 at section 18.10.3.1:

$$V_e = \Omega_v \omega_v V_u \leq 3V_u \tag{4}$$

Where, Ω_v is overstrength factor, ω_v is the dynamic amplification factor, and V_u is the shear force obtained from code lateral load analysis. The Overstrength factor Ω_v, is calculated as follows :

Table 19 Overstrength factor (Ω_v) at critical section

Slenderness	Value	Factor Ω_v	
$\frac{h_w}{l_w}$ (Flexure)	more than 1.5	Greater of	$\frac{M_{pr}}{M_u}$
			1.50
$\frac{h_w}{l_w}$ (Shear)	less than 1.5	1	

The slenderness is much greater than 1.5 for both NS and EW so M_{pr}, the probable moment capacity, must be found to calculate the overstrength factor. Slenderness is computed in Table 20.

Table 20 Slenderness factor $\frac{h_w}{l_w}$ for the west core wall

Earthquake	h_w (ft)	l_w (ft)	Slenderness
EW direction	1266 ft	30.5 ft	41.51
NS direction	1266 ft	30.167 ft	41.97

In capacity design, a P-M diagram is calculated using a reduction factor ϕ equal to 1 and an expected yield strength f_{yexp} equal to $1.25f_y$. The following figure shows the computation of M_{pr} for the EW and NS direction according to the two load combinations. For these P-M diagrams, M_{pr} is based on longitudinal steel ratio of 1.89% for NS and 1.92% for EW related to each axial combination.
For the given P_{umax}, maximum ultimate shear, the M_{pr} was found by finding what value leads to an intersection on the graph.

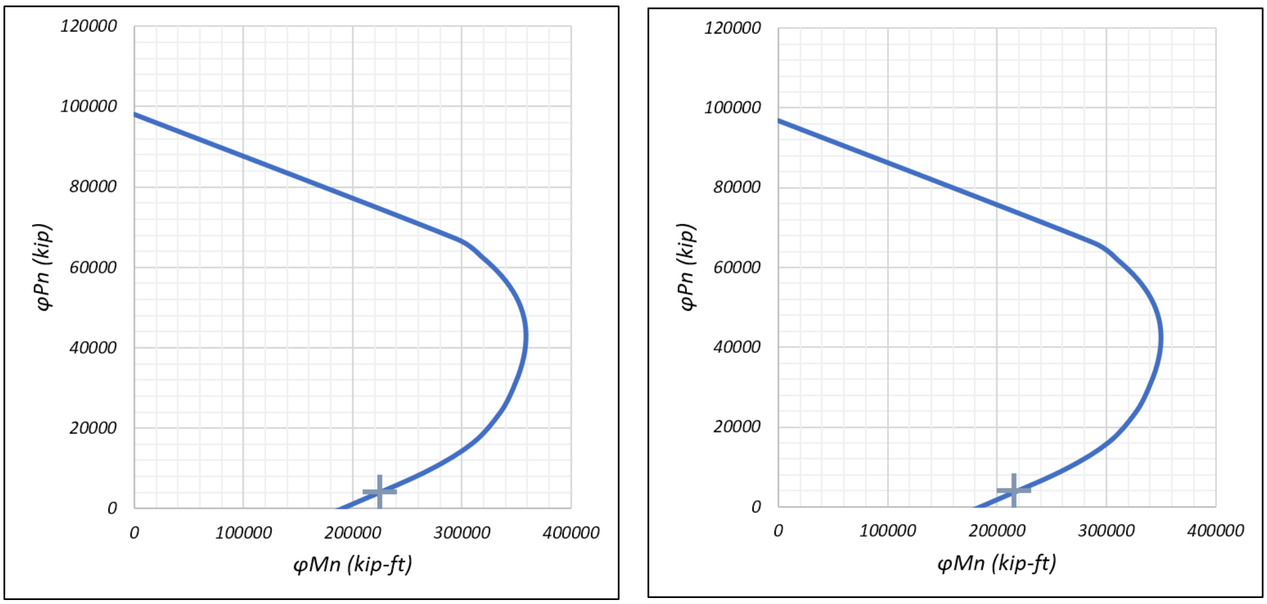


Figure 3 : (a) M_{pr} for EW direction. (b) M_{pr} for NS direction. For both cases, the reduction factor is considered as $\phi = 1$

Furthermore, the overstrength factors are calculated using the relation between M_u, ultimate moment, and M_{pr}, probable moment. Table 21 shows the values corresponding to EW direction meanwhile Table 22 displays those values in NS direction.

Table 21 Overstrength factor Ω_v of core wall (EQ in the EW direction)

Load Combination	P_u (kip)	M_u NS dir. (kip.ft)	M_{pr} NS dir. (kip.ft)	$\Omega_v = \left\lvert \frac{M_{pr}}{M_u} \right\rvert$
$(0.9 - 0.2S_{DS})D + 1.0E_x$	1617.42	144198.7	200000	1.387
$(1.2 + 0.2S_{DS})D + 0.5L + 1.0E_x$	4013.54	144198.7	225000	1.497

Table 22 Overstrength factor Ω_v of core wall (EQ in the NS direction)

Load Combination	P_u (kip)	M_u EW dir. (kip.ft)	M_{pr} EW dir. (kip.ft)	$\Omega_v = \left\lvert \frac{M_{pr}}{M_u} \right\rvert$
$(0.9 - 0.2S_{DS})D + 1.0E_y$	1511.22	144198.7	200000	1.387
$(1.2 + 0.2S_{DS})D + 0.5L + 1.0E_y$	3761.19	144198.7	215800	1.497

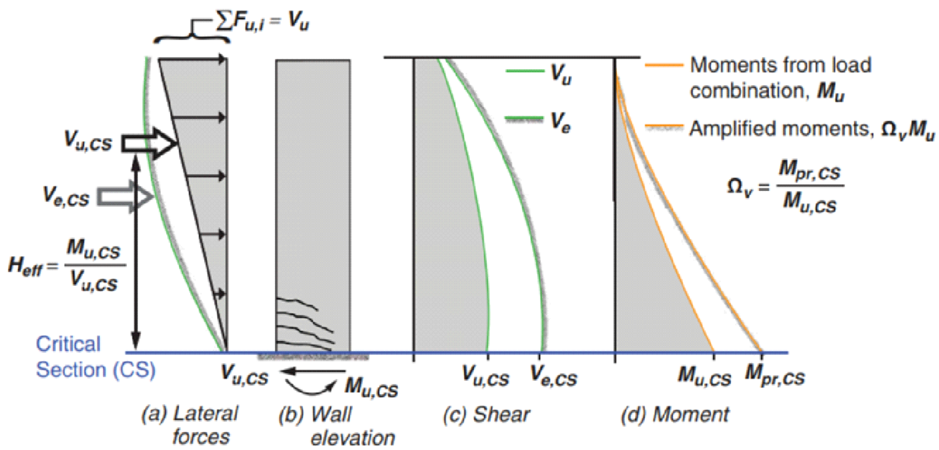


Figure 3.3 : Determination of shear demand for walls with $\frac{h_w}{l_w} \geq 2$

The dynamic amplification factor, ω_v, was calculated with equation 7 as the slenderness ratio was greater than 2, $\frac{h_w}{l_w} < 2$. n_s is the number of stories.

$$\omega_v = 0.9 + \frac{n_s}{10},\ n_s \leq 6 \tag{7}$$

According to equations 7 and 8, Table 23 and 24 displays the values of ω_v for EW and NS direction.

Table 23 Dynamic amplification factor ω_v of West core wall (EQ in the EW direction)

Direction	h_w	l_w	$\frac{h_w}{l_w}$	n_s	ω_v
EW	105.5 ft	30.5 ft	3.459	8	1.7

Table 24 Dynamic amplification factor ω_v of West core wall (EQ in the NS direction)

Direction	h_w	l_w	$\frac{h_w}{l_w}$	n_s	ω_v
NS	105.5 ft	30.167 ft	3.497	8	1.7

In Tables 25 and 26, V_e is computed considering that $\Omega_v \omega_v$ will be no more than 3 in all cases. From Tables 21, 22, 23 and 24, the product of Ω_v and ω_v is more than 3, therefore, V_e will be 3 times V_u. V_e was calculated for the load combination that produced the highest demand.

Table 25 Shear design force of core wall (EQ in the EW direction) on the flange

Load Combination	V_u [Kip]	Ω_v	ω_v	$\Omega_v \omega_v$ [max=3]	$\lvert V_e \rvert$ [kip]
$(1.2 + 0.2S_{DS})D + 0.5L + 1.0E_x$	2547.18	1.56	1.7	2.652	6756.63

Table 26 Shear design force of core wall (EQ in the NS direction) on the web

Load Combination	V_u [Kip]	Ω_v	ω_v	$\Omega_v \omega_v$ [max=3]	$\lvert V_e \rvert$ [kip]
$(1.2 + 0.2S_{DS})D + 0.5L + 1.0E_y$	2547.18	1.5	1.7	2.55	6495.32

Finally, we use the equation below to estimate the shear strength according to ACI 318-19. This was done only for the load combination that produces the greatest demand.

$$V_n = (\alpha_c \lambda \sqrt{f_c} + \rho_t f_{yt}) A_{cv} \tag{9}$$

where, α_c takes the value of 3 and 2 for $\frac{h_w}{l_w \le 1.5}$ and $\frac{h_w}{l_w \ge 2.0}$ respectively. It shall be permitted to linearly interpolate the value of α_c between 3 and 2 for

$$1.5 < \frac{h_w}{l_w} < 2.0$$

The transversal reinforcement ratio in equation 9, ρ_t, must be greater than 0.0025. Finally, the value of V_n shall not be taken greater than

$$V_n = 8\sqrt{f_c} A_{cv} \tag{10}$$

for vertical wall segments sharing a common lateral force, and

$$V_n = 10\sqrt{f_c} A_{cv} \tag{11}$$

for horizontal wall segments (interaction between structural walls and couple beams).

Table 27 Verification of shear capacity of core wall (EQ in the EW direction)

Load Combination	V_u [Kip]	V_n (kip) [eq. 9]	V_n (Kip) [eq. 10]	Min. V_n [Kip]	$\left\|\frac{V_u}{V_n}\right\|$
$\left(1.2 + 0.2S_{DS}\right)D + 0.5L + 1.0$	2547.18	14699.33	8333.23	8333.23	0.306

Table 28 Verification of shear capacity of core wall (EQ in the NS direction)

Load Combination	V_u [Kip]	V_n (kip) [eq. 9]	V_n (Kip) [eq. 10]	Min. V_n [Kip]	$\left\|\frac{V_u}{V_n}\right\|$
$\left(1.2 + 0.2S_{DS}\right)D + 0.5L + 1.0$	2547.18	14349.72	8242.16	8242.16	0.309

3.4 Longitudinal and transversal steel reinforcement

The required steel reinforcement spacing for shear in the web was calculated from the the required steel capacity and required reinforcement ratios. The initial spacing set for the web was 6” for both NS and EW. The spacing chosen for our design had to be less than the required spacing derived from the required steel ratio and the maximum spacing. The spacing of 6 in c.c. in the curtain reinforcement is satisfactory. Table 29 details the reinforcement in the web.

Table 29 Longitudinal Reinforcement Steel Bars

Description	Variable	Value
EW Core Wall		
Longitudinal Steel reinforcement (vertical)	Ls_{EW}	[#11 @ 6”]
Transversal steel reinforcement (horizontal)	Ts_{EW}	[#6 @ 8”]
EW Core Wall		
Longitudinal Steel reinforcement (vertical)	Ls_{NS}	[#11 @ 6”]
Transversal steel reinforcement (horizontal)	Ts_{NS}	[#6 @ 8”]

For the reinforcement at the confinement zones, the first step was to find the neutral axis depth, c, then perform a deflection and special boundary element check. It was determined that SBE was required, so the designed length of 60 in. was checked against the requirements and proved to be sufficient. Finally the maximum hoop spacing at the boundary elements and transverse reinforcement required were calculated. Table 30 displays the longitudinal reinforcement and shear reinforcement in the compression zones (Confinement columns at the corners of the wall).

Table 30 Reinforcement Steel Bars at confinement zones (Columns)

Description	Variable	Value
EW Core Wall		
Axial Steel reinforcement (vertical)	Cs_{EW}	[22 #11]
Rebar size for stirrups and spacing [Base of the wall]	Ss_{EW}	[#6 @ 4.5”]
EW Core Wall		

Description	Variable	Value
Axial Steel reinforcement (vertical)	Cs_{NS}	[22 #11]
Rebar size for stirrups and spacing [Base of the wall]	Ss_{NS}	[#6 @ 4.5”]

CONCLUSION AND RECOMMENDATIONS

Shear walls are one of the most important members in a building's design. Its ability to resist lateral forces, both wind and seismic, are paramount to strengthening a building when experiencing those loads. Therefore, the shear wall design process must be meticulously followed to ensure that the wall provides enough strength and that no consideration is left unnoticed.

The lateral force procedure consisted of a process outlined in ASCE 7-22. It is crucial that this process is followed correctly, otherwise the walls could be designed with a smaller capacity than is necessary. Given all of the weights of the building components, including the beams, columns, slabs and walls, it came out to a higher value than expected, at around 73300 kips. Using the site seismicity parameters, the effective base shear was calculated to be 20377 kips. At the end of the procedure, this gave a surprisingly large moment at the base of the building, well over 1 million kip-ft. Nevertheless, the graphs for the lateral force, story shear, and overturning moment distributions came out to be what we expected, having the same shapes as the example design. A lot of trial and error was used in the beginning of the calculations, due to information being left out, so it is imperative that all variables are accounted for.

To determine the interstory drift, the deflection contributions from each level were first added together at a given floor. Then the deflection of the story below the floor of consideration would be subtracted from the deflection, leaving a relative "drift" value for the floor. The maximum drift value was found to be 2.58 inches, which is within the 3-inch maximum limit for a 12.5-foot tall story (from the 2% drift limit for buildings of this height and seismic design category). After the deflections and drifts were calculated at every floor level for both the East-West and North-South directions, the drift requirements were found to have been met. When designing for drift, certain considerations were made. The original shear wall layout planned for only four shear walls in each direction. However, because the walls kept failing the drift check, additional walls of the same size were added. Adding double the walls per direction lowered the interstory drift substantially, allowing the design to pass the interstory drift checks with ease.

The special boundary element design provides additional stability to the wall. This also creates the need for more calculations to consider how it affects the walls' shear and flexural stability. The SBE was longer than expected at 60 in., but that gave enough space for the required amount of additional reinforcements to replace while also keeping the steel reinforcement ratio under the maximum amount and over the minimum amount. This initial length of 60 in. was helpful despite seeming long. I would set a long length for future projects as well. In the future, I would also consider putting in more reinforcement when doing the initial steel reinforcement design as it had to be redesigned numerous times due to an inadequate capacity. Thankfully though, the resultant design created for the P-M diagram passed all the spacing checks.

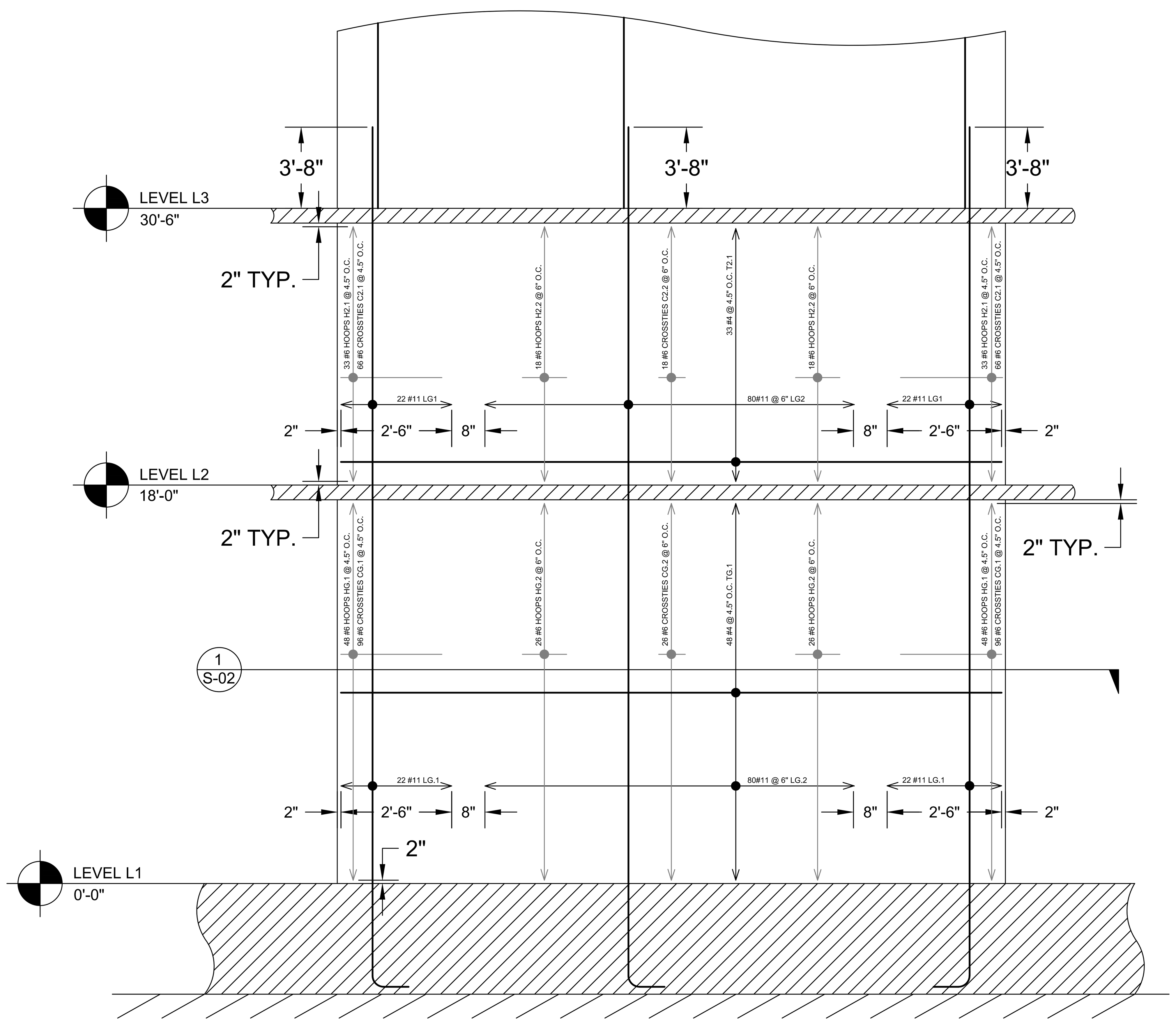
REFERENCES

ACI Committee 318. (2019). *Building code requirements for structural concrete (ACI 318-19) and commentary (ACI 318R-19)*. American Concrete Institute.

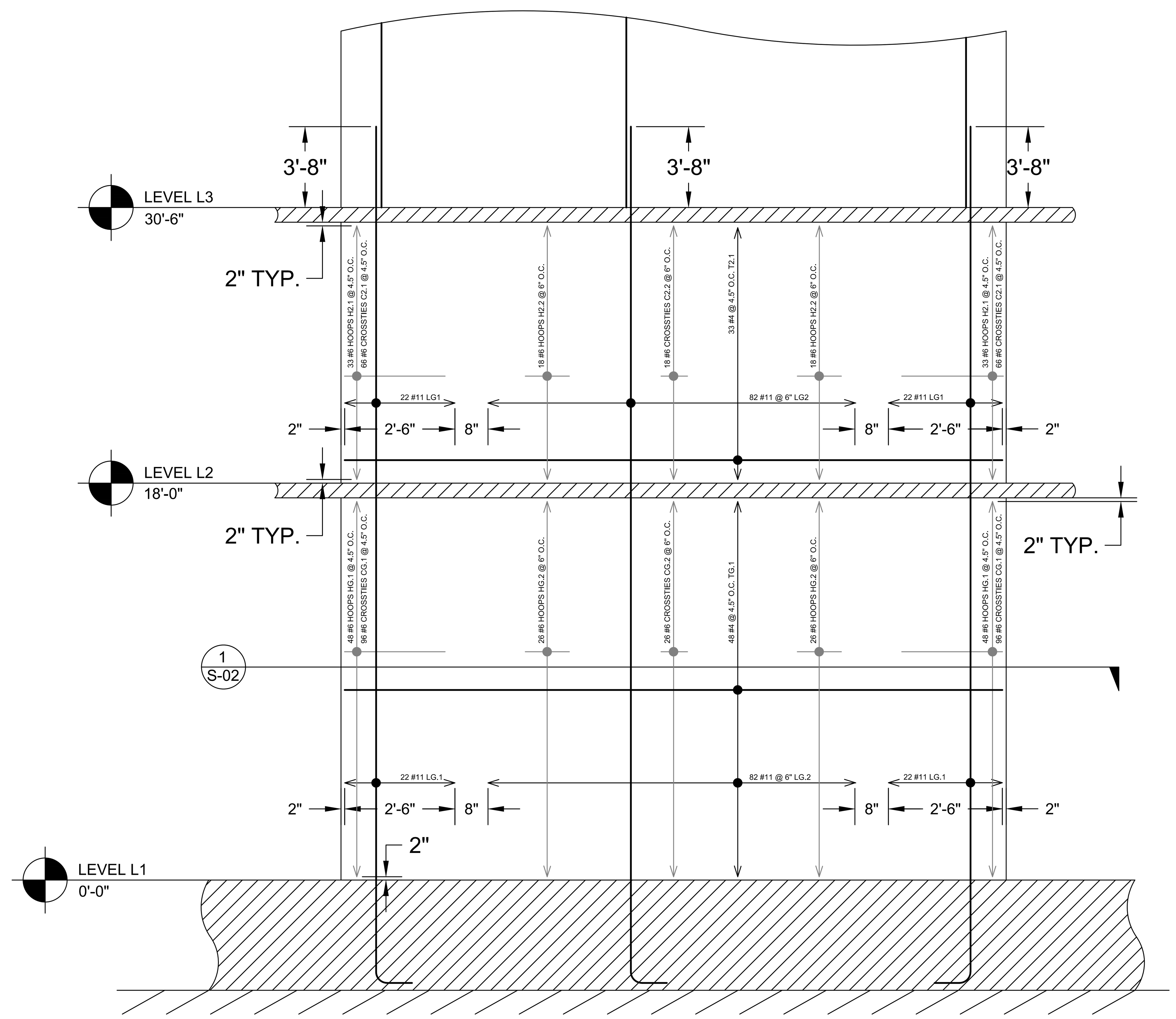
American Society of Civil Engineers. (2022). *Minimum design loads and associated criteria for buildings and other structures* (ASCE/SEI 7-22). https://doi.org/10.1061/9780784415788

APPENDIX A – Technical drawings

A1 - Wall Elevation (NS)
A2 - Wall Elevation (EW)
A3 - Wall Cross Section, with confinement zone (NS)
A4 - Wall Cross Section, with confinement zone (EW)
A5 - Reinforcing Bar Schedule



1 | ELEVATION WALL NS-1: LEVELS G0 TO L3
S-01 | 1/4" = 1'-0"



2 | ELEVATION WALL EW-1: LEVELS G0 TO L3
S-01 | 1/4" = 1'-0"

General Notes

WALL ELEVATION VIEWS

No.	Revision/Issue	Date

Firm Name and Address

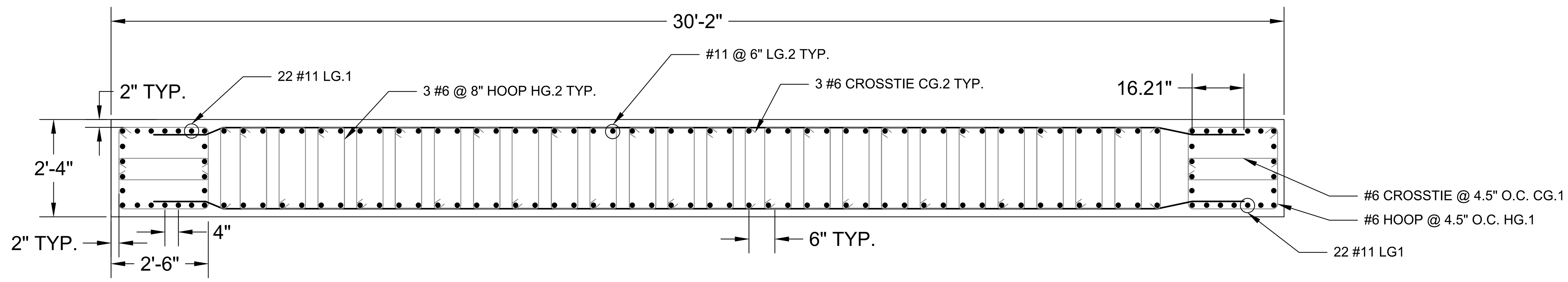
Project Name and Address

Project: SE 140B Project 3

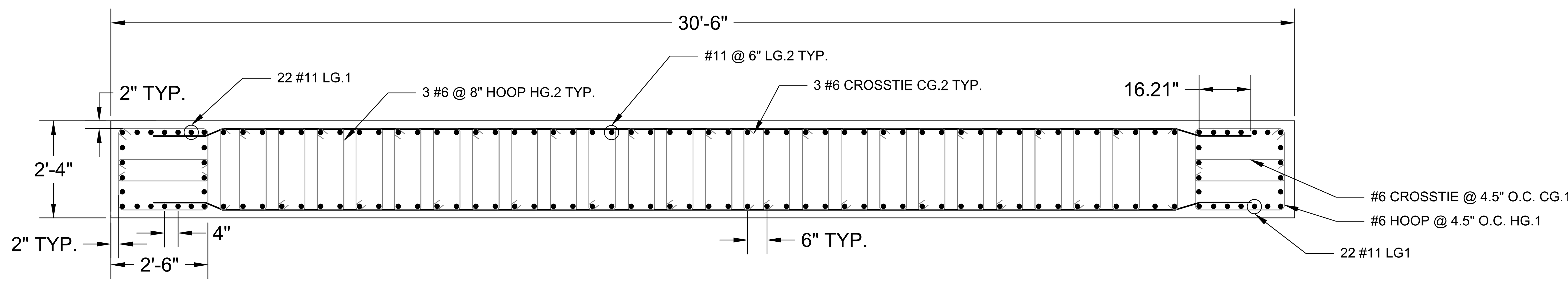
Sheet: S-01

Date: 6/5/2026

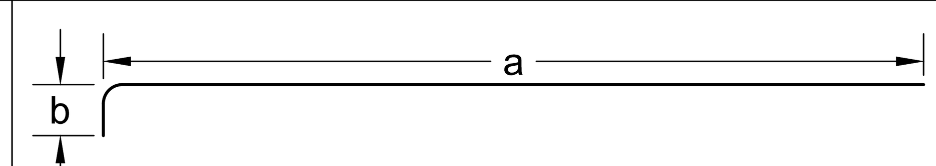
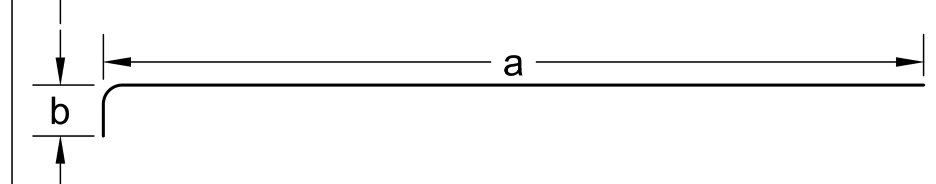
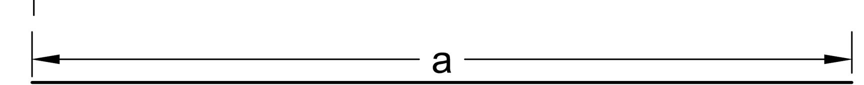
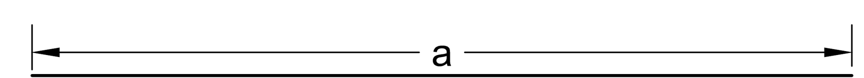
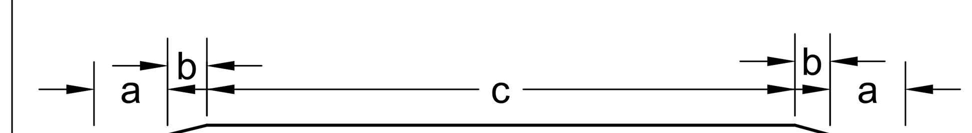
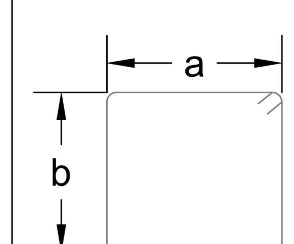
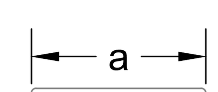
Scale: As Noted



1	STRUCTURAL WALL NS-1 CROSS-SECTION
S-02	1/2" = 1'-0"



2	STRUCTURAL WALL EW-1 CROSS-SECTION
S-02	1/2" = 1'-0"

MEMBER ID	TAG	LOCATION	BAR ID	a	b	c	QTY	NOTES	SHAPE
WALL NS/EW-1	LG.1	LEVELS G0-L2	#11	22'-10"	1'-4"		44	MECHANICAL COUPLER AT STRAIGHT END	
WALL NS-1 WALL EW-1	LG.2 LG.2	LEVELS G0-L2 LEVELS G0-L2	#11 #11	22'-10" 22'-10"	1'-4" 1'-4"		80 82	MECHANICAL COUPLER AT STRAIGHT END	
WALL NS/EW-1	L2.1	LEVELS L2-L3	#11	22'-10"			44	MECHANICAL COUPLER AT ONE END	
WALL NS-1 WALL EW-1	L2.2 L2.2	LEVELS L2-L3 LEVELS L2-L3	#11 #11	22'-10" 22'-10"			80 82	MECHANICAL COUPLER AT ONE END MECHANICAL COUPLER AT ONE END	
WALL NS/EW-1	TG.1 T2.1	LEVELS G0-L2 LEVELS L2-L3	#4 #4	1'-4" 1'-4"	0'-1.5" 0'-1.5"	24'-0" 24'-0"	48 33	1:6 CRANK AT b 1:6 CRANK AT b	
WALL NS/EW-1	HG.1 HG.2 H2.1 H2.2	LEVELS G0-L2 LEVELS G0-L2 LEVELS L2-L3 LEVELS L2-L3	#6 #6 #6 #6	2'-6" 2'-6" 2'-6" 2'-6"	2'-4" 2'-4" 2'-4" 2'-4"		96 52 66 36		
WALL NS/EW-1	CG.1 CG.2 C2.1 C2.2	LEVELS G0-L2 LEVELS G0-L2 LEVELS L2-L3 LEVELS L2-L3	#6 #6 #6 #6	2'-4" 2'-0" 2'-4" 2'-0"			192 26 132 18		

General Notes

CROSS SECTIONAL VIEWS

No.	Revision/Issue	Date

Firm Name and Address

Project Name and Address

Project: SE 140B Project 3	Sheet: S-02
Date: 6/5/2026	
Scale: As Noted	

APPENDIX B – Step-by-Step calculations

(B1) Input, Geometry, and Site Seismicity Calculations

[SE 140B] Project 3				
Codes: ACI 318-19, ASCE 7-22				
Phase A — Input, geometry & site seismicity				
1.1 Building Geometry and PID-based Dimensions				
h_1	Podium Floor Height	18	*ft*	
h_2-8	Story Floor Height	12.5	*ft*	
l_w, EW	Wall's length (EW direction): 30'-i" (i = 1st PID digit)	30.5	*ft*	
l_w, NS	Wall's length (NS direction): 30'-j" (j = 2nd PID digit)	30.16666667	*ft*	
1.2 Concrete and Steel Material Properties				
f'c	Compression Resistance of Concrete: [ksi — from PID digit k]	9	*ksi*	
f_y = f_yt	Yield Resistance of Steel bars	60	*ksi*	
γ_c	Weight per unit volume	0.150	*kcf*	
E_c	Young's Modulus: Ec = 57√f'c (ksi)	5407.495	*ksi*	
1.3 Wall and Section Preliminary Dimensions				
t_w	Wall Thickness	30	*in*	**(as a first trial)**
column	Column Dimensions	30	*in^2*	
t_s	Slab Thickness	8	*in*	
2.1 Site Seismicity Parameters from ATC Hazards Tool				
S_s	5% damped, spectral response acceleration parameter at short period	1.9	—	
S_1	5% damped, spectral response acceleration parameter at 1s period	0.57	—	
Site Class		D	—	
2.2 SDS, SD1 and Design Response Spectrum				
S_DS	Design spectral response accelertation parameter at short periods: SDS = (2/3)SMS	1.390	—	
S_D1	Design spectral response accelertation parameter at 1s periods: SD1 = (2/3)SM1	0.950	—	
Design Spectrum				
T_a	fundamental apporximate period: T_a = C_t*h_n^x	0.658	*s*	
T_S	T_s = S_D1/S_DS	0.683	*s*	
T_O	T_O = 0.2T_S	0.137	*s*	
T_L		4	*s*	
2.3 Risk Category, Ie, and Seismic Design Category (SDC)				
Risk Category		II	—	
I_e	Importance Factor	1.0	—	
SDC_SDS	Seismic Design Category from SDS	D	—	
SDC_SDS	Seismic Design Category from SD1	D	—	
2.4 Seismic Force-Resisting System Parameters				
R		5	—	
C_d		5	—	
Ω_o		2.5	—	
ρ		1.0	—	

(B2) Seismic Weight and Lateral Force Analysis Calculations

[SE 140B] Project 3				
Codes: ACI 318-19, ASCE 7-22				
Phase B — Seismic Weight and Lateral Force Analysis				
1. Initial Inputs				
1.1 Building Geometry and PID-based Dimensions				
h_pod	Podium Floor Height	18	ft	
h_floors	Story Floor Height	12.5	ft	
l_w, EW	Wall's length (EW direction): 30'-i" (i = 1st PID digit)	30.5	ft	
l_w, NS	Wall's length (NS direction): 30'-j" (j = 2nd PID digit)	30.167	ft	
L_bldg	Total length of the building	211.167	ft	
B_bldg	Total width of the building	156	ft	
A_bldg	Area of the building	31026.42	ft^2	
L_beams	Length of all the beams in x- and y-directions	2459	ft	
1.2 Concrete and Steel Material Properties				
f'c	Compression Resistance of Concrete: [ksi — from PID digit k]	9	ksi	
f_y = f_yt	Yield Resistance of Steel bars	60	ksi	
γ_c	Weight per unit volume	0.150	kcf	
E_c	Young's Modulus: Ec = 57√f'c (ksi)	5407.49	ksi	
E_v	Concrete Shear Modulus of Elasticity: 0.4 * Ec	2163.00	ksi	
1.3 Wall and Section Preliminary Dimensions				
t_w	Wall thickness	30	in	**(as a first trial)**
column	Column dimensions	30	in	
t_s	Slab thickness	8	in	
beam	Beam dimensions	864	in^2	
1.4 Building Loads				
w_floor_D	Distributed dead load applied to typical floors	30	psf	
w_floor_L	Distributed unreduced live load applied to typical floors	50	psf	
w_floor_L_gravity	Distributed reduced live load applied to typical floors	20	psf	
w_floor_L_seismic	Distributed reduced live load applied to typical floors for seismic weight	12.5	psf	
w_roof_D	Distributed dead load applied to the roof	30	psf	
w_roof_L	Distributed live load applied to the roof	20	psf	
ρ_wall	Area weight of one concrete wall	375	psf	
ρ_slab	Area weight of the concrete slab: (0.150 kcf) * (8 / 12 ft) = 0.100 ksf (area load)	100	psf	
ρ_columns	Area weight of one concrete column	375	psf	
W_beams	Weight of the beams per floor	2213.1	kip	
2. Seismic Site Parameters				
2.1 Site Seismicity Parameters from ATC Hazards Tool				
S_s	5% damped, spectral response acceleration parameter at short period	1.651	—	
S_1	5% damped, spectral response acceleration parameter at 1s period	0.575	—	
Site Class		D	—	
2.2 SDS, SD1 and Design Response Spectrum				
S_DS	Design spectral response accelertation parameter at short periods: SDS = (2/3)SMS	1.39	—	
S_D1	Design spectral response accelertation parameter at 1s periods: SD1 = (2/3)SM1	0.95	—	
Design Spectrum				
T_a	fundamental apporximate period: T_a = C_t*h_n^x	0.658	s	
T_S	T_s = S_D1/S_DS	0.683	s	
T_O	T_O = 0.2T_S	0.137	s	
T_L		4	s	
2.3 Risk Category, Ie, and Seismic Design Category (SDC)				
Risk Category		II	—	
I_e	Importance Factor	1.0	—	
SDC_SDS	Seismic Design Category from SDS	D	—	
SDC_SDS	Seismic Design Category from SD1	D	—	
2.4 Seismic Force-Resisting System Parameters				
R		5	—	
C_d		5	—	
Ω_o		2.5	—	
ρ		1.0	—	

3. Seismic Weight Calculation (W_x)				
L_total	Total length of all the structural walls: 8 in the N-S and E-W directions	485.33	ft	
3.1 Calculations for Level 1 (Podium)				
h_trib_1	Tributary height: =0.5*(h_pod) +0.5*(h_floors)	15.25	ft	
W_wall_1	Wall weight acting on the podium: L_total*h_trib_1*p_wall	2775.50	kip	
W_floor_1	Floor weight acting on the podium: (A_bldg)*(w_floor_D + 0.25*w_floor_L + p_slab) + P_beams + y_conc*h_col*A_col*(N_col)	7091.86	kip	
W_1	Total weight acting on podium: W_wall_1 + W_floor_1	9867.36	kip	
3.2 Calculations for Typical Floor (Levels 2 through 7)				
h_trib_typ	Tributary height: = h_floors /2*2	12.5	ft	
W_wall_typ	Wall weight acting on typical floor: L_total*h_trib_typ*p_wall	2275.00	kip	
W_floor_typ	Floor weight acting on typical floor: (A_bldg)*(w_floor_D + 0.25*w_floor_L + p_slab + p_col) + P_beams + y_conc*h_col*A_col*(N_col)	7009.36	kip	
W_typ	Total weight acting on podium: W_wall_typ + W_floor_typ	9284.36	kip	
3.3 Calculations for Roof				
h_trib_roof	Tributary height which includes bottom half of Story 8	6.25	ft	
W_wall_roof	Wall weight acting on the roof: L_total*h_trib_roof*p_wall	1137.50	kip	
W_floor_roof	Floor weight acting on the roof: (A_bldg)*(w_roof_D + 0.25*w_roof_L + p_slab) + P_beams	6589.17	kip	
W_roof	Total Weight acting on Roof: W_wall_roof + W_floor_roof	7726.67	kip	
3.4 Total Seismic Weight				
W	Total seismic weight: W_1 + 6*W_typ + W_roof	73300.22	kip	
3.5 Fundamental Period and Seismic Base Shear				
T_a	Fundamental Approximate Period determined using ASCE 7-16 Section 12.8.2.1	0.658	s	
C_s,eq	Seismic response coeffecient determined using Section 12.8.1.1	0.278	—	
C_s,max	Maximum allowable limit (T_a < T_L)	0.289	—	
C_s	Governing C_s	0.278	—	
V	Design seismic base shear: C_s*W	20377.46	kip	
3.6 Vertical Distribution of Seismic Forces				
		h_i (ft)	**W_i (kip)**	
Level 8 (Roof)		105.50	7726.67	
Level 7		93.00	9284.36	
Level 6		80.50	9284.36	
Level 5		68.00	9284.36	
Level 4		55.50	9284.36	
Level 3		43.00	9284.36	
Level 2		30.50	9284.36	
Level 1 (Podium)		18.00	9867.36	
k	Distribution exponent related to building's fundamental period: k = 1 for T_a < 0.5: (1.5 + T_a)/2	1.079	—	
∑W_i*h_i^k	Denominator for the lateral seismic force equation, F_x	6195242.41	kip-ft	
		F_i (kip)	**V_i (kips)**	**M_i (kip-ft)**
F_8	Lateral seismic force for Level 8: ((W_8*h_8^k)/∑W_8*h_8^k)*V	3877.45	3877.45	0.00
F_7	Lateral seismic force for Level 7: ((W_7*h_7^k)/∑W_7*h_7^k)*V	4066.31	7943.76	48468.16
F_6	Lateral seismic force for Level 6: ((W_6*h_6^k)/∑W_6*h_6^k)*V	3479.76	11423.52	147765.16
F_5	Lateral seismic force for Level 5: ((W_5*h_5^k)/∑W_5*h_5^k)*V	2900.41	14323.93	290559.16
F_4	Lateral seismic force for Level 4: ((W_4*h_4^k)/∑W_4*h_4^k)*V	2329.47	16653.40	469608.26
F_3	Lateral seismic force for Level 3: ((W_3*h_3^k)/∑W_3*h_3^k)*V	1768.71	18422.12	677775.78
F_2	Lateral seismic force for Level 2: ((W_2*h_2^k)/∑W_2*h_2^k)*V	1220.89	19643.01	908052.23
F_1	Lateral seismic force for Level 1: ((W_1*h_1^k)/∑W_1*h_1^k)*V	734.45	20377.46	1153589.83

(B3) Drift Calculations

[SE 140B] Project 3

Codes: ACI 318-19, ASCE 7-22

Phase C: Hand Calculations

1. Initial Inputs

1.1 Building Geometry and PID-based Dimensions

h_1	Podium Floor Height	18	*ft*	
h_2-8	Story Floor Height	12.5	*ft*	
l_w, EW	Wall's length (EW direction): 30'-i" (i = 1st PID digit)	30.5	*ft*	
l_w, NS	Wall's length (NS direction): 30'-j" (j = 2nd PID digit)	30.167	*ft*	
L_bldg	Total length of the building	211.167	*ft*	
B_bldg	Total width of the building	156	*ft*	

1.2 Concrete and Steel Material Properties

f'c	Compression Resistance of Concrete: [ksi — from PID digit k]	9	*ksi*	
f_y = f_yt	Yield Resistance of Steel bars	60	*ksi*	
γ_c	Weight per unit volume	0.150	*kcf*	
E_c	Young's Modulus: Ec = 57√f'c (ksi)	5407.49	*ksi*	
E_v	Concrete Shear Modulus of Elasticity: 0.4 * Ec	2163.00	*ksi*	

1.3 Wall and Section Preliminary Dimensions

t_w	Wall Thickness	30	*in*	**(as a first trial)**
column	Column Dimensions	30	*in^2*	
t_s	Slab Thickness	8	*in*	

2. Load Calculations

2.1 Effective Height

h_eff	h_efff = M_base / V_base	679.33	*in*

2.2. Section Properties of a EW Wall

Ig, EW	t_w * (l_w ^ 3)/12	122569740.00	*in^4*
Av, EW	t_w * l_w	10980.00	*in^2*
Icr, EW		61284870.00	*in^4*
ke, EW	1 / (h_eff^3 / (3 * Ec * Icr) + h_eff / (A_v * E_v))	2907.46	*kip/in*

2.2. Section Properties of a NS Wall

Ig, NS	t_w * (l_w ^ 3)/12	118594820.00	*in^4*
Av, NS	t_w * l_w	10860.00	*in^2*
Icr, NS	I_cr = (100/fy + Pu / (f'c*Ag)) * Ig	59297410.00	*in^4*
ke, NS	1 / (h_eff^3 / (3 * Ec * Icr) + h_eff / (A_v * E_v))	2818.27	*kip/in*

3. Shear Distribution

3.1 Direct Shear Distribution

Vu_wall Vu, global / 8

Mu_wall Mu, global / 8

Floor	**h_i (in)**	**F_wall (kip)**	**Vu_wall (kips)**	**Mu_wall (kip-ft)**
8	1266	484.68	484.68	0.00
7	1116	508.29	992.97	6058.52
6	966	434.97	1427.94	18470.65
5	816	362.55	1790.49	36319.90
4	666	291.18	2081.68	58701.03
3	516	221.09	2302.76	84721.97
2	366	152.61	2455.38	113506.53
1	216	91.81	2547.18	144198.73

4. EW Deflection at Floor # due to Floor $

δ_elastic	Elastic Deflection =IF(OR(x < h_j, x = h_j), (F_j * x^2 * (3*h_j - x)) / (6 * E_c * I_cr) + (F_j * x) / (A_v * E_v), (F_j * h_j^2 * (3*x - h_j)) / (6 * E_c * I_cr) + (F_j * h_j) / (A_v * E_v))
δ_inelastic	Inelastic Deflection = δ_elastic * Cd / Ie
Δ	Story Drift = δ_inelastic_i - δ_inelastic_i-1
Story Drift Ratio (%)	**Δ/ h * 100%**
Δa	Story Drift Limit = 0.02 * h_sx

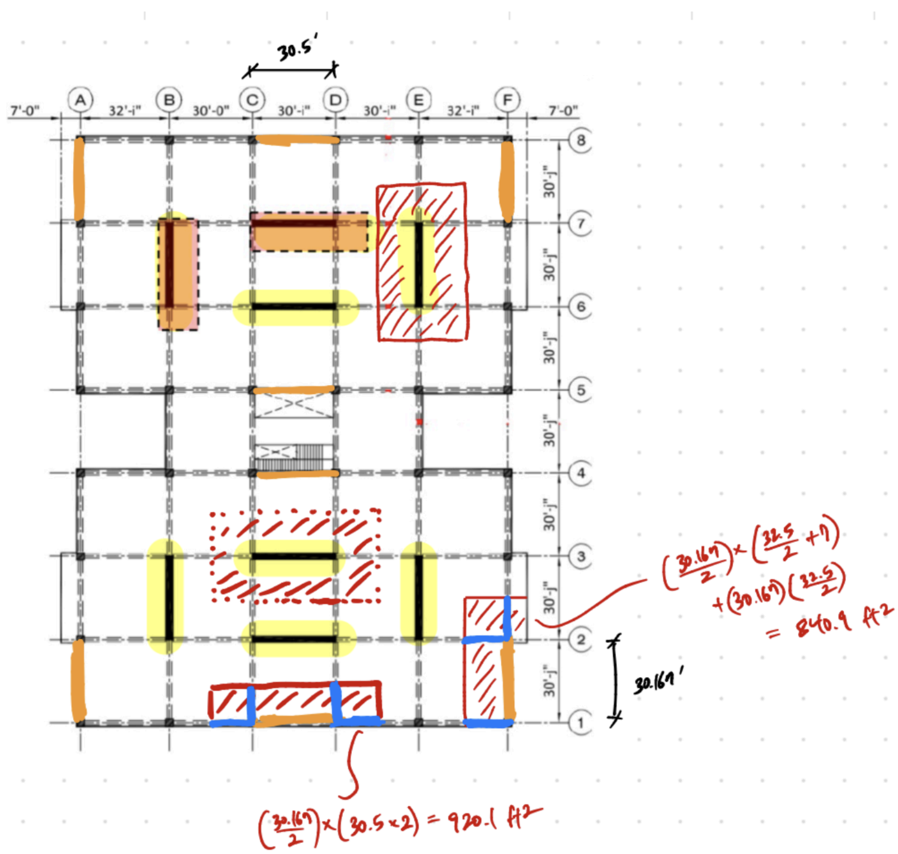
4.1 EW Deflection at Floor # due to Floor $

Due to Floor	**8**	**7**	**6**	**5**	**4**	**3**	**2**	**1**	*in*
	1266	**1116**	**966**	**81(**	**666**	**516**	**366**	**216**	*in*
8	1.015	0.837	0.664	0.5	0.352	0.224	0.120	0.045	*in*
7	0.878	0.734	0.589	0.4	0.318	0.204	0.110	0.042	*in*
6	0.596	0.504	0.412	0.3	0.229	0.148	0.081	0.031	*in*
5	0.374	0.320	0.265	0.2	0.154	0.102	0.056	0.022	*in*
4	0.212	0.182	0.153	0.1	0.095	0.064	0.037	0.015	*in*
3	0.102	0.089	0.075	0.0	0.049	0.035	0.021	0.009	*in*
2	0.038	0.033	0.028	0.0	0.019	0.015	0.010	0.005	*in*
1	0.009	0.008	0.007	0.0	0.005	0.004	0.003	0.002	*in*
Total δ at Floor	3.223	2.707	2.194	1.6	1.221	0.795	0.437	0.171	*in*

4.2 NS Deflection at Floor # due to Floor $

Due to Floor	**8**	**7**	**6**	**5**	**4**	**3**	**2**	**1**	*in*
	1266	**1116**	**966**	**81(**	**666**	**516**	**366**	**216**	*in*
8	1.015	0.864	0.685	0.5	0.364	0.231	0.123	0.047	*in*
7	0.878	0.734	0.608	0.4	0.329	0.210	0.113	0.043	*in*
6	0.596	0.504	0.412	0.3	0.236	0.153	0.083	0.032	*in*
5	0.374	0.320	0.265	0.2	0.159	0.105	0.058	0.023	*in*
4	0.212	0.182	0.153	0.1	0.095	0.066	0.038	0.015	*in*
3	0.102	0.089	0.075	0.0	0.049	0.035	0.022	0.009	*in*
2	0.038	0.033	0.028	0.0	0.019	0.015	0.010	0.005	*in*
1	0.009	0.008	0.007	0.0	0.005	0.004	0.003	0.002	*in*
Total δ at Floor	3.223	2.734	2.234	1.7	1.254	0.818	0.450	0.176	*in*

4.3 EW Deflection Limit Check								
Floor	**h_i (ft)**	**δ_elastic**	**δ_inelastic**	**Δ**	**Δa**	**Story Drift Ratio (%)**	**DCR: Δ < Δa**	**DCR Status**
8	105.5	3.223	16.114	2.5	3	1.72%	0.860	G
7	93	2.707	13.535	2.5	3	1.71%	0.856	G
6	80.5	2.194	10.968	2.5	3	1.67%	0.834	G
5	68	1.693	8.466	2.3	3	1.57%	0.787	G
4	55.5	1.221	6.104	2.1	3	1.42%	0.710	G
3	43	0.795	3.974	1.7	3	1.19%	0.597	G
2	30.5	0.437	2.185	1.3	3	0.89%	0.444	G
1	18	0.171	0.854	0.8	4.32	0.40%	0.198	G
4.4 NS Deflection Limit Check								
Floor	**h_i (ft)**	**δ_elastic**	**δ_inelastic**	**Δ**	**Δa**	**Story Drift Ratio (%)**	**DCR: Δ < Δa**	**DCR Status**
8	105.5	3.223	16.114	2.4	3	1.63%	0.814	G
7	93	2.734	13.671	2.5	3	1.67%	0.833	G
6	80.5	2.234	11.171	2.5	3	1.67%	0.834	G
5	68	1.734	8.670	2.3	3	1.60%	0.799	G
4	55.5	1.254	6.272	2.1	3	1.45%	0.727	G
3	43	0.818	4.091	1.8	3	1.23%	0.614	G
2	30.5	0.450	2.250	1.3	3	0.91%	0.457	G
1	18	0.176	0.879	0.8	4.32	0.41%	0.203	G



5. Governing Load Combinations and Ultimate Demands				
5.1 Unfactored Gravity Loads				
		EW	**NS**	
A_trib	Assumed tributary area	920.1	840.9	*ft^2*
beam_length	Tributary Beam Length: 4 for EW, 3 for NS (conservative)	60.67	47.58	*ft*
D_floors	A_trib * ((8 floors * 30 psf) / 1000	220.82	220.82	*kips*
D_slab	A_trib * (8 floors) * (8in /12) * (0.15 kcf)	736.07	672.72	*kips*
D_wall	L_wall * H_wall * (Wall Area Load) / 1000	1206.66	1193.47	*kips*
D_beams	(A_beam)*(y_conc)*(beam_length) * (8 floors)	436.80	342.60	
D_total	D_floors + D_slab + D_wall	2600.36	2429.62	*kips*
L_floors	A_trib * ((7 floors * 50 psf) / 1000	322.032725	322.032725	*kips*
L_roof	A_trib * ((20 psf) / 1000	18.40	18.40	*kips*
L_total	L_floors + L_roof	340.43	340.43	*kips*
5.2 Seismic Loads				
Eh	p*Q_E1	2547.18	2547.18	*kips*
Ev	0.2*SDS*D	722.90	675.43	*kips*
5.2 Max Compression				
Vu_max		2547.18	2547.18	*kips*
Mu_max		144198.73	144198.73	*kip-ft*
Pu_max	1.2D + Ev + 0.5L	4013.54	3761.19	*kips*
5.2 Min Compression				
Vu_min	Eh = p*Q_E1	2547.18	2547.18	*kips*
Mu_min		144198.73	144198.73	*kip-ft*
Pu_min	0.9D - Ev	1617.42	1511.22	*kips*

(B4) P-M Interaction Diagrams

EW Diagram

Rectangular Wall Interaction Diagram Spreadsheet

Calculations are based on the Fiber Discretization Numerical Solution of Rectangular Stress Block Flexure Theory Introduced in SE151A
Note: at any position of the Neutral Axis Depth, the difference between the Tensile and Compressive forces equal the external axial force
Axial force acts at the concrete section centroid y_g

1. Material properties

1 Input the specified material strengths

f'_c 9000 psi β_1 0.65
f_y 60000 psi E_s 29000 ksi ε_y 0.00207

2. Section diameter and Long bar layout

2 Input the wall length lw, width b and a trial for the ratio of the boundary element to wall length

l_w 366.0 in. l_b/l_w 0.21
b 30.0 in. ACI 318: l_b/l_w equal or greater than 0.15

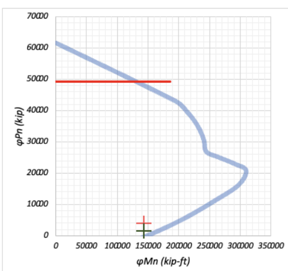
3. Section Analysis - PM Interaction Diagram

3 Input a trial reinforcement ratio for the wall ends where boundary elements are defined, and in the web (in between boundary elements)

Notes:
(i) ρ_ℓ is the total area of wall longitudinal reinforcement A_{st} divided by the wall gross section area l_w b
(ii) ρ_w is the total area of longitudinal reinforcement in the web A_{sw} divided by the web gross section area $(l_w - 2l_b)$ b
(ii) ρ_{be} is the total area of longitudinal reinforcement in a boundary element A_{sbe} divided by the gross section area of the boundary element (l_b/l_w) l_w b
(iii) $A_{st} = 2A_{be} + A_{sw}$
(iv) RECOMMENDATION make $\rho_{be} \leq 0.025$ to avoid congestion
(v) MANDATORY make $\rho_{be} \geq \rho_w$ (it makes no sense to have the web with denser longitudinal reinforcement than the boundary elements)
(vi) MANDATORY $\rho_\ell \geq 0.0025$ and $\rho_w \geq 0.0025$ (ACI 318 minimum wall longitudinal reinforcement)

ρ_ℓ 0.0190 ρ_w 0.0150 ρ_{be} 0.0245
A_{st} 208.6 in.^{2} A_{sw} 95.526 in.^{2} A_{sbe} 56.5 in.^{2}

Figure 1: PM - Interaction Diagram



a/lw	c/lw	ϕ	ϕP_n (kip)	ϕM_n (kip-ft)
0.05	0.059	0.90	-6048	72079
0.1	0.118	0.90	754	157481
0.15	0.176	0.90	5717	210661
0.2	0.235	0.90	11439	260557
0.25	0.294	0.90	16266	297516
0.3	0.353	0.87	20872	309593
0.35	0.412	0.77	22984	291196
0.4	0.471	0.71	25039	265832
0.45	0.529	0.65	26861	244863
0.5	0.588	0.65	30354	241088
0.55	0.647	0.65	33741	234522
0.6	0.706	0.65	37019	224541
0.65	0.765	0.65	39975	212021
0.7	0.824	0.65	43102	194740
∞			61697	0
ϕP_o			61697	0
ϕ$P_{n,max}$			49357	0

Figure 2: strength reduction factor ϕ vs φP_n diagram

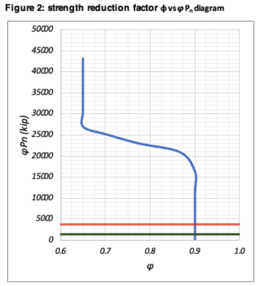
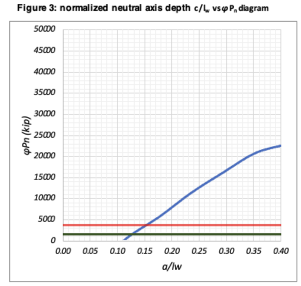


Figure 3: normalized neutral axis depth c/l_w vs φP_n diagram



3. Design

4 Input Pu max and Mu and change pl in cell H36 till the blue cross intersects the interaction diagram
5 Input Pu min and Mu and change pl in cell H36 till the green cross intersects the interaction diagram

Required strength	P_u (kip)	M_u (kip-ft)	ρ_ℓ	ρ_w	ρ_{be}	ϕ	c/l_w
$(1.2+0.2S_{DS})D+\rho Q_E+L+0.2S$	3761.19	144198.7	0.0190	0.0150	0.0245	0.9	0.256
$(0.9-0.2S_{DS})D+\rho Q_E$	1511.22	144198.7	0.0190	0.0150	0.0245	0.9	0.232
			Copy from C36, F36 and I36			Obtain from Figs 2 & 3	

5. Concrete fibers

Number of fibers 20

Fiber thickness 18.30 in.

Fiber	y-distance frombase (in.)	Width (in.)	Area (in.^{2})	A.y (in.^{3})
1	9.15	30.00	549.0	5023
2	27.45	30.00	549.0	15070
3	45.75	30.00	549.0	25117
4	64.05	30.00	549.0	35163
5	82.35	30.00	549.0	45210
6	100.65	30.00	549.0	55257
7	118.95	30.00	549.0	65304
8	137.25	30.00	549.0	75350
9	155.55	30.00	549.0	85397
10	173.85	30.00	549.0	95444
11	192.15	30.00	549.0	105490
12	210.45	30.00	549.0	115537
13	228.75	30.00	549.0	125584
14	247.05	30.00	549.0	135630
15	265.35	30.00	549.0	145677
16	283.65	30.00	549.0	155724
17	301.95	30.00	549.0	165771
18	320.25	30.00	549.0	175817
19	338.55	30.00	549.0	185864
20	356.85	30.00	549.0	195911

A_g 10980 in.^{2}

$\Sigma A.y$ 2009340 in.^{3}

y_g 183 in.

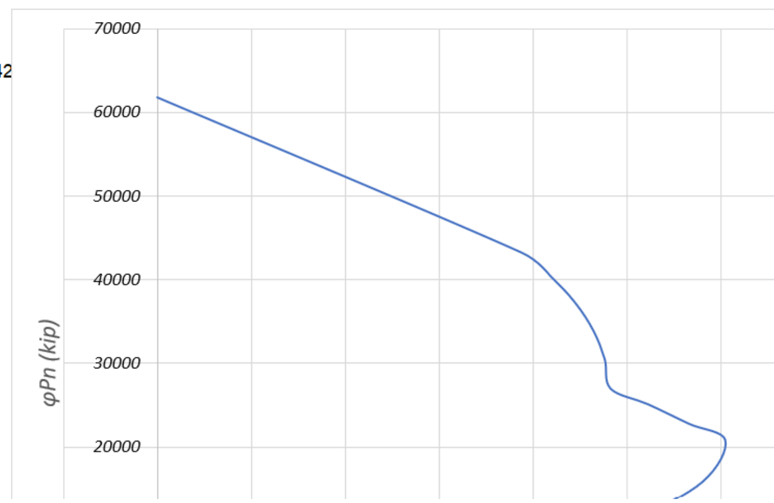
6. Reinforcement fibers: Areas and Location

Fiber	y-distance frombase (in.)	Area (in.^{2})	A.y (in.^{3})
1	38.4	56.55	2173
2	129.9	47.76	6206
3	236.1	47.76	11275
4	327.6	56.55	18523

Interaction Diagram with ACI Sign Convention: (+) Compression

4013.54

a/H	c/H	φP_n (kip)	φM_n (kip·ft)	
0.05	0.06	-5557	80490	Pu = 1617.42
0.1	0.12	-338	147827	c/lw =
0.15	0.18	5271	204814	0.16328
0.2	0.24	10994	253343	c =
0.25	0.29	15852	290575	59.7615
0.3	0.35	20771	302480	
0.35	0.41	22746	283156	
0.4	0.47	25090	261247	
0.45	0.53	26942	241423	
0.5	0.59	30462	238287	
0.55	0.65	33872	232241	
0.6	0.71	37169	222692	
0.65	0.76	40378	209488	
0.7	0.82	43194	194109	
∞		61753	0	
P_o		61753	0	



NS Diagram

1. Material properties

1 Input the specified material strengths

f'_c 9000 psi β_1 0.65

f_y 60000 psi E_s 29000 ksi ε_y 0.00207

2. Section diameter and Long bar layout

2 Input the wall length lw, width b and a trial for the ratio of the boundary element to wall length

l_w 362.0 in. l_b/l_w 0.17

b 30.0 in.   ACI 318: l_b/l_w equal or greater than 0.15

3. Section Analysis - PM Interaction Diagram

3 Input a trial reinforcement ratio for the wall ends where boundary elements are defined, and in the web (in between boundary elements)

Notes:

(i) ρ_ℓ is the total area of wall longitudinal reinforcement A_{st} divided by the wall gross section area l_w **b**

(ii) ρ_w is the total area of longitudinal reinforcement in the web A_{sw} divided by the web gross section area (l_w - 2l_b) **b**

(ii) ρ_{be} is the total area of longitudinal reinforcement in a boundary element A_{sbe} divided by the gross section area of the boundary element (l_b/l_w) l_w **b**

(iii) $A_{st} = 2A_{be} + A_{sw}$

(iv) RECOMMENDATION make $\rho_{be} \leq$ **0.025** to avoid congestion

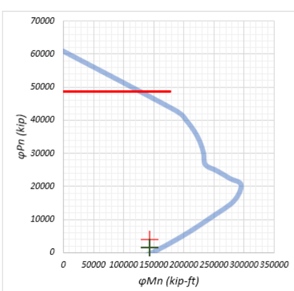
(v) MANDATORY make $\rho_{be} \geq \rho_w$ (it makes no sense to have the web with denser longitudinal reinforcement than the boundary elements)

(vi) MANDATORY $\rho_\ell \geq$ **0.0025** and $\rho_w \geq$ **0.0025** (ACI 318 minimum wall longitudinal reinforcement)

ρ_ℓ 0.0189 ρ_w 0.0172 ρ_{be} 0.0222

A_{st} 204.8 in.^{2} A_{sw} 124.7979 in.^{2} A_{sbe} 40.0 in.^{2}

Figure 1: PM - Interaction Diagram



a/lw	c/lw	φ	φP_n (kip)	φM_n (kip-ft)
0.05	0.059	0.90	-5403	77702
0.1	0.118	0.90	-253	143398
0.15	0.176	0.90	5256	198856
0.2	0.235	0.90	10909	246314
0.25	0.294	0.90	15705	282658
0.3	0.353	0.87	20559	294367
0.35	0.412	0.77	22500	275710
0.4	0.471	0.71	24800	254524
0.45	0.529	0.65	26618	235351
0.5	0.588	0.65	30087	232396
0.55	0.647	0.65	33449	226572
0.6	0.706	0.65	36701	217306
0.65	0.765	0.65	39868	204454
0.7	0.824	0.65	42653	189452
∞			60970	0
ϕP_o			60970	0
$\phi P_{n,max}$			48776	0
			48776	176620.25

Figure 2: strength reduction factor φ vs φ P_n diagram

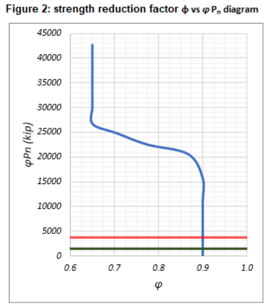
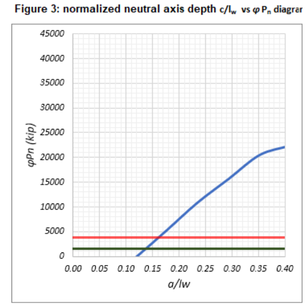


Figure 3: normalized neutral axis depth c/l_w vs φ P_n diagrar



48776 176620

3. Design

4 Input Pu max and Mu and change pl in cell H36 till the blue cross intersects the interaction diagram

5 Input Pu min and Mu and change ρl in cell H36 till the green cross intersects the interaction diagram

Required strength

$(1.2+0.2S_{DS})D+\rho Q_E+L+0.2S$

$(0.9-0.2S_{DS})D+\rho Q_E$

P_u (kip)	M_u (kip-ft)	ρ_ℓ	ρ_w	ρ_{be}	φ	c/l_w
3761.19	144199	0.0189	0.0172	0.0222	0.9	0.256
1511.22	144199	0.0189	0.0172	0.0222	0.9	0.232
		Copy from C36, F36 and I36			Obtain from Figs 2 & 3	

5. Concrete fibers

Number of fibers 20

Fiber thickness 18.10 in.

Fiber	y-distance from base (in.)	Width (in.)	Area ($in.^2$)	A.y ($in.^3$)
1	9.05	30.00	543.0	4914
2	27.15	30.00	543.0	14742
3	45.25	30.00	543.0	24571
4	63.35	30.00	543.0	34399
5	81.45	30.00	543.0	44227
6	99.55	30.00	543.0	54056
7	117.65	30.00	543.0	63884
8	135.75	30.00	543.0	73712
9	153.85	30.00	543.0	83541
10	171.95	30.00	543.0	93369
11	190.05	30.00	543.0	103197
12	208.15	30.00	543.0	113025
13	226.25	30.00	543.0	122854
14	244.35	30.00	543.0	132682
15	262.45	30.00	543.0	142510
16	280.55	30.00	543.0	152339
17	298.65	30.00	543.0	162167
18	316.75	30.00	543.0	171995
19	334.85	30.00	543.0	181824
20	352.95	30.00	543.0	191652

A_g 10860 $in.^2$

$\Sigma A.y$ 1965660 $in.^3$

y_g 181 in.

6. Reinforcement fibers: Areas and Location

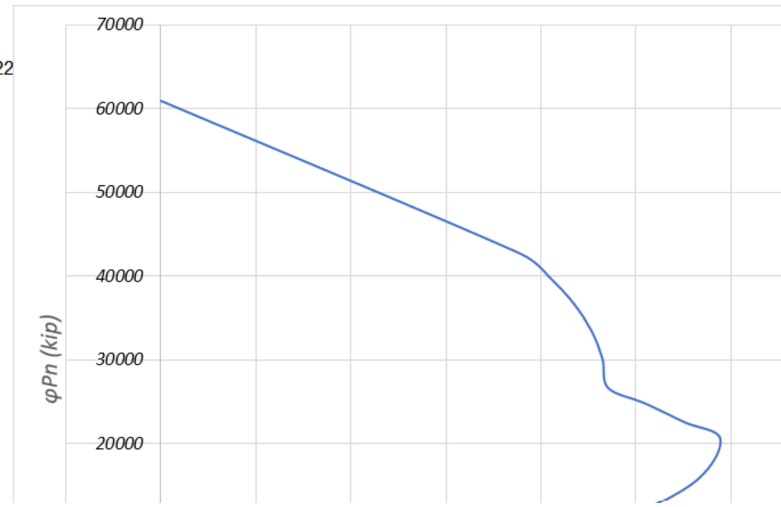
Fiber	y-distance from base (in.)	Area ($in.^2$)	A.y ($in.^3$)
1	30.0	40.00	1200
2	120.5	62.40	7519
3	241.5	62.40	15069
4	332.0	40.00	13280

$\Sigma A.y$ 37068 $in.^3$

y_s 181.0 in.

Interaction Diagram with ACI Sign Convention: (+) Compression

a/H	c/H	φPn (kip)	φMn (kip-ft)	
				3761.19
				Pu = 1511.22
0.05	0.06	-5403	77702	c/lw =
0.1	0.12	-253	143398	0.16051
0.15	0.18	5256	198856	
0.2	0.24	10909	246314	c =
0.25	0.29	15705	282658	58.7453
0.3	0.35	20559	294367	
0.35	0.41	22500	275710	
0.4	0.47	24800	254524	
0.45	0.53	26618	235351	
0.5	0.59	30087	232396	
0.55	0.65	33449	226572	
0.6	0.71	36701	217306	
0.65	0.76	39868	204454	
0.7	0.82	42653	189452	
∞		60970	0	
P_o		60970	0	



EW Mpr Diagram

1 Input the specified material strengths

1. Material properties

f'_c	9000	psi	β_1	0.65			
f_y	60000	psi	E_s	29000 ksi		ε_y	0.00259
f_{ypr}	75000	psi					

of the joint. Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcement with an effective yield stress equal to at least **1.25**f_y. Distribution of

2 Input the wall length l_w, width b and the boundary element to wall length ratio (as detailed)

2. Section diameter and Long bar layout

l_w	366.0	in.	l_b/l_w	0.16
b	30.0	in.		

3 Input the boundary element (one side only) and web reinforcement areas

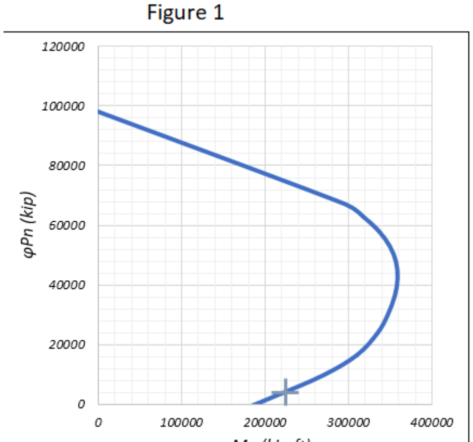
3. Reinforcement : Areas and Location

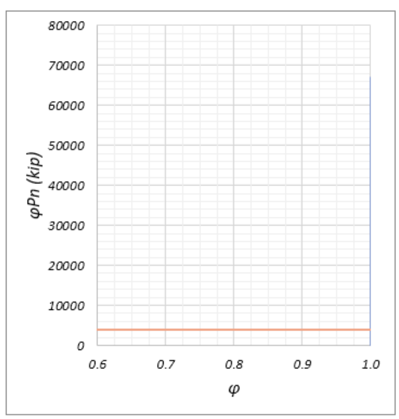
Asbe	40.00	in2	**Asw**	127.92	in2
ρ_{be}	0.022		ρ_w	0.017	

4 Input P_{umax} and vary ϕM_{pr} until the cross intersects the interaction diagram, then read the strength reduction factor in Fig 2 and enter the value in cell D33
Cell E33 will give you the M_{pr} as intended in ACI 318-19

P_{umax} (kip)	φM_{pr} (kip-ft)	φ	M_{pr} (kip-ft)
4013.54	225000	1	225000

Figure 1





a/lw	c/lw	φ	φP_n (kip)	φM_n (kip-ft)
0.05	0.059	1.00	-9872	80247
0.1	0.118	1.00	-4823	145524
0.15	0.176	1.00	2738	213371
0.2	0.235	1.00	9673	270314
0.25	0.294	1.00	16407	310151
0.3	0.353	1.00	23766	334513
0.35	0.412	1.00	29896	346701
0.4	0.471	1.00	36186	355018
0.45	0.529	1.00	41991	358369
0.5	0.588	1.00	47474	356566
0.55	0.647	1.00	52725	349269
0.6	0.706	1.00	57800	336249
0.65	0.765	1.00	62740	317350
0.7	0.824	1.00	66948	295673
∞			98000	0
ϕP_o			98000	0
$\phi P_{n,max}$			78400	0

NS Mpr Diagram

1 Input the specified material strengths

1. Material properties

f'_c	9000	psi	β_1	0.65		
f_y	60000	psi	E_s	29000 ksi	ε_y	0.00259
f_{ypr}	75000	psi				

of the joint. Moment strengths are to be determined using a strength reduction factor of 1.0 and reinforcement with an effective yield stress equal to at least $1.25f_y$. Distribution of

2 Input the wall length l_w, width b and the boundary element to wall length ratio (as detailed)

2. Section diameter and Long bar layout

l_w	362.0	in.	l_b/l_w	0.17
b	30.0	in.		

3 Input the boundary element (one side only) and web reinforcement areas

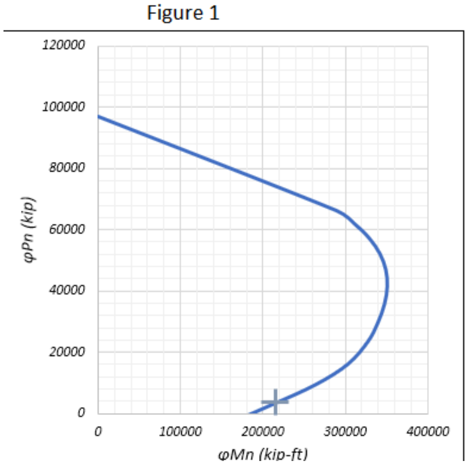
3. Reinforcement : Areas and Location

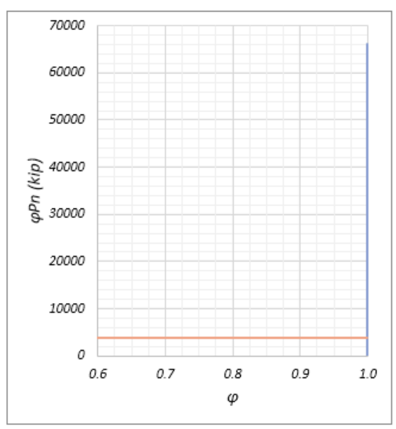
Asbe	40.00	in2	Asw	124.80	in2
ρ_{be}	0.022		ρ_w	0.017	

4 Input P_{umax} and vary ϕM_{pr} until the cross intersects the interaction diagram, then read the strength reduction factor in Fig 2 and enter the value in cell D33
Cell E33 will give you the M_{pr} as intended in ACI 318-19

P_{umax} (kip)	ϕM_{pr} (kip-ft)	φ	M_{pr} (kip-ft)
3761.19	215800	1	215800

Figure 1





a/lw	c/lw	φ	φP_n (kip)	φM_n (kip-ft)
0.05	0.059	1.00	-9722	78193
0.1	0.118	1.00	-4716	142175
0.15	0.176	1.00	2722	208285
0.2	0.235	1.00	9577	263907
0.25	0.294	1.00	16246	302792
0.3	0.353	1.00	23509	326716
0.35	0.412	1.00	29567	338705
0.4	0.471	1.00	35783	346938
0.45	0.529	1.00	41517	350261
0.5	0.588	1.00	46935	348534
0.55	0.647	1.00	52123	341424
0.6	0.706	1.00	57139	328712
0.65	0.765	1.00	62023	310245
0.7	0.824	1.00	66185	289023
∞			96872	0
ϕP_o			96872	0
$\phi P_{n,max}$			77498	0

(B5) Boundary Element Calculations

[SE 140B] Project 3

Codes: ACI 318-19, ASCE 7-22

Phase E: Hand Calculations

1. Initial Inputs

1.1 Building Geometry and PID-based Dimensions

h_1	Podium Floor Height	18	*ft*	
h_2-8	Story Floor Height	12.5	*ft*	
l_w, EW	Wall's length (EW direction): 30′-i″ (i = 1st PID digit)	30.5	*ft*	
l_w, NS	Wall's length (NS direction): 30′-j″ (j = 2nd PID digit)	30.167	*ft*	
L_bldg	Total length of the building	211.167	*ft*	
B_bldg	Total width of the building	156	*ft*	

1.2 Concrete and Steel Material Properties

f'c	Compression Resistance of Concrete: [ksi — from PID digit k]	9	*ksi*	
f_y = f_yt	Yield Resistance of Steel bars	60	*ksi*	
γ_c	Weight per unit volume	0.150	*kcf*	
E_c	Young's Modulus: Ec = 57√f'c (ksi)	5407.49	*ksi*	
ection Prelimina		2163.00	*ksi*	

1.3 Wall and Section Preliminary Dimensions

t_w	Wall Thickness	30	*in*	**(as a first trial)**
column	Column Dimensions	30	*in^2*	
t_s	Slab Thickness	8	*in*	

2. Final Neutral Axis Depth

c_NS	Neutral Axis Depth - NS Direction	58.7453	*in*
c_EW	Neutral Axis Depth - EW Direction	59.7154	*in*

3. Deflection & SBE Check

δ_xe	Elastic deflection at floor
δ_u	Inelastic Deflection

3.1 NS Direction

	h_i (ft)	δ_elastic	δ_inelastic	1.5*δ_u / h_w	l_w / 600*c	1.5*δ_u / h_w > l_w / 600*c	
Roof	105.5	3.223	16.114	0.0190927851	0.01027032517	**TRUE**	**SBE Required**

3.2 EW Direction

	h_i (ft)	δ_elastic	δ_inelastic	1.5*δ_u / h_w	l_w / 600*c	1.5*δ_u / h_w > l_w / 600*c	
Roof	105.5	3.223	16.114	0.0190927851	0.01021512039	**TRUE**	**SBE Required**

4 Length of Special Boundary Element (l_be)

l_be,req	l_be,req (in) = max(c - 0.1*l_w, c/2)	
	l_be,req (in)	**l_be < trial (75")**
NS Direction	29.37265	**TRUE**
EW Direction	29.8577	**TRUE**

5. Transverse Reinforcement Details						
5.1 Maximum Hoop Spacing at Boundary Elements (s)						
A_b	Area of #6 hoops			0.44	*in^2*	
d_b	Diameter of #6 hoops			0.75	*in*	
	t_w / 3 (in)	**6*d_b (in)**	**s = min(t_w/3, 6*d_b)**			
NS Direction	10	4.5	4.5			
EW Direction	10	4.5	4.5			
5.2 Transverse Reinforcement Required Area (A_sh)						
b_c	Cross-Sectional Core = t_w - 2(cover)			27	*in*	
A_sh	Required Cross-Section Area of Reinforcements > 0.09*s*b_c*(f'c / f_yt)					
	A_sh (in^2)	**Required Legs**	**Actual Amount of Legs (#)**	**4 legs of #4** transverse reinforcement at a vertical spacing of **4.5" center to center** throughout the Special Bounday Elements		
NS Direction	1.64025	3.727840909	**4**			
EW Direction	1.64025	3.727840909	**4**			
A_SBE	Actual Area of Steel Reinforcements (for both NS and EW)			1.76	*in^2*	

(B6) Shear Calculations

[SE 140B] Project 3					
Codes: ACI 318-19, ASCE 7-22					
Phase E: Hand Calculations					
1. Initial Inputs					
1.1 Building Geometry and PID-based Dimensions					
h_1	Podium Floor Height		18	*ft*	
h_2-8	Story Floor Height		12.5	*ft*	
l_w, EW	Wall's length (EW direction): 30'-i" (i = 1st PID digit)		30.5	*ft*	
l_w, NS	Wall's length (NS direction): 30'-j" (j = 2nd PID digit)		30.167	*ft*	
L_bldg	Total length of the building		211.167	*ft*	
B_bldg	Total width of the building		156	*ft*	
1.2 Concrete and Steel Material Properties					
f'c	Compression Resistance of Concrete: [ksi — from PID digit k]		9	*ksi*	
f_y = f_yt	Yield Resistance of Steel bars		60	*ksi*	
γ_c	Weight per unit volume		0.150	*kcf*	
E_c	Young's Modulus: Ec = 57√f'c (ksi)		5407.49	*ksi*	
ection Prelimina			2163.00	*ksi*	
1.3 Wall and Section Preliminary Dimensions					
t_w	Wall Thickness		30	*in*	**(as a first trial)**
column	Column Dimensions		30	*in^2*	
t_s	Slab Thickness		8	*in*	
1.4 Ultimate Demands					
NS	**V_u**	Max Shear	2547.18	*kips*	
	M_u	Max Moment	144198.73	*kip*ft*	
EW	**V_u**	Max Shear	2547.18	*kips*	
	M_u	Max Moment	144198.73	*kip*ft*	
2. Dynamic Shear Amplification Factor					
2.1 NS Flexural Overstrength Factor (Ω_v)					
M_pr	Probable Flexural Strength		215800	*kip*ft*	
Ω_v	Overstrength Factor = max(M_pr / M_u; 1.5)		1.5		
2.2 EW Flexural Overstrength Factor (Ω_v)					
M_pr	Probable Flexural Strength		225000	*kip*ft*	
Ω_v	Overstrength Factor = max(M_pr / M_u; 1.5)		1.560346938		
2.3 Amplification Factor (ω_v)					
n_s	Number of stories		8	*stories*	
ω_v	= 0.9 + n_s/10 ;		1.7		
2.3 Design Shear Force					
NS	**V_e**	Amplified Design Shear = Ω_v*ω_v*V_u < 3*V_u	6495.315468	*kips*	
EW	**V_e**	Amplified Design Shear = Ω_v*ω_v*V_u < 3*V_u	6756.630401	*kips*	

4. Distributed Web Reinforcement Design				
4.1 NS Shear Reinforcements				
V_s	Steel Shear Reinforcement = V_e/Φ - V_c	6599.88	*kips*	
ρ_t,req	Required Transverse Reinforcement Ratio = V_s / (A_cv*f_y)	0.010129	*in*	greater than ρ_t,min = 0.0025
s_req	Required Spacing from Steel Ratio Req = A_v / (ρ_t,req * t_w)	10.268	*in*	
s_max	Maximum Spacing = min(18", 3*t_w, l_w/5)	18	*in*	
s = 6" < s_req = 10.3" < s_max = 18"				
ρ_t,prov	Provided Transverse Reinforcement Ratio = A_v / (s * t_w)	0.0173	> 0.0025	**OK**
2 curtains of #11 bars at 6" c.c.				
4.1 EW Shear Reinforcements				
V_s	Steel Shear Reinforcement = V_e/Φ - V_c	6577.11	*kips*	
ρ_t,req	Required Transverse Reinforcement Ratio = V_s / (A_cv*f_y)	0.009983	*in*	greater than ρ_t,min = 0.0025
s_req	Required Spacing from Steel Ratio Req = A_v / (ρ_t,req * t_w)	10.417	*in*	
s_max	Maximum Spacing = min(18", 3*t_w, l_w/5)	18	*in*	
s = 6" < s_req = 10.4" < s_max = 18"				
ρ_t,prov	Provided Transverse Reinforcement Ratio = A_v / (s * t_w)	0.0173	> 0.0025	**OK**
2 curtains of #11 bars at 6" c.c.				

(B7) Reinforcement Schedule and Serviceability

[SE 140B] Project 3				
Codes: ACI 318-19, ASCE 7-22				
Phase G: Reinforcement schedule & serviceability				
1. Initial Inputs				
1.1 Building Geometry and PID-based Dimensions				
h_1	Podium Floor Height	18	*ft*	
h_2-8	Story Floor Height	12.5	*ft*	
l_w, EW	Wall's length (EW direction): 30′-i″ (i = 1st PID digit)	30.5	*ft*	
l_w, NS	Wall's length (NS direction): 30′-j″ (j = 2nd PID digit)	30.167	*ft*	
L_bldg	Total length of the building	211.167	*ft*	
B_bldg	Total width of the building	156	*ft*	
1.2 Concrete and Steel Material Properties				
f'c	Compression Resistance of Concrete: [ksi — from PID digit k]	9	*ksi*	
f_y = f_yt	Yield Resistance of Steel bars	60	*ksi*	
γ_c	Weight per unit volume	0.150	*kcf*	
E_c	Young's Modulus: Ec = 57√f'c (ksi)	5407.49	*ksi*	
ection Prelimina		2163.00	*ksi*	
1.3 Wall and Section Preliminary Dimensions				
t_w	Wall Thickness	30	*in*	**(as a first trial)**
column	Column Dimensions	30	*in^2*	
t_s	Slab Thickness	8	*in*	
1.4 Ultimate Demands				
V_u	Max Shear	2547.18	*kips*	
M_u	Max Moment	144198.73	*kip*ft*	
2. Dynamic Shear Amplification Factor				
2.1 Flexural Overstrength Factor (Ω_v)				
M_pr	Probable Flexural Strength	215800	*kip*ft*	
Ω_v	Overstrength Factor = M_pr / M_u	1.496545786		
2.2 Amplification Factor (ω_v)				
n_s	T_s = S_D1/S_DS	8	*stories*	
ω_v	= 0.9 + n_s/10 ;	1.7		
2.3 Design Shear Force				
V_e	Amplified Design Shear = Ω_v * ω_v * V_u < 3 * V_u	6480.357996	*kips*	
3. Concrete Shear Capacity				
3.1 NS Concrete Shear Capacity				
A_cv	= t_w * l_w	10860.00	*in^2*	
Aspect Ratio	= h_w / l_w	3.50		
α_c	Concrete Shear Coefficient (for h_w/l_w > 2.0) =	2.00		
V_c	Nominal Shear Capacity = α_c * λ * sqrt(f'c) * A_cv	2060.54	*kips*	
Φ V_c	(Shear Strength Reduction Factor Φ = 0.75)	1545.41	*kips*	
3.1 EW Concrete Shear Capacity				
A_cv	= t_w * l_w	10980.00	*in^2*	
Aspect Ratio	= h_w / l_w	3.46		
α_c	Concrete Shear Coefficient (for h_w/l_w > 2.0) =	2.00		
V_c	Nominal Shear Capacity = α_c * λ * sqrt(f'c) * A_cv	2083.31	*kips*	
ΦV_c	(Shear Strength Reduction Factor Φ = 0.75)	1562.48	*kips*	

4. Distributed Web Reinforcement Design				
4.1 NS Shear Reinforcements				
V_s	Steel Shear Reinforcement = V_e/Φ - V_c	6579.94	*kips*	
ρ_t,req	Required Transverse Reinforcement Ratio = V_s / (A_cv*f_y)	0.010098	*in*	greater than ρ_t,min = 0.0025
s_req	Required Spacing from Steel Ratio Req = A_v / (ρ_t,req * t_w)	10.299	*in*	
s_max	Maximum Spacing = min(18", 3*t_w, l_w/5)	18	*in*	
s = 6" < s_req = 10.3" < s_max = 18"				
ρ_t,prov	Provided Transverse Reinforcement Ratio = A_v / (s * t_w)	0.0173	> 0.0025	**OK**
2 curtains of #11 bars at 8" c.c.				
4.1 EW Shear Reinforcements				
V_s	Steel Shear Reinforcement = V_e/Φ - V_c	6557.17	*kips*	
ρ_t,req	Required Transverse Reinforcement Ratio = V_s / (A_cv*f_y)	0.009953	*in*	greater than ρ_t,min = 0.0025
s_req	Required Spacing from Steel Ratio Req = A_v / (ρ_t,req * t_w)	10.449	*in*	
s_max	Maximum Spacing = min(18", 3*t_w, l_w/5)	18	*in*	
s = 6" < s_req = 10.4" < s_max = 18"				
ρ_t,prov	Provided Transverse Reinforcement Ratio = A_v / (s * t_w)	0.0173	> 0.0025	**OK**
2 curtains of #11 bars at 8" c.c.				

5. Reinforcement schedule & serviceability				
5.1 Verify bar spacing limits				
s	Spacing	8	*in*	
d_b	Diameter #11 Bar	1.41	*in*	
s<18	Verify Spacing (NS)	8	< 18	**OK**
s<18	Verify Spacing (EW)	8	< 18	**OK**
s_min	Minimum Spacing = max (d_b, 1 in, 4/3*dagg)	1.41	*in*	
s_clear	Clear Spacing = s - d_b	6.59	*in*	
s_clear = 4.59" > 1.41"				**OK**
5.2 Verify concrete cover				
cover	Concrete Cover	2	*in*	
s_steel/cover	Steel/Conc Space = 2*(cover + d_b,transverse + d_b,longitudinal)	9.64	*in*	
gap	Wall Gap = t_w - s_steel/cover	20.36	*in*	
gap = 20.36" > 0"				**OK**
5.3 Calculate and verify development lengths ℓd for longitudinal bars				
f'c	Compression Resistance of Concrete: [psi — from PID digit k]	9,000	*psi*	
f_y = f_yt	Yield Resistance of Steel bars	60,000	*psi*	
Ψ	Reinforcement Modification Factor	1.0	-	
λ	Concrete Modification Factor	1.0	-	
l_d	development (straigh bars) = (f_y*Ψ)/(20*λ*√f'c)*d_b	44.59	*in*	
l_d = 44.59" > 12"				**OK**
l_dh	development (standard hooks) = (f_y*Ψ)/(55*λ*√f'c)*d_b	16.21	*in*	
l_dh, min	l_dh > max (8*d_b, 6)	11.28	*in*	
l_dh = 16.21" > 11.28"				**OK**
Space	Space = t_w - (2*cover)	26	*in*	
l_d, check	confirm straight bars fit within boundary element = l_d > s	44.59	> 26	**Fail**
l_dh, check	confirm hook bars fit within boundary element = l_dh > s	16.21	< 26	**OK**

APPENDIX C

Team reflections (See Section XX Design brief)

INDIVIDUAL REFLECTION PROMPTS

Isaac Kim

- My primary role in the project was to perform the calculations for Phases C and D, namely the drift calculations, extracting the wall demands at the first story, and performing the initial core wall flexure design. My secondary role involved going back through the earlier seismic calculations to account for any mistakes and correct anything. Due to it being the end of the year, there were moments when responsibilities shifted according to the schedules of other group members. We tried to be as accommodating as possible.
- One technical challenge that our team encountered was having mismatching seismic loads with the TAs. This primarily had to do with the fact that all of our calculations (this time around) were performed in Excel. Due to the simplicity of manually creating any formula and seeing it cleanly displayed along a set of cells, Excel was chosen. However, we encountered several issues with numbers not matching up. This is because the Excel cells are not unit-aware, so we had some conversion issues. More than this, there were times we simply did not account for loads (we did not include the slab loads or column loads initially). Also, because some of the loads were added manually in the formulas, it was difficult to parse the good portions of the loads from the errors. Our team worked through it by double-checking our calculations to make sure the values made sense, going back and cross-checking with our TAs, and re-running everything. In the future, we will go back to using some sort of automatic calculation software like BlockPad or MathCAD to avoid similar errors.
- Our team managed our schedule differences by being as communicative as possible during this last busy finals week. Because some members of our team had more work outside of class than others, those who had less during the week tried to pick up some slack and get as much work done as possible. We also tried to be vocal about who would complete what part of the project in the "Project Timeline" tab in the Excel spreadsheet.
- The most important thing I learned from this project, as well as this class as a whole, is the beauty of communication. Whenever we ran into an issue between the project's "phases", it was because we had different people working on different phases that did not know what was going on in another phase. I think this helped me to understand that it's vital for engineers to communicate calculations so that it's clear to the average person. I

can't just use numbers and expect people to get it. I need to explain the clear logical flow from A to B, since that is what I will be doing in the real world.

Chris Liu

- My primary role was to perform the seismic weight calculations and lateral force analysis. I also performed the drafting for the project. At first, it was not decided who would do what part, but I decided to take on the lateral force analysis and drafting because I had the most experience with it in the past, again playing to my strengths. As the school year was nearing to a close and group members got busy, some responsibilities shifted to those who had more time on their hands to help complete the project.
- A big technical challenge that occurred was with our lateral force analysis and seismic weight calculation. When checking with the TA during office hours, it was discovered that our forces did not match up by a large amount. After some back and forth, we realized that there were a lot of mistakes with the calculations, such as not including the beam weight, incorrect number of walls, and incorrect area of the building. We spent over an hour with the TAs, painstakingly going through our calculations and making sure values were correct. Perhaps moving forward, we will not use Excel for our calculations as it does not show the equations with the variables, nor does it take into account units.
- Our team managed disagreements over design choices, workload, and approaches by being honest and stressing the importance of communication. We were really good about not being afraid to ask for help, and people were willing to pick up for others and help wherever needed.
- The most important thing I learned from this project is how to manage conflicts, whether it be scheduling, disagreements with other group members, or issues with the calculations. Something that helped navigate this was effective communication. I know I keep mentioning it in this reflection, but communication really does make a difference when it comes to group work. It helps others know what is going on and if there's anything that needs to be addressed or focused on. Without communication, groups would fall apart. Another thing I learned is that not everyone works at the same pace. Some people might work really quickly and will want to finish their work as soon as possible, while others take their time, not necessarily because they are lazy, but because they might have other things they have to focus on or take things slower so that they

understand it better. It’s important that we, as a group, remain understanding and adaptable so that everyone is accounted for.

Hannah Ahn

- My primary role was to do the core wall design. I was originally assigned part E and F but there were complications with part D so I ended up doing parts D to F. A lot of my responsibilites were on the last leg of the project so I took on a lot of the miscellaneous responsibilities that had to be covered to finish the project. This made my role evolve significantly over the course of the project because a portion of the initial parts had to be redone to complete my portion of the project.

- On technical challenge was iterating the P-M diagram. The initial reinforcement design was not adequate so it had to be iterated multiple times before arriving at the final design. I had initially started on Phase E that requires the computing of the neutral axis, c. It relies on the P-M diagram so I worked through it all until I realized that the P-M diagram did not start from an initial design that produced the reinforcement ratios but the ratios were conjured. So I had to go back and do the reinforcement design and then the P-M diagram and then redo Phase E so that I could continue onto Phase F. This mishap took up a lot of time. There was also a slip up on my end where I designed for 6” c.c. for the web curtain reinforcement but wrote down 8” instead. I had to tell my teammate so that he could redo the drawing.

- The approach and roles for this final project was determined more by our schedules and availability than an onset decision. With finals and homework assignments, our schedule for working on this project was uncertain so we worked on it whenever we had time. I did not have time during the beginning of the project so I worked on the later part of the project. This was not an idea approach to working on the project but it is what worked for our team this time around. Next time, taking time to plan out a schedule for completing the project would be most helpful. planning around are uncertainty is definitely possible and would be a better choice then figuring it out as we go but we did this past time. The only issues that arose during this project was some communication uncertainties.This was resolved through clear communication. There were aspects of the project that had to be redone due to mistakes made. this was resolved through communication and reallocating the rules that had to be completed.

- The most important thing I learned from this project is the need for more clear communication at the beginning of a project to assign rules and to make sure that double checking of work can be done before submitting the project. There were a lot of changes that to be made for previous parts done that could have been avoided if our work was double checked earlier on in the project. As a team though, I'm quite satisfied with the work we produced on such a time crunch and with so many responsibilities from other classes piling up.

Brian Tran

- My primary role in the project was the input, geometry & site seismicity. It evolved over the three weeks after the complete project checklist was released I was also responsible for the reinforcement schedule & serviceability. There were moments where responsibilities shifted when we got closer to finals because group members got busy so we tried to help each other out wherever we could.
- One technical challenge that my team encountered was when I was obtaining site seismicity parameters from ATC Hazards tool, I used ASCE 7-22 instead of ASCE 7-16. Because of this, the SDS, SD1 I found were not accurate and caused our design response spectrum to be plotted incorrectly. Luckily one of my group members caught this mistake and gave me the correct values which fixed our design response spectrum.
- My team managed disagreements about design choices, workload, and approach by communicating with each other. We listened to what each other had to say and were open to change and trying different approaches. What worked was dividing the work before we started, so that everyone knew what part of the project they were responsible for.
- The most important thing I learned, technical or professional, from working on this project as a team was communication. A lot of the problems we had were solved by communicating with each other and helping each other understand the different parts each of us worked on individually.

TEAM SUMMARY

Our team started the project by dividing the various roles and responsibilities needed in order to complete this project. Brian was responsible for the input, geometry & site seismicity as well as reinforcement schedule & serviceability. Isaac was responsible for the hand calculations in addition to the core wall flexure design (P-M interaction). Chris was responsible for the seismic weight and lateral force analysis and also the technical drawings. Hannah was responsible for the report formatting, checking over the drawings, boundary element design, and shear design. Each of us chose which portions we wanted to work on. At different points throughout the duration of the project, we asked other members to take on other roles, but it was done through open and clear communication.

One concrete lesson our team will carry into future collaborative engineering work is the importance of communication. We would sometimes run into issues due to the fact that different group members were working on different parts of the project, so communication was key in helping us understand what was happening in each other's different phases of the project. Many of the problems that we encountered were solved by communicating with each other and helping each other come up with the appropriate solutions. Communication is also important when managing conflicts amongst our group, such as scheduling, disagreements, or issues with calculations, as mentioned time and time again. Overall, communication allows each of us to know what is happening with one another and if there is something that requires more attention and without communication we would fall apart.